ROBINSON & COLE LLP

KENNETH C. BALDWIN

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Hartford, CT 06103-3597
Main (860) 275-8200
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kbaldwin@rc.com
Direct (860) 275-8345

EM-VER-056-080521

ORIGINAL

May 21, 2008

Via Hand Delivery

S. Derek Phelps
Executive Director
Connecticut Siting Council
10 Franklin Square
New Britain, CT 06051

Re: **Notice of Exempt Modification**
30 Higley Road
Granby, Connecticut

Dear Mr. Phelps:

Cellco Partnership d/b/a Verizon Wireless ("Cellco") intends to install antennas on the existing 120-foot self-supporting monopole owned by AT&T at 30 Higley Road in Granby, Connecticut. Please accept this letter as notification pursuant to 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 letter is being sent to William F. Smith, Jr., Town Manager of the Town of Granby. Pursuant to a Council directive, a copy of this letter is also being sent to Janet & William Pease, the owners of the property on which the tower is located.

The facility consists of a 120-foot self-supporting monopole tower capable of supporting multiple carriers within a fenced compound at 30 Higley Road in Granby. The tower is currently shared by AT&T (formerly Cingular Wireless) with antennas located at the 120-foot level; and T-Mobile with antennas located at the 107-foot level on the tower. Cellco intends to install six (6) LPA-80080/6CF antennas and six (6) LPA-185080/12CF antennas at the 97-foot level on the tower. Associated equipment, including a diesel fueled back-up generator, will be located within a 12' x 30' equipment shelter on the ground adjacent to the tower. Attached behind Tab 1 are Project Plans for the proposed Cellco facility.

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

1. The proposed modification will not increase the overall height of the existing tower. Cellco's antennas will be mounted with their centerline at the 97-foot level on the 120-foot tower.

2. The proposed installation of associated equipment within the equipment shelter will not require an extension of the fenced compound or lease area.

3. The proposed installation will not increase the noise levels at the facility by six decibels or more.

4. The operation of the antennas will not increase radio frequency (RF) power density levels at the facility to a level at or above the Federal Communications Commission (FCC) adopted safety standard. The RF power density calculations for existing and Cellco antennas would be 31.89% of the FCC standard. A cumulative power density calculations table is included behind Tab 2.

Included behind Tab 3, is a Structural Analysis Report confirming that the tower can support the existing and Cellco antennas, and associated equipment.

For the foregoing reasons, Cellco respectfully submits that the proposed antenna installation at the facility constitutes an exempt modification under R.C.S.A. § 16-50j-72(b)(2).

Sincerely,

Kenneth C. Baldwin

Attachments
Copy to:
William F. Smith, Jr., Granby Town Manager
Janet & William Pease
Sandy M. Carter
Michelle Kababik

Cellco Partnership

d.b.a. verizon wireless

WIRELESS COMMUNICATIONS FACILITY

HUGGINS GORGE CT
30 HIGLEY ROAD
GRANBY, CT 06090

SITE DIRECTIONS

FROM: 99 EAST RIVER DRIVE, EAST HARTFORD, CONNECTICUT TO: 30 HIGLEY ROAD, GRANBY, CONNECTICUT

1. START AT 99 E RIVER DR TOWARD E RIVER DR	<0.1 MI.
2. TURN LEFT ON E RIVER DR	0.3 MI.
3. TURN LEFT ON CONNECTICUT BLVD(US-44)	0.2 MI.
4. TAKE RAMP ONTO I-84 W	0.3 MI.
5. TAKE EXIT #51/SPRINGFIELD ONTO I-91 N	9.4 MI.
6. TAKE EXIT #40/BRADLEY INTERNATIONAL AIRPORT ONTO CT-20 W	3.5 MI.
7. TAKE THE E. GRANBY/GRANBY EXIT ONTO CT-20	9.6 MI.
8. TURN RIGHT ON HIGLEY RD	0.3 MI.
9. ARRIVE AT 30 HIGLEY RD	

GENERAL NOTES

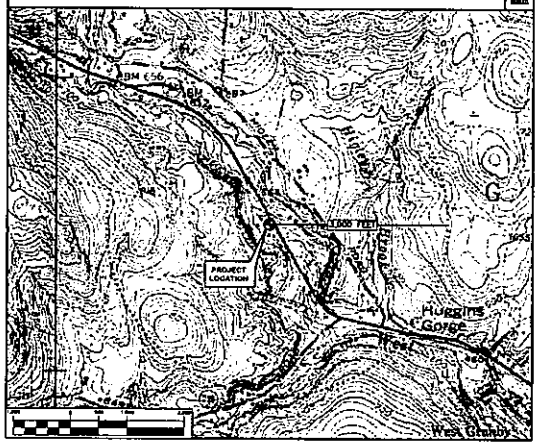
1. PROPOSED ANTENNA LOCATIONS AND HEIGHTS PROVIDED BY CELLCO PARTNERSHIP.

PROJECT SCOPE

1. THE PROPOSED SCOPE OF WORK GENERALLY INCLUDES THE INSTALLATION OF A 12'x30' PREFABRICATED WIRELESS EQUIPMENT SHELTER ON A CONCRETE FOUNDATION WITHIN THE EXISTING WIRELESS COMMUNICATIONS LEASE AREA.
2. A TOTAL OF TWELVE (12) DIRECTIONAL PANEL ANTENNAS ARE PROPOSED TO BE MOUNTED ON AN EXISTING 128' TALL MONOPOLE TOWER AT A RAD CENTER ELEVATION OF 87' ABOVE THE EXISTING TOWER BASE PLATE.
3. ELECTRIC AND TELCO UTILITIES SHALL BE ROUTED UNDERGROUND TO THE PROPOSED EQUIPMENT SHELTER FROM AN EXISTING UTILITY BACKBOARD LOCATED OUTSIDE THE FENCED COMPOUND.

VICINITY MAP

SCALE: 1" = 1000'



PROJECT SUMMARY

SITE NAME:	HUGGINS GORGE CT
SITE ADDRESS:	30 HIGLEY ROAD GRANBY, CT 06090
LESSEE/ TENANT:	CELLCO PARTNERSHIP d.b.a. VERIZON WIRELESS 99 EAST RIVER DRIVE EAST HARTFORD, CT 06108
CONTACT PERSON:	SANDY CARTER CELLCO PARTNERSHIP (860) 803-6210
TOWER COORDINATES:	LATITUDE: 41°-57'-56.8" LONGITUDE: 72°-51'-19.3" COORDINATES BASED ON CSC WEB LOG LAST UPDATED 5/23/07.

SHEET INDEX

SHT. NO.	DESCRIPTION	REV. NO.
T-1	TITLE SHEET	B
C-1	COMPOUND PLAN AND ELEVATION	B

REVISIONS		
A	10/16/07	CSC - REVIEW
B	05/15/08	CSC

Cellco Partnership d.b.a. verizon wireless

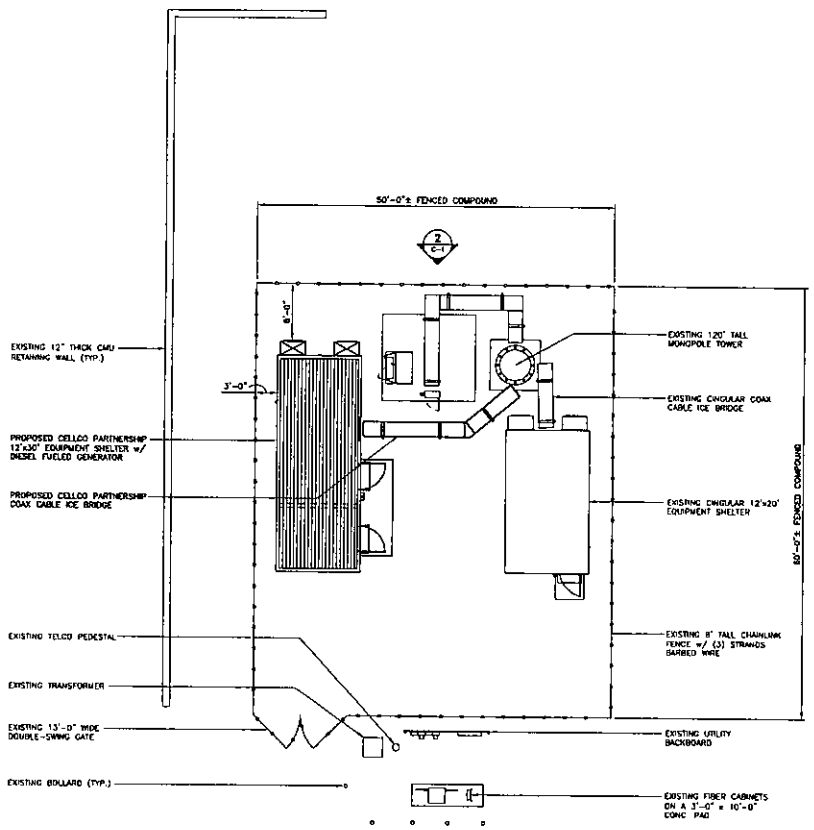
NATCOMM
CONSULTING ENGINEERS
p: 203.488.0580
f: 203.488.8587
w: natcomm.com
e: info@nat-eng.com
63-2 N. Branford Rd.
Branford, CT 06405

HUGGINS GORGE CT
30 HIGLEY ROAD
GRANBY, CT 06090

PROJECT NO:	07073
DRAWN BY:	DEB
CHECKED BY:	CFC
SCALE:	AS NOTED
DATE:	10/10/07

TITLE SHEET

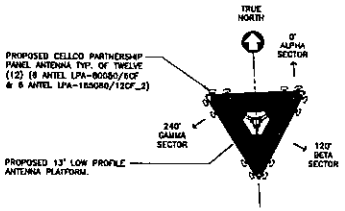
T-1
DWG. 1 OF 2



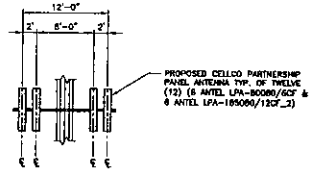
1 C-1 COMPOUND PLAN
SCALE: 1/8" = 1'-0"

TRUE NORTH

GRAPHIC SCALE
(IN FEET)
1/8 inch = 1 ft.

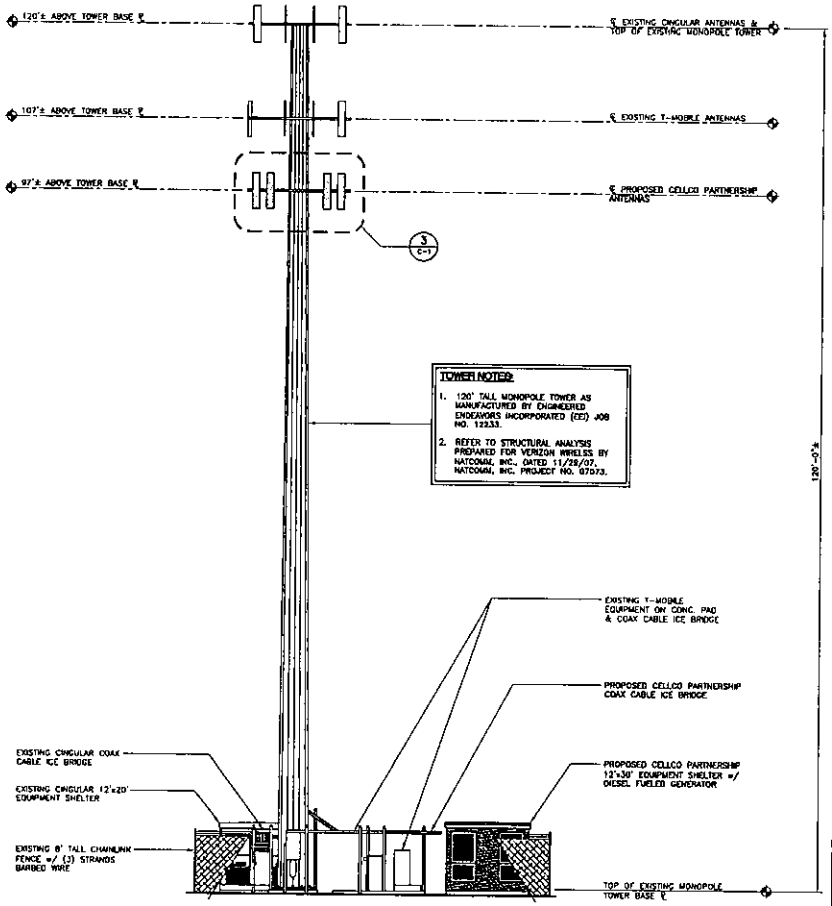


PLAN VIEW



TYPICAL SECTOR ELEVATION

3 C-1 ANTENNA MOUNTING CONFIGURATION
NOT TO SCALE



2 C-1 NORTH ELEVATION
SCALE: 1/8" = 1'-0"

GRAPHIC SCALE
(IN FEET)
1/8 inch = 1 ft.

REVISIONS		
A	10/16/07	CSC - REVIEW
B	09/15/08	CSC

Cellco Partnership
d.b.a. Verizon wireless

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HUGGINS GORGE CT

38 HIGLEY ROAD
GRANBY, CT 06090

PROJECT NO:	07073
DRAWN BY:	DEB
CHECKED BY:	CFC
SCALE:	AS NOTED
DATE:	10/10/07

COMPOUND PLAN AND ELEVATION

C-1

DWG. 2 OF 2

	General	Power	Density					
Site Name: Huggins Gorge (Granby)								
Tower Height: Verizon @ 97Ft.								
CARRIER	**# OF CHAN.**	**WATTS ERP**	**HEIGHT**	**CALC. POWER DENS**	**FREQ.**	**MAX. PERMISS. EXP.**	**FRACTION MPE**	**Total**
*Cingular	6	296	120	0.0443	880	0.5867	7.56%	
*Cingular	3	427	120	0.0320	1930	1.0000	3.20%	
*T-Mobile	8	150	107	0.0377	1935	1.0000	3.77%	
Verizon	**3**	**485**	**97**	**0.0556**	**869**	**1.0000**	**5.56%**	
Verizon	**9**	**200**	**97**	**0.0688**	**1900**	**0.583**	**11.80%**	
								31.89%
* Source: Siting Council								

NATCOMM INC.
CONSULTING ENGINEERS

Structural Analysis Report

120' Existing Monopole

30 Higley Road
Granby, CT

Natcomm Project No. 07073

Date: November 29, 2007

Prepared for:

Verizon Wireless
99 East River Road, 9th Floor
East Hartford, CT 06108

p: 203.488.0580
f: 203.488.8587
w: nat-eng.com
63-2 N. Branford Rd.
Branford, CT 06405

Table of Contents

Introduction

The purpose of this report is to summarize the results of the non-linear, P-Δ structural analysis of the antenna and platform installation proposed on the existing monopole (tower) located in Granby, Connecticut. The host tower is a 120-ft, three section, eighteen sided, tapered monopole originally designed and manufactured by Engineered Endeavors Incorporated (EEI); job no. 12233-E02 dated July 26, 2005 (revision 1). EEI's original structure and foundation design calculations are available for reference in Section 4 of this report.

Antenna and Appurtenance Summary

The existing tower was designed to support several communication antennas. The existing, proposed and future loads considered in this analysis consist of the following:

- CINGULAR (Existing):
  Antennas: Twelve (12) Powerwave 7770.00 panel antennas and twelve (12) Powerwave LGP21401 TMAs mounted on a 13-ft low profile platform with a RAD center elevation of 120-ft above the existing tower base plate .
  Coax Cables: Twelve (12) 1-5/8" ∅ coax cables running on the inside of the existing tower.
- T-MOBILE (Existing):
  Antennas: Nine (9) APX16PV-16PVL-X panel antennas and six (6) G20057A1 TMAs mounted on a 15-ft square platform with rails with a RAD center elevation of 107-ft above the existing tower base plate.
  Coax Cables: Twelve (12) 1-5/8" ∅ coax cables running on the inside of the existing tower.
- **VERIZON (Proposed):**
  Antennas: Six (6) Amphenol Antel, Inc. (Antel) LPA-80080/6CF and six (6) Antel LPA-185080/12CF_2 panel antennas mounted on a 13-ft low profile platform with a RAD center elevation of 97-ft above the existing tower base plate .
  Coax Cables: Twelve (12) 1-5/8" ∅ coax cables running on the inside of the existing tower.
- FUTURE CARRIER:
  Antennas: Twelve (12) Powerwave 7770.00 panel antennas mounted on a 13-ft low profile platform with a RAD center elevation of 90-ft above the existing tower base plate.
  Coax Cables: Twelve (12) 1-5/8" ∅ coax cables running on the inside of the existing tower.
- FUTURE CARRIER:
  Antennas: Twelve (12) Powerwave 7770.00 panel antennas mounted on a 13-ft low profile platform with a RAD center elevation of 80-ft above the existing tower base plate.
  Coax Cables: Twelve (12) 1-5/8" ∅ coax cables running on the inside of the existing tower.

- FUTURE CARRIER:
  Antennas: Twelve (12) Powerwave 7770.00 panel antennas mounted on a 13-ft low profile platform with a RAD center elevation of 70-ft above the existing tower base plate.
  Coax Cables: Twelve (12) 1-5/8" ∅ coax cables running on the inside of the existing tower.

Primary Assumptions Used in the Analysis

- The tower structure's theoretical capacity not including any assessment of the condition of the tower.
- The tower carries the horizontal and vertical loads due to the weight of antennas, ice load and wind.
- Tower is properly installed and maintained.
- Tower is in plumb condition.
- Tower loading for antennas and mounts as listed in this report.
- All bolts are appropriately tightened providing the necessary connection continuity.
- All welds are fabricated with ER-70S-6 electrodes.
- All members are assumed to be as specified in the original tower design documents.
- All members are "hot dipped" galvanized in accordance with ASTM A123 and ASTM A153 Standards.
- All member protective coatings are in good condition.
- All tower members were properly designed, detailed, fabricated, installed and have been properly maintained since erection.
- Any deviation from the analyzed antenna loading will require a new analysis for verification of structural adequacy.
- All coax cables to be installed within tower.
- A new porthole will not be required.

Analysis

The existing tower was analyzed using a comprehensive computer program entitled RISATower. The program analyzes the tower, considering the worst case loading condition. The tower is considered as loaded by concentric forces along the tower shaft, and the model assumes that the shaft members are subjected to bending, axial, and shear forces.

The existing tower was analyzed for 80mph basic wind speed (fastest mile) with no ice and 75% reduction of wind force with ½ inch accumulative ice to determine stresses in members as per guidelines of TIA/EIA-222-F-96 entitled "Structural Standards for Steel Antenna Towers and Antenna Supporting Structures", the American Institute of Steel Construction (AISC) and the Manual of Steel Construction; Allowable Stress Design (ASD).

Tower Loading

Tower loading was determined by the basic wind speed as applied to projected surface areas with modification factors per TIA/EIA-222-F, gravity loads of the tower structure and its components, and the application of ½" radial ice tower structure and its components.

Basic Wind Speed:	Hartford; v = 80 mph (fastest mile)	*[Section 16 of TIA/EIA-222-F-96]*
	Granby; v = 90 mph (3 second gust) equivalent to v = 75 mph (fastest mile)	*[Appendix K of the 2005 CT Building Code Supplement]*
	TIA/EIA wind speed Controls	
Load Cases:	Load Case 1; 80 mph wind speed w/ no ice plus gravity load – used in calculation of tower stresses and rotation. This load case typically controls the design.	*[Section 2.3.16 of TIA/EIA-222-F-96]*
	Load Case 2; 69 mph wind speed w/ ½" radial ice plus gravity load – used in calculation of tower stresses. The 69 mph wind speed velocity represents 75% of the wind pressure generated by the 80 mph wind speed.	*[Section 2.3.16 of TIA/EIA-222-F-96]*
	Load Case 3; Seismic – not checked	*[Section 1610.1.3 of State Bldg. Code 1999] does not control in the design of this structure type*

Tower Capacity

Tower stresses were calculated utilizing the structural analysis software RISATower. Allowable stresses were determined based on Table 5 of the TIA/EIA code with a 1/3 increase per Section 3.1.1.1 of the same code.

