



Daniel F. Caruso
Chairman

STATE OF CONNECTICUT

CONNECTICUT SITING COUNCIL

Ten Franklin Square, New Britain, CT 06051

Phone: (860) 827-2935 Fax: (860) 827-2950

E-Mail: siting.council@ct.gov

Internet: ct.gov/csc

March 25, 2009

Kenneth C. Baldwin, Esq.
Robinson & Cole LLP
280 Trumbull Street
Hartford, CT 06103-3597

RE: **EM-VER-096-090113** - Cellco Partnership d/b/a Verizon Wireless notice of intent to modify an existing telecommunications facility located at 586 Danbury Road, New Milford, Connecticut.

Dear Attorney Baldwin:

The Connecticut Siting Council (Council) hereby acknowledges your notice to modify this existing telecommunications facility, pursuant to Section 16-50j-73 of the Regulations of Connecticut State Agencies.

The proposed modifications are to be implemented as specified here and in your notice dated January 13, 2009, including the placement of all necessary equipment and shelters within the tower compound. The modifications are in compliance with the exception criteria in Section 16-50j-72 (b) of the Regulations of Connecticut State Agencies as changes to an existing facility site that would not increase tower height, extend the boundaries of the tower site, increase noise levels at the tower site boundary by six decibels, and increase the total radio frequencies electromagnetic radiation power density measured at the tower site boundary to or above the standard adopted by the State Department of Environmental Protection pursuant to General Statutes § 22a-162. This facility has also been carefully modeled to ensure that radio frequency emissions are conservatively below State and federal standards applicable to the frequencies now used on this tower.

This decision is under the exclusive jurisdiction of the Council. Please be advised that the validity of this action shall expire one year from the date of this letter. Any additional change to this facility will require explicit notice to this agency pursuant to Regulations of Connecticut State Agencies Section 16-50j-73. Such notice shall include all relevant information regarding the proposed change with cumulative worst-case modeling of radio frequency exposure at the closest point of uncontrolled access to the tower base, consistent with Federal Communications Commission, Office of Engineering and Technology, Bulletin 65. Any deviation from this format may result in the Council implementing enforcement proceedings pursuant to General Statutes § 16-50u including, without limitation, imposition of expenses resulting from such failure and of civil penalties in an amount not less than one thousand dollars per day for each day of construction or operation in material violation.

Thank you for your attention and cooperation.

Very truly yours,

S. Derek Phelps
Executive Director

SDP/MP/laf

c: The Honorable Patricia A. Murphy, Mayor, Town of New Milford
Hans Fiedler, T-Mobile USA, Inc.



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Ten Franklin Square, New Britain, CT 06051

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January 16, 2009

The Honorable Patricia A. Murphy
Mayor
Town of New Milford
Town Hall
10 Main Street
New Milford, CT 06776

RE: **EM-VER-096-090113** - Cellco Partnership d/b/a Verizon Wireless notice of intent to modify existing telecommunications facilities located at 586 Danbury Road, New Milford, Connecticut.

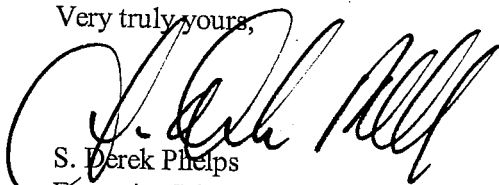
Dear Mayor Murphy:

The Connecticut Siting Council (Council) received this request to modify an existing telecommunications facility, pursuant to Regulations of Connecticut State Agencies Section 16-50j-72.

If you have any questions or comments regarding this proposal, please call me or inform the Council by January 30, 2009.

Thank you for your cooperation and consideration.

Very truly yours,



S. Derek Phelps
Executive Director

SDP/jb

Enclosure: Notice of Intent

EM-VER-096-090113

280 Trumbull Street
Hartford, CT 06103-3597
Main (860) 275-8200
Fax (860) 275-8299
kbaldwin@rc.com
Direct (860) 275-8345

ORIGINAL

January 13, 2009

Via Hand Delivery

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

RECEIVED
JAN 13 2009CONNECTICUT
SITING COUNCIL

Re: **Notice of Exempt Modification – Antenna Swap**
586 Danbury Road, New Milford, Connecticut

Dear Mr. Phelps:

Cellco Partnership d/b/a Verizon Wireless ("Cellco") currently maintains two antennas at the 77-foot level inside a "stealth" flagpole tower, owned by T-Mobile at 586 Danbury Road in New Milford, Connecticut. Cellco acquired this facility from Alltel Communications on May 30, 2008.

Cellco now intends to remove the 18.5" diameter concealment tube and the two (2) existing antennas and install a 30.5" diameter concealment tube and two (2) XDUO6-65 antennas at the same 77-foot level inside the flagpole tower. Attached behind Tab 1 are the specifications for the proposed replacement antennas.

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 Patricia A. Murphy, Mayor of the Town of New Milford. Pursuant to a Council directive, a copy of this letter is also being sent to Oskar G. Rogg and Anne Wynne Rogg, the owners of the property on which the flagpole tower is located.

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

1. The proposed modifications will not result in the increase in the overall height of the existing structure. Cellco's replacement antennas will be located at the 77-foot level of the 100-foot flagpole tower.



Law Offices

BOSTON

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www.rc.com

HART1-1482380-1

S. Derek Phelps
January 13, 2009
Page 2

2. The proposed antenna modifications will not require the extension of the site boundaries.

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

4. The operation of the replacement 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. A cumulative power density table for the facility is included behind Tab 2.

Also attached is a Structural Analysis Report confirming that the flagpole tower can support the proposed modifications. (See Tab 3).

For the foregoing reasons, Cellco respectfully submits that the proposed modifications to the above-referenced telecommunications facility constitutes an exempt modification under R.C.S.A. § 16-50j-72(b)(2).

Sincerely,



Kenneth C. Baldwin

Enclosures

Copy to:

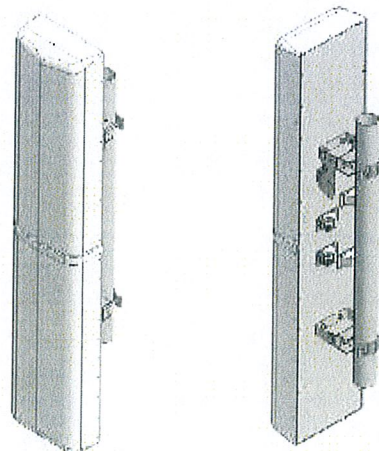
Patricia A. Murphy, New Milford Mayor
Oskar G. Rogg and Anne Wynne Rogg
Sandy M. Carter



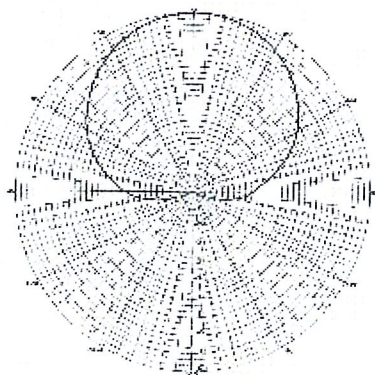
4 in 1 Design
Dual Band and Dual Polarized

Electrical Specifications

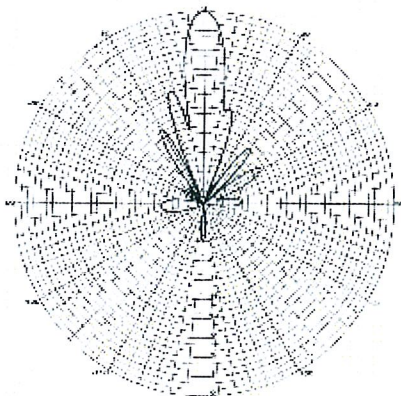
	* 824-896 MHz	1850-1990 MHz
Gain	16.1 dBi	18.6 dBi
Horizontal Beam (1/2 power)	65°	65°
Vertical Beam (1/2 power)	11°	5°
Electrical Downtilt Options	0, 2, 4 or 6 deg	0, 2, 4 or 6 deg
VSWR	≤1.40:1	≤1.40:1
VSWR (with -i option)	≤1.50:1	≤1.50:1
Front-to-Back Ratio	>24 dB	>30 dB
Upper Side Lobe Supp.	<-18 dB	<-20 dB
Polarization	±45°	±45°
Impedance	50 ohms	50 ohms
Power Input Rating (per port)	500 CW	250 CW
Intermodulation (2x20w)	typ. -150 dBc	typ. -150 dBc
Port to Port Isolation	>30 dB	>30 dB
Cross Polar Discrimination	>16 dB	>18 dB



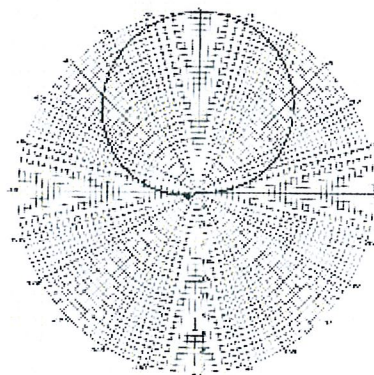
824-896 MHz
Horizontal pattern



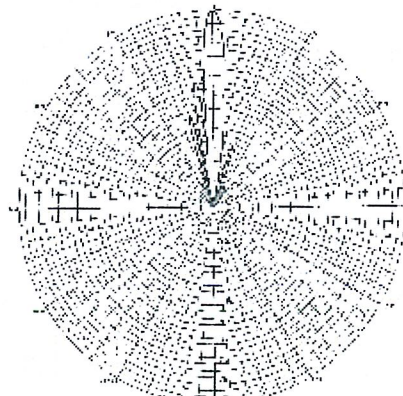
Vertical pattern



1850-1990 MHz
Horizontal pattern



Vertical pattern

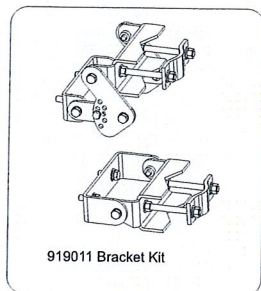


65° Horizontal Beams
11° & 5° Vertical Beams
16.1 & 18.6 dBi Gain

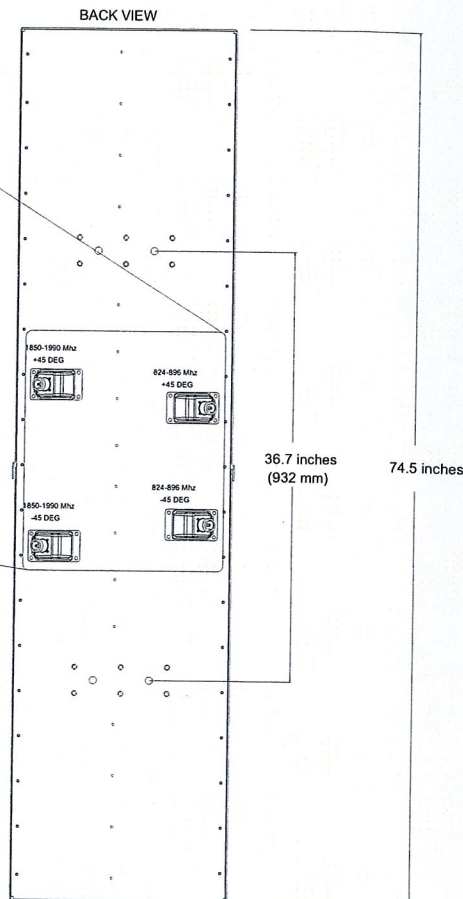
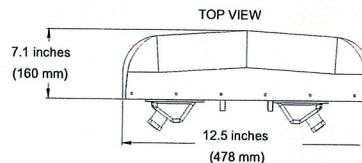
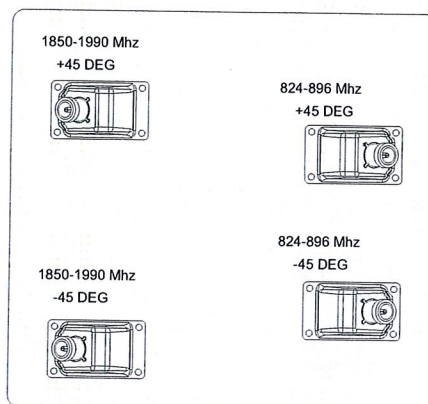


Directing our energies for you.

XDUO6-65



919011 Bracket Kit



Mechanical Specifications

Input Connectors (female)
Antenna Dimensions
Antenna Weight
Bracket Weight
Lightning Protection
RF Distribution
Radome
Weatherability
Radome Water Absorption
Environmental
Wind Survival
Front Wind Load at 100 mph
Front Flat Plate Equivalent
Mounting Brackets
Mechanical Downtilt Range
Clamps/Bolts

Four Back Mounted 7/16 DIN
74.5 x 12.5 x 7.1 Inches
37 lbs
13 lbs
Direct Ground
Printed Microstrip Substrate
Ultra High-Strength Luran
UV Stabilized, ASTM D1925
ASTM D570, 0.45%
MIL-STD-810E
150 mph
183 lbs
3.73 sq-ft. (c=2)
Fits 2.5 to 3 Inch Schedule 40 Pipe
0-12 Degrees in 1 Degree Increments
Hot Dip Galvanized Steel/Stainless Steel

Model Numbers Option Descriptions

XDUO6-65-xy x=Electrical Downtilt at 800 MHz in Degrees (0, 2, 4 or 6)
y=Electrical Downtilt at 1900 MHz in Degrees (0, 2, 4 or 6)
XDUO6-65-xyi Dual Band Combiner included as an internal device

Downtilt and Mounting Bracket Included.

* other frequency bands available

Published 091307
Subject to alteration.

CSS Antenna, Inc.
Tel: 410-612-0080 Fax: 410-612-0336
www.cssantenna.com



	General	Power	Density					
Site Name: Candlewood								
Tower Height: Verizon @ 77ft								
CARRIER	# OF CHAN.	WATTS ERP	HEIGHT	CALC. POWER DENS	FREQ.	MAX. PERMISS. EXP.	FRACTION MPE	Total
*VoiceStream	4	250	95	0.0398	1930	1.0000	3.98%	
*VoiceStream	4	250	86	0.0486	1930	1.0000	4.86%	
Verizon	9	410	77	0.2238	1970	1.0000	22.38%	
Verizon	9	345	77	0.1883	875	0.5830	32.30%	
* Source: Siting Council								
								63.52%



STRUCTURAL ANALYSIS REPORT

100' STEALTH FLAG POLE

586 Danbury Road, New Milford, CT
Verizon Site: Candlewood Alltel, CT

January 7, 2009
Revision 2

Prepared for:



Construction Services of Branford, LLC
63-3 North Branford Road
Branford, CT 06405



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 - **ANCHOR BOLT AND BASE PLATE ANALYSIS**
 - **FOUNDATION ANALYSIS**



1. EXECUTIVE SUMMARY

This report summarizes the structural analysis of the existing 100' self supporting steel flagpole structure located at 586 Danbury Road in New Milford, Connecticut. The analysis was conducted in accordance with the 2005 Connecticut State Building Code and the TIA/EIA-222-F standard for a wind velocity of 80 mph (fastest mile) and 69 mph (fastest mile) concurrent with ½" ice. The antenna loading considered in the analysis consists of all existing and proposed antennas, cables, and ancillary items as outlined in the Introduction Section of this report. The proposed Verizon additions are as follows:

Proposed Antenna and Mount	Carrier	Antenna Center Elevation
Remove: 18" diameter by 6' antenna concealment cylinder (2) Powerwave 7263.01 antennas and (4) 7/8" coax cables	Verizon	77' AGL
Install: (2) CSS XDUO6-65 antennas inside a new 30" diameter by 6' antenna concealment cylinder with (8) 1 5/8" coax cables	Verizon	77' AGL

The results of the analysis indicate that the flagpole structure is in compliance with the proposed loading conditions. **The flagpole and its foundation are considered structurally adequate with the wind load classification specified above with all the existing and proposed antenna loading.**

This analysis is based on:

- 1) The flagpole structure's theoretical capacity, not including any assessment of the condition of the flagpole.
- 2) Flagpole geometry and structural member sizes taken from original analysis report prepared by Paul J. Ford and Company, job number 31900-067, signed and sealed August 21, 2000.
- 3) Antenna and mount configuration as specified on the following page of this report.

This report is only valid per the assumptions and data utilized in this report for antenna inventory, mounts and associated cables. The user of this report shall field verify the assumption of the antenna and mount configuration as well as the physical condition of the flagpole and connections. Notify the engineer in writing immediately if any of the information in this report is found to be other than specified.

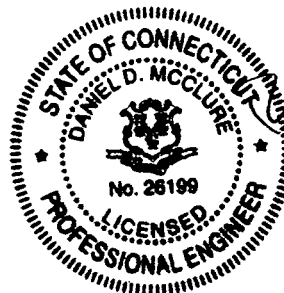
If you should have any questions, please call.

Sincerely,

JWB Tower Services, LLC

Daniel D. McClure, P.E.
Project Engineer

cc: File





2. INTRODUCTION

The subject flagpole is located at 586 Danbury Road in New Milford, Connecticut. The structure is a 100' self supporting steel flagpole designed by Paul J. Ford and Company.

The flagpole geometry and structure member sizes were taken from the original analysis report prepared by Paul J. Ford and Company, job number 31900-067, signed and sealed August 21 2000.

The inventory is summarized in the table below:

Antenna, Mount, and Cable	Carrier	Antenna Center Elevation
(2) Panel antennas inside a 18" diameter by 19' antenna concealment cylinder with (4) 7/8" coax cables	T-Mobile (existing)	95' AGL
(2) CSS XDU06-65 antennas inside a new 30" diameter by 6' antenna concealment cylinder with (8) 1 5/8" coax cables	Verizon (proposed)	77' AGL

This structural analysis of the communications flagpole was performed by JWB Tower Services LLC (JWB) for Construction Services of Branford LLC (CSB). The purpose of this analysis was to investigate the structural integrity of the existing flagpole with its existing and proposed antenna loads. This analysis was conducted to evaluate stress on the flagpole and subsequent reaction forces to the foundation of the flagpole resulting from the above flagpole inventory.



3. ANALYSIS METHODOLOGY AND LOADING CONDITIONS

The structural analysis was done in accordance with the 2005 Connecticut State Building Code, TIA/EIA-222-F—Structural Standard for Steel Antenna Towers and Antenna Supporting Structures, and the American Institute of Steel Construction (AISC) Manual of Steel Construction—Allowable Stress Design (ASD).

The analysis was conducted using RISA Tower 5.3. Two load conditions were evaluated as shown below which were compared to allowable stresses according to AISC and TIA/EIA.

Load Condition 1 = 80 mph (fastest mile) Wind Load (without ice) + Tower Dead Load
Load Condition 2 = 69 mph (fastest mile Wind Load (with ice) + Ice Load + Tower Dead Load

Please note that wind pressure is a function of velocity squared. Under Load Condition 2, a 25 percent reduction in wind pressure is allowed by code to account for the unlikelihood of the full wind pressure and ice load occurring at the same time. The same results may be achieved by utilizing a lower wind pressure without taking the 25 percent reduction, as shown above.

The TIA/EIA standard permits a one-third increase in allowable stresses for towers and monopoles less than 700 feet tall. For the purposes of this analysis, in computing the load capacity the allowable stresses of the flagpole members were increased by one-third.

4. FINDINGS AND EVALUATION

Stresses on the flagpole structure were evaluated to compare with allowable stresses in accordance with AISC. The calculated stresses under the proposed loading were below the allowable stresses. Detailed analysis and calculations for the proposed load condition are provided in section 6 of this report. Additionally, the anchor bolts and foundation were found to be structurally adequate.



5. CONCLUSIONS

The results of the analysis indicate that the flagpole structure is in compliance with the proposed loading conditions. **The flagpole and its foundation are structurally adequate under the wind load classification specified above and the proposed antenna loadings.**

Limitations/Assumptions:

This report is based on the following:

1. Flagpole inventory as listed in this report.
2. Flagpole is properly installed and maintained.
3. All members are as specified in the original design documents and are in good condition.
4. All required members are in place.
5. All bolts are in place and are properly tightened.
6. Flagpole is in plumb condition.
7. All member protective coatings are in good condition.
8. All flagpole members were properly designed, detailed, fabricated, and installed and have been properly maintained since erection.
9. Foundations were properly constructed to support original design loads as specified in the original design documents.
10. All coaxial cable is installed internally.
11. Stealth concealment cylinder support pipe material is A519-70 (obtained from Rich Whitefoot of Stealth Network Technologies 11/22/2008).

JWB is not responsible for any modifications completed prior to or hereafter in which JWB is not or was not directly involved. Modifications include but are not limited to:

- A. Adding antennas
- B. Removing/replacing antennas
- C. Adding coaxial cables

JWB hereby states that this document represents the entire report and that it assumes no liability for any factual changes that may occur after the date of this report. All representations, recommendations, and conclusions are based upon information contained and set forth herein. If you are aware of any information which conflicts with that which is contained herein, or you are aware of any defects arising from original design, material, fabrication, or erection deficiencies, you should disregard this report and immediately contact JWB. JWB disclaims all liability for any representation, recommendation, or conclusion not expressly stated herein.

Ongoing and Periodic Inspection and Maintenance:

After the Contractor has successfully completed the installation and the work has been accepted, the owner will be responsible for the ongoing and periodic inspection and maintenance of the flagpole.

The owner shall refer to TIA/EIA-222-F for recommendations for maintenance and inspection. The frequency of the inspection and maintenance intervals is to be determined by the owner based upon actual site and environmental conditions. It is recommended that a complete and thorough inspection of the entire tower structural system be performed at least yearly and more frequently as conditions warrant. According to TIA/EIA-222-F section 14.1, Note 1: It is recommended that the structure be inspected after severe wind and/or ice storms or other extreme loading conditions.



100' Flagpole
586 Danbury Road, New Milford CT
Verizon Site: Candlewood Alltel, CT

6. DRAWINGS AND DATA



100' Flagpole
586 Danbury Road, New Milford CT
Verizon Site: Candlewood Alltel, CT

RISA TOWER INPUT/OUTPUT SUMMARY

DESIGNED APPURTENANCE LOADING

TYPE	ELEVATION	TYPE	ELEVATION
Flag Pole Ball	99.5	18" Diameter x 6' Stealth Tube	89
18" Diameter x 7' Stealth Tube	95.5	18" Diameter x 6' Stealth Tube	83
4' Panel (T-Mobile)	95	CSS XDU06-65 (Verizon)	77
4' Panel (T-Mobile)	95	30" Diameter x 6' Stealth Tube	77
Flag Loading	93	CSS XDU06-65 (Verizon)	77

MATERIAL STRENGTH

GRADE	Fy	Fu	GRADE	Fy	Fu
A607-65	65 ksi	80 ksi			

TOWER DESIGN NOTES

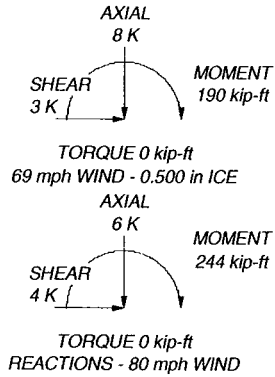
1. Tower designed for a 80 mph basic wind in accordance with the TIA/EIA-222-F Standard.
2. Tower is also designed for a 69 mph basic wind with 0.50 in ice.
3. Deflections are based upon a 50 mph wind.
4. 25,000 ft Stealth Canister Support is included for load transfer only.
5. TOWER RATING: 48.2%

Section	2	1
Length (ft)	48,000	28,500
Number of Sides	18	18
Thickness (in)	0.188	0.188
Lap Splice (ft)		3,000
Top Dia (in)	21,450	18,000
Bot Dia (in)	28,650	22,275
Grade	A607-65	
Weight (K)	2.4	1.2

74.0 ft

45.5 ft

0.5 ft



JWB Tower Services
148 Governor Street
New Britain, CT 06053
Phone: (800) 819-3084
FAX: (860) 256-8175

Job: 100' Flag Pole Structural Analysis			
Project: 586 Danbury Road, New Milford, CT			
Client: CSB	Drawn by: Staff	App'd:	
Code: TIA/EIA-222-F	Date: 01/06/09	Scale: NTS	
Path: C:\CSB\New Milford_CTR\Rev 2.01_06_2009\ERU Files\PJF 31500-067.dwg		Dwg No. E-1	

Section	1
Size	HSS5x.5
Length (ft)	25.000
Grade	A519-7
Weight (K)	0.6
	99.0 ft

DESIGNED APPURTENANCE LOADING

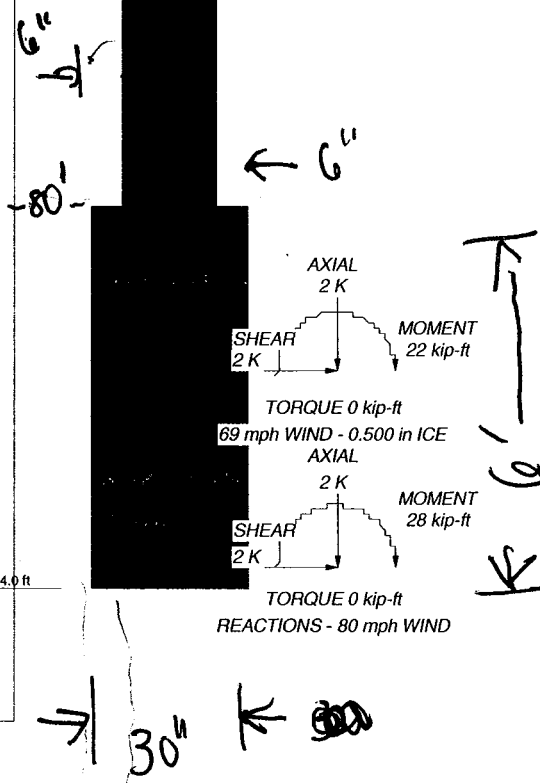
TYPE	ELEVATION	TYPE	ELEVATION
Flag Pole Ball	99.5	18" Diameter x 6' Stealth Tube	89
18" Diameter x 7' Stealth Tube	95.5	18" Diameter x 6' Stealth Tube	83
4' Panel (T-Mobile)	95	CSS XDUO6-65 (Verizon)	77
4' Panel (T-Mobile)	95	30" Diameter x 6' Stealth Tube	77
Flag Loading	93	CSS XDUO6-65 (Verizon)	77

MATERIAL STRENGTH

GRADE	Fy	Fu	GRADE	Fy	Fu
A519-70	70 ksi	85 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 50 mph wind.
4. TOWER RATING: 79%



JWB Tower Services		Job: 25' Flag Pole Mast Analysis	
148 Governor Street		Project: 586 Danbury Road, New Milford, CT	
New Britain, CT 06053		Client: CSB	Drawn by: Staff
Phone: (800) 819-3084		Code: TIA/EIA-222-F	Date: 01/06/09
FAX: (860) 256-8175		Path: C:\CSB\New Milford, CT\Rev 2.01.06.2009\ERI Files\PJF 31000-067 5in Mast.rvt	Scale: NTS
			Dwg No. E-1



100' Flagpole
586 Danbury Road, New Milford CT
Verizon Site: Candlewood Alltel, CT

RISA TOWER DETAILED OUTPUT

RISATower**JWB Tower Services**

148 Governor Street
New Britain, CT 06053
Phone: (800) 819-3084
FAX: (860) 256-8175

100' Flag Pole Structural Analysis

1 of 14

Project

586 Danbury Road, New Milford, CT

Date

21:19:12 01/06/09

Client

CSB

Designed by

Staff

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 0.500 in.

Ice density of 56 pcf.

A wind speed of 69 mph is used in combination with ice.

Deflections calculated using a wind speed of 50 mph.

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
Consider Moments - Horizontals
Consider Moments - Diagonals
Use Moment Magnification

√ Use Code Stress Ratios

√ Use Code Safety Factors - Guys

Escalate Ice

Always Use Max Kz

Use Special Wind Profile

Include Bolts In Member Capacity

Leg Bolts Are At Top Of Section

Secondary Horizontal Braces Leg

Use Diamond Inner Bracing (4 Sided)

Add IBC .6D+W Combination

Distribute Leg Loads As Uniform
Assume Legs Pinned

Assume Rigid Index Plate

Use Clear Spans For Wind Area

Use Clear Spans For KL/r

Retension Guys To Initial Tension

√ Bypass Mast Stability Checks

Use Azimuth Dish Coefficients

√ Project Wind Area of Appurt.

Autocalc Torque Arm Areas

SR Members Have Cut Ends

√ Sort Capacity Reports By Component

Triangulate Diamond Inner Bracing

Treat Feedline Bundles As Cylinder

Use ASCE 10 X-Brace Ly Rules

Calculate Redundant Bracing Forces

Ignore Redundant Members in FEA

SR Leg Bolts Resist Compression

All Leg Panels Have Same Allowable

Offset Girt At Foundation

√ Consider Feedline Torque

Include Angle Block Shear Check

Poles

Include Shear-Torsion Interaction

Always Use Sub-Critical Flow

Use Top Mounted Sockets

Tapered Pole Section Geometry

Section	Elevation ft	Section Length ft	Splice Length ft	Number of Sides	Top Diameter in	Bottom Diameter in	Wall Thickness in	Bend Radius in	Pole Grade
L1	74.000-45.500	28.500	3.000	18	18.000	22.275	0.188	0.750	A607-65 (65 ksi)
L2	45.500-0.500	48.000		18	21.450	28.650	0.188	0.750	A607-65 (65 ksi)

Tapered Pole Properties

Section	Tip Dia. in	Area in ²	I in ⁴	r in	C in	I/C in ³	J in ⁴	Iu/Q in ²	w in	w/t
L1	18.278	10.601	424.933	6.323	9.144	46.471	850.425	5.301	2.838	15.136

RISA Tower**JWB Tower Services**

148 Governor Street

New Britain, CT 06053

Phone: (800) 819-3084

FAX: (860) 256-8175

100' Flag Pole Structural Analysis

2 of 14

Project

586 Danbury Road, New Milford, CT

Date

21:19:12 01/06/09

Client

CSB

Designed by

Staff

Section	Tip Dia. in	Area in ²	I in ⁴	r in	C in	I/C in ³	J in ⁴	It/Q in ²	w in	w/t
L2	22.619	13.145	810.187	7.841	11.316	71.598	1621.440	6.574	3.590	19.149
	22.238	12.654	722.751	7.548	10.897	66.328	1446.453	6.328	3.445	18.374
	29.092	16.939	1733.663	10.104	14.554	119.118	3469.608	8.471	4.712	25.133

Tower Elevation	Gusset Area (per face)	Gusset Thickness	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
ft	ft ²	in						
L1 74.000- 45.500				1	1	1		
L2 45.500- 0.500				1	1	1		

Feed Line/Linear Appurtenances - Entered As Area

Description	Face or Leg	Allow Shield	Component Type	Placement ft	Total Number		C _A A _A ft ² /ft	Weight klf
7/8 (T-Mobile)	C	No	Inside Pole	74.000 - 0.500	4	No Ice	0.000	0.001
1 5/8 (Verizon)	C	No	Inside Pole	74.000 - 0.500	8	1/2" Ice	0.000	0.001
						No Ice	0.000	0.001
						1/2" Ice	0.000	0.001

Feed Line/Linear Appurtenances Section Areas

Tower Section	Tower Elevation ft	Face	A _R ft ²	A _F ft ²	C _A A _A In Face ft ²	C _A A _A Out Face ft ²	Weight K
L1	74.000-45.500	A	0.000	0.000	0.000	0.000	0.000
		B	0.000	0.000	0.000	0.000	0.000
		C	0.000	0.000	0.000	0.000	0.299
L2	45.500-0.500	A	0.000	0.000	0.000	0.000	0.000
		B	0.000	0.000	0.000	0.000	0.000
		C	0.000	0.000	0.000	0.000	0.472

Feed Line/Linear Appurtenances Section Areas - With Ice

Tower Section	Tower Elevation ft	Face or Leg	Ice Thickness in	A _R ft ²	A _F ft ²	C _A A _A In Face ft ²	C _A A _A Out Face ft ²	Weight K
L1	74.000-45.500	A	0.500	0.000	0.000	0.000	0.000	0.000
		B		0.000	0.000	0.000	0.000	0.000
		C		0.000	0.000	0.000	0.000	0.299
L2	45.500-0.500	A	0.500	0.000	0.000	0.000	0.000	0.000
		B		0.000	0.000	0.000	0.000	0.000
		C		0.000	0.000	0.000	0.000	0.472