Calculated stresses were found to be within allowable limits. In Load Case 1, per RISATower "Section Capacity Table", this tower was found to be at **78.1%** of its total capacity.

Foundation and Anchors

The existing foundation consists of a 7-ft square reinforced concrete pedestal with a 22-ft square reinforced concrete pad bearing directly on existing sub grade. The sub grade conditions used in the analysis of the existing foundation were obtained from Jaworski Geotech, Inc's (JGI) geotechnical report, JGI project No. 03735G dated January 6, 2004, available in section 4 of this report. The monopole tower is connected to the pedestal by means of twelve (12) 2 1/4" diameter, A615-GR75 anchor bolts embedded 7-ft into the concrete foundation structure.

Review of the foundation and anchor design consisted of verification of applied loads obtained from the tower design calculations and code checks of allowable stresses:

- The tower base reactions developed from the governing Load Case 1 were used in the verification of the foundation and its anchors:
  - Shear Force @ top of pedestal = **23.0** kips
  - Moment @ top of pedestal = **1,950.0** ft-kips
  - Axial Force @ top of pedestal = **31.0** kips
- The base plate, anchor bolts and the foundation are within allowable limits.
- Foundation resists two times the calculated wind load per the requirements of section 3108.4.2 of the 2005 CT State Building Code Supplement to the 2003 International Building Code (IBC).

Conclusions

This analysis shows that the subject tower **is adequate** to support the proposed antennas, platform and associated hardware.

The analysis is based, in part, on the information provided to this office by Verizon Wireless. If the existing conditions are different than the information in this report, Natcomm, Inc. must be contacted for resolution of any potential issues.

Please feel free to call with any questions or comments.

Respectfully Submitted by:

Carlo F. Centore, PE
Principal ~ Structural Engineer

Standard Conditions for Furnishing of Professional Engineering Services on Existing Structures

All engineering services are performed on the basis that the information used is current and correct. This information may consist of, but is not necessarily limited to:

- Information supplied by the client regarding the structure itself, its foundations, the soil conditions, the antenna and feed line loading on the structure and its components, or other relevant information.
- Information from the field and/or drawings in the possession of Natcomm, Inc. or generated by field inspections or measurements of the structure.
- It is the responsibility of the client to ensure that the information provide to Natcomm, Inc. and used in the performance of our engineering services is correct and complete. In the absence of information to the contrary, we assume that all structures were constructed in accordance with the drawings and specifications and are in an un-corroded condition and have not deteriorated. It is therefore assumed that its capacity has not significantly changed from the "as new" condition.
- All services will be performed to the codes specified by the client, and we do not imply to meet any other codes or requirements unless explicitly agreed in writing. If wind and ice loads or other relevant parameters are to be different from the minimum values recommended by the codes, the client shall specify the exact requirement. In the absence of information to the contrary, all work will be performed in accordance with the latest revision of ANSI/ASCE10 & ANSI/EIA-222
- All services performed, results obtained, and recommendations made are in accordance with generally accepted engineering principles and practices. Natcomm, Inc. is not responsible for the conclusions, opinions and recommendations made by others based on the information we supply.

GENERAL DESCRIPTION OF STRUCTURAL ANALYSIS PROGRAM

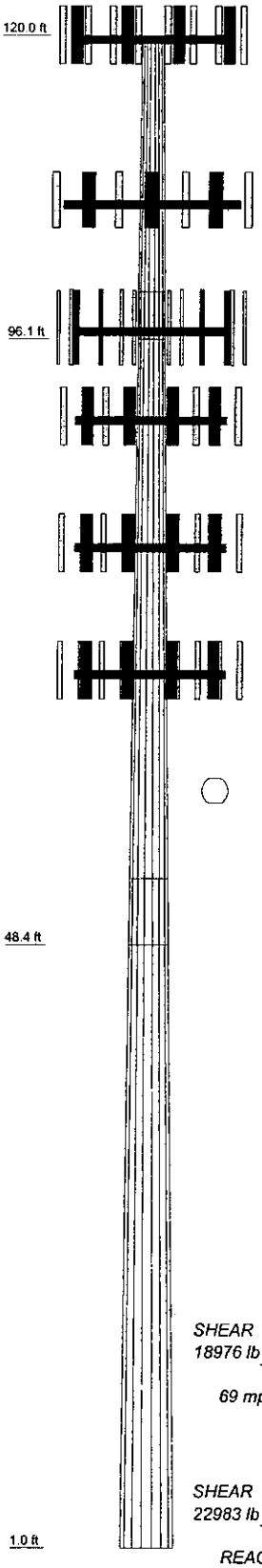
RISATower, is an integrated structural analysis and design software package for Designed specifically for the telecommunications industry, RISATower, formerly ERITower, automates much of the tower analysis and design required by the TIA/EIA 222 Standard.

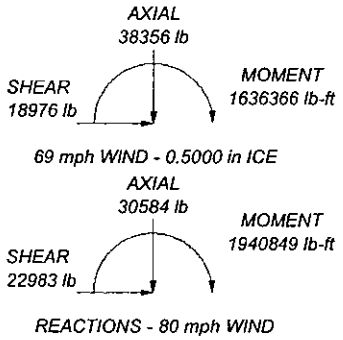
RISATower Features:

- RISATower can analyze and design 3- and 4-sided guyed towers, 3- and 4-sided self-supporting towers and either round or tapered ground mounted poles with or without guys.
- The program analyzes towers using the TIA-222-G (2005) standard or any of the previous TIA/EIA standards back to RS-222 (1959). Steel design is checked using the AISC ASD 9th Edition or the AISC LRFD specifications.
- Linear and non-linear (P-delta) analyses can be used in determining displacements and forces in the structure. Wind pressures and forces are automatically calculated.
- Extensive graphics plots include material take-off, shear-moment, leg compression, displacement, twist, feed line, guy anchor and stress plots.
- RISATower contains unique features such as True Cable behavior, hog rod take-up, foundation stiffness and much more.
-

Section	1	2	3
Length (ft)	23.88	51.50	52.62
Number of Sides	18	18	18
Thickness (in)	0.1875	0.2500	0.3125
Lap Splice (ft)	3.75	5.25	
Top Dia (in)	19.0000	24.0621	35.9355
Bot Dia (in)	25.4500	37.8400	50.0000
Grade	A572-65		
Weight (lb)	1065.7	4269.1	7575.5

Total Weight: 12910.4





DESIGNED APPURTENANCE LOADING

TYPE	ELEVATION	TYPE	ELEVATION
13' Low Profile Platform (Cingular)	120	APX16PV-16PVL-X (T-Mobile (exist))	107
7770.00 (Cingular (exist))	120	(2) G20057A1 TMA (T-Mobile (exist))	107
(2) LPG21401 TMA (Cingular (exist))	120	(2) APX16PV-16PVL-X (T-Mobile (future))	107
7770.00 (Cingular (future))	120	13' Low Profile Platform (Verizon)	97
7770.00 (Cingular (future))	120	LPA-80080/6CF (Verizon)	97
7770.00 (Cingular (exist))	120	LPA-185080/12CF (Verizon)	97
(2) LPG13519 Diplexer (Cingular (exist))	120	LPA-185080/12CF (Verizon)	97
7770.00 (Cingular (exist))	120	LPA-80080/6CF (Verizon)	97
(2) LPG21401 TMA (Cingular (exist))	120	LPA-80080/6CF (Verizon)	97
7770.00 (Cingular (future))	120	LPA-185080/12CF (Verizon)	97
7770.00 (Cingular (future))	120	LPA-185080/12CF (Verizon)	97
7770.00 (Cingular (exist))	120	LPA-80080/6CF (Verizon)	97
(2) LPG13519 Diplexer (Cingular (exist))	120	LPA-80080/6CF (Verizon)	97
		LPA-185080/12CF (Verizon)	97
7770.00 (Cingular (exist))	120	LPA-185080/12CF (Verizon)	97
(2) LPG21401 TMA (Cingular (exist))	120	LPA-80080/6CF (Verizon)	97
7770.00 (Cingular (future))	120	13' Low Profile Platform (Future)	90
7770.00 (Cingular (future))	120	(4) 7770.00 (Future)	90
7770.00 (Cingular (exist))	120	(4) 7770.00 (Future)	90
(2) LPG13519 Diplexer (Cingular (exist))	120	(4) 7770.00 (Future)	90
15' Square Platform w/rails (T-Mobile)	107	13' Low Profile Platform (Future)	80
APX16PV-16PVL-X (T-Mobile (exist))	107	(4) 7770.00 (Future)	80
(2) G20057A1 TMA (T-Mobile (exist))	107	(4) 7770.00 (Future)	80
(2) APX16PV-16PVL-X (T-Mobile (future))	107	(4) 7770.00 (Future)	80
		13' Low Profile Platform (Future)	70
APX16PV-16PVL-X (T-Mobile (exist))	107	(4) 7770.00 (Future)	70
(2) G20057A1 TMA (T-Mobile (exist))	107	(4) 7770.00 (Future)	70
(2) APX16PV-16PVL-X (T-Mobile (future))	107	(4) 7770.00 (Future)	70

MATERIAL STRENGTH

GRADE	Fy	Fu	GRADE	Fy	Fu
A572-65	65 ksi	80 ksi			

TOWER DESIGN NOTES

1. Tower designed for a 80 mph basic wind in accordance with the TIA/EIA-222-F Standard.
2. Tower is also designed for a 69 mph basic wind with 0.50 in ice.
3. Deflections are based upon a 60 mph wind.
4. Tower members are "hot dipped" galvanized in accordance with ASTM A123 and ASTM A153 Standards.
5. Welds are fabricated with ER-70S-6 electrodes.
6. TOWER RATING: 78.1%

NATCOMM
63-2 N. Branford Rd.
Branford, CT 06405
Phone: (203) 488-0580
FAX: (203) 488-8587

Job: *120' EEI Monopole*			
Project: *07073 - 30 Higley Road, Granby, CT*			
Client: Verizon	Drawn by: Staff	App'd:	
Code: TIA/EIA-222-F	Date: 10/15/07	Scale: NTS	
Path: C:\Users\wmah214\Documents\Natcomm\07073 Granby\ERI Feed\120 EEI Granby.eji		Dwg No. E-1	

Tower Input Data

There is a pole section.
This tower is designed using the TIA/EIA-222-F standard.
The following design criteria apply:

- Basic wind speed of 80 mph.
- Nominal ice thickness of 0.5000 in.
- Ice density of 56 pcf.
- A wind speed of 69 mph is used in combination with ice.
- Temperature drop of 50 °F.
- Deflections calculated using a wind speed of 60 mph.
- Tower members are "hot dipped" galvanized in accordance with ASTM A123 and ASTM A153 Standards..
- Welds are fabricated with ER-70S-6 electrodes..
- A non-linear (P-delta) analysis was used.
- Pressures are calculated at each section.
- Stress ratio used in pole design is 1.333.
- Local bending stresses due to climbing loads, feedline supports, and appurtenance mounts are not considered.

Options

	Consider Moments - Legs		Distribute Leg Loads As Uniform		Treat Feedline Bundles As Cylinder
	Consider Moments - Horizontals		Assume Legs Pinned		Use ASCE 10 X-Brace Ly Rules
	Consider Moments - Diagonals	√	Assume Rigid Index Plate		Calculate Redundant Bracing Forces
	Use Moment Magnification		Use Clear Spans For Wind Area		Ignore Redundant Members in FEA
√	Use Code Stress Ratios		Use Clear Spans For KL/r		SR Leg Bolts Resist Compression
√	Use Code Safety Factors - Guys		Retension Guys To Initial Tension		All Leg Panels Have Same Allowable
	Escalate Ice		Bypass Mast Stability Checks		Offset Girt At Foundation
	Always Use Max Kz		Use Azimuth Dish Coefficients	√	Consider Feedline Torque
	Use Special Wind Profile	√	Project Wind Area of Appurt.		Include Angle Block Shear Check
	Include Bolts In Member Capacity		Autocalc Torque Arm Areas		Poles
	Leg Bolts Are At Top Of Section		SR Members Have Cut Ends		Include Shear-Torsion Interaction
	Secondary Horizontal Braces Leg	√	Sort Capacity Reports By Component		Always Use Sub-Critical Flow
	Use Diamond Inner Bracing (4 Sided)		Triangulate Diamond Inner Bracing		Use Top Mounted Sockets
	Add IBC .6D+W Combination				

Tapered Pole Section Geometry

Section	*Elevation*	*Section Length*	*Splice Length*	*Number of Sides*	*Top Diameter*	*Bottom Diameter*	*Wall Thickness*	*Bend Radius*	*Pole Grade*
	ft	*ft*	*ft*		*in*	*in*	*in*	*in*	
L1	120.00-96.12	23.88	3.75	18	19.0000	25.4500	0.1875	0.7500	A572-65 (65 ksi)
L2	96.12-48.37	51.50	5.25	18	24.0621	37.8400	0.2500	1.0000	A572-65 (65 ksi)
L3	48.37-1.00	52.62		18	35.9355	50.0000	0.3125	1.2500	A572-65 (65 ksi)

Tapered Pole Properties

Section	Tip Dia. in	Area in²	I in⁴	r in	C in	I/C in³	J in⁴	It/Q in²	w in	w/t
L1	19.2931	11.1958	500.5935	6.6784	9.6520	51.8642	1001.8456	5.5990	3.0140	16.075
	25.8426	15.0343	1212.2010	8.9682	12.9286	93.7612	2425.9970	7.5186	4.1492	22.129
L2	25.4520	18.8949	1353.5638	8.4533	12.2236	110.7340	2708.9087	9.4493	3.7949	15.18
	38.4237	29.8277	5324.7762	13.3445	19.2227	277.0043	10656.5589	14.9167	6.2198	24.879
L3	37.9147	35.3335	5664.7973	12.6462	18.2552	310.3112	11337.0485	17.6701	5.7746	18.479
	50.7713	49.2838	15372.1931	17.6391	25.4000	605.2045	30764.6134	24.6466	8.2500	26.4

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
L1 120.00-96.12				1	1	1		
L2 96.12-48.37				1	1	1		
L3 48.37-1.00				1	1	1		

Feed Line/Linear Appurtenances - Entered As Area

Description	Face or Leg	Allow Shield	Component Type	Placement ft	Total Number		C_AA_A ft²/ft	Weight plf
1 5/8 (Cingular)	A	No	Inside Pole	120.00 - 4.00	12	No Ice	0.00	1.04
						1/2" Ice	0.00	1.04
1 5/8 (T-Mobile)	B	No	Inside Pole	107.00 - 4.00	12	No Ice	0.00	1.04
						1/2" Ice	0.00	1.04
1 5/8 (Verizon)	C	No	Inside Pole	97.00 - 4.00	12	No Ice	0.00	1.04
						1/2" Ice	0.00	1.04
1 5/8 (Future)	A	No	Inside Pole	90.00 - 4.00	12	No Ice	0.00	1.04
						1/2" Ice	0.00	1.04
1 5/8 (Future)	B	No	Inside Pole	80.00 - 4.00	12	No Ice	0.00	1.04
						1/2" Ice	0.00	1.04
1 5/8 (Future)	C	No	Inside Pole	70.00 - 4.00	12	No Ice	0.00	1.04
						1/2" Ice	0.00	1.04

Feed Line/Linear Appurtenances Section Areas

Tower Section	Tower Elevation ft	Face	A_R ft²	A_F ft²	C_AA_A In Face ft²	C_AA_A Out Face ft²	Weight lb
L1	120.00-96.12	A	0.000	0.000	0.000	0.000	298.02
		B	0.000	0.000	0.000	0.000	135.78
		C	0.000	0.000	0.000	0.000	10.98
L2	96.12-48.37	A	0.000	0.000	0.000	0.000	1115.46
		B	0.000	0.000	0.000	0.000	990.66
		C	0.000	0.000	0.000	0.000	865.86
L3	48.37-1.00	A	0.000	0.000	0.000	0.000	1107.48
		B	0.000	0.000	0.000	0.000	1107.48
		C	0.000	0.000	0.000	0.000	1107.48

Feed Line/Linear Appurtenances Section Areas - With Ice

Tower Section	Tower Elevation ft	Face or Leg	Ice Thickness in	A_R ft²	A_F ft²	C_AA_A In Face ft²	C_AA_A Out Face ft²	Weight lb
L1	120.00-96.12	A	0.500	0.000	0.000	0.000	0.000	298.02
		B		0.000	0.000	0.000	0.000	135.78
		C		0.000	0.000	0.000	0.000	10.98
L2	96.12-48.37	A	0.500	0.000	0.000	0.000	0.000	1115.46
		B		0.000	0.000	0.000	0.000	990.66
		C		0.000	0.000	0.000	0.000	865.86
L3	48.37-1.00	A	0.500	0.000	0.000	0.000	0.000	1107.48
		B		0.000	0.000	0.000	0.000	1107.48
		C		0.000	0.000	0.000	0.000	1107.48

Feed Line Center of Pressure

Section	Elevation ft	CP_X in	CP_Z in	CP_X Ice in	CP_Z Ice in
L1	120.00-96.12	0.0000	0.0000	0.0000	0.0000
L2	96.12-48.37	0.0000	0.0000	0.0000	0.0000
L3	48.37-1.00	0.0000	0.0000	0.0000	0.0000

Discrete Tower Loads

Description	Face or Leg	Offset Type	Offsets: Horz Lateral ft / Vert ft	Azimuth Adjustment °	Placement ft		C_AA_A Front ft²	C_AA_A Side ft²	Weight lb
13' Low Profile Platform (Cingular)	C	None		0.0000	120.00	No Ice	15.70	15.70	1300.00
						1/2" Ice	20.10	20.10	1765.00
7770.00 (Cingular (exist))	A	From Face	4.00 -6.00 0.00	0.0000	120.00	No Ice 1/2" Ice	5.88 6.31	2.93 3.27	35.00 67.63
(2) LPG21401 TMA (Cingular (exist))	A	From Face	4.00 -6.00 0.00	0.0000	120.00	No Ice 1/2" Ice	0.95 1.09	0.37 0.48	17.50 23.31
7770.00 (Cingular (future))	A	From Face	4.00 -2.00 0.00	0.0000	120.00	No Ice 1/2" Ice	5.88 6.31	2.93 3.27	35.00 67.63
7770.00 (Cingular (future))	A	From Face	4.00 2.00 0.00	0.0000	120.00	No Ice 1/2" Ice	5.88 6.31	2.93 3.27	35.00 67.63
7770.00 (Cingular (exist))	A	From Face	4.00 6.00 0.00	0.0000	120.00	No Ice 1/2" Ice	5.88 6.31	2.93 3.27	35.00 67.63
(2) LPG13519 Diplexer (Cingular (exist))	A	From Face	4.00 6.00 0.00	0.0000	120.00	No Ice 1/2" Ice	0.27 0.34	0.18 0.25	5.30 7.71
7770.00	B	From Face	4.00	0.0000	120.00	No Ice	5.88	2.93	35.00