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Feed Line Center of Pressure

Section	Elevation	CP _X	CP _Z	CP _X Ice	CP _Z Ice
	ft	in	in	in	in
L1	74.000-45.500	0.000	0.000	0.000	0.000
L2	45.500-0.500	0.000	0.000	0.000	0.000

Antenna Pole Forces Stealth Canister Support

Length of Pole	I _x	I _y	Modulus E	Antenna Pole C _A A _A	Antenna Pole Weight	Length of Beacon	Beacon C _A A _A	Beacon Weight
ft	in ⁴	in ⁴	ksi	ft ² /ft	klf	ft	ft ²	K
25.000	17.210	17.210	29000.000	No Ice With Ice	0.000 0.000	0.024 0.024	0.000 0.000	0.000 0.000

User Defined Loads

Description	Elevation	Offset From Centroid	Azimuth Angle	Weight	F _x	F _z	Wind Force	C _A A _C
	ft	ft	°	K	K	K	K	ft ²
Flag Loading	93.000	0.000	0.000	No Ice Ice Service	0.100 0.200 0.000	0.350 0.232 0.000	0.000 0.000 0.000	0.000 0.000 0.000

Discrete Tower Loads

Description	Face or Leg	Offset Type	Offsets: Horz Lateral Vert	Azimuth Adjustment	Placement	C _A A _A Front	C _A A _A Side	Weight
			ft ft ft	°	ft	ft ²	ft ²	K
18" Diameter x 7' Stealth Tube	C	None		0.000	95.500	No Ice 1/2" Ice	8.400 8.972	0.125 0.213
18" Diameter x 6' Stealth Tube	C	None		0.000	89.000	No Ice 1/2" Ice	7.200 7.706	0.125 0.202
18" Diameter x 6' Stealth Tube	C	None		0.000	83.000	No Ice 1/2" Ice	7.200 7.706	0.125 0.202
30" Diameter x 6' Stealth Tube	C	None		0.000	77.000	No Ice 1/2" Ice	12.000 12.572	0.500 0.636
4' Panel (T-Mobile)	A	From Leg	0.500 0.000 0.000	0.000	95.000	No Ice 1/2" Ice	0.000 0.000	0.035 0.035
4' Panel (T-Mobile)	C	From Leg	0.500 0.000 0.000	0.000	95.000	No Ice 1/2" Ice	0.000 0.000	0.035 0.035

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Description	Face or Leg	Offset Type	Offsets: Horz Lateral Vert ft ft ft	Azimuth Adjustment °	Placement ft		C _A A _A Front ft ²	C _A A _A Side ft ²	Weight K
CSS XDU06-65 (Verizon)	A	From Leg	0.500 0.000 0.000	0.000	77.000	No Ice 1/2" Ice	0.000 0.000	0.000 0.000	0.035 0.035
CSS XDU06-65 (Verizon)	C	From Leg	0.500 0.000 0.000	0.000	77.000	No Ice 1/2" Ice	0.000 0.000	0.000 0.000	0.035 0.035

Dishes

Description	Face or Leg	Dish Type	Offset Type	Offsets: Horz Lateral Vert ft	Azimuth Adjustment °	3 dB Beam Width °	Elevation ft	Outside Diameter ft	Aperture Area ft ²	Weight K
Flag Pole Ball		Paraboloid w/Radome	None		Worst		99.500	1.000	No Ice 1/2" Ice	0.050 0.065

Tower Pressures - No Ice

$$G_H = 1.690$$

Section Elevation	z	K _Z	q _z	A _G	F a c e	A _F	A _R	A _{leg}	Leg %	C _A A _A In Face ft ²	C _A A _A Out Face ft ²
ft	ft		ksf	ft ²		ft ²	ft ²	ft ²			
L1 74.000- 45.500	59.246	1.182	0.019	47.827	A	0.000	47.827	47.827	100.00	0.000	0.000
					B	0.000	47.827		100.00	0.000	0.000
					C	0.000	47.827		100.00	0.000	0.000
L2 45.500- 0.500	22.045	1	0.016	94.781	A	0.000	94.781	94.781	100.00	0.000	0.000
					B	0.000	94.781		100.00	0.000	0.000
					C	0.000	94.781		100.00	0.000	0.000

Tower Pressure - With Ice

$$G_H = 1.690$$

Section Elevation	z	K _Z	q _z	t _z	A _G	F a c e	A _F	A _R	A _{leg}	Leg %	C _A A _A In Face ft ²	C _A A _A Out Face ft ²
ft	ft		ksf	in	ft ²		ft ²	ft ²	ft ²			
L1 74.000- 45.500	59.246	1.182	0.015	0.500	50.202	A	0.000	50.202	50.202	100.00	0.000	0.000
					B	0.000	50.202		100.00	0.000	0.000	0.000
					C	0.000	50.202		100.00	0.000	0.000	0.000
L2 45.500-0.500	22.045	1	0.012	0.500	98.531	A	0.000	98.531	98.531	100.00	0.000	0.000

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Section Elevation	z	K _Z	q _z	t _z	A _G	F a c e	A _F	A _R	A _{leg}	Leg %	C _A A _A In Face ft ²	C _A A _A Out Face ft ²
ft	ft		ksf	in	ft ²		ft ²	ft ²	ft ²			
						B	0.000	98.531		100.00	0.000	0.000
						C	0.000	98.531		100.00	0.000	0.000

Tower Pressure - Service

$$G_H = 1.690$$

Section Elevation	z	K _Z	q _z	A _G	F a c e	A _F	A _R	A _{leg}	Leg %	C _A A _A In Face ft ²	C _A A _A Out Face ft ²
ft	ft		ksf	ft ²		ft ²	ft ²	ft ²			
L1 74.000- 45.500	59.246	1.182	0.008	47.827	A	0.000	47.827	47.827	100.00	0.000	0.000
					B	0.000	47.827		100.00	0.000	0.000
					C	0.000	47.827		100.00	0.000	0.000
L2 45.500- 0.500	22.045	1	0.006	94.781	A	0.000	94.781	94.781	100.00	0.000	0.000
					B	0.000	94.781		100.00	0.000	0.000
					C	0.000	94.781		100.00	0.000	0.000

Tower Forces - No Ice - Wind Normal To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000- 45.500	0.299	1.151	A	1	0.65	1	1	1	47.827	1.017	0.036	C
			B	1	0.65	1	1	1	47.827			
			C	1	0.65	1	1	1	47.827			
L2 45.500- 0.500	0.472	2.417	A	1	0.65	1	1	1	94.781	1.713	0.038	C
			B	1	0.65	1	1	1	94.781			
			C	1	0.65	1	1	1	94.781			
Sum Weight:	0.770	3.568						OTM	96.667 kip-ft	2.730		

Tower Forces - No Ice - Wind 60 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000- 45.500	0.299	1.151	A	1	0.65	1	1	1	47.827	1.017	0.036	C
			B	1	0.65	1	1	1	47.827			
			C	1	0.65	1	1	1	47.827			
L2 45.500- 0.500	0.472	2.417	A	1	0.65	1	1	1	94.781	1.713	0.038	C
			B	1	0.65	1	1	1	94.781			
			C	1	0.65	1	1	1	94.781			
Sum Weight:	0.770	3.568						OTM	96.667 kip-ft	2.730		

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Tower Forces - No Ice - Wind 90 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000- 45.500	0.299	1.151	A	1	0.65	1	1	1	47.827	1.017	0.036	C
			B	1	0.65	1	1	1	47.827			
			C	1	0.65	1	1	1	47.827			
L2 45.500- 0.500	0.472	2.417	A	1	0.65	1	1	1	94.781	1.713	0.038	C
			B	1	0.65	1	1	1	94.781			
			C	1	0.65	1	1	1	94.781			
Sum Weight:	0.770	3.568						OTM	96.667 kip-ft	2.730		

Tower Forces - With Ice - Wind Normal To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000- 45.500	0.299	1.514	A	1	0.65	1	1	1	50.202	0.801	0.028	C
			B	1	0.65	1	1	1	50.202			
			C	1	0.65	1	1	1	50.202			
L2 45.500- 0.500	0.472	3.133	A	1	0.65	1	1	1	98.531	1.335	0.030	C
			B	1	0.65	1	1	1	98.531			
			C	1	0.65	1	1	1	98.531			
Sum Weight:	0.770	4.647						OTM	75.821 kip-ft	2.136		

Tower Forces - With Ice - Wind 60 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000- 45.500	0.299	1.514	A	1	0.65	1	1	1	50.202	0.801	0.028	C
			B	1	0.65	1	1	1	50.202			
			C	1	0.65	1	1	1	50.202			
L2 45.500- 0.500	0.472	3.133	A	1	0.65	1	1	1	98.531	1.335	0.030	C
			B	1	0.65	1	1	1	98.531			
			C	1	0.65	1	1	1	98.531			
Sum Weight:	0.770	4.647						OTM	75.821 kip-ft	2.136		

Tower Forces - With Ice - Wind 90 To Face

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Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000-45.500	0.299	1.514	A	1	0.65	1	1	1	50.202	0.801	0.028	C
			B	1	0.65	1	1	1	50.202			
			C	1	0.65	1	1	1	50.202			
L2 45.500-0.500	0.472	3.133	A	1	0.65	1	1	1	98.531	1.335	0.030	C
			B	1	0.65	1	1	1	98.531			
			C	1	0.65	1	1	1	98.531			
Sum Weight:	0.770	4.647						OTM	75.821 kip-ft	2.136		

Tower Forces - Service - Wind Normal To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000-45.500	0.299	1.151	A	1	0.65	1	1	1	47.827	0.397	0.014	C
			B	1	0.65	1	1	1	47.827			
			C	1	0.65	1	1	1	47.827			
L2 45.500-0.500	0.472	2.417	A	1	0.65	1	1	1	94.781	0.669	0.015	C
			B	1	0.65	1	1	1	94.781			
			C	1	0.65	1	1	1	94.781			
Sum Weight:	0.770	3.568						OTM	37.761 kip-ft	1.066		

Tower Forces - Service - Wind 60 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000-45.500	0.299	1.151	A	1	0.65	1	1	1	47.827	0.397	0.014	C
			B	1	0.65	1	1	1	47.827			
			C	1	0.65	1	1	1	47.827			
L2 45.500-0.500	0.472	2.417	A	1	0.65	1	1	1	94.781	0.669	0.015	C
			B	1	0.65	1	1	1	94.781			
			C	1	0.65	1	1	1	94.781			
Sum Weight:	0.770	3.568						OTM	37.761 kip-ft	1.066		

Tower Forces - Service - Wind 90 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 74.000-45.500	0.299	1.151	A	1	0.65	1	1	1	47.827	0.397	0.014	C
			B	1	0.65	1	1	1	47.827			

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Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L2 45.500- 0.500	0.472	2.417	C A B C	1 1 1 1	0.65 0.65 0.65 0.65	1 1 1 1	1 1 1 1	1 1 1 1	47.827 94.781 94.781 94.781	0.669	0.015	C
Sum Weight:	0.770	3.568						OTM	37.761 kip-ft	1.066		

Force Totals

Load Case	Vertical Forces	Sum of Forces X	Sum of Forces Z	Sum of Overturning Moments, M _x	Sum of Overturning Moments, M _z	Sum of Torques
	K	K	K	kip-ft	kip-ft	kip-ft
Leg Weight	3.568					
Bracing Weight	0.000					
Total Member Self-Weight	3.568			-0.044	0.076	
Total Weight	6.106			-0.044	0.076	
Wind 0 deg - No Ice		0.350	-4.018	-206.407	-32.299	0.000
Wind 30 deg - No Ice		2.359	-3.480	-178.759	-135.481	0.000
Wind 60 deg - No Ice		3.830	-2.009	-103.225	-211.015	0.000
Wind 90 deg - No Ice		4.368	0.000	-0.044	-238.662	0.000
Wind 120 deg - No Ice		3.830	2.009	103.138	-211.015	0.000
Wind 150 deg - No Ice		2.359	3.480	178.672	-135.481	0.000
Wind 180 deg - No Ice		0.350	4.018	206.319	-32.299	0.000
Wind 210 deg - No Ice		-1.659	3.480	178.672	70.882	0.000
Wind 240 deg - No Ice		-3.130	2.009	103.138	146.416	0.000
Wind 270 deg - No Ice		-3.668	0.000	-0.044	174.064	0.000
Wind 300 deg - No Ice		-3.130	-2.009	-103.225	146.416	0.000
Wind 330 deg - No Ice		-1.659	-3.480	-178.759	70.882	0.000
Member Ice	1.079					
Total Weight Ice	7.678			-0.044	0.076	
Wind 0 deg - Ice		0.232	-3.165	-163.516	-21.384	0.000
Wind 30 deg - Ice		1.814	-2.741	-141.615	-103.120	0.000
Wind 60 deg - Ice		2.973	-1.582	-81.780	-162.955	0.000
Wind 90 deg - Ice		3.397	0.000	-0.044	-184.856	0.000
Wind 120 deg - Ice		2.973	1.582	81.692	-162.955	0.000
Wind 150 deg - Ice		1.814	2.741	141.527	-103.120	0.000
Wind 180 deg - Ice		0.232	3.165	163.428	-21.384	0.000
Wind 210 deg - Ice		-1.350	2.741	141.527	60.352	0.000
Wind 240 deg - Ice		-2.509	1.582	81.692	120.187	0.000
Wind 270 deg - Ice		-2.933	0.000	-0.044	142.088	0.000
Wind 300 deg - Ice		-2.509	-1.582	-81.780	120.187	0.000
Wind 330 deg - Ice		-1.350	-2.741	-141.615	60.352	0.000
Total Weight	6.006			-0.044	0.076	
Wind 0 deg - Service		0.000	-1.570	-80.654	0.076	0.000
Wind 30 deg - Service		0.785	-1.359	-69.854	-40.229	0.000
Wind 60 deg - Service		1.359	-0.785	-40.349	-69.735	0.000
Wind 90 deg - Service		1.570	0.000	-0.044	-80.535	0.000
Wind 120 deg - Service		1.359	0.785	40.261	-69.735	0.000
Wind 150 deg - Service		0.785	1.359	69.767	-40.229	0.000
Wind 180 deg - Service		0.000	1.570	80.567	0.076	0.000
Wind 210 deg - Service		-0.785	1.359	69.767	40.381	0.000
Wind 240 deg - Service		-1.359	0.785	40.261	69.886	0.000
Wind 270 deg - Service		-1.570	0.000	-0.044	80.686	0.000
Wind 300 deg - Service		-1.359	-0.785	-40.349	69.886	0.000

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Load Case	Vertical Forces K	Sum of Forces X K	Sum of Forces Z K	Sum of Overturning Moments, M_x kip-ft	Sum of Overturning Moments, M_z kip-ft	Sum of Torques kip-ft
Wind 330 deg - Service		-0.785	-1.359	-69.854	40.381	0.000

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
15	Dead+Wind 0 deg+Ice
16	Dead+Wind 30 deg+Ice
17	Dead+Wind 60 deg+Ice
18	Dead+Wind 90 deg+Ice
19	Dead+Wind 120 deg+Ice
20	Dead+Wind 150 deg+Ice
21	Dead+Wind 180 deg+Ice
22	Dead+Wind 210 deg+Ice
23	Dead+Wind 240 deg+Ice
24	Dead+Wind 270 deg+Ice
25	Dead+Wind 300 deg+Ice
26	Dead+Wind 330 deg+Ice
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 Comb.	Force K	Major Axis Moment kip-ft	Minor Axis Moment kip-ft
LI	74 - 45.5	Pole	Max Tension	1	0.000	0.000	0.000
			Max. Compression	14	-3.868	0.076	0.044
			Max. M_x	5	-2.976	-76.856	0.045

KISA TOWER JWB Tower Services 148 Governor Street New Britain, CT 06053 Phone: (800) 819-3084 FAX: (860) 256-8175	100' Flag Pole Structural Analysis		10 of 14
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Section No.	Elevation ft	Component Type	Condition	Gov. Load Comb.	Force K	Major Axis Moment kip-ft	Minor Axis Moment kip-ft
L2	45.5 - 0.5	Pole Antenna	Max. My	2	-2.996	-15.918	60.993
			Max. Vy	5	2.629	-76.856	0.045
			Max. Vx	2	-2.264	-15.918	60.993
			Max. Torque	8			0.104
			Max Tension	18	0.000	0.000	0.000
			Max. Compression	14	-0.603	0.000	0.000
			Max. Mx	5	-0.602	-0.302	0.000
			Max. My	2	-0.602	-0.055	0.247
			Max. Vy	5	0.024	-0.302	0.000
			Max. Vx	2	-0.020	-0.055	0.247
		Pole	Max. Torque	8			0.000
			Max Tension	1	0.000	0.000	0.000
			Max. Compression	14	-7.678	0.076	0.044
			Max. Mx	5	-6.102	-244.162	0.046
			Max. My	2	-6.103	-33.195	211.029
			Max. Vy	5	4.373	-244.162	0.046
			Max. Vx	2	-4.023	-33.195	211.029
			Max. Torque	8			0.104

Maximum Reactions

Location	Condition	Gov. Load Comb.	Vertical K	Horizontal, X K	Horizontal, Z K
Pole	Max. Vert	18	7.678	-3.397	0.000
	Max. H _x	11	6.106	3.668	0.000
	Max. H _z	2	6.106	-0.350	4.018
	Max. M _x	2	211.029	-0.350	4.018
	Max. M _z	5	244.162	-4.368	0.000
	Max. Torsion	8	0.104	-0.350	-4.018
	Min. Vert	1	6.106	0.000	0.000
	Min. H _x	5	6.106	-4.368	0.000
	Min. H _z	8	6.106	-0.350	-4.018
	Min. M _x	8	-210.938	-0.350	-4.018
	Min. M _z	11	-177.799	3.668	0.000
	Min. Torsion	2	-0.104	-0.350	4.018

Tower Mast Reaction Summary

Load Combination	Vertical K	Shear _x K	Shear _z K	Overturning Moment, M _x kip-ft	Overturning Moment, M _z kip-ft	Torque kip-ft
Dead Only	6.106	0.000	0.000	-0.044	0.076	0.000
Dead+Wind 0 deg - No Ice	6.106	0.350	-4.018	-211.029	-33.195	0.104
Dead+Wind 30 deg - No Ice	6.106	2.359	-3.480	-182.756	-138.682	0.089
Dead+Wind 60 deg - No Ice	6.106	3.830	-2.009	-105.531	-215.900	0.051
Dead+Wind 90 deg - No Ice	6.106	4.368	0.000	-0.046	-244.162	-0.001
Dead+Wind 120 deg - No Ice	6.106	3.830	2.009	105.440	-215.900	-0.053
Dead+Wind 150 deg - No Ice	6.106	2.359	3.480	182.665	-138.682	-0.091
Dead+Wind 180 deg - No Ice	6.106	0.350	4.018	210.938	-33.195	-0.104
Dead+Wind 210 deg - No Ice	6.106	-1.659	3.480	182.677	72.299	-0.089
Dead+Wind 240 deg - No Ice	6.106	-3.130	2.009	105.452	149.530	-0.051
Dead+Wind 270 deg - No Ice	6.106	-3.668	0.000	-0.046	177.799	0.001

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Load Combination	Vertical K	Shear _x K	Shear _y K	Overturning Moment, M _x kip-ft	Overturning Moment, M _y kip-ft	Torque kip-ft
Dead+Wind 300 deg - No Ice	6.106	-3.130	-2.009	-105.543	149.530	0.053
Dead+Wind 330 deg - No Ice	6.106	-1.659	-3.480	-182.768	72.299	0.090
Dead+Ice	7.678	0.000	0.000	-0.044	0.076	0.000
Dead+Wind 0 deg+Ice	7.678	0.232	-3.165	-168.150	-22.131	0.054
Dead+Wind 30 deg+Ice	7.678	1.814	-2.741	-145.626	-106.181	0.046
Dead+Wind 60 deg+Ice	7.678	2.973	-1.582	-84.095	-167.708	0.026
Dead+Wind 90 deg+Ice	7.678	3.397	0.000	-0.046	-190.228	-0.001
Dead+Wind 120 deg+Ice	7.678	2.973	1.582	84.003	-167.708	-0.028
Dead+Wind 150 deg+Ice	7.678	1.814	2.741	145.534	-106.181	-0.047
Dead+Wind 180 deg+Ice	7.678	0.232	3.165	168.058	-22.131	-0.054
Dead+Wind 210 deg+Ice	7.678	-1.350	2.741	145.539	61.922	-0.046
Dead+Wind 240 deg+Ice	7.678	-2.509	1.582	84.009	123.455	-0.026
Dead+Wind 270 deg+Ice	7.678	-2.933	0.000	-0.046	145.978	0.001
Dead+Wind 300 deg+Ice	7.678	-2.509	-1.582	-84.100	123.455	0.028
Dead+Wind 330 deg+Ice	7.678	-1.350	-2.741	-145.631	61.922	0.047
Dead+Wind 0 deg - Service	6.106	0.000	-1.570	-82.473	0.079	-0.001
Dead+Wind 30 deg - Service	6.106	0.785	-1.359	-71.430	-41.135	-0.001
Dead+Wind 60 deg - Service	6.106	1.359	-0.785	-41.259	-71.305	-0.001
Dead+Wind 90 deg - Service	6.106	1.570	0.000	-0.046	-82.348	-0.000
Dead+Wind 120 deg - Service	6.106	1.359	0.785	41.168	-71.305	0.000
Dead+Wind 150 deg - Service	6.106	0.785	1.359	71.339	-41.135	0.000
Dead+Wind 180 deg - Service	6.106	0.000	1.570	82.382	0.079	0.001
Dead+Wind 210 deg - Service	6.106	-0.785	1.359	71.339	41.292	0.001
Dead+Wind 240 deg - Service	6.106	-1.359	0.785	41.168	71.463	0.001
Dead+Wind 270 deg - Service	6.106	-1.570	0.000	-0.046	82.506	0.000
Dead+Wind 300 deg - Service	6.106	-1.359	-0.785	-41.259	71.463	0.000
Dead+Wind 330 deg - Service	6.106	-0.785	-1.359	-71.430	41.292	-0.000

Solution Summary

Load Comb.	Sum of Applied Forces			Sum of Reactions			% Error
	PX K	PY K	PZ K	PX K	PY K	PZ K	
1	0.000	-6.106	0.000	0.000	6.106	0.000	0.000%
2	0.350	-6.106	-4.018	-0.350	6.106	4.018	0.000%
3	2.359	-6.106	-3.480	-2.359	6.106	3.480	0.000%
4	3.830	-6.106	-2.009	-3.830	6.106	2.009	0.000%
5	4.368	-6.106	0.000	-4.368	6.106	0.000	0.000%
6	3.830	-6.106	2.009	-3.830	6.106	-2.009	0.000%
7	2.359	-6.106	3.480	-2.359	6.106	-3.480	0.000%
8	0.350	-6.106	4.018	-0.350	6.106	-4.018	0.000%
9	-1.659	-6.106	3.480	1.659	6.106	-3.480	0.000%
10	-3.130	-6.106	2.009	3.130	6.106	-2.009	0.000%
11	-3.668	-6.106	0.000	3.668	6.106	0.000	0.000%
12	-3.130	-6.106	-2.009	3.130	6.106	2.009	0.000%
13	-1.659	-6.106	-3.480	1.659	6.106	3.480	0.000%
14	0.000	-7.678	0.000	0.000	7.678	0.000	0.000%
15	0.232	-7.678	-3.165	-0.232	7.678	3.165	0.000%
16	1.814	-7.678	-2.741	-1.814	7.678	2.741	0.000%
17	2.973	-7.678	-1.582	-2.973	7.678	1.582	0.000%
18	3.397	-7.678	0.000	-3.397	7.678	0.000	0.000%
19	2.973	-7.678	1.582	-2.973	7.678	-1.582	0.000%
20	1.814	-7.678	2.741	-1.814	7.678	-2.741	0.000%
21	0.232	-7.678	3.165	-0.232	7.678	-3.165	0.000%
22	-1.350	-7.678	2.741	1.350	7.678	-2.741	0.000%
23	-2.509	-7.678	1.582	2.509	7.678	-1.582	0.000%
24	-2.933	-7.678	0.000	2.933	7.678	0.000	0.000%
25	-2.509	-7.678	-1.582	2.509	7.678	1.582	0.000%

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Load Comb.	Sum of Applied Forces			Sum of Reactions			% Error
	PX K	PY K	PZ K	PX K	PY K	PZ K	
26	-1.350	-7.678	-2.741	1.350	7.678	2.741	0.000%
27	0.000	-6.106	-1.570	0.000	6.106	1.570	0.000%
28	0.785	-6.106	-1.359	-0.785	6.106	1.359	0.000%
29	1.359	-6.106	-0.785	-1.359	6.106	0.785	0.000%
30	1.570	-6.106	0.000	-1.570	6.106	0.000	0.000%
31	1.359	-6.106	0.785	-1.359	6.106	-0.785	0.000%
32	0.785	-6.106	1.359	-0.785	6.106	-1.359	0.000%
33	0.000	-6.106	1.570	0.000	6.106	-1.570	0.000%
34	-0.785	-6.106	1.359	0.785	6.106	-1.359	0.000%
35	-1.359	-6.106	0.785	1.359	6.106	-0.785	0.000%
36	-1.570	-6.106	0.000	1.570	6.106	0.000	0.000%
37	-1.359	-6.106	-0.785	1.359	6.106	0.785	0.000%
38	-0.785	-6.106	-1.359	0.785	6.106	1.359	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	0.00058842
3	Yes	5	0.00000001	0.00004580
4	Yes	5	0.00000001	0.00004109
5	Yes	4	0.00000001	0.00023527
6	Yes	5	0.00000001	0.00004096
7	Yes	5	0.00000001	0.00004573
8	Yes	4	0.00000001	0.00058774
9	Yes	4	0.00000001	0.00053770
10	Yes	4	0.00000001	0.00068452
11	Yes	4	0.00000001	0.00011197
12	Yes	4	0.00000001	0.00068646
13	Yes	4	0.00000001	0.00053837
14	Yes	4	0.00000001	0.00000001
15	Yes	4	0.00000001	0.00036884
16	Yes	5	0.00000001	0.00003759
17	Yes	4	0.00000001	0.00097161
18	Yes	4	0.00000001	0.00023527
19	Yes	4	0.00000001	0.00096794
20	Yes	5	0.00000001	0.00003752
21	Yes	4	0.00000001	0.00036827
22	Yes	4	0.00000001	0.00044038
23	Yes	4	0.00000001	0.00051615
24	Yes	4	0.00000001	0.00012325
25	Yes	4	0.00000001	0.00051795
26	Yes	4	0.00000001	0.00044109
27	Yes	4	0.00000001	0.00003113
28	Yes	4	0.00000001	0.00008163
29	Yes	4	0.00000001	0.00008200
30	Yes	4	0.00000001	0.00003093
31	Yes	4	0.00000001	0.00008145
32	Yes	4	0.00000001	0.00008135
33	Yes	4	0.00000001	0.00003099
34	Yes	4	0.00000001	0.00008231
35	Yes	4	0.00000001	0.00008194
36	Yes	4	0.00000001	0.00003118
37	Yes	4	0.00000001	0.00008250
38	Yes	4	0.00000001	0.00008260

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Maximum Tower Deflections - Service Wind

Section No.	Elevation ft	Horz. Deflection in	Gov. Load Comb.	Tilt °	Twist °
Pole	99 - 74	10.374	37	0.742	0.000
Antenna					
L1	74 - 45.5	6.510	37	0.727	0.000
L2	48.5 - 0.5	3.028	37	0.556	0.000

Critical Deflections and Radius of Curvature - Service Wind

Elevation ft	Appurtenance	Gov. Load Comb.	Deflection in	Tilt °	Twist °	Radius of Curvature ft
99.500	Flag Pole Ball	37	10.374	0.742	0.000	182916
95.500	18" Diameter x 7' Stealth Tube	37	9.828	0.747	0.000	182916
95.000	4' Panel	37	9.750	0.747	0.000	182916
93.000	Flag Loading	37	9.439	0.749	0.000	152430
89.000	18" Diameter x 6' Stealth Tube	37	8.817	0.752	0.000	91458
83.000	18" Diameter x 6' Stealth Tube	37	7.888	0.750	0.000	57161
77.000	30" Diameter x 6' Stealth Tube	37	6.967	0.738	0.000	40896

Maximum Tower Deflections - Design Wind

Section No.	Elevation ft	Horz. Deflection in	Gov. Load Comb.	Tilt °	Twist °
Pole	99 - 74	31.979	5	2.319	0.001
Antenna					
L1	74 - 45.5	19.906	5	2.272	0.001
L2	48.5 - 0.5	9.145	5	1.699	0.001

Critical Deflections and Radius of Curvature - Design Wind

Elevation ft	Appurtenance	Gov. Load Comb.	Deflection in	Tilt °	Twist °	Radius of Curvature ft
99.500	Flag Pole Ball	5	31.979	2.319	0.001	46624
95.500	18" Diameter x 7' Stealth Tube	5	30.270	2.331	0.001	46624
95.000	4' Panel	5	30.026	2.332	0.001	46624
93.000	Flag Loading	5	29.051	2.338	0.001	38853
89.000	18" Diameter x 6' Stealth Tube	5	27.105	2.345	0.001	23311
83.000	18" Diameter x 6' Stealth Tube	5	24.202	2.339	0.001	14569
77.000	30" Diameter x 6' Stealth Tube	5	21.328	2.304	0.001	10467

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Staff**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 K	Allow. P _a K	Ratio P P _a
L1	74 - 45.5 (2)	TP22.275x18x0.188	28.500	0.000	0.0	39.000	12.877	-2.976	502.204	0.006
L2	45.5 - 0.5 (3)	TP28.65x21.45x0.188	48.000	0.000	0.0	38.807	16.939	-6.102	657.338	0.009

Pole Bending Design Data

Section No.	Elevation ft	Size	Actual M _x kip-ft	Actual f _{bx} ksi	Allow. F _{bx} ksi	Ratio f _{bx} F _{bx}	Actual M _y kip-ft	Actual f _{by} ksi	Allow. F _{by} ksi	Ratio f _{by} F _{by}
L1	74 - 45.5 (2)	TP22.275x18x0.188	76.856	-13.425	39.000	0.344	0.000	0.000	39.000	0.000
L2	45.5 - 0.5 (3)	TP28.65x21.45x0.188	244.162	-24.597	38.807	0.634	0.000	0.000	38.807	0.000

Pole Interaction Design Data

Section No.	Elevation ft	Size	Ratio P P _a	Ratio f _{bx} F _{bx}	Ratio f _{by} F _{by}	Comb. Stress Ratio	Allow. Stress Ratio	Criteria
L1	74 - 45.5 (2)	TP22.275x18x0.188	0.006	0.344	0.000	0.350 ✓	1.333	H1-3 ✓
L2	45.5 - 0.5 (3)	TP28.65x21.45x0.188	0.009	0.634	0.000	0.643 ✓	1.333	H1-3 ✓

Section Capacity Table

Section No.	Elevation ft	Component Type	Size	Critical Element	P K	SF*P _{allow} K	% Capacity	Pass Fail
L1	74 - 45.5	Pole	TP22.275x18x0.188	2	-2.976	669.438	26.3	Pass
L2	45.5 - 0.5	Pole	TP28.65x21.45x0.188	3	-6.102	876.232	48.2	Pass
							Summary	
							Pole (L2)	Pass
							RATING = 48.2	Pass

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Job

25' Flag Pole Mast Analysis

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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 0.500 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 50 mph.