(Cingular (exist))			-6.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(2) LPG21401 TMA	B	From Face	4.00	0.0000	120.00	No Ice	0.95	0.37	17.50
(Cingular (exist))			-6.00			1/2" Ice	1.09	0.48	23.31
			0.00						
7770.00	B	From Face	4.00	0.0000	120.00	No Ice	5.88	2.93	35.00
(Cingular (future))			-2.00			1/2" Ice	6.31	3.27	67.63
			0.00						
7770.00	B	From Face	4.00	0.0000	120.00	No Ice	5.88	2.93	35.00
(Cingular (future))			2.00			1/2" Ice	6.31	3.27	67.63
			0.00						
7770.00	B	From Face	4.00	0.0000	120.00	No Ice	5.88	2.93	35.00
(Cingular (exist))			6.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(2) LPG13519 Diplexer	B	From Face	4.00	0.0000	120.00	No Ice	0.27	0.18	5.30
(Cingular (exist))			6.00			1/2" Ice	0.34	0.25	7.71
			0.00						
7770.00	C	From Face	4.00	0.0000	120.00	No Ice	5.88	2.93	35.00
(Cingular (exist))			-6.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(2) LPG21401 TMA	C	From Face	4.00	0.0000	120.00	No Ice	0.95	0.37	17.50
(Cingular (exist))			-6.00			1/2" Ice	1.09	0.48	23.31
			0.00						
7770.00	C	From Face	4.00	0.0000	120.00	No Ice	5.88	2.93	35.00
(Cingular (future))			-2.00			1/2" Ice	6.31	3.27	67.63
			0.00						
7770.00	C	From Face	4.00	0.0000	120.00	No Ice	5.88	2.93	35.00
(Cingular (future))			2.00			1/2" Ice	6.31	3.27	67.63
			0.00						
7770.00	C	From Face	4.00	0.0000	120.00	No Ice	5.88	2.93	35.00
(Cingular (exist))			6.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(2) LPG13519 Diplexer	C	From Face	4.00	0.0000	120.00	No Ice	0.27	0.18	5.30
(Cingular (exist))			6.00			1/2" Ice	0.34	0.25	7.71
			0.00						
15' Square Platform w/rails	C	None		0.0000	107.00	No Ice	25.00	25.00	2000.00
(T-Mobile)						1/2" Ice	30.00	30.00	2500.00
APX16PV-16PVL-X	A	From Face	5.00	0.0000	107.00	No Ice	6.70	2.00	40.00
(T-Mobile (exist))			0.00			1/2" Ice	7.13	2.33	71.34
			0.00						
(2) G20057A1 TMA	A	From Face	5.00	0.0000	107.00	No Ice	0.82	0.39	11.00
(T-Mobile (exist))			0.00			1/2" Ice	0.95	0.49	16.41
			0.00						
(2) APX16PV-16PVL-X	A	From Face	5.00	0.0000	107.00	No Ice	6.70	2.00	40.00
(T-Mobile (future))			0.00			1/2" Ice	7.13	2.33	71.34
			0.00						
APX16PV-16PVL-X	B	From Face	5.00	0.0000	107.00	No Ice	6.70	2.00	40.00
(T-Mobile (exist))			0.00			1/2" Ice	7.13	2.33	71.34
			0.00						
(2) G20057A1 TMA	B	From Face	5.00	0.0000	107.00	No Ice	0.82	0.39	11.00
(T-Mobile (exist))			0.00			1/2" Ice	0.95	0.49	16.41
			0.00						
(2) APX16PV-16PVL-X	B	From Face	5.00	0.0000	107.00	No Ice	6.70	2.00	40.00
(T-Mobile (future))			0.00			1/2" Ice	7.13	2.33	71.34
			0.00						
APX16PV-16PVL-X	C	From Face	5.00	0.0000	107.00	No Ice	6.70	2.00	40.00
(T-Mobile (exist))			0.00			1/2" Ice	7.13	2.33	71.34
			0.00						
(2) G20057A1 TMA	C	From Face	5.00	0.0000	107.00	No Ice	0.82	0.39	11.00
(T-Mobile (exist))			0.00			1/2" Ice	0.95	0.49	16.41
			0.00						
(2) APX16PV-16PVL-X	C	From Face	5.00	0.0000	107.00	No Ice	6.70	2.00	40.00
(T-Mobile (future))			0.00			1/2" Ice	7.13	2.33	71.34
			0.00						

13' Low Profile Platform	C	None		0.0000	97.00	No Ice	15.70	15.70	1300.00
(Verizon)						1/2" Ice	20.10	20.10	1765.00
LPA-80080/6CF	A	From Face	4.00	0.0000	97.00	No Ice	4.33	9.09	21.00
(Verizon)			-6.00			1/2" Ice	4.76	9.64	69.24
			0.00						
LPA-185080/12CF	A	From Face	4.00	0.0000	97.00	No Ice	3.53	4.57	11.00
(Verizon)			-4.00			1/2" Ice	3.96	5.01	37.49
			0.00						
LPA-185080/12CF	A	From Face	4.00	0.0000	97.00	No Ice	3.53	4.57	11.00
(Verizon)			4.00			1/2" Ice	3.96	5.01	37.49
			0.00						
LPA-80080/6CF	A	From Face	4.00	0.0000	97.00	No Ice	4.33	9.09	21.00
(Verizon)			6.00			1/2" Ice	4.76	9.64	69.24
			0.00						
LPA-80080/6CF	B	From Face	4.00	0.0000	97.00	No Ice	4.33	9.09	21.00
(Verizon)			-6.00			1/2" Ice	4.76	9.64	69.24
			0.00						
LPA-185080/12CF	B	From Face	4.00	0.0000	97.00	No Ice	3.53	4.57	11.00
(Verizon)			-4.00			1/2" Ice	3.96	5.01	37.49
			0.00						
LPA-185080/12CF	B	From Face	4.00	0.0000	97.00	No Ice	3.53	4.57	11.00
(Verizon)			4.00			1/2" Ice	3.96	5.01	37.49
			0.00						
LPA-80080/6CF	B	From Face	4.00	0.0000	97.00	No Ice	4.33	9.09	21.00
(Verizon)			6.00			1/2" Ice	4.76	9.64	69.24
			0.00						
LPA-80080/6CF	C	From Face	4.00	0.0000	97.00	No Ice	4.33	9.09	21.00
(Verizon)			-6.00			1/2" Ice	4.76	9.64	69.24
			0.00						
LPA-185080/12CF	C	From Face	4.00	0.0000	97.00	No Ice	3.53	4.57	11.00
(Verizon)			-4.00			1/2" Ice	3.96	5.01	37.49
			0.00						
LPA-185080/12CF	C	From Face	4.00	0.0000	97.00	No Ice	3.53	4.57	11.00
(Verizon)			4.00			1/2" Ice	3.96	5.01	37.49
			0.00						
LPA-80080/6CF	C	From Face	4.00	0.0000	97.00	No Ice	4.33	9.09	21.00
(Verizon)			6.00			1/2" Ice	4.76	9.64	69.24
			0.00						
13' Low Profile Platform	C	None		0.0000	90.00	No Ice	15.70	15.70	1300.00
(Future)						1/2" Ice	20.10	20.10	1765.00
(4) 7770.00	A	From Face	4.00	0.0000	90.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(4) 7770.00	B	From Face	4.00	0.0000	90.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(4) 7770.00	C	From Face	4.00	0.0000	90.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63
			0.00						
13' Low Profile Platform	C	None		0.0000	80.00	No Ice	15.70	15.70	1300.00
(Future)						1/2" Ice	20.10	20.10	1765.00
(4) 7770.00	A	From Face	4.00	0.0000	80.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(4) 7770.00	B	From Face	4.00	0.0000	80.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(4) 7770.00	C	From Face	4.00	0.0000	80.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63
			0.00						
13' Low Profile Platform	C	None		0.0000	70.00	No Ice	15.70	15.70	1300.00
(Future)						1/2" Ice	20.10	20.10	1765.00
(4) 7770.00	A	From Face	4.00	0.0000	70.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63

			0.00						
(4) 7770.00	B	From Face	4.00	0.0000	70.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63
			0.00						
(4) 7770.00	C	From Face	4.00	0.0000	70.00	No Ice	5.88	2.93	35.00
(Future)			0.00			1/2" Ice	6.31	3.27	67.63
			0.00						

Tower Pressures - No Ice

$G_H = 1.690$

Section Elevation ft	z ft	K_z	q_z psf	A_G ft²	Face	A_F ft²	A_R ft²	A_{leg} ft²	Leg %	C_AA_A In Face ft²	C_AA_A Out Face ft²
L1	107.48	1.401	23	44.228	A	0.000	44.228	44.228	100.00	0.000	0.000
120.00-96.12					B	0.000	44.228		100.00		
					C	0.000	44.228		100.00		
L2 96.12-48.37	71.19	1.246	20	125.155	A	0.000	125.155	125.155	100.00	0.000	0.000
					B	0.000	125.155		100.00		
					C	0.000	125.155		100.00		
L3 48.37-1.00	23.70	1	17	172.385	A	0.000	172.385	172.385	100.00	0.000	0.000
					B	0.000	172.385		100.00		
					C	0.000	172.385		100.00		

Tower Pressure - With Ice

$G_H = 1.690$

Section Elevation ft	z ft	K_z	q_z psf	t_z in	A_G ft²	Face	A_F ft²	A_R ft²	A_{leg} ft²	Leg %	C_AA_A In Face ft²	C_AA_A Out Face ft²
L1 120.00-96.12	107.48	1.401	17	0.5000	46.218	A	0.000	46.218	46.218	100.00	0.000	0.000
						B	0.000	46.218		100.00		
						C	0.000	46.218		100.00		
L2 96.12-48.37	71.19	1.246	15	0.5000	129.135	A	0.000	129.135	129.135	100.00	0.000	0.000
						B	0.000	129.135		100.00		
						C	0.000	129.135		100.00		
L3 48.37-1.00	23.70	1	12	0.5000	176.332	A	0.000	176.332	176.332	100.00	0.000	0.000
						B	0.000	176.332		100.00		
						C	0.000	176.332		100.00		

Tower Pressure - Service

$G_H = 1.690$

Section Elevation ft	z ft	K_z	q_z psf	A_G ft²	Face	A_F ft²	A_R ft²	A_{leg} ft²	Leg %	C_AA_A In Face ft²	C_AA_A Out Face ft²
L1 120.00-96.12	107.48	1.401	13	44.228	A	0.000	44.228	44.228	100.00	0.000	0.000
					B	0.000	44.228		100.00		
					C	0.000	44.228		100.00		
L2 96.12-48.37	71.19	1.246	11	125.155	A	0.000	125.155	125.155	100.00	0.000	0.000
					B	0.000	125.155		100.00		
					C	0.000	125.155		100.00		
L3 48.37-1.00	23.70	1	9	172.385	A	0.000	172.385	172.385	100.00	0.000	0.000
					B	0.000	172.385		100.00		
					C	0.000	172.385		100.00		

Tower Forces - No Ice - Wind Normal To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1 120.00-96.12	444.79	1065.71	A	1	0.65	1	1	1	44.228	1115.41	46.71	C
			B	1	0.65	1	1	1	44.228			
			C	1	0.65	1	1	1	44.228			
L2 96.12-48.37	2971.99	4269.15	A	1	0.65	1	1	1	125.155	2791.61	58.46	C
			B	1	0.65	1	1	1	125.155			
			C	1	0.65	1	1	1	125.155			
L3 48.37-1.00	3322.43	7575.54	A	1	0.65	1	1	1	172.385	3141.33	66.31	C
			B	1	0.65	1	1	1	172.385			
			C	1	0.65	1	1	1	172.385			
Sum Weight:	6739.20	12910.40						OTM	386022.85 lb-ft	7048.35		

Tower Forces - No Ice - Wind 60 To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1 120.00-96.12	444.79	1065.71	A	1	0.65	1	1	1	44.228	1115.41	46.71	C
			B	1	0.65	1	1	1	44.228			
			C	1	0.65	1	1	1	44.228			
L2 96.12-48.37	2971.99	4269.15	A	1	0.65	1	1	1	125.155	2791.61	58.46	C
			B	1	0.65	1	1	1	125.155			
			C	1	0.65	1	1	1	125.155			
L3 48.37-1.00	3322.43	7575.54	A	1	0.65	1	1	1	172.385	3141.33	66.31	C
			B	1	0.65	1	1	1	172.385			
			C	1	0.65	1	1	1	172.385			
Sum Weight:	6739.20	12910.40						OTM	386022.85 lb-ft	7048.35		

Tower Forces - No Ice - Wind 90 To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1	444.79	1065.71	A	1	0.65	1	1	1	44.228	1115.41	46.71	C
120.00-96.12			B	1	0.65	1	1	1	44.228			
			C	1	0.65	1	1	1	44.228			
L2	2971.99	4269.15	A	1	0.65	1	1	1	125.155	2791.61	58.46	C
96.12-48.37			B	1	0.65	1	1	1	125.155			
			C	1	0.65	1	1	1	125.155			
L3 48.37-1.00	3322.43	7575.54	A	1	0.65	1	1	1	172.385	3141.33	66.31	C
			B	1	0.65	1	1	1	172.385			
			C	1	0.65	1	1	1	172.385			
Sum Weight:	6739.20	12910.40						OTM	386022.85 lb-ft	7048.35		

Tower Forces - With Ice - Wind Normal To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1	444.79	1400.63	A	1	0.65	1	1	1	46.218	874.20	36.61	C
120.00-96.12			B	1	0.65	1	1	1	46.218			
			C	1	0.65	1	1	1	46.218			
L2	2971.99	5210.78	A	1	0.65	1	1	1	129.135	2160.27	45.24	C
96.12-48.37			B	1	0.65	1	1	1	129.135			
			C	1	0.65	1	1	1	129.135			
L3 48.37-1.00	3322.43	8866.84	A	1	0.65	1	1	1	176.332	2409.94	50.87	C
			B	1	0.65	1	1	1	176.332			
			C	1	0.65	1	1	1	176.332			
Sum Weight:	6739.20	15478.25						OTM	299421.98 lb-ft	5444.41		

Tower Forces - With Ice - Wind 60 To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1	444.79	1400.63	A	1	0.65	1	1	1	46.218	874.20	36.61	C
120.00-96.12			B	1	0.65	1	1	1	46.218			
			C	1	0.65	1	1	1	46.218			
L2	2971.99	5210.78	A	1	0.65	1	1	1	129.135	2160.27	45.24	C
96.12-48.37			B	1	0.65	1	1	1	129.135			
			C	1	0.65	1	1	1	129.135			
L3 48.37-1.00	3322.43	8866.84	A	1	0.65	1	1	1	176.332	2409.94	50.87	C
			B	1	0.65	1	1	1	176.332			
			C	1	0.65	1	1	1	176.332			
Sum Weight:	6739.20	15478.25						OTM	299421.98 lb-ft	5444.41		

Tower Forces - With Ice - Wind 90 To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1	444.79	1400.63	A	1	0.65	1	1	1	46.218	874.20	36.61	C
120.00-96.12			B	1	0.65	1	1	1	46.218			
			C	1	0.65	1	1	1	46.218			
L2	2971.99	5210.78	A	1	0.65	1	1	1	129.135	2160.27	45.24	C
96.12-48.37			B	1	0.65	1	1	1	129.135			
			C	1	0.65	1	1	1	129.135			
L3 48.37-1.00	3322.43	8866.84	A	1	0.65	1	1	1	176.332	2409.94	50.87	C
			B	1	0.65	1	1	1	176.332			
			C	1	0.65	1	1	1	176.332			
Sum Weight:	6739.20	15478.25						OTM	299421.98 lb-ft	5444.41		

Tower Forces - Service - Wind Normal To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1	444.79	1065.71	A	1	0.65	1	1	1	44.228	627.42	26.27	C
120.00-96.12			B	1	0.65	1	1	1	44.228			
			C	1	0.65	1	1	1	44.228			
L2	2971.99	4269.15	A	1	0.65	1	1	1	125.155	1570.28	32.89	C
96.12-48.37			B	1	0.65	1	1	1	125.155			
			C	1	0.65	1	1	1	125.155			
L3 48.37-1.00	3322.43	7575.54	A	1	0.65	1	1	1	172.385	1767.00	37.30	C
			B	1	0.65	1	1	1	172.385			
			C	1	0.65	1	1	1	172.385			
Sum Weight:	6739.20	12910.40						OTM	217137.85 lb-ft	3964.69		

Tower Forces - Service - Wind 60 To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1	444.79	1065.71	A	1	0.65	1	1	1	44.228	627.42	26.27	C
120.00-96.12			B	1	0.65	1	1	1	44.228			
			C	1	0.65	1	1	1	44.228			
L2	2971.99	4269.15	A	1	0.65	1	1	1	125.155	1570.28	32.89	C
96.12-48.37			B	1	0.65	1	1	1	125.155			
			C	1	0.65	1	1	1	125.155			
L3 48.37-1.00	3322.43	7575.54	A	1	0.65	1	1	1	172.385	1767.00	37.30	C
			B	1	0.65	1	1	1	172.385			
			C	1	0.65	1	1	1	172.385			
Sum Weight:	6739.20	12910.40						OTM	217137.85 lb-ft	3964.69		

Tower Forces - Service - Wind 90 To Face

Section Elevation ft	Add Weight lb	Self Weight lb	Face	e	C_F	R_R	D_F	D_R	A_E ft²	F lb	w plf	Ctrl. Face
L1	444.79	1065.71	A	1	0.65	1	1	1	44.228	627.42	26.27	C
120.00-96.12			B	1	0.65	1	1	1	44.228			
			C	1	0.65	1	1	1	44.228			
L2	2971.99	4269.15	A	1	0.65	1	1	1	125.155	1570.28	32.89	C
96.12-48.37			B	1	0.65	1	1	1	125.155			
			C	1	0.65	1	1	1	125.155			
L3 48.37-1.00	3322.43	7575.54	A	1	0.65	1	1	1	172.385	1767.00	37.30	C
			B	1	0.65	1	1	1	172.385			
			C	1	0.65	1	1	1	172.385			
Sum Weight:	6739.20	12910.40						OTM	217137.85 lb-ft	3964.69		

Force Totals

Load Case	Vertical Forces lb	Sum of Forces X lb	Sum of Forces Z lb	Sum of Overturning Moments, M_x lb-ft	Sum of Overturning Moments, M_z lb-ft	Sum of Torques lb-ft
Leg Weight	12910.40					
Bracing Weight	0.00					
Total Member Self-Weight	12910.40			0.00	0.00	
Total Weight	30584.40			0.00	0.00	
Wind 0 deg - No Ice		0.00	-22983.32	-1887277.37	0.00	0.00
Wind 30 deg - No Ice		11491.66	-19904.14	-1634430.15	-943638.69	0.00
Wind 60 deg - No Ice		19904.14	-11491.66	-943638.69	-1634430.15	0.00
Wind 90 deg - No Ice		22983.32	0.00	0.00	-1887277.37	0.00
Wind 120 deg - No Ice		19904.14	11491.66	943638.69	-1634430.15	0.00
Wind 150 deg - No Ice		11491.66	19904.14	1634430.15	-943638.69	0.00
Wind 180 deg - No Ice		0.00	22983.32	1887277.37	0.00	0.00
Wind 210 deg - No Ice		-11491.66	19904.14	1634430.15	943638.69	0.00
Wind 240 deg - No Ice		-19904.14	11491.66	943638.69	1634430.15	0.00
Wind 270 deg - No Ice		-22983.32	0.00	0.00	1887277.37	0.00
Wind 300 deg - No Ice		-19904.14	-11491.66	-943638.69	1634430.15	0.00
Wind 330 deg - No Ice		-11491.66	-19904.14	-1634430.15	943638.69	0.00
Member Ice	2567.86					
Total Weight Ice	38355.93			0.00	0.00	
Wind 0 deg - Ice		0.00	-18976.40	-1574663.74	0.00	0.00
Wind 30 deg - Ice		9488.20	-16434.05	-1363698.80	-787331.87	0.00
Wind 60 deg - Ice		16434.05	-9488.20	-787331.87	-1363698.80	0.00
Wind 90 deg - Ice		18976.40	0.00	0.00	-1574663.74	0.00
Wind 120 deg - Ice		16434.05	9488.20	787331.87	-1363698.80	0.00
Wind 150 deg - Ice		9488.20	16434.05	1363698.80	-787331.87	0.00
Wind 180 deg - Ice		0.00	18976.40	1574663.74	0.00	0.00
Wind 210 deg - Ice		-9488.20	16434.05	1363698.80	787331.87	0.00
Wind 240 deg - Ice		-16434.05	9488.20	787331.87	1363698.80	0.00
Wind 270 deg - Ice		-18976.40	0.00	0.00	1574663.74	0.00
Wind 300 deg - Ice		-16434.05	-9488.20	-787331.87	1363698.80	0.00
Wind 330 deg - Ice		-9488.20	-16434.05	-1363698.80	787331.87	0.00
Total Weight	30584.40			0.00	0.00	
Wind 0 deg - Service		0.00	-12928.12	-1061593.52	0.00	0.00
Wind 30 deg - Service		6464.06	-11196.08	-919366.96	-530796.76	0.00
Wind 60 deg - Service		11196.08	-6464.06	-530796.76	-919366.96	0.00