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		

Pole Section Geometry

Section	Elevation	Section Length	Pole Size	Pole Grade	Socket Length
	ft	ft			ft
L1	99.000-74.000	25.000	HSS5x.5	A519-70 (70 ksi)	

Tower Elevation	Gusset Area (per face)	Gusset Thickness	Gusset Grade	Adjust. Factor A_f	Adjust. Factor A_r	Weight Mult.	Double Angle Stitch Bolt Spacing Diagonals	Double Angle Stitch Bolt Spacing Horizontals
ft	ft ²	in					in	in
L1 99.000- 74.000				1	1	1		

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25' Flag Pole Mast Analysis

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Feed Line/Linear Appurtenances Section Areas

Tower Section	Tower Elevation ft	Face	A_R ft^2	A_F ft^2	C_{AA} In Face ft^2	C_{AA} Out Face ft^2	Weight K
L1	99,000-74,000	A	0.000	0.000	0.000	0.000	0.000
		B	0.000	0.000	0.000	0.000	0.000
		C	0.000	0.000	0.000	0.000	0.000

Feed Line/Linear Appurtenances Section Areas - With Ice

Tower Section	Tower Elevation ft	Face or Leg	Ice Thickness in	A_R ft^2	A_F ft^2	C_{AA} In Face ft^2	C_{AA} Out Face ft^2	Weight K
L1	99,000-74,000	A	0.500	0.000	0.000	0.000	0.000	0.000
		B		0.000	0.000	0.000	0.000	0.000
		C		0.000	0.000	0.000	0.000	0.000

User Defined Loads

Description	Elevation ft	Offset From Centroid ft	Azimuth Angle °	Weight K	F_x K	F_z K	Wind Force K	C_{AA} ft^2
Flag Loading	93,000	0.000	0.000	No Ice	0.100	0.350	0.000	0.000
				Ice	0.200	0.232	0.000	0.000
				Service	0.000	0.000	0.000	0.000

Discrete Tower Loads

Description	Face or Leg	Offset Type	Offsets: Horz Lateral Vert ft ft ft	Azimuth Adjustment °	Placement ft	C_{AA} Front ft^2	C_{AA} Side ft^2	Weight K
18" Diameter x 7' Stealth Tube	C	None		0.000	95.500	No Ice 8.400 1/2" Ice 8.972	8.400 8.972	0.125 0.213
18" Diameter x 6' Stealth Tube	C	None		0.000	89.000	No Ice 7.200 1/2" Ice 7.706	7.200 7.706	0.125 0.202
18" Diameter x 6' Stealth Tube	C	None		0.000	83.000	No Ice 7.200 1/2" Ice 7.706	7.200 7.706	0.125 0.202
30" Diameter x 6' Stealth Tube	C	None		0.000	77.000	No Ice 12.000 1/2" Ice 12.572	12.000 12.572	0.500 0.636
4' Panel (T-Mobile)	A	From Leg	0.500 0.000 0.000	0.000	95.000	No Ice 0.000 1/2" Ice 0.000	0.000 0.000	0.035 0.035
4' Panel (T-Mobile)	C	From Leg	0.500 0.000	0.000	95.000	No Ice 0.000 1/2" Ice 0.000	0.000 0.000	0.035 0.035

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Description	Face or Leg	Offset Type	Offsets: Horz Lateral Vert ft ft ft	Azimuth Adjustment °	Placement ft		C _A A _A Front ft ²	C _A A _A Side ft ²	Weight K
CSS XDUO6-65 (Verizon)	A	From Leg	0.000 0.500 0.000 0.000	0.000	77.000	No Ice 1/2" Ice	0.000 0.000	0.000 0.000	0.035 0.035
CSS XDUO6-65 (Verizon)	C	From Leg	0.500 0.000 0.000	0.000	77.000	No Ice 1/2" Ice	0.000 0.000	0.000 0.000	0.035 0.035

Dishes

Description	Face or Leg	Dish Type	Offset Type	Offsets: Horz Lateral Vert ft	Azimuth Adjustment °	3 dB Beam Width °	Elevation ft	Outside Diameter ft	Aperture Area ft ²	Weight K
Flag Pole Ball		Paraboloid w/Radome	None		0.000		99.500	1.000	No Ice 1/2" Ice	0.786 0.922

Tower Pressures - No Ice

$$G_H = 1.690$$

Section Elevation ft	z ft	K _Z	q _z ksf	A _G ft ²	F a c e	A _F ft ²	A _R ft ²	A _{leg} ft ²	Leg %	C _A A _A In Face ft ²	C _A A _A Out Face ft ²
L1 99.000- 74.000	86.500	1.317	0.022	10.417	A	0.000	10.417	10.417	100.00	0.000	0.000
					B	0.000	10.417		100.00	0.000	0.000
					C	0.000	10.417		100.00	0.000	0.000

Tower Pressure - With Ice

$$G_H = 1.690$$

Section Elevation ft	z ft	K _Z	q _z ksf	t _z in	A _G ft ²	F a c e	A _F ft ²	A _R ft ²	A _{leg} ft ²	Leg %	C _A A _A In Face ft ²	C _A A _A Out Face ft ²
L1 99.000- 74.000	86.500	1.317	0.016	0.500	12.500	A	0.000	12.500	12.500	100.00	0.000	0.000
						B	0.000	12.500		100.00	0.000	0.000
						C	0.000	12.500		100.00	0.000	0.000

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Tower Pressure - Service

$$G_H = 1.690$$

Section Elevation	z	K _Z	q _z	A _G	F a c e	A _F	A _R	A _{leg}	Leg %	C _A A _A In Face ft ²	C _A A _A Out Face ft ²
ft	ft		ksf	ft ²		ft ²	ft ²	ft ²			
L1 99.000- 74.000	86.500	1.317	0.008	10.417	A	0.000	10.417	10.417	100.00	0.000	0.000
					B	0.000	10.417		100.00	0.000	0.000
					C	0.000	10.417		100.00	0.000	0.000

Tower Forces - No Ice - Wind Normal To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000- 74.000	0.000	0.564	A	1	1.139	1	1	1	10.417	0.433	0.017	C
			B	1	1.139	1	1	1	10.417			
			C	1	1.139	1	1	1	10.417			
Sum Weight:	0.000	0.564						OTM	5.408 kip- ft	0.433		

Tower Forces - No Ice - Wind 60 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000- 74.000	0.000	0.564	A	1	1.139	1	1	1	10.417	0.433	0.017	C
			B	1	1.139	1	1	1	10.417			
			C	1	1.139	1	1	1	10.417			
Sum Weight:	0.000	0.564						OTM	5.408 kip- ft	0.433		

Tower Forces - No Ice - Wind 90 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000- 74.000	0.000	0.564	A	1	1.139	1	1	1	10.417	0.433	0.017	C
			B	1	1.139	1	1	1	10.417			
			C	1	1.139	1	1	1	10.417			
Sum Weight:	0.000	0.564						OTM	5.408 kip- ft	0.433		

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Project

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Tower Forces - With Ice - Wind Normal To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000-74.000	0.000	0.648	A	1	1.2	1	1	1	12.500	0.410	0.016	C
			B	1	1.2	1	1	1	12.500			
			C	1	1.2	1	1	1	12.500			
Sum Weight:	0.000	0.648						OTM	5.128 kip-ft	0.410		

Tower Forces - With Ice - Wind 60 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000-74.000	0.000	0.648	A	1	1.2	1	1	1	12.500	0.410	0.016	C
			B	1	1.2	1	1	1	12.500			
			C	1	1.2	1	1	1	12.500			
Sum Weight:	0.000	0.648						OTM	5.128 kip-ft	0.410		

Tower Forces - With Ice - Wind 90 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000-74.000	0.000	0.648	A	1	1.2	1	1	1	12.500	0.410	0.016	C
			B	1	1.2	1	1	1	12.500			
			C	1	1.2	1	1	1	12.500			
Sum Weight:	0.000	0.648						OTM	5.128 kip-ft	0.410		

Tower Forces - Service - Wind Normal To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000-74.000	0.000	0.564	A	1	1.2	1	1	1	10.417	0.178	0.007	C
			B	1	1.2	1	1	1	10.417			
			C	1	1.2	1	1	1	10.417			
Sum Weight:	0.000	0.564						OTM	2.226 kip-ft	0.178		

KISA Tower JWB Tower Services 148 Governor Street New Britain, CT 06053 Phone: (800) 819-3084 FAX: (860) 256-8175	25' Flag Pole Mast Analysis					Page 6 of 12
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Tower Forces - Service - Wind 60 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000- 74.000	0.000	0.564	A	1	1.2	1	1	1	10.417	0.178	0.007	C
			B	1	1.2	1	1	1	10.417			
			C	1	1.2	1	1	1	10.417			
Sum Weight:	0.000	0.564						OTM	2.226 kip-ft	0.178		

Tower Forces - Service - Wind 90 To Face

Section Elevation	Add Weight	Self Weight	F a c e	e	C _F	R _R	D _F	D _R	A _E	F	w	Ctrl. Face
ft	K	K							ft ²	K	klf	
L1 99.000- 74.000	0.000	0.564	A	1	1.2	1	1	1	10.417	0.178	0.007	C
			B	1	1.2	1	1	1	10.417			
			C	1	1.2	1	1	1	10.417			
Sum Weight:	0.000	0.564						OTM	2.226 kip-ft	0.178		

Force Totals

Load Case	Vertical Forces	Sum of Forces X	Sum of Forces Z	Sum of Overturning Moments, M _x	Sum of Overturning Moments, M _z	Sum of Torques
	K	K	K	kip-ft	kip-ft	kip-ft
Leg Weight	0.564					
Bracing Weight	0.000					
Total Member Self-Weight	0.564					
Total Weight	1.729			-0.025	0.043	
Wind 0 deg - No Ice				-0.025	0.043	
Wind 30 deg - No Ice		0.350	-1.721	-20.439	-6.607	0.000
Wind 60 deg - No Ice		1.210	-1.490	-17.704	-16.814	0.000
Wind 90 deg - No Ice		1.840	-0.860	-10.232	-24.286	0.000
Wind 120 deg - No Ice		2.071	0.000	-0.025	-27.021	0.000
Wind 150 deg - No Ice		1.840	0.860	10.182	-24.286	0.000
Wind 180 deg - No Ice		1.210	1.490	17.654	-16.814	0.000
Wind 210 deg - No Ice		0.350	1.721	20.389	-6.607	0.000
Wind 240 deg - No Ice		-0.510	1.490	17.654	3.600	0.000
Wind 270 deg - No Ice		-1.140	0.860	10.182	11.072	0.000
Wind 300 deg - No Ice		-1.371	0.000	-0.025	13.807	0.000
Wind 330 deg - No Ice		-1.140	-0.860	-10.232	11.072	0.000
Member Ice		-0.510	-1.490	-17.704	3.600	0.000
Total Weight Ice	0.084					
Wind 0 deg - Ice	2.306			-0.025	0.043	
Wind 30 deg - Ice		0.232	-1.439	-17.217	-4.365	0.000
Wind 60 deg - Ice		0.951	-1.246	-14.913	-12.961	0.000
Wind 90 deg - Ice		1.478	-0.719	-8.621	-19.254	0.000
Wind 120 deg - Ice		1.671	0.000	-0.025	-21.557	0.000
Wind 150 deg - Ice		1.478	0.719	8.571	-19.254	0.000
		0.951	1.246	14.864	-12.961	0.000

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Load Case	Vertical Forces K	Sum of Forces X K	Sum of Forces Z K	Sum of Overturning Moments, M_x kip-ft	Sum of Overturning Moments, M_z kip-ft	Sum of Torques kip-ft
Wind 180 deg - Ice		0.232	1.439	17.167	-4.365	0.000
Wind 210 deg - Ice		-0.487	1.246	14.864	4.231	0.000
Wind 240 deg - Ice		-1.014	0.719	8.571	10.523	0.000
Wind 270 deg - Ice		-1.207	0.000	-0.025	12.827	0.000
Wind 300 deg - Ice		-1.014	-0.719	-8.621	10.523	0.000
Wind 330 deg - Ice		-0.487	-1.246	-14.913	4.231	0.000
Total Weight	1.629			-0.025	0.043	
Wind 0 deg - Service		0.000	-0.681	-8.112	0.043	0.000
Wind 30 deg - Service		0.341	-0.590	-7.029	-4.001	0.000
Wind 60 deg - Service		0.590	-0.341	-4.069	-6.961	0.000
Wind 90 deg - Service		0.681	0.000	-0.025	-8.045	0.000
Wind 120 deg - Service		0.590	0.341	4.019	-6.961	0.000
Wind 150 deg - Service		0.341	0.590	6.979	-4.001	0.000
Wind 180 deg - Service		0.000	0.681	8.063	0.043	0.000
Wind 210 deg - Service		-0.341	0.590	6.979	4.087	0.000
Wind 240 deg - Service		-0.590	0.341	4.019	7.047	0.000
Wind 270 deg - Service		-0.681	0.000	-0.025	8.131	0.000
Wind 300 deg - Service		-0.590	-0.341	-4.069	7.047	0.000
Wind 330 deg - Service		-0.341	-0.590	-7.029	4.087	0.000

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

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Comb. No.	Description
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 Comb.	Force K	Major Axis Moment kip-ft	Minor Axis Moment kip-ft
L1	99 - 74	Pole	Max Tension	1	0.000	0.000	0.000
			Max. Compression	14	-2.306	0.044	0.025
			Max. M _x	5	-1.719	-27.764	0.026
			Max. M _y	2	-1.722	-6.810	20.992
			Max. V _y	5	2.079	-27.764	0.026
			Max. V _x	2	-1.727	-6.810	20.992
			Max. Torque	8			0.001

Maximum Reactions

Location	Condition	Gov. Load Comb.	Vertical K	Horizontal, X K	Horizontal, Z K
Pole	Max. Vert	18	2.306	-1.671	0.000
	Max. H _x	11	1.729	1.371	0.000
	Max. H _z	2	1.729	-0.350	1.721
	Max. M _x	2	20.992	-0.350	1.721
	Max. M _z	5	27.764	-2.071	0.000
	Max. Torsion	8	0.001	-0.350	-1.721
	Min. Vert	1	1.729	0.000	0.000
	Min. H _x	5	1.729	-2.071	0.000
	Min. H _z	8	1.729	-0.350	-1.721
	Min. M _x	8	-20.941	-0.350	-1.721
	Min. M _z	11	-14.162	1.371	0.000
	Min. Torsion	2	-0.001	-0.350	1.721

Tower Mast Reaction Summary

Load Combination	Vertical K	Shear _x K	Shear _z K	Overturning Moment, M _x kip-ft	Overturning Moment, M _z kip-ft	Torque kip-ft
Dead Only	1.729	0.000	0.000	-0.025	0.044	0.000
Dead+Wind 0 deg - No Ice	1.729	0.350	-1.721	-20.992	-6.810	0.001
Dead+Wind 30 deg - No Ice	1.729	1.210	-1.490	-18.179	-17.289	0.001
Dead+Wind 60 deg - No Ice	1.729	1.840	-0.860	-10.505	-24.958	0.001
Dead+Wind 90 deg - No Ice	1.729	2.071	0.000	-0.026	-27.764	0.000
Dead+Wind 120 deg - No Ice	1.729	1.840	0.860	10.454	-24.958	-0.000
Dead+Wind 150 deg - No Ice	1.729	1.210	1.490	18.128	-17.289	-0.001
Dead+Wind 180 deg - No Ice	1.729	0.350	1.721	20.941	-6.810	-0.001
Dead+Wind 210 deg - No Ice	1.729	-0.510	1.490	18.136	3.674	-0.001