Wind 90 deg - Service	12928.12	0.00	0.00	-1061593.52	0.00
Wind 120 deg - Service	11196.08	6464.06	530796.76	-919366.96	0.00
Wind 150 deg - Service	6464.06	11196.08	919366.96	-530796.76	0.00
Wind 180 deg - Service	0.00	12928.12	1061593.52	0.00	0.00
Wind 210 deg - Service	-6464.06	11196.08	919366.96	530796.76	0.00
Wind 240 deg - Service	-11196.08	6464.06	530796.76	919366.96	0.00
Wind 270 deg - Service	-12928.12	0.00	0.00	1061593.52	0.00
Wind 300 deg - Service	-11196.08	-6464.06	-530796.76	919366.96	0.00
Wind 330 deg - Service	-6464.06	-11196.08	-919366.96	530796.76	0.00

Load Combinations

Comb. No.	*Description*
1	Dead Only
2	Dead+Wind 0 deg - No Ice
3	Dead+Wind 30 deg - No Ice
4	Dead+Wind 60 deg - No Ice
5	Dead+Wind 90 deg - No Ice
6	Dead+Wind 120 deg - No Ice
7	Dead+Wind 150 deg - No Ice
8	Dead+Wind 180 deg - No Ice
9	Dead+Wind 210 deg - No Ice
10	Dead+Wind 240 deg - No Ice
11	Dead+Wind 270 deg - No Ice
12	Dead+Wind 300 deg - No Ice
13	Dead+Wind 330 deg - No Ice
14	Dead+Ice+Temp
15	Dead+Wind 0 deg+Ice+Temp
16	Dead+Wind 30 deg+Ice+Temp
17	Dead+Wind 60 deg+Ice+Temp
18	Dead+Wind 90 deg+Ice+Temp
19	Dead+Wind 120 deg+Ice+Temp
20	Dead+Wind 150 deg+Ice+Temp
21	Dead+Wind 180 deg+Ice+Temp
22	Dead+Wind 210 deg+Ice+Temp
23	Dead+Wind 240 deg+Ice+Temp
24	Dead+Wind 270 deg+Ice+Temp
25	Dead+Wind 300 deg+Ice+Temp
26	Dead+Wind 330 deg+Ice+Temp
27	Dead+Wind 0 deg - Service
28	Dead+Wind 30 deg - Service
29	Dead+Wind 60 deg - Service
30	Dead+Wind 90 deg - Service
31	Dead+Wind 120 deg - Service
32	Dead+Wind 150 deg - Service
33	Dead+Wind 180 deg - Service
34	Dead+Wind 210 deg - Service
35	Dead+Wind 240 deg - Service
36	Dead+Wind 270 deg - Service
37	Dead+Wind 300 deg - Service
38	Dead+Wind 330 deg - Service

Maximum Member Forces

Section No.	*Elevation ft*	*Component Type*	*Condition*	*Gov. Load*	*Force*	*Major Axis Moment*	*Minor Axis Moment*

				Comb.	lb	lb-ft	lb-ft
L1	120 - 96.12	Pole	Max Tension	1	0.00	0.00	0.00
			Max. Compression	14	-7531.97	0.00	0.00
			Max. Mx	5	-5109.23	-91357.30	-0.00
			Max. My	8	-5109.23	0.00	-91357.30
			Max. Vy	5	6843.38	-91357.30	-0.00
			Max. Vx	8	6843.38	0.00	-91357.30
			Max. Torque	16			-0.00
L2	96.12 - 48.37	Pole	Max Tension	1	0.00	0.00	0.00
			Max. Compression	14	-25195.04	0.00	0.00
			Max. Mx	5	-18137.21	-805580.04	-0.00
			Max. My	2	-18137.21	0.00	805580.04
			Max. Vy	5	20151.36	-805580.04	-0.00
			Max. Vx	2	-20151.36	0.00	805580.04
			Max. Torque	16			-0.00
L3	48.37 - 1	Pole	Max Tension	1	0.00	0.00	0.00
			Max. Compression	14	-38355.93	0.00	0.00
			Max. Mx	5	-30566.24	-1940846.98	0.00
			Max. My	2	-30566.24	0.00	1940846.98
			Max. Vy	5	23007.46	-1940846.98	0.00
			Max. Vx	8	23007.46	0.00	-1940846.98
			Max. Torque	16			-0.01

Maximum Reactions

Location	Condition	Gov. Load Comb.	Vertical lb	Horizontal, X lb	Horizontal, Z lb
Pole	Max. Vert	15	38355.93	0.00	18976.41
	Max. H_x	11	30584.40	22983.32	0.00
	Max. H_z	2	30584.40	0.00	22983.32
	Max. M_x	2	1940846.98	0.00	22983.32
	Max. M_z	5	1940846.98	-22983.32	0.00
	Max. Torsion	26	0.01	9488.20	16434.05
	Min. Vert	1	30584.40	0.00	0.00
	Min. H_x	5	30584.40	-22983.32	0.00
	Min. H_z	8	30584.40	0.00	-22983.32
	Min. M_x	8	-1940846.98	0.00	-22983.32
	Min. M_z	11	-1940846.98	22983.32	0.00
	Min. Torsion	16	-0.01	-9488.20	16434.05

Tower Mast Reaction Summary

Load Combination	Vertical lb	$Shear_x$ lb	$Shear_z$ lb	Overturning Moment, M_x lb-ft	Overturning Moment, M_z lb-ft	Torque lb-ft
Dead Only	30584.40	0.00	0.00	0.00	0.00	0.00
Dead+Wind 0 deg - No Ice	30584.40	0.00	-22983.32	-1940846.98	0.00	0.00
Dead+Wind 30 deg - No Ice	30584.40	11491.66	-19904.14	-1680824.79	-970424.65	0.00
Dead+Wind 60 deg - No Ice	30584.40	19904.14	-11491.66	-970424.65	-1680824.79	-0.00
Dead+Wind 90 deg - No Ice	30584.40	22983.32	0.00	0.00	-1940846.98	0.00

Dead+Wind 120 deg - No Ice	30584.40	19904.14	11491.66	970424.65	-1680824.79	0.00
Dead+Wind 150 deg - No Ice	30584.40	11491.66	19904.14	1680824.79	-970424.65	-0.00
Dead+Wind 180 deg - No Ice	30584.40	0.00	22983.32	1940846.98	0.00	0.00
Dead+Wind 210 deg - No Ice	30584.40	-11491.66	19904.14	1680824.79	970424.65	0.00
Dead+Wind 240 deg - No Ice	30584.40	-19904.14	11491.66	970424.65	1680824.79	-0.00
Dead+Wind 270 deg - No Ice	30584.40	-22983.32	0.00	0.00	1940846.98	0.00
Dead+Wind 300 deg - No Ice	30584.40	-19904.14	-11491.66	-970424.65	1680824.79	0.00
Dead+Wind 330 deg - No Ice	30584.40	-11491.66	-19904.14	-1680824.79	970424.65	-0.00
Dead+Ice+Temp	38355.93	0.00	0.00	0.00	0.00	0.00
Dead+Wind 0 deg+Ice+Temp	38355.93	0.00	-18976.41	-1636366.44	0.00	0.00
Dead+Wind 30 deg+Ice+Temp	38355.93	9488.20	-16434.05	-1417134.92	-818183.24	0.01
Dead+Wind 60 deg+Ice+Temp	38355.93	16434.05	-9488.20	-818183.24	-1417134.92	-0.01
Dead+Wind 90 deg+Ice+Temp	38355.93	18976.41	0.00	0.00	-1636366.44	0.00
Dead+Wind 120 deg+Ice+Temp	38355.93	16434.05	9488.20	818183.24	-1417134.92	0.01
Dead+Wind 150 deg+Ice+Temp	38355.93	9488.20	16434.05	1417134.92	-818183.24	-0.01
Dead+Wind 180 deg+Ice+Temp	38355.93	0.00	18976.41	1636366.44	0.00	0.00
Dead+Wind 210 deg+Ice+Temp	38355.93	-9488.20	16434.05	1417134.92	818183.24	0.01
Dead+Wind 240 deg+Ice+Temp	38355.93	-16434.05	9488.20	818183.24	1417134.92	-0.01
Dead+Wind 270 deg+Ice+Temp	38355.93	-18976.41	0.00	0.00	1636366.44	0.00
Dead+Wind 300 deg+Ice+Temp	38355.93	-16434.05	-9488.20	-818183.24	1417134.92	0.01
Dead+Wind 330 deg+Ice+Temp	38355.93	-9488.20	-16434.05	-1417134.92	818183.24	-0.01
Dead+Wind 0 deg - Service	30584.40	0.00	-12928.12	-1092239.26	0.00	0.00
Dead+Wind 30 deg - Service	30584.40	6464.06	-11196.08	-945908.22	-546120.36	0.00
Dead+Wind 60 deg - Service	30584.40	11196.08	-6464.06	-546120.36	-945908.22	-0.00
Dead+Wind 90 deg - Service	30584.40	12928.12	0.00	0.00	-1092239.26	0.00
Dead+Wind 120 deg - Service	30584.40	11196.08	6464.06	546120.36	-945908.22	0.00
Dead+Wind 150 deg - Service	30584.40	6464.06	11196.08	945908.22	-546120.36	-0.00
Dead+Wind 180 deg - Service	30584.40	0.00	12928.12	1092239.26	0.00	0.00
Dead+Wind 210 deg - Service	30584.40	-6464.06	11196.08	945908.22	546120.36	0.00
Dead+Wind 240 deg - Service	30584.40	-11196.08	6464.06	546120.36	945908.22	-0.00
Dead+Wind 270 deg - Service	30584.40	-12928.12	0.00	0.00	1092239.26	0.00
Dead+Wind 300 deg - Service	30584.40	-11196.08	-6464.06	-546120.36	945908.22	0.00
Dead+Wind 330 deg - Service	30584.40	-6464.06	-11196.08	-945908.22	546120.36	-0.00

Solution Summary

Load Comb.	Sum of Applied Forces			Sum of Reactions			% Error
	PX	PY	PZ	PX	PY	PZ	
	lb	lb	lb	lb	lb	lb	
1	0.00	-30584.40	0.00	0.00	30584.40	0.00	0.000%
2	0.00	-30584.40	-22983.32	0.00	30584.40	22983.32	0.000%
3	11491.66	-30584.40	-19904.14	-11491.66	30584.40	19904.14	0.000%
4	19904.14	-30584.40	-11491.66	-19904.14	30584.40	11491.66	0.000%
5	22983.32	-30584.40	0.00	-22983.32	30584.40	0.00	0.000%
6	19904.14	-30584.40	11491.66	-19904.14	30584.40	-11491.66	0.000%
7	11491.66	-30584.40	19904.14	-11491.66	30584.40	-19904.14	0.000%
8	0.00	-30584.40	22983.32	0.00	30584.40	-22983.32	0.000%
9	-11491.66	-30584.40	19904.14	11491.66	30584.40	-19904.14	0.000%
10	-19904.14	-30584.40	11491.66	19904.14	30584.40	-11491.66	0.000%
11	-22983.32	-30584.40	0.00	22983.32	30584.40	0.00	0.000%
12	-19904.14	-30584.40	-11491.66	19904.14	30584.40	11491.66	0.000%
13	-11491.66	-30584.40	-19904.14	11491.66	30584.40	19904.14	0.000%
14	0.00	-38355.93	0.00	0.00	38355.93	0.00	0.000%
15	0.00	-38355.93	-18976.40	0.00	38355.93	18976.41	0.000%
16	9488.20	-38355.93	-16434.05	-9488.20	38355.93	16434.05	0.000%
17	16434.05	-38355.93	-9488.20	-16434.05	38355.93	9488.20	0.000%
18	18976.40	-38355.93	0.00	-18976.41	38355.93	0.00	0.000%
19	16434.05	-38355.93	9488.20	-16434.05	38355.93	-9488.20	0.000%
20	9488.20	-38355.93	16434.05	-9488.20	38355.93	-16434.05	0.000%

Load Comb.	Sum of Applied Forces			Sum of Reactions			% Error
	PX lb	PY lb	PZ lb	PX lb	PY lb	PZ lb	
1	0.00	-30584.40	0.00	0.00	30584.40	0.00	0.000%
21	0.00	-38355.93	18976.40	0.00	38355.93	-18976.41	0.000%
22	-9488.20	-38355.93	16434.05	9488.20	38355.93	-16434.05	0.000%
23	-16434.05	-38355.93	9488.20	16434.05	38355.93	-9488.20	0.000%
24	-18976.40	-38355.93	0.00	18976.41	38355.93	0.00	0.000%
25	-16434.05	-38355.93	-9488.20	16434.05	38355.93	9488.20	0.000%
26	-9488.20	-38355.93	-16434.05	9488.20	38355.93	16434.05	0.000%
27	0.00	-30584.40	-12928.12	0.00	30584.40	12928.12	0.000%
28	6464.06	-30584.40	-11196.08	-6464.06	30584.40	11196.08	0.000%
29	11196.08	-30584.40	-6464.06	-11196.08	30584.40	6464.06	0.000%
30	12928.12	-30584.40	0.00	-12928.12	30584.40	0.00	0.000%
31	11196.08	-30584.40	6464.06	-11196.08	30584.40	-6464.06	0.000%
32	6464.06	-30584.40	11196.08	-6464.06	30584.40	-11196.08	0.000%
33	0.00	-30584.40	12928.12	0.00	30584.40	-12928.12	0.000%
34	-6464.06	-30584.40	11196.08	6464.06	30584.40	-11196.08	0.000%
35	-11196.08	-30584.40	6464.06	11196.08	30584.40	-6464.06	0.000%
36	-12928.12	-30584.40	0.00	12928.12	30584.40	0.00	0.000%
37	-11196.08	-30584.40	-6464.06	11196.08	30584.40	6464.06	0.000%
38	-6464.06	-30584.40	-11196.08	6464.06	30584.40	11196.08	0.000%

Non-Linear Convergence Results

Load Combination	Converged?	Number of Cycles	Displacement Tolerance	Force Tolerance
1	Yes	4	0.00000001	0.00000001
2	Yes	4	0.00000001	0.00005305
3	Yes	5	0.00000001	0.00024546
4	Yes	5	0.00000001	0.00024546
5	Yes	4	0.00000001	0.00005305
6	Yes	5	0.00000001	0.00024546
7	Yes	5	0.00000001	0.00024546
8	Yes	4	0.00000001	0.00005305
9	Yes	5	0.00000001	0.00024546
10	Yes	5	0.00000001	0.00024546
11	Yes	4	0.00000001	0.00005305
12	Yes	5	0.00000001	0.00024546
13	Yes	5	0.00000001	0.00024546
14	Yes	4	0.00000001	0.00000001
15	Yes	5	0.00000001	0.00010631
16	Yes	5	0.00000001	0.00045080
17	Yes	5	0.00000001	0.00045080
18	Yes	5	0.00000001	0.00010631
19	Yes	5	0.00000001	0.00045080
20	Yes	5	0.00000001	0.00045080
21	Yes	5	0.00000001	0.00010631
22	Yes	5	0.00000001	0.00045080
23	Yes	5	0.00000001	0.00045080
24	Yes	5	0.00000001	0.00010631
25	Yes	5	0.00000001	0.00045080
26	Yes	5	0.00000001	0.00045080
27	Yes	4	0.00000001	0.00003241
28	Yes	5	0.00000001	0.00006166
29	Yes	5	0.00000001	0.00006166
30	Yes	4	0.00000001	0.00003241
31	Yes	5	0.00000001	0.00006166
32	Yes	5	0.00000001	0.00006166

Load Combination	Converged?	Number of Cycles	Displacement Tolerance	Force Tolerance
33	Yes	4	0.00000001	0.00003241
34	Yes	5	0.00000001	0.00006166
35	Yes	5	0.00000001	0.00006166
36	Yes	4	0.00000001	0.00003241
37	Yes	5	0.00000001	0.00006166
38	Yes	5	0.00000001	0.00006166

Maximum Tower Deflections - Service Wind

Section No.	Elevation	Horz. Deflection	Gov. Load Comb.	Tilt	Twist
	ft	in		°	°
L1	120 - 96.12	31.282	36	2.1955	0.0000
L2	99.87 - 48.37	22.237	36	2.0525	0.0000
L3	53.62 - 1	6.226	36	1.1056	0.0000

Critical Deflections and Radius of Curvature - Service Wind

Elevation	Appurtenance	Gov. Load Comb.	Deflection	Tilt	Twist	Radius of Curvature
ft			in	°	°	ft
120.00	13' Low Profile Platform	36	31.282	2.1955	0.0000	19753
107.00	15' Square Platform w/rails	36	25.379	2.1259	0.0000	7597
97.00	13' Low Profile Platform	36	21.006	2.0114	0.0000	4469
90.00	13' Low Profile Platform	36	18.098	1.8857	0.0000	3664
80.00	13' Low Profile Platform	36	14.231	1.6646	0.0000	2915
70.00	13' Low Profile Platform	36	10.775	1.4282	0.0000	2421

Maximum Tower Deflections - Design Wind

Section No.	Elevation	Horz. Deflection	Gov. Load Comb.	Tilt	Twist
	ft	in		°	°
L1	120 - 96.12	55.549	5	3.8995	0.0000
L2	99.87 - 48.37	39.493	5	3.6456	0.0000
L3	53.62 - 1	11.062	5	1.9643	0.0000

Critical Deflections and Radius of Curvature - Design Wind

Elevation	Appurtenance	Gov. Load Comb.	Deflection	Tilt	Twist	Radius of Curvature
ft			in	°	°	ft
120.00	13' Low Profile Platform	5	55.549	3.8995	0.0000	11202
107.00	15' Square Platform w/rails	5	45.070	3.7711	0.0000	4308
97.00	13' Low Profile Platform	5	37.306	3.5769	0.0000	2532
90.00	13' Low Profile Platform	5	32.144	3.3684	0.0000	2074

80.00	13' Low Profile Platform	5	25.277	2.9984	0.0000	1648
70.00	13' Low Profile Platform	5	19.141	2.5887	0.0000	1367

Compression Checks

Pole Design Data

Section No.	Elevation ft	Size	L ft	L_u ft	Kl/r	F_a ksi	A in^2	Actual P lb	Allow. P_a lb	Ratio $\frac{P}{P_a}$
L1	120 - 96.12 (1)	TP25.45x19x0.1875	23.88	119.00	165.9	5.427	14.4316	-5109.22	78320.40	0.065
L2	96.12 - 48.37 (2)	TP37.84x24.0621x0.25	51.50	119.00	111.2	12.084	28.7132	-18137.20	346977.00	0.052
L3	48.37 - 1 (3)	TP50x35.9355x0.3125	52.62	119.00	81.0	21.369	49.2838	-30566.20	1053160.00	0.029

Pole Bending Design Data

Section No.	Elevation ft	Size	Actual M_x lb-ft	Actual f_{bx} ksi	Allow. F_{bx} ksi	Ratio $\frac{f_{bx}}{F_{bx}}$	Actual M_y lb-ft	Actual f_{by} ksi	Allow. F_{by} ksi	Ratio $\frac{f_{by}}{F_{by}}$
L1	120 - 96.12 (1)	TP25.45x19x0.1875	91357.50	-12.693	39.000	0.325	0.00	0.000	39.000	0.000
L2	96.12 - 48.37 (2)	TP37.84x24.0621x0.25	805581.67	-37.670	39.000	0.966	0.00	0.000	39.000	0.000
L3	48.37 - 1 (3)	TP50x35.9355x0.3125	1940850.00	-38.483	38.001	1.013	0.00	0.000	38.001	0.000

Pole Interaction Design Data

Section No.	Elevation ft	Size	Ratio $\frac{P}{P_a}$	Ratio $\frac{f_{bx}}{F_{bx}}$	Ratio $\frac{f_{by}}{F_{by}}$	Comb. Stress Ratio	Allow. Stress Ratio	Criteria
L1	120 - 96.12 (1)	TP25.45x19x0.1875	0.065	0.325	0.000	0.391 ✔	1.333	H1-3 ✔
L2	96.12 - 48.37 (2)	TP37.84x24.0621x0.25	0.052	0.966	0.000	1.018 ✔	1.333	H1-3 ✔
L3	48.37 - 1 (3)	TP50x35.9355x0.3125	0.029	1.013	0.000	1.042 ✔	1.333	H1-3 ✔

Section Capacity Table

Section No.	Elevation ft	Component Type	Size	Critical Element	P lb	$SF*P_{allow}$ lb	% Capacity	Pass Fail
L1	120 - 96.12	Pole	TP25.45x19x0.1875	1	-5109.22	104401.09	29.3	Pass
L2	96.12 - 48.37	Pole	TP37.84x24.0621x0.25	2	-18137.20	462520.32	76.4	Pass
L3	48.37 - 1	Pole	TP50x35.9355x0.3125	3	-30566.20	1403862.22	78.1	Pass

	Summary	
Pole (L3)	78.1	Pass
RATING =	**78.1**	**Pass**

NATCOMM

Job	120' EEI Monopole – Granby, CT	**Project No.**	07073	**Page**	1 of 9
Description	Foundation Analysis	**Computed by**	JEK	**Date**	10/15/2007

MONOPOLE FOUNDATION ANALYSIS

TOWER FORCES:

Moment Caused by Tower	$M_t := 1950 \cdot ft \cdot kips$
Shear at Base of Tower	$S_t := 23kip$
Max Compressive Force	$C_t := 31 \cdot kip$
Height of Tower	$H_t := 120 \cdot ft$
Base Plate Bolt Circle	$MP := 59in$

FOOTING DIMENSIONS:

Overall Depth of Footing	$D_f := 7ft$
Length of Pier	$L_p := 5 \cdot ft$
Extension of Pier Above Grade	$L_{pag} := 1.0 \cdot ft$
Diameter of Pier	$d_p := 7 \cdot ft$
Thickness of Footing	$T_f := 3 \cdot ft$
Width of Footing:	$W_f := 22ft$
Length of Anchor Bolts:	$L_{st} := 96in$
Projection of anchor bolts above pier	$A_{BP} := 12 \cdot in$

PROPERTIES:

Compressive Strength of Concrete	$f'c := 3000psi$
Yield Strength of Steel Reinforcement	$fy := 60000 \cdot psi$
Yield Strength of Anchor Bolt	$fya := 75000 \cdot psi$
Internal Friction Angle of Soil	$\phi_s := 0 \cdot deg$
Allowable Bearing Capacity	$q_s := 6000 \cdot psf$
Unit Weight of Soil	$\gamma_s := 120 \cdot pcf$
Unit Weight of Concrete	$\gamma_c := 150 \cdot pcf$
Depth to Neglect	$n := 0ft$
Cohesion of Clay Type Soil Note: Use 0 for Sandy Soil	$c := 0 \cdot ksf$
Seismic Zone Factor: UBC Fig 23-2	$Z := 2$
Coefficient of Friction between Concrete:	$\mu := 0.45$
Clear Cover of Reinforcement Pier:	$Cvr_{pier} := 3 \cdot in$
Clear Cover of Reinforcement Pad:	$Cvr_{pad} := 3 \cdot in$
Anchor Bolt Diameter	$d_{anchor} := 2.25in$
Anchor bolt area	$A_{anchor} := 3.97 \cdot in^2$

PIER REINFORCEMENT:

Bar Size	$BSpier := 8$	Bar Diameter	$d_{bpier} := 1.000 \cdot in$
Number of Bars	$NBpier := 30$	Bar Area	$A_{bpier} := 0.790 \cdot in^2$

PAD REINFORCEMENT:

TOP:	Bar Size	$BS_{top} := 8$	Bar Diameter	$d_{btop} := 1.000 \cdot in$
	Number of Bars	$NB_{top} := 24$	Bar Area	$A_{btop} := 0.790 \cdot in^2$
BOTTOM:	Bar Size	$BS_{bot} := 8$	Bar Diameter	$d_{bbot} := 1.000 \cdot in$
	Number of Bars	$NB_{bot} := 24$	Bar Area	$A_{bot} := 0.790 \cdot in^2$

Coefficient of Lateral Soil Pressure: $K_p := \frac{1 + \sin(\phi_s)}{1 - \sin(\phi_s)}$ $K_p = 1$

Load Factor (EIA 3.1.1): $LF := if\left[H_t \le 700 \cdot ft, 1.3, if\left[H_t \ge 1200, 1.7, 1.3 + \left(\frac{H_t - 700}{1200 - 700}\right) \cdot 0.4\right]\right]$ $LF = 1.3$

CHECK ANCHOR STEEL EMBEDMENT

Depth:

$D_{ab} := L_{st} - A_{BP}$ $D_{ab} = 7 \cdot ft$ $L_{anchor} := \frac{(0.11 \cdot fy) \cdot in}{\sqrt{f'c \cdot psi}}$ $L_{anchor} = 10.0416 \cdot ft$

$DepthCheck := if(D_{ab} \geq L_{anchor}, "Okay", "No Good")$

DepthCheck = "No Good" **Note:** anchor plate is provided

STABILITY OF FOOTING

Passive Pressure:

$P_{pn} := K_p \cdot \gamma_s \cdot n + c \cdot 2 \cdot \sqrt{K_p}$ $P_{pn} = 0 \cdot ksf$

$P_{pt} := K_p \cdot \gamma_s \cdot (D_f - T_f) + c \cdot 2 \cdot \sqrt{K_p}$ $P_{pt} = 0.48 \cdot ksf$

$P_{top} := if[n < (D_f - T_f), P_{pt}, P_{pn}]$ $P_{top} = 0.48 \cdot ksf$

$P_{bot} := K_p \cdot \gamma_s \cdot D_f + c \cdot 2 \cdot \sqrt{K_p}$ $P_{bot} = 0.84 \cdot ksf$

$P_{ave} := \frac{P_{top} + P_{bot}}{2}$ $P_{ave} = 0.66 \cdot ksf$

$T_p := if[n < (D_f - T_f), T_f, (D_f - n)]$ $T_p = 3 \cdot ft$

$A_p := W_f \cdot T_p$ $A_p = 66 \cdot ft^2$

Ultimate Shear:

$S_u := P_{ave} \cdot A_p$ $S_u = 43.56 \cdot kip$

Weight of Concrete Pad:

$WT_c := [(W_f^2 \cdot T_f) + d_p^2 L_p] \cdot \gamma_c$ $WT_c = 254.55 \cdot kip$

Weight of Soil: above Footing:

$WT_{s1} := \left[W_f^2 \cdot (|L_p - L_{pag}|) - \frac{d_p^2 \cdot \pi}{4} \cdot (|L_p - L_{pag}|) \right] \cdot \gamma_s$ $WT_{s1} = 213.8474 \cdot kip$

Weight of Soil Wedge at back face:

$WT_{s2} := \left(\frac{D_f \cdot \tan(\phi_s)}{2} \cdot W_f \right) \cdot \gamma_s$ $WT_{s2} = 0 \cdot kip$

Total Weight:

$WT_{tot} := WT_c + WT_{s1} + C_t$ $WT_{tot} = 499.3974 \cdot kip$

Resisting Moment:

$M_r := (WT_{tot}) \cdot \frac{W_f}{2} + S_u \cdot \frac{T_f}{3} + WT_{s2} \cdot \left(W_f + \frac{D_f \cdot \tan(\phi_s)}{3} \right)$ $M_r = 5536.9318 \cdot kip \cdot ft$

Overturning Moment:

$M_{ot} := M_t + S_t \cdot (L_p + T_f)$ $M_{ot} = 2134 \cdot kip \cdot ft$

Factor of Safety:

$FS := \frac{M_r}{M_{ot}}$ $FS_{req} := 2$ $FS = 2.59$

$SafetyCheck := if(FS > FS_{req}, "Okay", "No Good")$ SafetyCheck = "Okay"

SHEAR CAPACITY IN PIER

$FS := 2$

$$S_p := \frac{P_{ave} \cdot A_p + \mu \cdot WT_{tot}}{FS}$$

$S_p = 134.1444 \cdot kips$

$ShearCheck := if(S_p > S_t, "Okay", "No Good")$

$ShearCheck = "Okay"$

BEARING PRESSURE CAUSED BY FOOTING

$A_{mat} := W_f^2$ $\quad A_{mat} = 484 \cdot ft^2$

$S := \frac{W_f^3}{6}$ $\quad S = 1774.6667 \cdot ft^3$

$P_{max} := \frac{WT_{tot}}{A_{mat}} + \frac{M_{ot}}{S}$ $\quad P_{max} = 2.2343 \cdot ksf$

$P_{min} := \frac{WT_{tot}}{A_{mat}} - \frac{M_{ot}}{S}$ $\quad P_{min} = -0.1707 \cdot ksf$

$MaxPressure := if(P_{max} < q_s, "Okay", "No Good")$ $\quad MaxPressure = "Okay"$

$MinPressure := if[(P_{min} \geq 0) \cdot (P_{min} < q_s), "Okay", "No Good"]$ $\quad MinPressure = "No Good"$

Distance to Resultant of Pressure Distribution:

$$X_p := \frac{P_{max}}{\frac{P_{max} - P_{min}}{W_f}} \cdot \frac{1}{3}$$

$X_p = 6.8129 \cdot ft$

Distance to Kern: $X_k := \frac{W_f}{6}$ $\quad X_k = 3.6667 \cdot ft$

Since Resultant Force is Not in Kern, Area to which Pressure is Applied Must be Reduced.

Eccentricity: $e := \frac{M_{ot}}{WT_{tot}}$ $\quad e = 4.2731$

Adjusted Soil Pressure:

$$P_a := \frac{2 \cdot WT_{tot}}{3 \cdot W_f \cdot \left(\frac{W_f}{2} - e\right)}$$

$P_a = 2.2497 \cdot ksf$

$$q_{adj} := if\left(P_{min} < 0, P_a, \frac{P_{max}}{ft^2}\right)$$

$q_{adj} = 2.2497 \cdot ksf$

$PressureCheck := if(q_{adj} < q_s, "Okay", "No Good")$ $\quad PressureCheck = "Okay"$

CONCRETE BEARING CAPACITY (ACI 10.17)

$\phi_c := 0.75$ (ACI 9.3.2.2)

$P_b := \phi_c \cdot 0.85 \cdot f'c \cdot \frac{d_p^{\,2} \cdot \pi}{4}$ $P_b = 10598.6341 \cdot kip$

$BearingCheck := if(P_b > LF \cdot C_t, "Okay", "No Good")$ $BearingCheck = "Okay"$

SHEAR STRENGTH OF CONCRETE

Beam Shear: (Critical section located at a distance d from the face of Pier) (ACI 11.3.1.1)

$\phi_{VBA} := .85$ (ACI 9.3.2.3)

$d := T_f - Cvr_{pad} - d_{bbot}$ $d = 32 \cdot in$

$d_1 := \frac{W_f}{2} - \frac{d_p}{2}$ $d_1 = 7.5 \cdot ft$

$d_2 := d_1 - d$ $d_2 = 4.8333 \cdot ft$

$L := \left(\frac{W_f}{2} - e\right) \cdot 3$ $L = 20.1806 \cdot ft$

$Slope := if\left(L > W_f, \frac{P_{max} - P_{min}}{W_f}, \frac{q_{adj}}{L}\right)$ $Slope = 0.1115 \cdot kcf$

$V_{req} := LF \cdot \left[\left(q_{adj} - Slope \cdot d_1\right) + \left(\frac{Slope \cdot d_1}{2}\right)\right] \cdot W_f \cdot d_1$ $V_{req} = 392.8864 \cdot kip$

ACI 11.3.1.1 $V_{Avail} := \phi_c \cdot 2 \cdot \sqrt{f'c \cdot psi} \cdot W_f \cdot d$ $V_{Avail} = 786.6172 \cdot kip$

$BeamShearCheck := if(V_{req} < V_{Avail}, "Okay", "No Good")$ $BeamShearCheck = "Okay"$

Punching Shear: (Critical Section Located at a distance of d/2 from the face of pier) (ACI 11.12.2.1)

$b_o := (d_p + d) \cdot \pi$ $b_o = 30.3687 \cdot ft$

Area included inside bo: $A_{bo} := \frac{\pi \cdot (d_p + d)^2}{4}$ $A_{bo} = 73.3911 \cdot ft^2$

Area outside of bo: $A_{out} := A_{mat} - A_{bo}$ $A_{out} = 410.6089 \cdot ft^2$

Guess Value: $v_u := 1\,ksf$ (From "Foundation Analysis and design", By Joseph Bowles, Eq. 8-9)

Given $d^2 + d_p \cdot d = \frac{WT_{tot}}{\pi \cdot v_u}$

$v_u := \text{Find}(v_u)$ $v_u = 6.1667 \cdot ksf$

$V_u := v_u \cdot d \cdot W_f$ $V_u = 361.7782 \cdot kips$

$V_{req} := LF \cdot V_u$ $V_{req} = 470.3116 \cdot kips$

$V_{Avail} := \phi_c \cdot 4 \cdot \sqrt{f'c \cdot psi} \cdot b_o \cdot d$ $V_{Avail} = 2171.6878 \cdot kips$

$PunchingShearCheck := \text{if}(V_{req} < V_{Avail}, \text{"Okay"}, \text{"No Good"})$ $PunchingShearCheck = \text{"Okay"}$

STEEL REINFORCEMENT IN THE PAD $\phi_m := .90$ ACI 9.3.2.2

Take Maximum Bending at face of Pier:

$q_b := q_{adj} - d_1 \cdot Slope$ $q_b = 1.4136 \cdot ksf$

$M_n := \frac{LF}{\phi_m} \cdot \left[(q_{adj} - q_b) \cdot \frac{{d_1}^2}{3} + q_b \cdot \frac{{d_1}^2}{2} \right] \cdot W_f$ $M_n = 1761.5681 \cdot kip \cdot ft$

ACI 10.2.7.3 $\beta := \text{if}\left[f'c \le 4000 \cdot psi, .85, \text{if}\left[f'c \ge 8000 \cdot psi, .65, .85 - \left(\frac{\frac{f'c}{psi} - 4000}{1000} \right) \cdot .05 \right] \right]$ $\beta = 0.85$

$R_u := \frac{M_n}{\phi_m \cdot W_f \cdot d^2}$ $R_u = 12511.1\ lbf$

$\rho := \frac{0.85 \cdot f'c}{fy} \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f'c}} \right)$ $\rho = 0.0015$

$\rho_{min} := 1.333 \cdot \rho$ $\rho_{min} = 0.00196$

Temperature and Shrinkage: $\rho_{sh} := if(fy \geq 60000 \cdot psi, 0.0018, 0.0020)$ $\rho_{sh} = 0.0018$

(ACI 7.12.2.1b)

FOR BOTTOM BARS: $As := \max(\rho, \rho_{min}, \rho_{sh}) \cdot W_f \cdot d$ $As = 16.5944 \cdot in^2$

$As_{prov} := A_{bot} \cdot NB_{bot}$ $As_{prov} = 18.96 \cdot in^2$

$PadReinforcement := if(As_{prov} > As, "Okay", "No Good")$ $PadReinforcement = "Okay"$

FOR TOP BARS: $As := \rho_{sh} \cdot (W_f \cdot d)$ $As = 15.2064 \cdot in^2$

$As_{prov} := A_{btop} \cdot NB_{top}$ $As_{prov} = 18.96 \cdot in^2$

$PadReinforcement := if(As_{prov} > As, "Okay", "No Good")$ $PadReinforcement = "Okay"$

TENSION (ACI 12.2.3) **DEVELOPMENT LENGTH OF PAD REINFORCEMENT**

Bar Spacing: $B_{sPad} := \frac{W_f - 2 \cdot Cvr_{pad} - NB_{bot} \cdot d_{bbot}}{NB_{bot} - 1}$ $B_{sPad} = 10.1739 \cdot in$

Development Length Factors:

Reinforcement Location Factor	$\alpha := 1.0$	
Coating Factor	$\beta := 1.0$	
Concrete strength Factor	$\lambda := 1.0$	
Reinforcement Size Factor	$\gamma := 1.0$	

Spacing or Cover Dimension: $c := if\left(Cvr_{pad} < \frac{B_{sPad}}{2}, Cvr_{pad}, \frac{B_{sPad}}{2}\right)$ $c = 3 \cdot in$

Transverse Reinforcement Index: As allowed by ACI 12.2.4 $k_{tr} := 0$

$$L_{dbt} := \frac{3}{40} \cdot \frac{fy}{\sqrt{f'c \cdot psi}} \cdot \frac{\alpha \cdot \beta \cdot \gamma \cdot \lambda}{\frac{c + k_{tr}}{d_{bbot}}} \cdot d_{bbot}$$ $L_{dbt} = 27.3861 \cdot in$

$L_{dbmin} := 12 \cdot in$

Minimum Development Length: (ACI 12.2.1) $L_{dbtCheck} := if(L_{dbt} \geq L_{dbmin}, "Use L.dbt", "Use L.dbmin")$ $L_{dbtCheck} = "Use L.dbt"$

Available Length in Pad: $L_{Pad} := \frac{W_f}{2} - \frac{d_p}{2} - Cvr_{pad}$ $L_{Pad} = 87 \cdot in$

$LpadTension := if(L_{Pad} > L_{dbt}, "Okay", "No Good")$ $LpadTension = "Okay"$

REINFORCEMENT IN PIER

Pier Area: $A_{pv} := \frac{\pi \cdot d_p^2}{4}$ $A_p = 5541.7694 \cdot in^2$

(ACI 10.8.4 and 10.9.1) $A_{smin} := 0.01 \cdot 0.05 \cdot A_p$ $A_{smin} = 2.7709 \cdot in^2$

$A_{sprov} := NBpier \cdot A_{bpier}$ $A_{sprov} = 23.7 \cdot in^2$

$SteelAreaCheck := if(A_{sprov} > A_{smin}, "Okay", "No Good")$ $SteelAreaCheck = "Okay"$

NOTE: Anchor Bolts are not accounted for in reinforcement calculation and will provide additional reinforcement to satisfy minimum requirement of steel.