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Load Combination	Vertical K	Shear _x K	Shear _y K	Overturning Moment, M _x kip-ft	Overturning Moment, M _y kip-ft	Torque kip-ft
Dead+Wind 240 deg - No Ice	1.729	-1.140	0.860	10.462	11.351	-0.000
Dead+Wind 270 deg - No Ice	1.729	-1.371	0.000	-0.026	14.162	-0.000
Dead+Wind 300 deg - No Ice	1.729	-1.140	-0.860	-10.513	11.351	0.000
Dead+Wind 330 deg - No Ice	1.729	-0.510	-1.490	-18.187	3.674	0.001
Dead+Ice+Temp	2.306	0.000	0.000	-0.025	0.044	0.000
Dead+Wind 0 deg+Ice+Temp	2.306	0.232	-1.439	-17.891	-4.559	0.001
Dead+Wind 30 deg+Ice+Temp	2.306	0.951	-1.246	-15.496	-13.490	0.001
Dead+Wind 60 deg+Ice+Temp	2.306	1.478	-0.719	-8.957	-20.026	0.000
Dead+Wind 90 deg+Ice+Temp	2.306	1.671	0.000	-0.026	-22.418	0.000
Dead+Wind 120 deg+Ice+Temp	2.306	1.478	0.719	8.905	-20.026	-0.000
Dead+Wind 150 deg+Ice+Temp	2.306	0.951	1.246	15.444	-13.490	-0.000
Dead+Wind 180 deg+Ice+Temp	2.306	0.232	1.439	17.839	-4.559	-0.001
Dead+Wind 210 deg+Ice+Temp	2.306	-0.487	1.246	15.448	4.374	-0.001
Dead+Wind 240 deg+Ice+Temp	2.306	-1.014	0.719	8.909	10.915	-0.000
Dead+Wind 270 deg+Ice+Temp	2.306	-1.207	0.000	-0.026	13.309	-0.000
Dead+Wind 300 deg+Ice+Temp	2.306	-1.014	-0.719	-8.960	10.915	0.000
Dead+Wind 330 deg+Ice+Temp	2.306	-0.487	-1.246	-15.500	4.374	0.000
Dead+Wind 0 deg - Service	1.729	0.000	-0.681	-8.337	0.044	0.000
Dead+Wind 30 deg - Service	1.729	0.341	-0.590	-7.223	-4.111	0.000
Dead+Wind 60 deg - Service	1.729	0.590	-0.341	-4.181	-7.153	0.000
Dead+Wind 90 deg - Service	1.729	0.681	0.000	-0.026	-8.267	0.000
Dead+Wind 120 deg - Service	1.729	0.590	0.341	4.130	-7.153	0.000
Dead+Wind 150 deg - Service	1.729	0.341	0.590	7.172	-4.111	-0.000
Dead+Wind 180 deg - Service	1.729	0.000	0.681	8.286	0.044	-0.000
Dead+Wind 210 deg - Service	1.729	-0.341	0.590	7.172	4.200	-0.000
Dead+Wind 240 deg - Service	1.729	-0.590	0.341	4.130	7.242	-0.000
Dead+Wind 270 deg - Service	1.729	-0.681	0.000	-0.026	8.355	-0.000
Dead+Wind 300 deg - Service	1.729	-0.590	-0.341	-4.181	7.242	0.000
Dead+Wind 330 deg - Service	1.729	-0.341	-0.590	-7.223	4.200	0.000

Solution Summary

Load Comb.	Sum of Applied Forces			Sum of Reactions			% Error
	PX K	PY K	PZ K	PX K	PY K	PZ K	
1	0.000	-1.729	0.000	0.000	1.729	0.000	0.000%
2	0.350	-1.729	-1.721	-0.350	1.729	1.721	0.000%
3	1.210	-1.729	-1.490	-1.210	1.729	1.490	0.000%
4	1.840	-1.729	-0.860	-1.840	1.729	0.860	0.000%
5	2.071	-1.729	0.000	-2.071	1.729	0.000	0.000%
6	1.840	-1.729	0.860	-1.840	1.729	-0.860	0.000%
7	1.210	-1.729	1.490	-1.210	1.729	-1.490	0.000%
8	0.350	-1.729	1.721	-0.350	1.729	-1.721	0.000%
9	-0.510	-1.729	1.490	0.510	1.729	-1.490	0.000%
10	-1.140	-1.729	0.860	1.140	1.729	-0.860	0.000%
11	-1.371	-1.729	0.000	1.371	1.729	0.000	0.000%
12	-1.140	-1.729	-0.860	1.140	1.729	0.860	0.000%
13	-0.510	-1.729	-1.490	0.510	1.729	1.490	0.000%
14	0.000	-2.306	0.000	0.000	2.306	0.000	0.000%
15	0.232	-2.306	-1.439	-0.232	2.306	1.439	0.000%
16	0.951	-2.306	-1.246	-0.951	2.306	1.246	0.000%
17	1.478	-2.306	-0.719	-1.478	2.306	0.719	0.000%
18	1.671	-2.306	0.000	-1.671	2.306	0.000	0.000%
19	1.478	-2.306	0.719	-1.478	2.306	-0.719	0.000%
20	0.951	-2.306	1.246	-0.951	2.306	-1.246	0.000%
21	0.232	-2.306	1.439	-0.232	2.306	-1.439	0.000%
22	-0.487	-2.306	1.246	0.487	2.306	-1.246	0.000%
23	-1.014	-2.306	0.719	1.014	2.306	-0.719	0.000%

JWB Tower Services 148 Governor Street New Britain, CT 06053 Phone: (800) 819-3084 FAX: (860) 256-8175	25' Flag Pole Mast Analysis		10 of 12
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Load Comb.	Sum of Applied Forces			Sum of Reactions			% Error
	PX K	PY K	PZ K	PX K	PY K	PZ K	
24	-1.207	-2.306	0.000	1.207	2.306	0.000	0.000%
25	-1.014	-2.306	-0.719	1.014	2.306	0.719	0.000%
26	-0.487	-2.306	-1.246	0.487	2.306	1.246	0.000%
27	0.000	-1.729	-0.681	0.000	1.729	0.681	0.000%
28	0.341	-1.729	-0.590	-0.341	1.729	0.590	0.000%
29	0.590	-1.729	-0.341	-0.590	1.729	0.341	0.000%
30	0.681	-1.729	0.000	-0.681	1.729	0.000	0.000%
31	0.590	-1.729	0.341	-0.590	1.729	-0.341	0.000%
32	0.341	-1.729	0.590	-0.341	1.729	-0.590	0.000%
33	0.000	-1.729	0.681	0.000	1.729	-0.681	0.000%
34	-0.341	-1.729	0.590	0.341	1.729	-0.590	0.000%
35	-0.590	-1.729	0.341	0.590	1.729	-0.341	0.000%
36	-0.681	-1.729	0.000	0.681	1.729	0.000	0.000%
37	-0.590	-1.729	-0.341	0.590	1.729	0.341	0.000%
38	-0.341	-1.729	-0.590	0.341	1.729	0.590	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	0.00075216
3	Yes	5	0.00000001	0.00000001
4	Yes	5	0.00000001	0.00000001
5	Yes	4	0.00000001	0.00024472
6	Yes	5	0.00000001	0.00000001
7	Yes	5	0.00000001	0.00000001
8	Yes	4	0.00000001	0.00074788
9	Yes	4	0.00000001	0.00025642
10	Yes	4	0.00000001	0.00039078
11	Yes	4	0.00000001	0.00000001
12	Yes	4	0.00000001	0.00039433
13	Yes	4	0.00000001	0.00025814
14	Yes	4	0.00000001	0.00000001
15	Yes	5	0.00000001	0.00000001
16	Yes	5	0.00000001	0.00000001
17	Yes	5	0.00000001	0.00000001
18	Yes	4	0.00000001	0.00082881
19	Yes	5	0.00000001	0.00000001
20	Yes	5	0.00000001	0.00000001
21	Yes	5	0.00000001	0.00000001
22	Yes	4	0.00000001	0.00072308
23	Yes	4	0.00000001	0.00086096
24	Yes	4	0.00000001	0.00039962
25	Yes	4	0.00000001	0.00086711
26	Yes	4	0.00000001	0.00072716
27	Yes	4	0.00000001	0.00000001
28	Yes	4	0.00000001	0.00000001
29	Yes	4	0.00000001	0.00000001
30	Yes	4	0.00000001	0.00000001
31	Yes	4	0.00000001	0.00000001
32	Yes	4	0.00000001	0.00000001
33	Yes	4	0.00000001	0.00000001
34	Yes	4	0.00000001	0.00000001
35	Yes	4	0.00000001	0.00000001
36	Yes	4	0.00000001	0.00000001
37	Yes	4	0.00000001	0.00000001

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Yes

4

0.00000001

0.00000001

Maximum Tower Deflections - Service Wind

Section No.	Elevation ft	Horz. Deflection in	Gov. Load Comb.	Tilt °	Twist °
L1	99 - 74	4.518	37	1.156	0.000

Critical Deflections and Radius of Curvature - Service Wind

Elevation ft	Appurtenance	Gov. Load Comb.	Deflection in	Tilt °	Twist °	Radius of Curvature ft
99.500	Flag Pole Ball	37	4.518	1.156	0.000	Inf
95.500	18" Diameter x 7' Stealth Tube	37	3.885	1.156	0.000	Inf
95.000	4' Panel	37	3.795	1.156	0.000	Inf
93.000	Flag Loading	37	3.434	1.156	0.000	Inf
89.000	18" Diameter x 6' Stealth Tube	37	2.711	1.156	0.000	Inf
83.000	18" Diameter x 6' Stealth Tube	37	1.626	1.156	0.000	Inf
77.000	30" Diameter x 6' Stealth Tube	37	0.542	1.156	0.000	Inf

Maximum Tower Deflections - Design Wind

Section No.	Elevation ft	Horz. Deflection in	Gov. Load Comb.	Tilt °	Twist °
L1	99 - 74	15.490	5	3.960	0.001

Critical Deflections and Radius of Curvature - Design Wind

Elevation ft	Appurtenance	Gov. Load Comb.	Deflection in	Tilt °	Twist °	Radius of Curvature ft
99.500	Flag Pole Ball	5	15.490	3.960	0.001	Inf
95.500	18" Diameter x 7' Stealth Tube	5	13.321	3.568	0.001	Inf
95.000	4' Panel	5	13.012	3.511	0.001	Inf
93.000	Flag Loading	5	11.772	3.287	0.001	Inf
89.000	18" Diameter x 6' Stealth Tube	5	9.294	2.838	0.001	Inf
83.000	18" Diameter x 6' Stealth Tube	5	5.576	2.165	0.000	Inf
77.000	30" Diameter x 6' Stealth Tube	5	1.859	1.492	0.000	Inf

Compression Checks

KISA Tower JWB Tower Services 148 Governor Street New Britain, CT 06053 Phone: (800) 819-3084 FAX: (860) 256-8175	Job	25' Flag Pole Mast Analysis	Page	12 of 12
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Pole Design Data

Section No.	Elevation ft	Size	L ft	L _u ft	Kl/r	F _a ksi	A in ²	Actual P K	Allow. P _a K	Ratio P P _a
L1	99 - 74 (1)	HSS5x.5	25.000	0.000	0.0	42.000	6.625	-1.719	278.246	0.006

Pole Bending Design Data

Section No.	Elevation ft	Size	Actual M _x kip-ft	Actual f _{bx} ksi	Allow. F _{bx} ksi	Ratio f _{bx} F _{bx}	Actual M _y kip-ft	Actual f _{by} ksi	Allow. F _{by} ksi	Ratio f _{by} F _{by}
L1	99 - 74 (1)	HSS5x.5	27.764	-48.397	46.200	1.048	0.000	0.000	46.200	0.000

Pole Interaction Design Data

Section No.	Elevation ft	Size	Ratio P P _a	Ratio f _{bx} F _{bx}	Ratio f _{by} F _{by}	Comb. Stress Ratio	Allow. Stress Ratio	Criteria
L1	99 - 74 (1)	HSS5x.5	0.006	1.048	0.000	1.054 ✓	1.333	H1-3 ✓

Section Capacity Table

Section No.	Elevation ft	Component Type	Size	Critical Element	P K	SF*P _{allow} K	% Capacity	Pass Fail
L1	99 - 74	Pole	HSS5x.5	1	-1.719	370.902	79.0	Pass
Summary								
Pole (L1)								Pass
RATING =								Pass



100' Flagpole
586 Danbury Road, New Milford CT
Verizon Site: Candlewood Alltel, CT

FLANGE PLATE ANALYSIS

FLANGE BOLT AND PLATE ANALYSIS

Input Data

Mast Reactions:

Overturning Moment:	OM := 28·kips·ft	user input
Shear Force:	Shear := 2.0·kips	user input
Axial Force:	Axial := 2.0·kips	user input

Bolt Data:

Use ASTM A325N

Number of Anchor Bolts = N	$N_w := 6$	user input
Bolt Ultimate Strength:	$F_u := 120\text{-ksi}$	user input
Bolt Allowable Strength:	$F_y := 92\text{-ksi}$	user input
Diameter Of Anchor Bolts	$D := 1.0\text{in}$	user input
Threaded length per inch	$n := 8.0$	user input
Bolt "Column" Distance:	$L_w := 0.1\text{in}$	user input
Bolt Modulus:	$E := 29000\text{-ksi}$	user input

Flange Plate Data:

Plate Yield Strength:	$F_{y_{bp}} := 36\text{-ksi}$	user input
Base Plate Thickness:	PlateThicknessProvide := 2.5·in	user input

Geometric Layout Data:

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

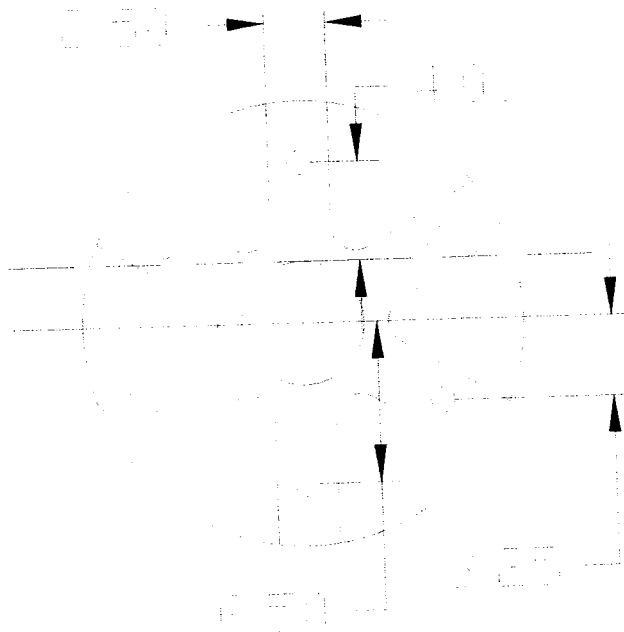
Distances for loading condition (see detail):

$$d_1 := 6.0 \cdot \text{in} \quad \text{user input}$$

$$\text{MomentArm}_1 := 4.0 \cdot \text{in} \quad \text{user input}$$

$$d_2 := 3.25 \cdot \text{in} \quad \text{user input}$$

$$\text{EffectiveWidth} := 2.5 \cdot \text{in} \quad \text{user input}$$

**DETAIL - ANCHOR BOLT AND PLATE**

Job 100' Flag Pole - New Milford, CT

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Description Flange Plate Analysis

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Bolt Section Properties:Polar Moment of Inertia (J) divided by Area (A) = Σd

$$\Sigma d := (d_1)^2 \cdot 2 + (d_2)^2 \cdot 4$$

$$\Sigma d = 114.25 \cdot \text{in}^2$$

Gross Area of Bolt:

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

$$A_g = 0.79 \cdot \text{in}^2$$

Net Area of Bolt:

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

$$A_{\text{net}} = 0.61 \cdot \text{in}^2$$

Net Diameter:

$$D_n := \frac{2 \cdot \sqrt{A_{\text{net}}}}{\sqrt{\pi}}$$

$$D_n = 0.88 \cdot \text{in}$$

Radius of Gyration of Bolt:

$$r := \frac{D_n}{4}$$

$$r = 0.22 \cdot \text{in}$$

Section Modulus of Bolt:

$$S_x := \frac{\pi \cdot D_n^3}{32}$$

$$S_x = 0.07 \cdot \text{in}^3$$

Bolt Bending Stress:

Maximum Applied Bending:

$$M_x := \left(\frac{\text{Shear}}{N} \right) \cdot l$$

$$M_x = 2.78 \times 10^{-3} \cdot \text{kips} \cdot \text{ft}$$

$$f_{bx} := \frac{M_x}{S_x}$$

$$f_{bx} = 0.5 \cdot \text{ksi}$$

Allowable Bending

$$F_{bx} := 1.333 \cdot 0.60 \cdot F_y$$

$$F_{bx} = 73.58 \cdot \text{ksi}$$

Note: 1.333 increase allowed per TIA/EIA

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Bolt Tensile Stress Check:

Maximum Tensile Force (Gross Area):

$$\text{AllowableTension} := 1.333 \cdot (0.33 \cdot A_g \cdot F_u) \quad \text{AllowableTension} = 41.46 \text{ kips}$$

Note: 1.333 increase allowed per TIA/EIA

Maximum Tensile Force (Net Area):

$$F_{\text{net.area}} := 1.333 \cdot (0.60 \cdot A_{\text{net}} \cdot F_y) \quad F_{\text{net.area}} = 44.57 \text{ kips}$$

Note: 1.333 increase allowed per TIA/EIA

Maximum Applied Tension:

$$\text{MaxTension} := \frac{\text{OM} \cdot d_1}{\sum d} - \frac{\text{Axial}}{N} \quad \text{MaxTension} = 17.31 \text{ kips}$$