Bar Spacing In Pier: $B_{sPier} := \frac{d_p \cdot \pi}{NBpier} - d_{bpier}$ $B_{sPier} = 7.7965 \cdot in$

Diamter of Reinforcement Cage: $Diam_{cage} := d_p - 2 \cdot Cvr_{pier}$ $Diam_{cage} = 78 \cdot in$

Maximum Moment in Pier: $M_p := \left[M_t + S_t \cdot \left(L_p + \frac{A_{BP}}{2} \right) \right] \cdot LF$ $M_p = 32393.4 \cdot in \cdot kips$

Pier Check evaluated from outside program and results are listed below;

(defined variables) $(f'_c \ f_y \ cl \ Spiral) = (3 \ 60 \ 3 \ 0)$

The required input is column diameter in inches, number of reinforcing bars, bar size number, factored axial load in kips and moment in kip inches: $(D \ N \ n \ P_u \ M_{xu}) := (84 \ 30 \ 8 \ 31 \ 32394)$

Clears any previous output: $(\phi P_n \ \phi M_{xn} \ f_{sp} \ \rho) := (0 \ 0 \ 0 \ 0)$

$(\phi P_n \ \phi M_{xn} \ f_{sp} \ \rho) := \phi P'_n(D, N, n, P_u, M_{xu})^T$

The Output is given as useable axial load in kips, moment capacity in kip inches, splicing stress in ksi, and reinforcement ratio: $(\phi P_n \ \phi M_{xn} \ f_{sp} \ \rho) = (45.2371 \ 47271.2755 \ -60 \ 0.0043)$

Column size and reinforcement may be changed to match capacity to the applied load.

$AxialLoadCheck := if(\phi P_n \geq P_u, "Okay", "No Good")$ $AxialLoadCheck = "Okay"$

$BendingCheck := if(\phi M_{xn} \geq M_{xu}, "Okay", "No Good")$ $BendingCheck = "Okay"$

DEVELOPMENT LENGTH OF PIER REINFORCEMENT

TENSION (ACI 12.2.3)

Factors for development:

Reinforcement Location Factor $\alpha := 1.0$

Coating Factor $\beta := 1.0$

Concrete strength Factor $\lambda := 1.0$

Reinforcement Size Factor $\gamma := 1.0$

Spacing or Cover Dimension: $c := \text{if}\left(Cvr_{pier} < \frac{B_{sPier}}{2}, Cvr_{pier}, \frac{B_{sPier}}{2}\right)$ $c = 3 \cdot in$

Transverse Reinforcement: As allowed by ACI 12.2.4 $k_{tr} := 0$

$$L_{dbt} := \frac{3}{40} \cdot \frac{fy}{\sqrt{f'c \cdot psi}} \cdot \frac{\alpha \cdot \beta \cdot \gamma \cdot \lambda}{\frac{c + k_{tr}}{d_{bpier}}} \cdot d_{bpier} \qquad L_{dbt} = 27.3861 \cdot in$$

Minimum Development Length: (ACI 12.2.1) $L_{dbmin} := 12 \cdot in$

Pier reinforcement bars are standard 90 degree hooks and therefore developement in the pad is computed as follows:

$$L_{dh} := \frac{1200 \cdot d_{bpier}}{\sqrt{\frac{f'c}{psi}}} \cdot .7 \qquad L_{dh} = 15.3362 \cdot in$$

$$L_{db} := \max\left(L_{dbt}, L_{dbmin}\right) \qquad L_{db} = 27.3861 \cdot in$$

COMPRESSION: (ACI 12.3.2)

$$L_{dbc1} := \frac{.02 \cdot d_{bpier} \cdot fy}{\sqrt{f'c \cdot psi}} \qquad L_{dbc1} = 21.9089 \cdot in$$

$$L_{dbmin} := 0.0003 \cdot \frac{in^2}{lb} \cdot \left(d_{bpier} \cdot fy\right) \qquad L_{dbmin} = 18 \cdot in$$

$$L_{dbc} := \text{if}\left(L_{dbc1} \geq L_{dbmin}, L_{dbc1}, L_{dbmin}\right) \qquad L_{dbc} = 21.9089 \cdot in$$

Available Length in Foundation:

$$L_{pier} := L_p - Cvr_{pier} \qquad L_{pier} = 57 \cdot in$$

$$L_{pad} := T_f - Cvr_{pad} \qquad L_{pad} = 33 \cdot in$$

$$L_{tension} := \text{if}\left(L_{pier} + L_{pad} > L_{dbt}, \text{"Okay"}, \text{"No Good"}\right) = \text{"Okay"} \qquad L_{tension} = \text{"Okay"}$$

$$L_{compression} := \text{if}\left(L_{pier} + L_{pad} > L_{dbc}, \text{"Okay"}, \text{"No Good"}\right) \qquad L_{compression} = \text{"Okay"}$$

NOTE: Anchor bolts and plate provided, OK

TIE SIZE AND SPACING IN COLUMN

Minimum Tie Size:

$Tie_{min} := if(BSpier \le 10, 3, 4)$ $\quad Tie_{min} = 3$

Used #4 Ties $\quad d_{Tie} := 4$

Seismic factor:
(ACI 21.10.5)

$z := if(Z \le 2, 1, 0.5)$ $\quad z = 1$

$s_{lim1} := 16 \cdot d_{bpier} \cdot z$ $\quad s_{lim1} = 16 \cdot in$

$s_{lim2} := \frac{48 \cdot d_{Tie} \cdot in}{8} \cdot z$ $\quad s_{lim2} = 24 \cdot in$

$s_{lim3} := D_f \cdot z$ $\quad s_{lim3} = 84 \cdot in$

$s_{lim4} := 18in$ $\quad s_{lim4} = 18 \cdot in$

Maximum Spacing:

$$s_{tie} := \min\left(\begin{pmatrix} s_{lim1} \\ s_{lim2} \\ s_{lim3} \\ s_{lim4} \end{pmatrix}\right) \quad s_{tie} = 16 \cdot in$$

Number of Ties Required:

$$n_{tie} := \frac{L_{pier} - 3 \cdot in}{s_{tie}} + 1 \quad n_{tie} = 4.375$$

ANCHOR BOLT AND BASE PLATE ANALYSIS

Input Data

Tower Reactions:

Overturning Moment:	$OM := 1950 \cdot ft \cdot kips$	*user input*
Shear Force:	$Shear := 23 \cdot kips$	*user input*
Axial Force:	$Axial := 31 \cdot kips$	*user input*

Anchor Bolt Data:

Use ASTM A615 Grade 75	*user input*	
Number of Anchor Bolts = N	$N := 12$	*user input*
Diameter of Bolt Circle:	$D_{bc} := 59in$	*user input*
Bolt "Column" Distance:	$l := 3.0in$	*user input*
Bolt Ultimate Strength:	$F_u := 100 \cdot ksi$	*user input*
Bolt Yield Strength:	$Fy := 75 \cdot ksi$	*user input*
Bolt Modulus:	$E := 29000 \cdot ksi$	*user input*
Anchor Bolt Diameter	$D := 2.25in$	*user input*
Threads per Inch:	$n := 4.5$	*user input*

Base Plate Data:

Use ASTM A572 (60 ksi)		*user input*
Plate Yield Strength:	$Fy_{bp} := 60 \cdot ksi$	*user input*
Base Plate Thickness:	$PlateThickness := 2.25 \cdot in$	*user input*
Base Plate Diameter:	$D_{bp} := 65 \cdot in$	*user input*
Outer Pole Diameter:	$D_{pole} := 50in$	*user input*

Geometric Layout Data:

Distance from the center of gravity of the group to bolt in question = d(i)

Radius of Bolt Circle: $R_{bc} := \frac{D_{bc}}{2}$

Distance to Bolts: $i := 1..N$

$$d_i := \left| \begin{array}{l} \theta \leftarrow 2 \cdot \pi \cdot \left(\frac{i}{N}\right) \\ d \leftarrow R_{bc} \cdot \sin(\theta) \end{array} \right.$$

$d_1 = 14.75 \cdot in$ $d_7 = -14.75 \cdot in$

$d_2 = 25.55 \cdot in$ $d_8 = -25.55 \cdot in$

$d_3 = 29.50 \cdot in$ $d_9 = -29.50 \cdot in$

$d_4 = 25.55 \cdot in$ $d_{10} = -25.55 \cdot in$

$d_5 = 14.75 \cdot in$ $d_{11} = -14.75 \cdot in$

$d_6 = 0.00 \cdot in$ etc.

Critical Distances For Bending in Plate:

Outer Pole Radius: $R_{pole} := \frac{D_{pole}}{2}$ $R_{pole} = 25.00 \cdot in$

Moment Arms of Bolts about Neutral Axis: $MA_i := if\left(d_i \geq R_{pole}, d_i - R_{pole}, 0in\right)$

$MA_1 = 0.00 \cdot in$ $MA_7 = 0.00 \cdot in$

$MA_2 = 0.55 \cdot in$ $MA_8 = 0.00 \cdot in$

$MA_3 = 4.50 \cdot in$ $MA_9 = 0.00 \cdot in$

$MA_4 = 0.55 \cdot in$ $MA_{10} = 0.00 \cdot in$

$MA_5 = 0.00 \cdot in$ $MA_{11} = 0.00 \cdot in$

$MA_6 = 0.00 \cdot in$ etc.

Effective Width of Baseplate for Bending: $EffectiveWidth := .8 \cdot 2 \cdot \sqrt{\left(\frac{D_{bp}}{2}\right)^2 - \left(\frac{D_{pole}}{2}\right)^2}$ $EffectiveWidth = 33.23 \cdot in$

Anchor Bolt Analysis:

Polar Moment of Inertia I_p:

$$I_p := \sum_i \left(d_i\right)^2 \qquad I_p = 5.221 \times 10^3 \cdot in^2$$

Gross Area of Bolt:

$$A_g := \frac{\pi}{4} \cdot D^2 \qquad A_g = 3.976 \cdot in^2$$

Net Area of Bolt:

$$A_n := \frac{\pi}{4} \cdot \left(D - \frac{0.9743 \cdot in}{n}\right)^2 \qquad A_n = 3.248 \cdot in^2$$

Net Diameter:

$$D_n := \frac{2 \cdot \sqrt{A_n}}{\sqrt{\pi}} \qquad D_n = 2.03 \cdot in$$

Radius of Gyration of Bolt:

$$r := \frac{D_n}{4} \qquad r = 0.51 \cdot in$$

Section Modulus of Bolt:

$$S_x := \frac{\pi \cdot D_n^{\,3}}{32} \qquad S_x = 0.826 \cdot in^3$$

Anchor Bolt Bending Stress:

Maximum Applied Bending:

$$M_x := \left(\frac{Shear}{N}\right) \cdot l \qquad M_x = 0.479 \cdot ft \cdot kips$$

$$f_{bx} := \frac{M_x}{S_x} \qquad f_{bx} = 7.0 \cdot ksi$$

Allowable Bending

$$F_{bx} := 1.333 \cdot 0.60 \cdot Fy \qquad F_{bx} = 60.0 \cdot ksi$$

Note: 1.333 increase allowed per TIA/EIA

Check Tensile Forces:

Maximum Tensile Force (Gross Area):

$AllowableTension := 1.333 \cdot (0.33 \cdot A_g \cdot F_u)$ $\quad AllowableTension = 174.9 \cdot kips$

Note: 1.333 increase allowed per TIA/EIA

Maximum Tensile Force (Net Area):

$F_{net.area} := 1.333 \cdot (0.60 \cdot A_n \cdot Fy)$ $\quad F_{net.area} = 194.8 \cdot kips$

Note: 1.333 increase allowed per TIA/EIA

Applied Tension:

$$MaxTension := \frac{OM \cdot R_{bc}}{I_p} - \frac{Axial}{N} \qquad MaxTension = 129.6 \cdot kips$$

Check Stresses:

Note: Bolts supplied are "upset bolts." Use net area for checking per AISC

$$\frac{MaxTension}{F_{net.area}} = 0.67$$

$$Condition := if\left(\frac{MaxTension}{F_{net.area}} \le 1.00, "OK", "Overstressed"\right)$$

Condition = "OK"

Check Compression & Combined Stresses (if required):

Check to see if a complete combined stress analysis is required:

Per ASCE Manual 72: "If the clearance between the base plate and concrete does not exceed two times the bolt diameter a bending stress analysis of the bolts is NOT normally required."

Set the clear space between the plate and bolt to zero and remove bending stresses if a combined stress analysis is not required:

$$l := \begin{cases} l & \text{if } l > 2 \cdot D_n \\ 0.00\,in & \text{otherwise} \end{cases} \qquad l = 0.00\,in \qquad f_{bx} := \begin{cases} f_{bx} & \text{if } l > 2 \cdot D_n \\ 0.0\,ksi & \text{otherwise} \end{cases} \qquad f_{bx} = 0.0\,ksi$$

Allowable Compressive Force:

$$K := 0.65$$

$$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{Fy}} \qquad C_c = 87.36$$

$$F_a := \begin{cases} \dfrac{\left[1 - \dfrac{\left(\frac{K \cdot l}{r}\right)^2}{2 \cdot C_c^{\,2}}\right] \cdot Fy}{\dfrac{5}{3} + \dfrac{3 \cdot \left(\frac{K \cdot l}{r}\right)}{8 \cdot C_c} - \dfrac{\left(\frac{K \cdot l}{r}\right)^3}{8 \cdot C_c^{\,3}}} & \text{if } \dfrac{K \cdot l}{r} \le C_c \\ \dfrac{12 \cdot \pi^2 \cdot E}{23 \cdot \left(\frac{K \cdot l}{r}\right)^2} & \text{if } \dfrac{K \cdot l}{r} > C_c \end{cases} \qquad F_a = 45.0\,ksi$$

$$F_a := 1.333 \cdot F_a \quad \text{Note: 1.333 increase allowed per TIA/EIA} \qquad F_a = 60.0\,ksi$$

Applied Compressive Force:

$$MaxCompression := \frac{OM \cdot R_{bc}}{I_p} + \frac{Axial}{N} \qquad MaxCompression = 134.8\,kips$$

$$f_a := \frac{MaxCompression}{A_n} \qquad f_a = 41.5\,ksi$$

Check Combined Stresses:

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} = 0.69$$

$$Condition := if\left(\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} \le 1.00, "OK", "Overstressed"\right) \qquad \boxed{Condition = "OK"}$$

Base Plate Analysis:

Force from Bolt(s):

$$C_i := \frac{OM \cdot d_i}{I_p} + \frac{Axial}{N}$$

$C_1 = 68.7 \cdot kips$ $C_7 = -63.5 \cdot kips$

$C_2 = 117.1 \cdot kips$ $C_8 = -111.9 \cdot kips$

$C_3 = 134.8 \cdot kips$ $C_9 = -129.6 \cdot kips$

$C_4 = 117.1 \cdot kips$ $C_{10} = -111.9 \cdot kips$

$C_5 = 68.7 \cdot kips$ $C_{11} = -63.5 \cdot kips$

$C_6 = 2.6 \cdot kips$ etc.

Bending Stress in Plate:

$$f_{bp} := \sum_i \frac{6 \cdot C_i \cdot MA_i}{EffectiveWidth \cdot PlateThickness^2} \qquad f_{bp} = 26.2 \cdot ksi$$

Check Stresses:

$$\frac{f_{bp}}{1.333 \cdot 0.75 Fy_{bp}} = 0.44$$

$$Condition := if\left(\frac{f_{bp}}{1.333 \cdot 0.75 Fy_{bp}} < 1.00, "OK", "Overstressed"\right)$$

Condition = "OK"

NatComm
Structure & Foundation
Design Calculations
120' Monopole
Site: West Granby/CT-810
EEI Job #: 12233-E02

ENGINEERED ENDEAVORS INCORPORATED
The Experienced Point of View

Customer: NATCOMM/CINGULAR
Description: 120' MONOPOLE
EEI Job Number: 12233

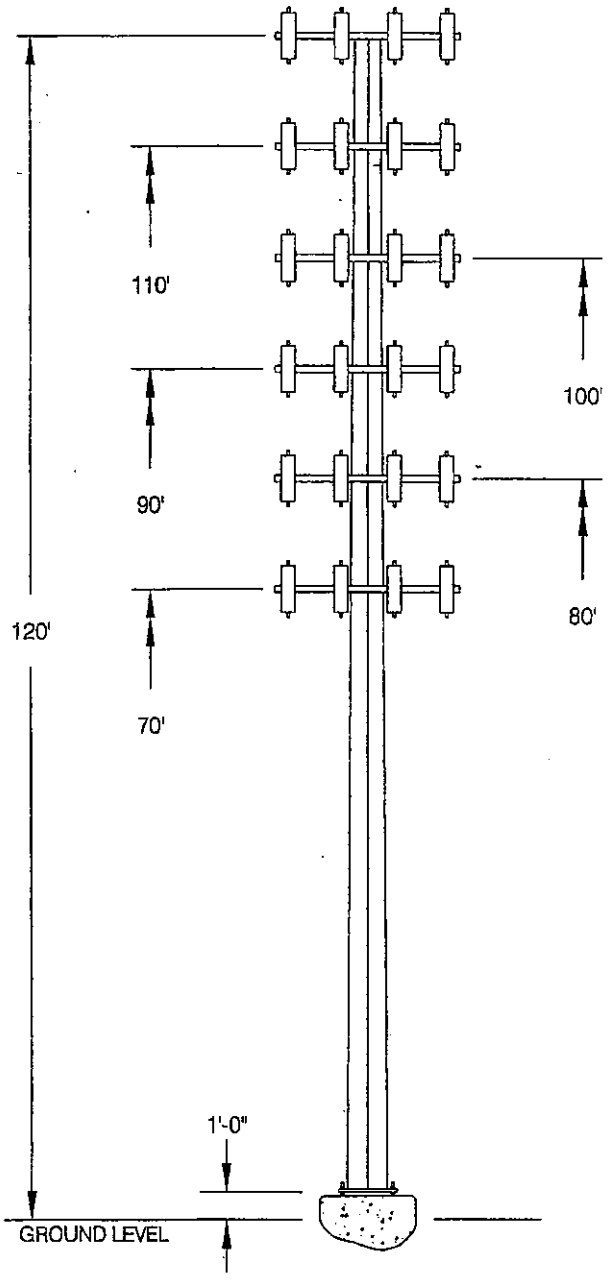
EMERSON™ Network Power

SITE INFORMATION

Location: HARTFORD COUNTY, CT
Site Name: WEST GRANBY
Site Number: N/A

DESIGN INFORMATION

Designed By: N. UNGER
Design Date: 7/26/2005
Status: REVISION 1



ANTENNA LOADING

(12) 7770.00 PANEL ANTENNAS MOUNTED ON LOW PROFILE PLATFORM AT 120' (CINGULAR)

(12) 7770.00 PANEL ANTENNAS MOUNTED ON LOW PROFILE PLATFORM AT 110' (FUTURE)

(12) 7770.00 PANEL ANTENNAS MOUNTED ON LOW PROFILE PLATFORM AT 100' (FUTURE)

(12) 7770.00 PANEL ANTENNAS MOUNTED ON LOW PROFILE PLATFORM AT 90' (FUTURE)

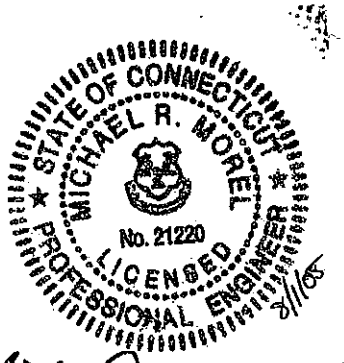
(12) 7770.00 PANEL ANTENNAS MOUNTED ON LOW PROFILE PLATFORM AT 80' (FUTURE)

(12) 7770.00 PANEL ANTENNAS MOUNTED ON LOW PROFILE PLATFORM AT 70' (FUTURE)

DESIGN CRITERIA

DESIGNED IN ACCORDANCE WITH THE TIA/EIA 222-F FOR 80 MPH FASTEST MILE WIND SPEED AND 1/2" RADIAL ICE (NON-SIMULTANEOUS)