Check Stresses:

$$\text{AnchorBoltStress} := \text{if}(F_{\text{net.area}} > \text{MaxTension}, \text{"Not Overstressed"}, \text{"Overstressed"})$$

$$\text{AnchorBoltStress} = \text{"Not Overstressed"}$$

$$\text{PercentStressed} := 100 \cdot \frac{\text{MaxTension}}{F_{\text{net.area}}}$$

$$\text{PercentStressed} = 38.84$$

Note: Shear Stress is negligible

Job 100' Flag Pole - New Milford, CT
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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 if a combined stress analysis is not required and set the bending stress to zero:

$$l_w := \begin{cases} 1 & \text{if } l > 2 \cdot D_n \\ 0.0 \text{ in} & \text{otherwise} \end{cases} \quad l = 0$$

$$f_{bx} := \begin{cases} f_{bx} & \text{if } l > 2 \cdot D_n \\ 0.0 \text{ ksi} & \text{otherwise} \end{cases} \quad f_{bx} = 0 \cdot \text{ksi}$$

Allowable Compressive Force:

$$K_w := 0.65$$

$$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{F_y}} \quad C_c = 78.88$$

$$F_a := \begin{cases} \left[1 - \frac{\left(\frac{K \cdot l}{r} \right)^2}{2 \cdot C_c^2} \right] \cdot F_y & \text{if } \frac{K \cdot l}{r} \leq C_c \\ \frac{5}{3} + \frac{3 \cdot \left(\frac{K \cdot l}{r} \right)}{8 \cdot C_c} - \frac{\left(\frac{K \cdot l}{r} \right)^3}{8 \cdot C_c^3} & \\ \frac{12 \cdot \pi^2 \cdot E}{23 \cdot \left(\frac{K \cdot l}{r} \right)^2} & \text{if } \frac{K \cdot l}{r} > C_c \end{cases} \quad F_a = 55.2 \cdot \text{ksi}$$

$$F_{ax} := 1.33 \cdot F_a \quad \text{Note: 1.33 increase allowed per TIA/EIA} \quad F_a = 73.42 \cdot \text{ksi}$$

Applied Compressive Force:

$$\text{MaxCompression} := \frac{OM \cdot d_l}{\Sigma d} + \frac{\text{Axial}}{N} \quad \text{MaxCompression} = 17.98 \cdot \text{kips}$$

$$f_a := \frac{\text{MaxCompression}}{A_{net}} \quad f_a = 29.68 \cdot \text{ksi}$$

Check Combined Stresses:

$$\text{StressRatio} := \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} \quad \text{StressRatio} = 0.40$$

Condition := if(StressRatio ≤ 1.0, "Not Overstressed", "Overstressed")

Condition = "Not Overstressed"

JWB

Job

100' Flag Pole - New Milford, CT

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Description

Flange Plate Analysis

Computed by

Staff

Date 01/06/09

Checked by

Date

Flange Plate Analysis:

Force From Bolt(s):

$$C_1 := \frac{OM \cdot d_1}{\Sigma d} + \frac{Axial}{N}$$

$$C_1 = 17.98 \times 10^3 \cdot lb$$

Bending Stress In Plate:

$$f_{bp} := \frac{6 \cdot (C_1 \cdot MomentArm_1)}{EffectiveWidth \cdot PlateThicknessProvide^2}$$

$$f_{bp} = 27.62 \cdot ksi$$

Check Stresses:

$$PlateRatio := \frac{f_{bp}}{1.333 \cdot 0.75 F_{y_{bp}}}$$

$$PlateRatio = 0.77$$

PlateStress := if(PlateRatio < 1, "Not Over Stress", "Is Over Stress")

PlateStress = "Not Over Stress"



100' Flagpole
586 Danbury Road, New Milford CT
Verizon Site: Candlewood Alltel, CT

ANCHOR BOLT AND BASE PLATE ANALYSIS

ANCHOR BOLT AND BASEPLATE ANALYSIS

Input Data

Tower Reactions:

Overturning Moment:	OM := 244-kips·ft	user input
Shear Force:	Shear := 4.0-kips	user input
Axial Force:	Axial := 6.0-kips	user input

Anchor Bolt Data:

Use ASTM 615 Grade 75

Number of Anchor Bolts = N	$N_b := 4$	user input
----------------------------	------------	------------

Bolt Ultimate Strength:	$F_u := 100\text{-ksi}$	user input
-------------------------	-------------------------	------------

Bolt Allowable Strength:	$F_y := 75\text{-ksi}$	user input
--------------------------	------------------------	------------

Diameter Of Anchor Bolts	$D := 2.25\text{in}$	user input
--------------------------	----------------------	------------

Threaded length per inch	$n := 4.5$	user input
--------------------------	------------	------------

Bolt "Column" Distance:	$l_w := 3.5\text{in}$	user input
-------------------------	-----------------------	------------

Bolt Modulus:	$E := 29000\text{-ksi}$	user input
---------------	-------------------------	------------

Base Plate Data:

Plate Yield Strength:	$F_{ybp} := 55\text{-ksi}$	user input
-----------------------	----------------------------	------------

Base Plate Thickness:	PlateThicknessProvide := 2·in	user input
-----------------------	-------------------------------	------------

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Geometric Layout Data:

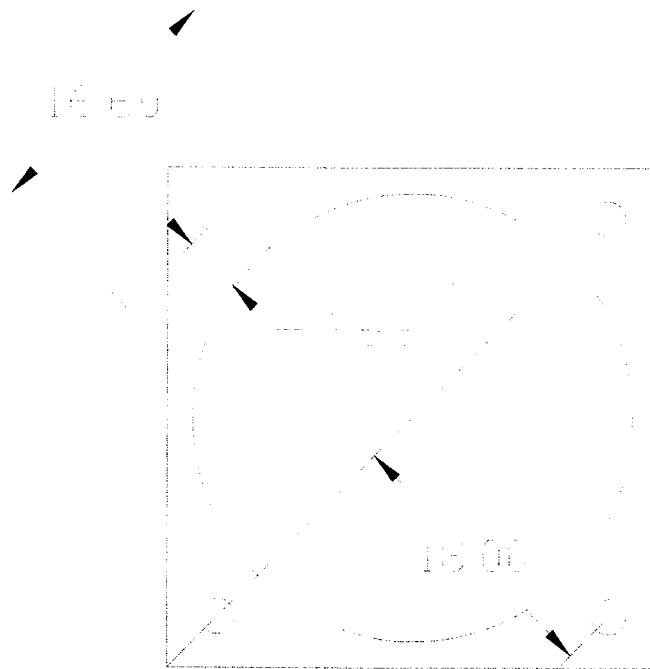
Distance from the center of gravity of the group to bolt in question = $d(i)$

Distances for loading condition (see detail):

$$d_1 := 18.00\text{-in} \quad \text{user input}$$

$$\text{MomentArm}_1 := 3.68\text{-in} \quad \text{user input}$$

$$\text{EffectiveWidth} := 16.60\text{-in} \quad \text{user input}$$



DETAIL - ANCHOR BOLT AND PLATE

Anchor Bolt Section Properties:Polar Moment of Inertia (J) divided by Area (A) = Σd

$$\Sigma d := (d_1)^2 \cdot 2$$

$$\Sigma d = 648 \cdot \text{in}^2$$

Gross Area of Bolt:

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

$$A_g = 3.98 \cdot \text{in}^2$$

Net Area of Bolt:

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

$$A_{\text{net}} = 3.25 \cdot \text{in}^2$$

Net Diameter:

$$D_n := \frac{2 \cdot \sqrt{A_{\text{net}}}}{\sqrt{\pi}}$$

$$D_n = 2.03 \cdot \text{in}$$

Radius of Gyration of Bolt:

$$r := \frac{D_n}{4}$$

$$r = 0.51 \cdot \text{in}$$

Section Modulus of Bolt:

$$S_x := \frac{\pi \cdot D_n^3}{32}$$

$$S_x = 0.83 \cdot \text{in}^3$$

Anchor Bolt Bending Stress:

Maximum Applied Bending:

$$M_x := \left(\frac{\text{Shear}}{N} \right) \cdot l$$

$$M_x = 0.29 \cdot \text{kips} \cdot \text{ft}$$

$$f_{bx} := \frac{M_x}{S_x}$$

$$f_{bx} = 4.24 \cdot \text{ksi}$$

Allowable Bending

$$F_{bx} := 1.333 \cdot 0.60 \cdot F_y$$

$$F_{bx} = 59.98 \cdot \text{ksi}$$

Note: 1.333 increase allowed per TIA/EIA

Anchor Bolt Tensile Stress Check:

Maximum Tensile Force (Gross Area):

$$\text{AllowableTension} := 1.333 \cdot (0.33 \cdot A_g \cdot F_u)$$

$$\text{AllowableTension} = 174.9 \text{ kips}$$

Note: 1.333 increase allowed per TIA/EIA

Maximum Tensile Force (Net Area):

$$F_{\text{net.area}} := 1.333 \cdot (0.60 \cdot A_{\text{net}} \cdot F_y)$$

$$F_{\text{net.area}} = 194.81 \text{ kips}$$

Note: 1.333 increase allowed per TIA/EIA

Maximum Applied Tension:

$$\text{MaxTension} := \frac{\text{OM} \cdot d_1}{\Sigma d} - \frac{\text{Axial}}{N}$$

$$\text{MaxTension} = 79.83 \text{ kips}$$

Check Stresses:

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

$$\text{AnchorBoltStress} := \text{if}(F_{\text{net.area}} > \text{MaxTension}, \text{"Not Overstressed"}, \text{"Overstressed"})$$

$$\text{AnchorBoltStress} = \text{"Not Overstressed"}$$

$$\text{PercentStressed} := 100 \cdot \frac{\text{MaxTension}}{F_{\text{net.area}}}$$

$$\text{PercentStressed} = 40.98$$

Note: Shear Stress is negligible

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 if a combined stress analysis is not required and set the bending stress to zero:

$$l_{ww} := \begin{cases} 1, & \text{if } l > 2 \cdot D_n \\ 0.0 \text{ in} & \text{otherwise} \end{cases} \quad l = 0 \quad f_{bx} := \begin{cases} f_{bx} & \text{if } l > 2 \cdot D_n \\ 0.0 \text{ ksi} & \text{otherwise} \end{cases} \quad f_{bx} = 0 \cdot \text{ksi}$$

Allowable Compressive Force:

$$K := 0.65$$

$$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{F_y}} \quad C_c = 87.36$$

$$F_a := \begin{cases} \frac{\left[1 - \frac{\left(\frac{K \cdot l}{r} \right)^2}{2 \cdot C_c^2} \right] \cdot F_y}{\frac{5}{3} + \frac{3 \cdot \left(\frac{K \cdot l}{r} \right)}{8 \cdot C_c} - \frac{\left(\frac{K \cdot l}{r} \right)^3}{8 \cdot C_c^3}} & \text{if } \frac{K \cdot l}{r} \leq C_c \\ \frac{12 \cdot \pi^2 \cdot E}{23 \cdot \left(\frac{K \cdot l}{r} \right)^2} & \text{if } \frac{K \cdot l}{r} > C_c \end{cases} \quad F_a = 45 \cdot \text{ksi}$$

$$F_{aw} := 1.33 \cdot F_a \quad \text{Note: 1.33 increase allowed per TIA/EIA} \quad F_a = 59.85 \cdot \text{ksi}$$

Applied Compressive Force:

$$\text{MaxCompression} := \frac{OM \cdot d_1}{\Sigma d} + \frac{\text{Axial}}{N} \quad \text{MaxCompression} = 82.83 \cdot \text{kips}$$

$$f_a := \frac{\text{MaxCompression}}{A_{net}} \quad f_a = 25.51 \cdot \text{ksi}$$

Check Combined Stresses:

$$\text{StressRatio} := \frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} \quad \text{StressRatio} = 0.43$$

Condition := if(StressRatio ≤ 1.0, "Not Overstressed", "Overstressed")

Condition = "Not Overstressed"

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Base Plate Analysis:

Force From Bolt(s):

$$C_1 := \frac{OM \cdot d_1}{\Sigma d} + \frac{Axial}{N}$$

$$C_1 = 82.83 \times 10^3 \cdot lb$$

Bending Stress In Plate:

$$f_{bp} := \frac{6 \cdot (C_1 \cdot MomentArm_1)}{EffectiveWidth \cdot PlateThicknessProvide^2}$$

$$f_{bp} = 27.54 \cdot ksi$$

Check Stresses:

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

$$BasePlateRatio = 0.5$$

BasePlateStress := if(BasePlateRatio < 1, "Not Over Stress" , "Is Over Stress")

BasePlateStress = "Not Over Stress"



100' Flagpole
586 Danbury Road, New Milford CT
Verizon Site: Candlewood Alltel, CT

FOUNDATION ANALYSIS

MONOPOLE FOUNDATION ANALYSIS

TOWER FORCES:

Moment Caused by Tower $M_t := 244 \text{ ft-kips}$
 Shear at Base of Tower $S_t := 4.0 \text{ kip}$
 Max Compressive Force $C_t := 6.0 \text{ kip}$
 Height of Tower $H_t := 100 \text{ ft}$
 Base Plate Bolt Circle $MP := 36 \text{ in}$

FOOTING DIMENSIONS:

Overall Depth of Footing $D_f := 10 \text{ ft}$
 Length of Pier $L_p := 7.5 \text{ ft}$
 Extension of Pier Above Grade $L_{pag} := 0.5 \text{ ft}$
 Diameter of Pier $d_p := 5.0 \text{ ft}$
 Thickness of Footing $T_f := 3.0 \text{ ft}$
 Width of Footing: $W_f := 13.0 \text{ ft}$
 Length of Anchor Bolts: $L_{st} := 72 \text{ in}$
 Projection of anchor bolts above pier $A_{BP} := 12 \text{ in}$

PROPERTIES:

Compressive Strength of Concrete $f_c := 3000 \text{ psi}$
 Yield Strength of Steel Reinforcement $f_y := 60000 \text{ psi}$
 Yield Strength of Anchor Bolt $f_{ya} := 75000 \text{ psi}$
 Internal Friction Angle of Soil $\phi_s := 30 \text{ deg}$
 Allowable Bearing Capacity $q_s := 4000 \text{ psf}$
 Unit Weight of Soil $\gamma_s := 100 \text{ pcf}$
 Unit Weight of Concrete $\gamma_c := 150 \text{ pcf}$
 Depth to Neglect $n := 0 \text{ ft}$
 Cohesion of Clay Type Soil $c_m := 0 \text{ ksf}$
 Note: Use 0 for Sandy Soil
 Seismic Zone Factor: $Z := 2$
 UBC Fig 23-2
 Coefficient of Friction between Concrete: $\mu := 0.45$
 Clear Cover of Reinforcement Pier: $C_{vr_pier} := 3 \text{ in}$
 Clear Cover of Reinforcement Pad: $C_{vr_pad} := 3 \text{ in}$
 Anchor Bolt Diameter $d_{anchor} := 2.25 \text{ in}$
 Anchor bolt area $A_{anchor} := 3.97 \text{ in}^2$

PIER REINFORCEMENT:

Bar Size $BS_{pier} := 9$ Bar Diameter $d_{bpier} := 1.125 \text{ in}$
 Number of Bars $NB_{pier} := 20$ Bar Area $A_{bpier} := 1.000 \text{ in}^2$

PAD REINFORCEMENT:

TOP: Bar Size $BS_{top} := 9$ Bar Diameter $d_{btop} := 1.125 \text{ in}$
 Number of Bars $NB_{top} := 13$ Bar Area $A_{btop} := 1.000 \text{ in}^2$
 BOTTOM: Bar Size $BS_{bot} := 9$ Bar Diameter $d_{bbot} := 1.125 \text{ in}$
 Number of Bars $NB_{bot} := 13$ Bar Area $A_{bot} := 1.000 \text{ in}^2$

Coefficient of Lateral Soil Pressure: $K_p := \frac{1 + \sin(\phi_s)}{1 - \sin(\phi_s)} K_p = 3$