DESIGN MEETS THE REQUIREMENTS OF SECTIONS 1609 AND 3108 OF THE 2000 AND 2003 INTERNATIONAL BUILDING CODES FOR 100 MPH 3-SECOND GUST WIND SPEED



Michael R. Morel

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Communications Structure Nonlinear Analysis and Design Program

16:03:16 07-28-2005
Revision 1.3 - 1/22/01
Engineer: ngu

Customer **NATCOMM/CINGULAR**
Job Name **12233-120**
Structure **120' MONOPOLE**
Location **HARTFORD COUNTY, CT**
Site **WEST GRANBY**

OD BOT	OD TOP	NUM. SIDES	THICK INCH	TAPER IN/FT	LENGTH FT	JOINT INCH	JOINT TYPE	YIELD KSI	WEIGHT LBS	JOINT HEIGHT
25.45	19.00	18	0.1875	0.270	23.88	45.00	SLIP	65.0	1052.	97.00
37.84	23.93	18	0.2500	0.270	51.50	63.00	SLIP	65.0	4208.	50.00
50.00	35.79	18	0.3125	0.270	52.63	0.00	BASEPL	65.0	7472.	0.00
			TOTAL TUBE WEIGHT				12732.	POUNDS		
			POLE SHAFT LENGTH				119.00	FEET		

E = 29600.0 KSI
UNIT WGT = 0.283 LBS/CU IN
AISC constants are used for stress reductions.
TUBE SECTIONS HAVE 18 SIDES AND ARE TREATED AS ROUND
Internal bend radius = 3 X T
Tube diameters are measured flat to flat.
Tube diameters are increased by 1.020 for wind across points.
Drag coefficients are increase by 1.300 for steps on the pole.
AISC Tube Shape Coefficient of 1.000 is applied.
ORIGINAL DATA FILE NAME T:\ENG5\JOBS12\12233120
REVISED DATA FILE NAME T:\ENG5\JOBS12\12233120

APPURTENANCES

DESCRIPTION	NUM.	ELEV.	Kz	AREA (WITHOUT ICE)	WGT (WITHOUT ICE)	Ca (WITHOUT ICE)	AREA (WITH ICE)	WGT (WITH ICE)	Ca (WITH ICE)	FACTOR
7770.00	12	119.	1.443	4.20	35.	1.4000	4.67	68.	1.4000	0.93
LOW PROF. PLATF.	1	119.	1.443	7.50	2100.	2.0000	9.00	3250.	2.0000	1.00
7770.00	12	109.	1.407	4.20	35.	1.4000	4.67	68.	1.4000	0.93
LOW PROF. PLATF.	1	109.	1.407	7.50	2100.	2.0000	9.00	3250.	2.0000	1.00
7770.00	12	99.	1.369	4.20	35.	1.4000	4.67	68.	1.4000	0.93
LOW PROF. PLATF.	1	99.	1.369	7.50	2100.	2.0000	9.00	3250.	2.0000	1.00
7770.00	12	89.	1.328	4.20	35.	1.4000	4.67	68.	1.4000	0.93
LOW PROF. PLATF.	1	89.	1.328	7.50	2100.	2.0000	9.00	3250.	2.0000	1.00
7770.00	12	79.	1.283	4.20	35.	1.4000	4.67	68.	1.4000	0.93
LOW PROF. PLATF.	1	79.	1.283	7.50	2100.	2.0000	9.00	3250.	2.0000	1.00
7770.00	12	69.	1.235	4.20	35.	1.4000	4.67	68.	1.4000	0.93
LOW PROF. PLATF.	1	69.	1.235	7.50	2100.	2.0000	9.00	3250.	2.0000	1.00

LOAD CASE 1

BASIC LOADING

DEAD LOAD FACTOR 1.00 WIND PSF REDUCTION 1.00 RADIAL ICE 0.00 IN.

WIND VELOCITY 80 BOTTOM 16.52 PSF TOP 23.54 PSF
MAX BASE ROTATION 0.00 DEG

APPLIED APPURTENANCE FORCES

	ELEVATION FT	WEIGHT KIPS	WIND KIPS
7770.00	119.00	0.420	2.621
LOW PROF. PLATF.	119.00	2.100	0.599
7770.00	109.00	0.420	2.556
LOW PROF. PLATF.	109.00	2.100	0.584
7770.00	99.00	0.420	2.487
LOW PROF. PLATF.	99.00	2.100	0.568
7770.00	89.00	0.420	2.412
LOW PROF. PLATF.	89.00	2.100	0.551
7770.00	79.00	0.420	2.332
LOW PROF. PLATF.	79.00	2.100	0.533
7770.00	69.00	0.420	2.243
LOW PROF. PLATF.	69.00	2.100	0.513

TUBE PROPERTIES			MEMBER FORCES			STRESSES			STRESS	TOTAL	
ELEV FT	DIAM IN	WALL IN	SHEAR K	BENDING K-FT	AXIAL K	AXIAL KSI	BEND. KSI	ALLOW KSI	RATIOS	DEFL IN	TILT DEG
119.00	19.00	0.1875	3.63	0.01	2.44	0.22	0.00	50.60	0.00	64.2	4.61
109.00	21.70	0.1875	3.63	36.15	2.44	0.19	6.42	48.83	0.13	54.7	4.53
99.00	24.40	0.1875	7.40	109.90	5.12	0.36	15.39	47.45	0.33	45.4	4.30
97.00	24.94	0.1875	10.90	131.64	7.69	0.53	17.64	47.21	0.38	43.6	4.24
TYPE OF JOINT: SLIP JOINT											
97.00	24.44	0.2500	11.18	131.66	8.26	0.43	13.89	51.12	0.28	43.6	4.24
89.00	26.60	0.2500	11.18	220.84	8.26	0.40	19.62	49.92	0.40	36.8	4.00
79.00	29.30	0.2500	14.74	367.88	11.25	0.49	26.87	48.68	0.56	28.8	3.61
69.00	32.00	0.2500	18.23	549.83	14.45	0.58	33.60	47.64	0.72	21.7	3.15
59.00	34.70	0.2500	21.57	765.23	17.81	0.66	39.70	46.76	0.86	15.6	2.65
50.00	37.13	0.2500	21.99	962.92	18.85	0.65	43.57	46.08	0.96	11.1	2.16
TYPE OF JOINT: SLIP JOINT											
50.00	36.50	0.3125	22.43	962.92	20.69	0.58	36.25	48.72	0.76	11.1	2.16
40.00	39.20	0.3125	22.43	1187.08	20.69	0.54	38.68	47.87	0.82	7.0	1.71
30.00	41.90	0.3125	22.87	1415.73	22.76	0.56	40.32	47.13	0.87	3.9	1.26
20.00	44.60	0.3125	23.30	1648.73	24.15	0.56	41.39	46.48	0.90	1.7	0.82
10.00	47.30	0.3125	23.75	1886.25	25.64	0.56	42.05	45.90	0.93	0.4	0.40
0.00	50.00	0.3125	24.56	2128.51	28.02	0.57	42.42	45.39	0.95	0.0	0.00

REACTION COMPONENTS (KIPS AND FT-KIPS)

TRANSVERSE SHEAR	VERTICAL FORCE	WIND SHEAR	MOMENT ABOUT TRANSVERSE	MOMENT ABOUT VERTICAL	MOMENT ABOUT WIND AXIS
0.000	28.023	-24.561	2128.511	0.000	0.000

LOAD CASE 2

BASIC LOADING PLUS ICE

DEAD LOAD FACTOR 1.00 WIND PSF REDUCTION 0.75 RADIAL ICE 0.50 IN.

WIND VELOCITY 80 BOTTOM 12.39 PSF TOP 17.66 PSF
MAX BASE ROTATION 0.00 DEG

APPLIED APPURTENANCE FORCES

	ELEVATION FT	WEIGHT KIPS	WIND KIPS
7770.00	119.00	0.811	2.186
LOW PROF. PLATF.	119.00	3.250	0.539
7770.00	109.00	0.811	2.132
LOW PROF. PLATF.	109.00	3.250	0.526
7770.00	99.00	0.811	2.074
LOW PROF. PLATF.	99.00	3.250	0.512
7770.00	89.00	0.811	2.012
LOW PROF. PLATF.	89.00	3.250	0.496
7770.00	79.00	0.811	1.944
LOW PROF. PLATF.	79.00	3.250	0.480
7770.00	69.00	0.811	1.871
LOW PROF. PLATF.	69.00	3.250	0.461

TUBE PROPERTIES			MEMBER FORCES			STRESSES			STRESS	TOTAL	
ELEV FT	DIAM IN	WALL IN	SHEAR K	BENDING K-FT	AXIAL K	AXIAL KSI	BEND. KSI	ALLOW KSI	RATIOS	DEFL IN	TILT DEG
119.00	19.00	0.1875	3.17	0.00	4.06	0.37	0.00	50.60	0.00	55.1	3.97
109.00	21.70	0.1875	3.17	31.60	4.06	0.32	5.61	48.83	0.12	46.9	3.90
99.00	24.40	0.1875	6.44	95.84	8.35	0.59	13.42	47.45	0.29	38.9	3.70
97.00	24.94	0.1875	9.49	114.78	12.52	0.86	15.38	47.21	0.34	37.4	3.65
TYPE OF JOINT: SLIP JOINT											
97.00	24.44	0.2500	9.68	114.75	13.09	0.69	12.11	51.12	0.25	37.4	3.65
89.00	26.60	0.2500	9.68	192.01	13.09	0.63	17.06	49.92	0.35	31.5	3.43
79.00	29.30	0.2500	12.73	319.10	17.67	0.77	23.31	48.68	0.49	24.6	3.10
69.00	32.00	0.2500	15.68	475.73	22.44	0.90	29.07	47.64	0.63	18.6	2.70
59.00	34.70	0.2500	18.48	660.40	27.35	1.01	34.26	46.76	0.75	13.3	2.26
50.00	37.13	0.2500	18.72	828.77	28.35	0.98	37.50	46.08	0.83	9.5	1.85
TYPE OF JOINT: SLIP JOINT											
50.00	36.50	0.3125	18.99	828.77	30.70	0.86	31.20	48.72	0.66	9.5	1.85
40.00	39.20	0.3125	18.99	1018.62	30.70	0.80	33.19	47.87	0.71	6.0	1.46
30.00	41.90	0.3125	19.51	1211.10	33.40	0.82	34.49	47.13	0.75	3.3	1.07
20.00	44.60	0.3125	19.78	1406.15	34.88	0.80	35.30	46.48	0.77	1.5	0.70
10.00	47.30	0.3125	20.07	1603.94	36.46	0.79	35.75	45.90	0.79	0.4	0.34
0.00	50.00	0.3125	20.33	1804.64	37.27	0.76	35.96	45.39	0.81	0.0	0.00

REACTION COMPONENTS (KIPS AND FT-KIPS)

TRANSVERSE SHEAR	VERTICAL FORCE	WIND SHEAR	MOMENT ABOUT TRANSVERSE	MOMENT ABOUT VERTICAL	MOMENT ABOUT WIND AXIS
0.000	37.270	-20.327	1804.637	0.000	0.000

SUMMARY TABLE

ELEV	STRESS RATIO	AXIAL	BENDING	LOADING
119.00	0.01	4.06	0.0	2 BASIC LOADING PLUS ICE
109.00	0.13	2.44	36.1	1 BASIC LOADING
99.00	0.33	5.12	109.9	1 BASIC LOADING
97.00	0.38	7.69	131.6	1 BASIC LOADING
89.00	0.40	8.26	220.8	1 BASIC LOADING
79.00	0.56	11.25	367.9	1 BASIC LOADING
69.00	0.72	14.45	549.8	1 BASIC LOADING
59.00	0.86	17.81	765.2	1 BASIC LOADING
50.00	0.96	18.85	962.9	1 BASIC LOADING
40.00	0.82	20.69	1187.1	1 BASIC LOADING
30.00	0.87	22.76	1415.7	1 BASIC LOADING
20.00	0.90	24.15	1648.7	1 BASIC LOADING
10.00	0.93	25.64	1886.3	1 BASIC LOADING
0.00	0.95	28.02	2128.5	1 BASIC LOADING

MAXIMUM SUPPORT MOMENT K-FT	2128.51
CORRESPONDING AXIAL FORCE KIPS	28.02
CORRESPONDING SHEAR FORCE KIPS	24.56

BASE PLATE AT ELEVATION 0.00 FEET

TUBE DIAMETER	50.00	INCHES
DESIGN MOMENT	2128.5	KIP FT
DESIGN MOMENT IS	0.	DEGREES FROM THE WIND DIRECTION
BOLTS ARE ON THE KNUCKLES OF THE TUBE		
APPLIED AXIAL FORCE	28.0	KIPS
APPLIED SHEAR	24.56	KIPS

BOLT DATA

BOLT TYPE	A615 GR75	
BOLTS ARE EVENLY SPACED		
DIAMETER	2.250	INCHES
EFFECTIVE AREA	3.250	SQ IN
TOTAL LENGTH	8.0	FEET
End plates are required.		
MINIMUM EMBEDMENT	7.0	FEET
NUMBER OF BOLTS	12	
BOLT CIRCLE DIAMETER	59.00	INCHES
ALLOWABLE STRESS	60.0	KSI
APPLIED AXIAL STRESS	45.1	KSI
MAX BOLT FORCE	146.6	KIPS
BOLT BENDING STRESS	3.0	KSI
COMBINED BOLT STRESS	48.1	KSI
CLEARANCE UNDER PLATE	3.25	INCHES
BOLT WEIGHT	1353.6	POUNDS

PLATE DATA

DIAMETER OF PLATE	65.00	INCHES
MATERIAL	A572 GR60	
PROVIDED THICKNESS	2.250	INCHES
REQUIRED THICKNESS	1.474	INCHES
BOLT HOLE DIAMETER	2.625	INCHES
CENTER HOLE SIZE	40.00	INCHES
NET WEIGHT	1271.4	POUNDS
RAW STOCK WEIGHT	2690.3	POUNDS
SURFACE AREA	27.73	SQ FT
ALLOWABLE STRESS	59.99	KSI
MAX APPLIED STRESS	25.76	KSI
CONCRETE STRENGTH	3000.	PSI

Base Plate - use 65.00 inch ROUND x 2.250 inch A572 GR60 with (12) 2.250 diameter x 8.00 foot caged A615 GR75 bolts on a 59.00 inch bolt circle. End plates are required.

FOUNDATION DESIGN CALCULATIONS
FOR
SPREAD FOOTING FOUNDATION

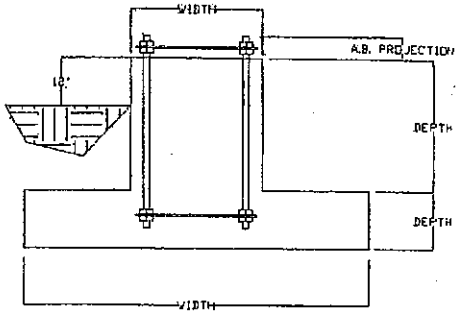
ENGINEERED ENDEAVORS INC.
*7610 Jentner Drive * Mentor, Ohio 44060*
*Tel:(216)918-1101 * Fax:(216)918-1108*

29-Jul-05
02:11 PM

CUSTOMER	NATCOMM/CINGULAR
STRUCTURE	120' MONOPOLE
EEI PROJECT	12233
LOCATION	HARTFORD, CT
SITE NAME	WEST CRANBY

SERVICE LOADS AT BASE OF THE MONOPOLE

	Design Loading
Moment, kip-ft	2128.5
Shear, kips	24.56
Axial Load, kips	28.0



Anchor Bolts	Quantity	12.0
	Length, ft	8.0
	Circle Dia., in	59.0
	Projection, in	12.0
Foundation Parameters		
Pedestal Min. Width, in		77.00
Pedestal Projection, in		12.0
Found. Min Height, ft		7.5
	Height, ft	Width, ft
Footing	3.00	22.00
Pedestal	5.00	7.00

Soil Unit Wt., pcf	110.00
Concrete Unit Wt., pcf	150.00
Slope of backfill, degrees	0.00

Foundation Weight, kips	254.55
Concrete, cub.yd.	62.85
Soil Weight, kips	191.40
Total Vertical Load, kips	473.95
Kern of Eccentricity, ft	3.67
Actual Eccentricity, ft	4.91
Overturning Moment, kip-ft	2324.98
Resisting Moment, kip-ft	5213.45
Allowable Gross Soil Pressure, ksf	N/A
Allowable Net Soil Pressure, ksf	6.0
Gross Soil Pressure, (Service Load), ksf	

H=	4.00	
B=	22.00	
	(gross)	(net)
max q=	2.36	1.44
min q=	0.00	

Safety Factor	Sf=	2.24

ULTIMATE STRENGTH DESIGN OF FOOTING

CONCRETE, psi	3000
STEEL, KSI	60

SHEAR IN FOOTING

1. CASE I -DEAD LOAD, TWO-WAY SHEAR — U= 1.4*D

Ultimate Vertical Load, kips	663.53	
Ultimate Pressure, ksf	1.37	
Ultimate shear V, kips	539.80	
Design shear Vn, kips	2547.57	O.K.

2. CASE II - WIND LOAD, ONE-WAY SHEAR — U=0.9*D+1.3*W

Ultimate Moment, kip-ft		3022.47	
Ultimate Vertical Load, kips		426.56	
Eccentricity, ft		7.09	
Ultimate Pressure, ksf	qult=	3.30	
Dist. from edge to critical sect., ft		5.00	
Pressure distance ft	c=	11.74	
Pressure @ critical section, ksf		1.90	
Ultimate Shear, kips		285.92	
Design Shear, kips		737.45	O.K.

FLEXURE STRENGTH DESIGN

Ultimate Moment, kip-ft		Case I	848.26		
		Case II	2478.31	q1=	1.19
Coefficient of Resistance		Rn=	139.1		
Reinforcement Ratio		r=	0.00238		
Min. Reinforcement Ratio		r min	0.00180		
Min. Steel Area, sq.in.		A1	18.89		
Type of Bars		#	8		
		Ab,in^2=	0.79		
BOTTOM	Min. Number of Bars		23.91		
	Actual Number of Bars		24.00		
	Actual Steel Area, sq.in.		18.96		
	Steel Ratio Actual	ra=	0.00239		
	Revised Coef. of Resist	Rn=	143.63		
	Design Moment, kip-ft		**2559.42**		
	Horizontal Spacing, in	shor=	11.22		
TOP	Min. Steel Area, sq.in		14.26		
	Min. Number of Bars		18.05		
	Actual Number of Bars		24.00		
	Top Steel Area, sq.in		18.96		
	Horizontal Spacing, in	shor=	11.22		

PEDESTAL DESIGN

Pedestal Width, in		84	Ultim. Moment	2926.7	
Concrete, ksi		3			
Reinforcement, ksi		60			
Actual Rebars , #8	Q-ty	30	Area, sq.in	0.79	
Design Rebars	Q-ty	12	Area, sq.in	1.98	
Minimum reinforcement ratio		0.0033	Rebar space, ir	7.85	
Actual reinforcement ratio		0.0034			
Concrete cover , in		4			
Rebar layout radius, in		37.50			

Bending about the major axis

No.	Angle, deg	Coord., in	Edge Dist., in	No.	Angle, deg	Coord., in	Edge Dist., in
1	0	37.50	4.50	7	180	-37.50	79.50
2	30	32.48	9.52	8	210	-32.48	74.48
3	60	18.75	23.25	9	240	-18.75	60.75
4	90	0.00	42.00	10	270	0.00	42.00
5	120	-18.75	60.75	11	300	18.75	23.25
6	150	-32.48	74.48	12	330	32.48	9.52

Location of neutral axis c=, ir 6.48
Compression zone, a= 5.51

	No.	e	Force *kips*
	1	0.0009	47.47
eu= 0.003			
Concrete, kips			1179.81
Total compression			**1227.28**

Tension zone	No.	e	Force *kips*
	2	0.0014	80.72
	3	0.0078	118.50
ey= 0.00207	4	0.0164	118.50
	5	0.0251	118.50
	6	0.0315	118.50
	7	0.0338	118.50
	8	0.0315	118.50
	9	0.0251	118.50
	10	0.0164	118.50
	11	0.0078	118.50
	12	0.0014	80.72
Total tension, kips			**1227.93**

Moment due to compression

Rebars	Force *kips*	Mom. Arm. *in*	Moment *k-ft*
1	47.47	37.50	148.33
2	0.00	32.48	0.00
12	0.00	32.48	0.00
Concrete	1179.81	39.25	3858.58
Total in compression			**4006.91**

Moment due to tension

Rebars	Force *kips*	Mom. Arm. *in*	Moment *k-ft*
2	80.72	32.48	-218.45
3	118.50	18.75	-185.16
4	118.50	0.00	0.00
5	118.50	-18.75	185.16
6	118.50	-32.48	320.70
7	118.50	-37.50	370.31
8	118.50	-32.48	320.70
9	118.50	-18.75	185.16
10	118.50	0.00	0.00
11	118.50	18.75	-185.16
12	80.72	32.48	-218.45
Total in tension			**574.82**

Design moment about the major axis, kip-ft 4123.56

JAWORSKI
GEOTECH, INC.