Load Factor (EIA 3.1.1): $LF := \text{if} \left[H_t \leq 700 \text{ ft}, 1.333, \text{if} \left[H_t \geq 1200, 1.7, 1.333 + \left(\frac{H_t - 700}{1200 - 700} \right) \cdot 0.4 \right] \right] LF = 1.333$

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CHECK ANCHOR STEEL EMBEDMENT

Depth: $D_{ab} := L_{st} - A_{BP}$ $D_{ab} = 5 \cdot \text{ft}$ $L_{\text{anchor}} := \frac{(0.11 \cdot f_y) \cdot \text{in}}{\sqrt{f_c \cdot \text{psi}}}$ $L_{\text{anchor}} = 10.0416 \cdot \text{ft}$

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

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

STABILITY OF FOOTING

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

$P_{pt} := K_p \cdot \gamma_s \cdot (D_f - T_f) + c \cdot 2 \cdot \sqrt{K_p}$ $P_{pt} = 2.1 \cdot \text{ksf}$

$P_{top} := \text{if}[n < (D_f - T_f), P_{pt}, P_{pn}]$ $P_{top} = 2.1 \cdot \text{ksf}$

$P_{bot} := K_p \cdot \gamma_s \cdot D_f + c \cdot 2 \cdot \sqrt{K_p}$ $P_{bot} = 3 \cdot \text{ksf}$

$P_{ave} := \frac{P_{top} + P_{bot}}{2}$ $P_{ave} = 2.55 \cdot \text{ksf}$

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

$A_p := W_f \cdot T_p$ $A_p = 39 \cdot \text{ft}^2$

Ultimate Shear: $S_u := P_{ave} \cdot A_p$ $S_u = 99.45 \cdot \text{kip}$

Weight of Concrete Pad: $WT_c := \left[(W_f^2 \cdot T_f) + d_p^2 \cdot L_p \right] \cdot \gamma_c$ $WT_c = 104.175 \cdot \text{kip}$

Weight of Soil above Footing: $WT_{s1} := \left[W_f^2 \cdot (|L_p - L_{pag}|) - \frac{d_p^2 \cdot \pi}{4} \cdot (|L_p - L_{pag}|) \right] \cdot \gamma_s$ $WT_{s1} = 104.5555 \cdot \text{kip}$

Weight of Soil Wedge at back face: $WT_{s2} := \left(\frac{D_f^2 \cdot \tan(\phi_s)}{2} \cdot W_f \right) \cdot \gamma_s$ $WT_{s2} = 37.5278 \cdot \text{kip}$

Total Weight: $WT_{tot} := WT_c + WT_{s1} + C_t$ $WT_{tot} = 214.7305 \cdot \text{kip}$

Resisting Moment: $M_r := (WT_{tot}) \cdot \frac{W_f}{2} + S_u \cdot \frac{T_f}{3} + WT_{s2} \cdot \left(W_f + \frac{D_f \cdot \tan(\phi_s)}{3} \right)$ $M_r = 2055.2817 \cdot \text{kip} \cdot \text{ft}$

Overturning Moment: $M_{ot} := M_t + S_t \cdot (L_p + T_f)$ $M_{ot} = 286 \cdot \text{kip} \cdot \text{ft}$

Factor of Safety: $FS := \frac{M_r}{M_{ot}}$ $FS_{req} := 2$ $FS = 7.19$

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

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SHEAR CAPACITY IN PIER

$$FS_s := 2$$

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

$$S_p = 98.0394 \text{ kips}$$

$$\text{ShearCheck} := \text{if}(S_p > S_t, \text{"Okay"}, \text{"No Good"})$$

$$\text{ShearCheck} = \text{"Okay"}$$

BEARING PRESSURE CAUSED BY FOOTING

$$A_{mat} := W_f^2$$

$$A_{mat} = 169 \cdot \text{ft}^2$$

$$S := \frac{W_f^3}{6}$$

$$S = 366.1667 \cdot \text{ft}^3$$

$$P_{max} := \frac{W_{T_{tot}}}{A_{mat}} + \frac{M_{ot}}{S}$$

$$P_{max} = 2.0517 \cdot \text{ksf}$$

$$P_{min} := \frac{W_{T_{tot}}}{A_{mat}} - \frac{M_{ot}}{S}$$

$$P_{min} = 0.4895 \cdot \text{ksf}$$

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

$$\text{MaxPressure} = \text{"Okay"}$$

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

$$\text{MinPressure} = \text{"Okay"}$$

Distance to Resultant of Pressure Distribution:

$$X_p := \frac{\frac{P_{max}}{P_{max} - P_{min}} \cdot \frac{1}{3}}{\frac{W_f}{6}}$$

$$X_p = 5.6913 \cdot \text{ft}$$

Distance to Kern:

$$X_k := \frac{W_f}{6}$$

$$X_k = 2.1667 \cdot \text{ft}$$

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

Eccentricity:

$$e := \frac{M_{ot}}{W_{T_{tot}}}$$

$$e = 1.3319$$

Adjusted Soil Pressure:

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

$$P_a = 2.1307 \cdot \text{ksf}$$

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

$$q_{adj} = 2.0517 \cdot \text{ksf}$$

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

$$\text{PressureCheck} = \text{"Okay"}$$

CONCRETE BEARING CAPACITY (ACI 10.17)

$$\phi_c := 0.75 \quad (\text{ACI 9.3.2.2})$$

$$P_b := \phi_c \cdot 0.85 \cdot f_c \cdot \frac{d_p^2 \cdot \pi}{4} \quad P_b = 5407.4664 \cdot \text{kip}$$

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

SHEAR STRENGTH OF CONCRETE

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

$$\phi_{\text{shear}} := .85 \quad (\text{ACI 9.3.2.3})$$

$$d := T_f - C_{\text{vr}} - d_{\text{bot}} \quad d = 31.875 \cdot \text{in}$$

$$d_1 := \frac{W_f}{2} - \frac{d_p}{2} \quad d_1 = 4 \cdot \text{ft}$$

$$d_2 := d_1 - d \quad d_2 = 1.3438 \cdot \text{ft}$$

$$L := \left(\frac{W_f}{2} - e \right) \cdot 3 \quad L = 15.5043 \cdot \text{ft}$$

$$\text{Slope} := \text{if} \left(L > W_f, \frac{P_{\text{max}} - P_{\text{min}}}{W_f}, \frac{q_{\text{adj}}}{L} \right) \quad \text{Slope} = 0.1202 \cdot \text{kcf}$$

$$V_{\text{req}} := LF \cdot \left[(q_{\text{adj}} - \text{Slope} \cdot d_1) + \left(\frac{\text{Slope} \cdot d_1}{2} \right) \right] \cdot W_f \cdot d_1 \quad V_{\text{req}} = 125.5543 \cdot \text{kip}$$

$$\text{ACI 11.3.1.1} \quad V_{\text{Avail}} := \phi_c \cdot 2 \cdot \sqrt{f_c \cdot \text{psi}} \cdot W_f \cdot d \quad V_{\text{Avail}} = 463.0036 \cdot \text{kip}$$

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

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

$$b_o := (d_p + d) \cdot \pi \quad b_o = 24.0528 \cdot \text{ft}$$

$$\text{Area included inside } b_o: \quad A_{bo} := \frac{\pi \cdot (d_p + d)^2}{4} \quad A_{bo} = 46.0386 \cdot \text{ft}^2$$

$$\text{Area outside of } b_o: \quad A_{\text{out}} := A_{\text{mat}} - A_{bo} \quad A_{\text{out}} = 122.9614 \cdot \text{ft}^2$$

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

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

$$v_u := \text{Find}(v_u)$$

$$v_u = 3.3609 \cdot \text{ksf}$$

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

$$V_u = 116.057 \cdot \text{kips}$$

$$V_{\text{req}} := LF \cdot V_u$$

$$V_{\text{req}} = 154.7039 \cdot \text{kips}$$

$$V_{\text{Avail}} := \phi_c \cdot 4 \cdot \sqrt{f_c \cdot \text{psi}} \cdot b_o \cdot d$$

$$V_{\text{Avail}} = 1713.314 \cdot \text{kips}$$

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

STEEL REINFORCEMENT IN THE PAD

$$\phi_m := .90 \text{ ACI 9.3.2.2}$$

Take Maximum Bending at face of Pier:

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

$$q_b = 1.571 \cdot \text{ksf}$$

$$M_n := \frac{1}{LF \cdot \phi_m} \left[(q_{\text{adj}} - q_b) \cdot \frac{d_1^2}{3} + q_b \cdot \frac{d_1^2}{2} \right] \cdot W_f$$

$$M_n = 163.9659 \cdot \text{kip} \cdot \text{ft}$$

ACI 10.2.7.3

$$\beta := \text{if} \left[f_c \leq 4000 \cdot \text{psi}, .85, \text{if} \left[f_c \geq 8000 \cdot \text{psi}, .65, .85 - \left(\frac{\frac{f_c}{\text{psi}} - 4000}{1000} \right) \cdot .05 \right] \right] \quad \beta = 0.85$$

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

$$R_u = 1986.2 \cdot \text{lbf}$$

$$\rho := \frac{0.85 \cdot f_c}{f_y} \left(1 - \sqrt{1 - \frac{2 \cdot R_u}{0.85 \cdot f_c}} \right)$$

$$\rho = 0.0002$$

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

$$\rho_{\text{min}} = 0.00031$$

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

(ACI 7.12.2.1b)

FOR BOTTOM BARS: $A_s := \max(\rho, \rho_{min}, \rho_{sh}) \cdot W_f \cdot d$ $A_s = 8.9505 \cdot \text{in}^2$
 $A_{s_{prov}} := A_{bot} \cdot NB_{bot}$ $A_{s_{prov}} = 13 \cdot \text{in}^2$
 $PadReinforcement := \text{if}(A_{s_{prov}} > A_s, "Okay", "No Good")$ $PadReinforcement = "Okay"$

FOR TOP BARS: $A_s := \rho_{sh} \cdot (W_f \cdot d)$ $A_s = 8.9505 \cdot \text{in}^2$
 $A_{s_{prov}} := A_{bot} \cdot NB_{top}$ $A_{s_{prov}} = 13 \cdot \text{in}^2$
 $PadReinforcement := \text{if}(A_{s_{prov}} > A_s, "Okay", "No Good")$ $PadReinforcement = "Okay"$

TENSION (ACI 12.2.3)

DEVELOPMENT LENGTH OF PAD REINFORCEMENT

Bar Spacing: $B_{sPad} := \frac{W_f - 2 \cdot C_{vr_{pad}} - NB_{bot} \cdot d_{bbot}}{NB_{bot} - 1}$ $B_{sPad} = 11.2813 \cdot \text{in}$

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: $c := \text{if}\left(C_{vr_{pad}} < \frac{B_{sPad}}{2}, C_{vr_{pad}}, \frac{B_{sPad}}{2}\right)$ $c = 3 \cdot \text{in}$

Transverse Reinforcement Index: $k_{tr} := 0$

$$L_{dbt} := \frac{3}{40} \cdot \frac{f_y}{\sqrt{f_c \cdot \text{psi}}} \cdot \frac{\alpha \cdot \beta \cdot \gamma \cdot \lambda}{c + k_{tr}} \cdot d_{bbot}$$

$$L_{dbt} = 34.6606 \cdot \text{in}$$

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

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

Available Length in Pad: $L_{Pad} := \frac{W_f}{2} - \frac{d_p}{2} - C_{vr_{pad}}$ $L_{Pad} = 45 \cdot \text{in}$

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

REINFORCEMENT IN PIER

$$\begin{aligned} \text{Pier Area:} \quad A_p &:= \frac{\pi \cdot d_p^2}{4} & A_p &= 2827.4334 \cdot \text{in}^2 \\ (\text{ACI 10.8.4 and 10.9.1}) \quad A_{smin} &:= 0.01 \cdot 0.05 \cdot A_p & A_{smin} &= 1.4137 \cdot \text{in}^2 \\ A_{sprov} &:= \text{NBpier} \cdot A_{bpier} & A_{sprov} &= 20 \cdot \text{in}^2 \\ \text{SteelAreaCheck} &:= \text{if}(A_{sprov} > A_{smin}, \text{"Okay"}, \text{"No Good"}) & \text{SteelAreaCheck} &= \text{"Okay"} \end{aligned}$$

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

$$\begin{aligned} \text{Bar Spacing In Pier:} \quad B_{sPier} &:= \frac{d_p \cdot \pi}{\text{NBpier}} - d_{bpier} & B_{sPier} &= 8.2998 \cdot \text{in} \\ \text{Diamter of Reinforcement Cage:} \quad \text{Diam}_{cage} &:= d_p - 2 \cdot C_{vrpier} & \text{Diam}_{cage} &= 54 \cdot \text{in} \\ \text{Maximum Moment in Pier:} \quad M_p &:= \left[M_t + S_t \cdot \left(L_p + \frac{A_{BP}}{2} \right) \right] \cdot LF & M_p &= 4414.896 \cdot \text{in-kips} \end{aligned}$$

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

(defined variables)

$$(f_c \ f_y \ c1 \ \text{Spiral}) = (3 \ 60 \ 3 \ 0)$$

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:

$$(D \ N \ n \ P_u \ M_{xu}) := (60 \ 20 \ 9 \ 6 \ 4500)$$

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:

$$(\phi P_n \ \phi M_{xn} \ f_{sp} \ \rho) = (35.542 \ 26656.4659 \ -60 \ 0.0071)$$

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

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

$$\text{AxialLoadCheck} = \text{"Okay"}$$

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

$$\text{BendingCheck} = \text{"Okay"}$$

DEVELOPMENT LENGTH OF PIER REINFORCEMENT

TENSION (ACI 12.2.3)

Factors for development: Reinforcement Location Factor $\alpha_w := 1.0$

Coating Factor $\beta_w := 1.0$

Concrete strength Factor $\lambda_w := 1.0$

Reinforcement Size Factor $\gamma_w := 1.0$

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

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

$$L_{dbt} := \frac{3}{40} \cdot \frac{f_y}{\sqrt{f_c \cdot \text{psi}}} \cdot \frac{\alpha \cdot \beta \cdot \gamma \cdot \lambda}{c + k_{tr}} \cdot d_{bpier} \quad L_{dbt} = 34.6606 \cdot \text{in}$$

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

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

$$L_{dh} := \frac{1200 \cdot d_{bpier}}{\sqrt{\frac{f_c}{\text{psi}}}} \cdot .7 \quad L_{dh} = 17.2533 \cdot \text{in}$$

$$L_{db} := \max(L_{dbt}, L_{dbmin}) \quad L_{db} = 34.6606 \cdot \text{in}$$

COMPRESSION: (ACI 12.3.2)

$$L_{dbc1} := \frac{.02 \cdot d_{bpier} \cdot f_y}{\sqrt{f_c \cdot \text{psi}}} \quad L_{dbc1} = 24.6475 \cdot \text{in}$$

$$L_{dbmin} := 0.0003 \cdot \frac{\text{in}^2}{\text{lb}} \cdot (d_{bpier} \cdot f_y) \quad L_{dbmin} = 20.25 \cdot \text{in}$$

$$L_{dbc} := \text{if}(L_{dbc1} \geq L_{dbmin}, L_{dbc1}, L_{dbmin}) \quad L_{dbc} = 24.6475 \cdot \text{in}$$

Available Length in Foundation:

$$L_{pier} := L_p - C_{vr_pier} \quad L_{pier} = 87 \cdot \text{in}$$

$$L_{pad} := T_f - C_{vr_pad} \quad L_{pad} = 33 \cdot \text{in}$$

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

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

NOTE: Anchor bolts and plate provided, OK

TIE SIZE AND SPACING IN COLUMN

Minimum Tie Size:

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

$$Tie_{min} = 3$$

Used #4 Ties

$$d_{Tie} := 4$$

Seismic factor:
(ACI 21.10.5)

$$z := \text{if}(Z \leq 2, 1, 0.5)$$

$$z = 1$$

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

$$s_{lim1} = 18 \cdot \text{in}$$

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

$$s_{lim2} = 24 \cdot \text{in}$$

$$s_{lim3} := D_f \cdot z$$

$$s_{lim3} = 120 \cdot \text{in}$$

$$s_{lim4} := 18 \text{in}$$

$$s_{lim4} = 18 \cdot \text{in}$$

Maximum Spacing:

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

$$s_{tie} = 18 \cdot \text{in}$$

Number of Ties Required:

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

$$n_{tie} = 5.6667$$