SERVICES
- Geotechnical
- Environmental
- Construction Monitoring
- Materials Testing

January 6, 2004

Ms. Jennifer Coombs
NATCOMM, LLC
63-2 North Branford Road
Branford, CT 06405

Advance Copy by Fax

Re: AT&T West Granby II
30 Higley Road
West Granby, Connecticut

Project No. 03735G

Dear Ms. Coombs:

The following report, prepared by Jaworski Geotech, Inc. (JGI), presents our geotechnical engineering evaluation of subsurface conditions as they relate to foundation design and earthwork construction for the proposed communication tower development. Our services were conducted in accordance with our general proposal, dated April 1, 2003, and are subject to the limitations contained herein.

SITE AND PROJECT DESCRIPTION

The proposed lease site is located at 30 Higley Road in West Granby, Connecticut. The site is undeveloped grassland. The lease area is relatively flat, at approximate Elevation 592 feet, as shown on plans provided by NATCOMM, LLC of Branford, Connecticut.

The project involves constructing a 120-foot tall, self-supported, steel monopole communications tower within an approximate 100-foot square lease area. Current plans indicate that several equipment cabinets and prefabricated shelters will be constructed within the tower compound. An access road will extend from Higley Road to the lease area. The proposed tower development, existing conditions and boring and probe locations are shown on Figure 1, Subsurface Exploration Location Plan.

☐ 77 Sundial Avenue, Suite 401W
Manchester, NH 03103
(603) 647-9700 Fax 647-4432

☑ 114 Woodlawn Road
Berlin, CT 06037-1535
(860) 829-1725 Fax 829-1745

Internet Address: http://www.jgi-geo.com

SUBSURFACE EXPLORATIONS AND CONDITIONS

JGI monitored the advancement of one test boring (JB-1), drilled by New England Borings Inc. of Glastonbury, Connecticut on December 31, 2003. The test boring was advanced with a 3¼-inch diameter solid stem auger at the tower center to a depth of 26 feet below the existing ground surface and continued to a final depth of approximately 26 feet using a roller bit. Soil samples were obtained where possible with a standard 2.0-inch outside diameter split-barrel sampler. Standard Penetration Tests were performed at sampling intervals in general accordance with ASTM D-1586.

The subsurface profile at the boring location consists of topsoil and subsoil underlain by a glacial till deposit. The topsoil has a thickness of about 1 foot and consists primarily of brown, coarse to fine sand with some silt and trace root matter. The subsoil extends to a depth of about 2 feet and consists of loose, brown, medium to fine sand, some silt, trace gravel and root matter. The glacial till was encountered below the subsoil, extends to a depth of at least 25 feet 10 inches. The glacial till consists of dense to very dense, gray-brown, coarse to fine sand, some silt, little gravel with occasional cobble.

At the time of the exploration program, groundwater was observed at 6 feet. However, groundwater levels vary depending upon season, precipitation and other conditions that may be different from those at the time of drilling. Further, groundwater could temporarily perch above the glacial till.

Following drilling, *in situ* soil resistivity testing was completed. Resistivity testing was performed in accordance with ASTM G-57 by the Wenner Four Probe Method using a 16gl Earth Resistivity Meter. Two resistivity lines were completed with electrodes spaced at 2, 4, 8, 16, 25 and 40 feet across the planned lease area. At the time of resistivity testing, the surficial soil was relatively moist. The location and orientation of resistivity lines are shown on Figure 1. The resistivity test results are summarized below:

	Resistivity (ohm-cm)	
Electrode Spacing (ft)	Line #1	Line #2
2	99,580	61,701
4	133,361	96,669
8	170,512	153,047
16	163,005	198,547
25	170,914	174,265
40	169,286	134,050

FOUNDATION TYPE AND DESIGN RECOMMENDATIONS

Tower

The tower should be supported on either a monolithic mat or a pier and pad foundation bearing on the glacial till or on compacted structural fill or crushed stone, placed on the till. The tower foundation can be designed based on a net allowable bearing pressure of 6 kips per square foot (ksf). The allowable bearing pressure may be increased by one third for transient loadings, such as wind and seismic. The bearing pressure is unlikely to govern the design, with overturning determining the size of the foundation. We estimate that settlement of the footing would be less than 1 inch, and depending on the actual size of the footing, the settlement may be less than ½ inch.

An ultimate friction factor (tan δ) of 0.45 may be used for calculation of the sliding resistance between the glacial till and concrete surfaces. A factor of safety of at least 1.5 should be applied to the sliding resistance. A dry unit weight of 120 pounds per cubic foot and an ultimate passive earth pressure coefficient, K_p, of 3.0 should be used for the calculation of passive resistance provided by compacted backfill adjacent to the tower foundation. Passive pressures calculated by these parameters should be reduced by a factor of safety of 3, to reflect movement required to mobilize passive resistance.

The underside of the tower foundation should be located at least 3.5 feet below exterior grade to reduce the likelihood of frost heave. To increase the overturning resistance of the footing, the mat or pad could be located deeper and extended to the surface with a pier. This would reduce the size of the foundation footprint and utilize the mass of the fill placed above the footing to increase overturning resistance. The excavation around and above the foundations should be backfilled with compacted fill.

Control of backfill compaction above and around the foundation will be required to provide uplift and lateral resistance. Care should be exercised during excavation for the tower foundation to minimize disturbance to the materials surrounding the excavation; disturbance to the adjacent materials will influence resistance to lateral loads.

Equipment Cabinets

The equipment cabinet pads may be supported on either concrete pier foundations or slabs-on-grade. We recommend that equipment cabinet foundations bear on the natural glacial till or on compacted structural fill or crushed stone, placed on the till. The footings may be designed using a maximum net allowable bearing pressure of 6 ksf. Settlements will be small, likely less than ½ inch, most of which will occur as the load is applied. Strip footings should have a minimum width of 12 inches. Piers should have a minimum side dimension/diameter of 12 inches.

A modulus of subgrade reaction (k_s) of 250 pounds per cubic inch may be used for design of slabs-on-grade bearing on the a minimum 12 inch layer of compacted structural fill or crushed stone, placed on the till. Consideration should be given to using dense insulation boards (Dow Styrofoam Highload, or similar) under and adjacent to lightly loaded slabs-on-grade, to provide the equivalent of 3.5 feet of earth cover, reducing frost penetration. Settlements will likely be less than ½ inch.

Air entraining admixtures should be used for concrete exposed to freezing. To reduce the likelihood of frost heave, the underside of foundation elements should be at least 3.5 feet below finished ground surface, unless adequately protected by insulation boards.

Seismic Design Criteria

The subsurface conditions were reviewed in accordance with the Connecticut State Building Code, which includes the *BOCA National Building Code, 13th Edition*. For calculation of the lateral seismic forces on the structure, the soil profile is considered to be Type S_1. Accordingly, the recommended site coefficient (S) for seismic design is 1.0. The site is not susceptible to liquefaction in the event of an earthquake.

EARTHWORK AND CONSTRUCTION RECOMMENDATIONS

Compacted Structural Fill

Excavated glacial till may selectively be reused as structural fill adjacent to and over the tower foundation, provided it can be adequately compacted. The glacial till has an elevated silt content and may be difficult to compact, especially during wet weather. Cobbles and boulders with a minimum dimension greater than 8 inches should be removed. Structural fill originating from an off-site source, if required, should conform to the gradation requirements for Bank or Crushed Gravel (M.2.06, Grading B) as defined by the *State of Connecticut Department of Transportation Standard Specifications*. Where fill is required, it should be placed in loose lifts not exceeding 8 inches in thickness. Structural fill placed below, and backfill adjacent to and over the foundations should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D-1557.

Mat and Equipment Foundation Subgrades

The foundation bearing subgrades should be prepared by the contractor as outlined herein and observed by the geotechnical engineer, prior to foundation construction. Fill or concrete should not be placed on frozen subgrades. Frozen materials should not be used as fill. Upon the completion of excavation for the foundations and slabs-on-grade, the subgrade should be proofrolled with at least six passes of a vibratory roller or plate compactor. Unstable subgrades should be removed and replaced with compacted structural fill or crushed stone, as necessary.

Temporary Excavation and Dewatering

Excavations deeper than 4 feet will be required for construction of the tower foundation. Temporary construction slopes in the glacial till should be designed in compliance with recent governing regulations. Construction slopes should be cut to a stable incline or braced, depending upon the excavation depth and encountered subsurface conditions.

Construction slopes should be reviewed for signs of mass movement. If movement and/or potential stability problems are observed, work should cease; the geotechnical engineer should be immediately contacted. The responsibility for excavation safety and stability of temporary construction slopes should lie solely with the contractor.

We do not anticipate significant construction dewatering as the groundwater was encountered at a depth of 6 feet. The contractor should prevent groundwater, if encountered, and surface water runoff from collecting in excavations. We anticipate that dewatering may be accomplished using multiple filtered sump. Subgrade soils that become unstable because of water and/or reworking by construction activity should be replaced with compacted granular structural fill or crushed stone, as necessary.

LIMITATIONS

The analyses and recommendations submitted in this report are based upon the data obtained from the test boring and auger probes. The nature and extent of variations from the conditions observed within the explorations may not become evident until construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations of this report.

We request the opportunity to review final design drawings and specifications to evaluate the appropriate implementation of our recommendations. In the event that changes in the nature, design, or location of the proposed areas are planned, the conclusions and recommendations contained in this report shall not be considered valid unless we review the changes, and conclusions of the report are modified or verified by us in writing.

A geotechnical engineer should be retained to provide testing and monitoring services during the earthwork phases of the project. This is to observe compliance with our design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

This report has been prepared for the exclusive use of NATCOMM, LLC. in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made. This report has been prepared for preliminary design purposes and may be limited

specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

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If you have questions, please contact us. It was a pleasure working with you on this project. We look forward to working with you in the future.

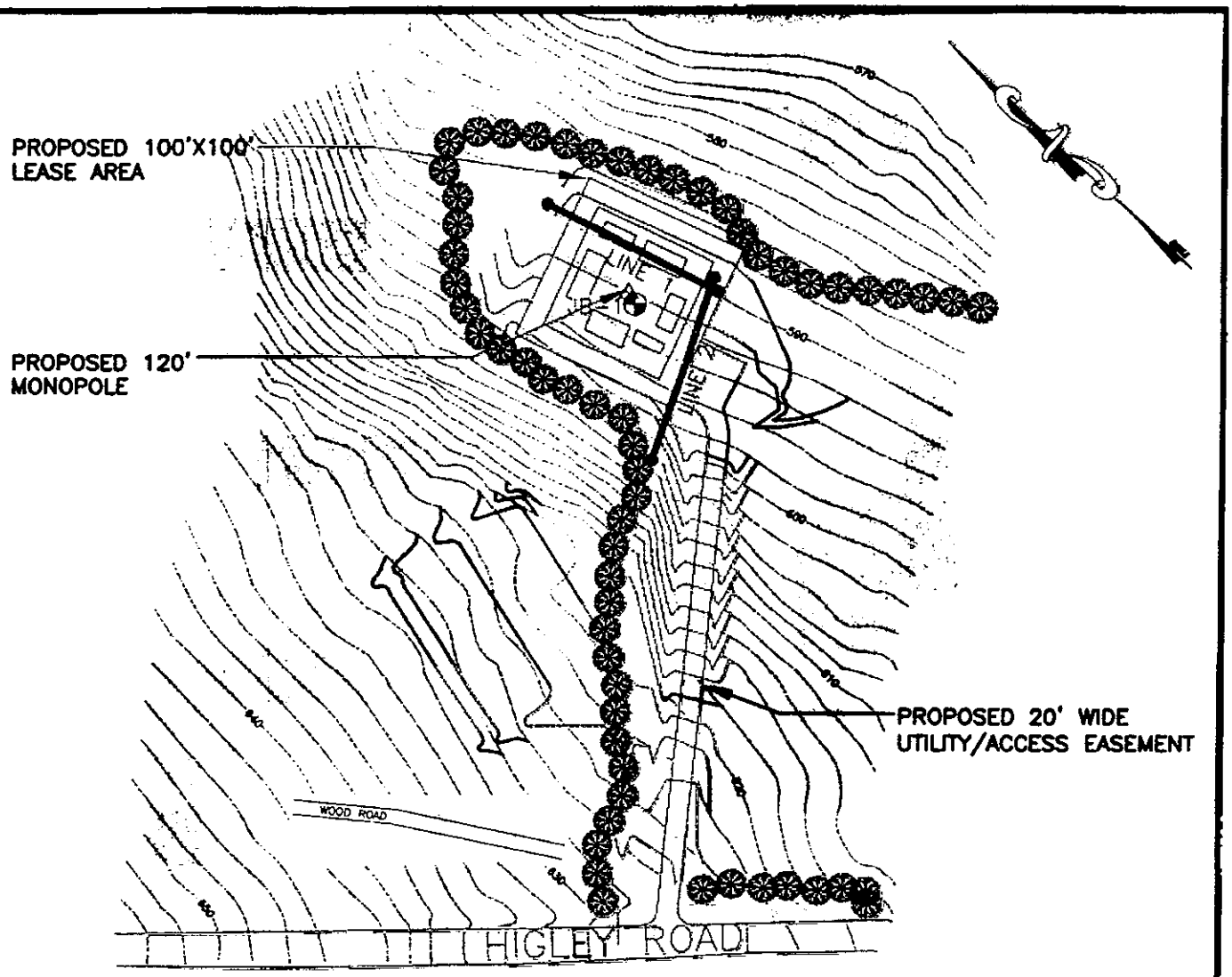
Very truly yours,

JAWORSKI GEOTECH, INC.

Ryan R. Roy, P.E.
Senior Engineer/Associate

/md/03735G

Attachments: Figure 1 – Subsurface Exploration Location Plan
Test Boring Log, JB-1



NOTES:

1. THIS PLAN WAS PREPARED FROM NATCOMM, LLC DRAWING #907-007-810E-SC3, TITLED SITE ACCESS MAP, DATED 10/01/03.
2. THE EXPLORATION SHOWN AS JB-1 WAS ADVANCED ON DECEMBER 31, 2003, UNDER THE DIRECTION OF JGI WITH EQUIPMENT OWNED AND OPERATED BY NEW ENGLAND BORING CONTRACTORS INC. OF GLASTONBURY, CT.
3. SOIL RESISTIVITY TESTS WERE PERFORMED ON DECEMBER 31, 2003 BY JGI.
3. THE APPROXIMATE LOCATION OF THE SUBSURFACE EXPLORATION WAS TAPED FROM STAKED TOWER LOCATION PLACED BY OTHERS. THE LOCATION SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
4. USE OF THIS PLAN IS LIMITED TO THE ILLUSTRATION OF THE APPROXIMATE LOCATION OF THE SUBSURFACE EXPLORATION AND OTHER PERTINENT SITE FEATURES. ANY OTHER USE OF THIS PLAN WITHOUT PERMISSION FROM JGI IS PROHIBITED.

LEGEND

JB-1 TEST BORING LOCATION

LINE #1 SOIL RESISTIVITY TEST

GRAPHIC SCALE

50 0 50 100 200

(IN FEET)

AT&T WEST GRANBY II

WEST GRANBY, CONNECTICUT

PREPARED FOR:

NATCOMM, LLC
63-2 NORTH BRANFORD ROAD
BRANFORD, CT 06405

DATE: JANUARY 2004
SCALE: 1" = 100'
03735

FIGURE 1

SUBSURFACE EXPLORATION LOCATION PLAN

JAWORSKI GEOTECH, INC.
114 WOODLAWN ROAD
BERLIN, CONNECTICUT 06037

03735

TEST BORING LOG

PROJ. NAME:	AT&T West Granby II	HAMMER		SAMPLER	CASING	SHEET 1 OF 1	
LOCATION:	West Granby, CT	TYPE:	Safety	SS	HSA	BORING:	JB-1
PROJECT NO.:	03735	SIZE:	140 lbs.	2" OD	3-1/4"ID	LOCATION:	See Plan
DATE START:	December 31, 2003	FALL:	30"	Drop Method:	Winch/Cable		
DATE END:	December 31, 2003					SURF. EL.:	592' +/-
BORING CO.:	New England Borings	GROUNDWATER OBSERVATIONS					
CO. LOCATION:	Glastonbury, CT	DATE	DEPTH	CASING AT	DURATION AFTER DRILLING		
FOREMAN:	Tim Carpenter	12/31/03	6'	None	15 min.		
JGI REP:	Greg Wingard						

Depth (ft)	SAMPLING No.	Depth (ft.)	Blows/ 6"	Penet./ Rec. (in)	Sample Description	Strata Change Depth (ft)	Notes
	SS-1A	0-1	1-1	24/18	SS-1A: Topsoil with Root Matter.	1.0	
	SS-1B	1-2	1-1		SS-B: Loose, brown, medium to fine SAND, some Silt, trace Gravel. (Subsoil)	2.0	
	SS-2	2-4	7-15	24/16	SS-2: Dense, gray-brown, medium to fine SAND, some Silt, trace Gravel.		
			19-15				
5							
	SS-3	5-7	14-19	24/18	SS-3: Similar to SS-2.		
			21-31				
	SS-4	7-9	29-41	24/20	SS-4: Very dense, gray-brown, coarse to fine SAND, some Silt, little Gravel, occasional Cobbles.		
			40-43				
10							
	SS-5	10-11	41-83	12/10	SS-5: Similar to SS-4.		
15							
	SS-6	15-15.6	52-100/1"	7/6	SS-6: Very dense, gray-brown, coarse to fine SAND, some Silt, little occasional Cobbles.		
20							
	SS-7	20-20.5	100/6"	6/5	SS-7: Similar to SS-6.		
25							
	SS-8	25-25.8	47-100/4"	10/10	SS-8: Similar to SS-6. (Till)		
					Exploration Terminated at 25.8'		
30							

JAWORSKI GEOTECH, INC.

Notes:

Proportions Used: trace (1-10%), little (10-20%), some (20-35%), and (35-50%).

Cohesive Consistency (Blows/ft.)		Cohesionless Relative Density (Blows/ft)	
very soft	0-2	very loose	0-4
soft	2-4	loose	4-10
medium stiff	4-8	medium dense	10-30
stiff	8-15	dense	30-50
very stiff	15-30	very dense	50+
hard	30+		

Boring No. JB-1