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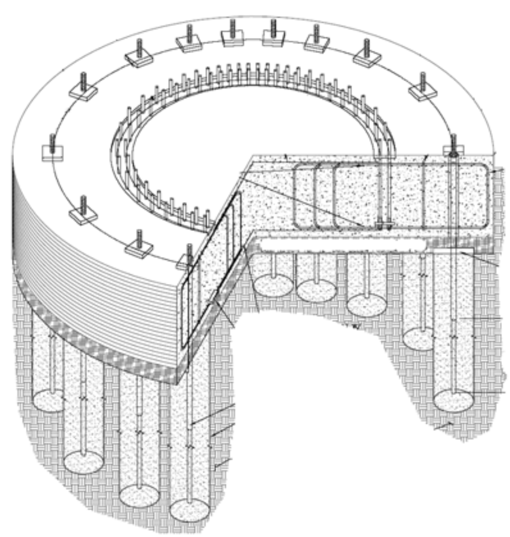
ENGINEERING DESIGN AND ANALYSIS OF THE PATRICK AND HENDERSON ROCK ANCHOR FOUNDATION

Stetson Mountain Wind Project, Danforth, Maine

38 – GE Wind 1.5sle 1.5MW on 80-m Hub Height Towers

14 – 40 to 50-foot Rock Anchors on 20-foot Ring with 24-foot Diameter Cap
using Williams 2-1/2-in diameter Grade 150 ksi Anchors or Equal

September 16, 2007



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ESSW Job No: 09824-55
P&H Job No. 07-036

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The Patrick and Henderson (P&H) rock anchor foundation is a proprietary (patent pending) foundation used to support wind turbines on monopole steel towers. This narrative presents the engineering design and analyses of the structural and geotechnical strength, stability and stiffness of the rock anchor foundation. The internal structural analysis of the cap is presented from Patrick and Henderson, Inc. *This document is for limited distribution as it contains confidential and proprietary information subject to confidentiality restrictions. This document and all related calculations and correspondence are the intellectual property of Earth Systems Southwest and is copyrighted with all rights reserved.*

Description

The P&H rock anchor foundation consists of a 5-foot thick, 24-foot diameter, reinforced concrete mat (cap) supported by 14 – mostly 40 feet deep (minimum), rock anchors aligned within a 20-foot diameter circle. For turbine sites with rock at depths of 10 feet or less from top of the foundation, 40 foot long rock anchors are adequate and the mud mat should extend to the top of rock. **For Turbine Sites T4 and T33 with the depth to rock at about 12.9 and 9.8 feet below expected mud mat elevations, respectively, the anchor lengths should be extended to 50 feet.** *The rock anchor foundation size and anchor lengths are based in part on the successfull completion of the Mars Hills, Maine project using the same foundation system with the same wind turbine and similar geologic conditions.*

Williams 2 ½ inch diameter rods are used within the rock anchors. The number and length of anchors are based on the magnitude of applied loads at the top of the foundation as well as the soil and rock conditions. The rock anchors are installed by drilling a shaft and filling the shaft with the anchor rod and grout. Post-tensioning is used to develop an internal tension force in each anchor. The intention is to have enough tension in the rod to reduce foundation deflections from normal wind loading.

The vertical loads are transferred through a base plate connection at the top of the circular plate with a double bolt circle (inner and outer bolt rings). Axial loads are transferred along the circular plate circumference via the concrete slab to the subgrade and circumferential rock anchors. The rock anchors then transfer the axial loads into the soil or rock through skin friction. Lateral loads are resisted by base shear friction at the bottom of the cap and lateral pressures along the anchors.

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Overturning moments are resisted by skin friction along the length of the rock anchors. The anchors are pre-tensioned to create a compression stress on the subgrade beneath the cap. The prestress effect resists the overturning moment by creating a built-in clamping force to keep the cap in place. The higher the pretensioning of the anchors results in a higher moment capacity of the foundation. The optimum point of prestress is a function of the resisting moment versus demand and factor of safety. The prestress in the anchor is calculated to cover the load imposed by maximum wind forces given by the turbine manufacturer. Therefore, normal operational loads will not produce stress reversals in the subgrade/concrete interface or anchor. There is no substantial cyclic degradation of the soil or rock.

Design Criteria and Assumptions

Referenced Code, Standards, and Industry Recommended Practices

The design of P&H rock anchor foundations uses a combination of national and international codes and standards. In general, the design of wind turbines foundations should be in accordance with the referenced national standards, codes of practice, legislation, licenses and consents applicable to the design and construction of the wind turbine foundation at each particular wind turbine location. In addition, the design of the wind turbine foundations should also be in accordance with the requirements of the International Electrotechnical Committee ("IEC"). The structural design loads and factors within the national codes of the United States (e.g. the International Building Code, the Uniform Building Code and ASCE-7, *Minimum Design Loads for Buildings & Structures*) were developed specifically for building structures. These codes consider wind turbines as non-building structures. The IEC developed the IEC 61400-1 standard specifically for wind turbines through extensive research.

The governing code for most jurisdictions in the United States is the International Building Code (IBC). Structural loads and factors for the foundation are derived in combination with the IEC 61400-1, "*Wind Turbine Generator Systems, Part 1: Safety Requirements*" and from Chapter 16 of the IBC that has its basis from the ASCE 7 standard. Chapter 18 of the IBC addresses foundation design in a general manner, but without definitive design equations applicable for the geotechnical design of the P&H anchor foundation. The structural design is in accordance with Chapter 19 of the IBC that has its basis on the requirements of the American Concrete Institute's "*Building Code Requirements for Structural Concrete*" (ACI 318). The design of the anchor themselves are based on the Post-Tensioning Institute, "Recommendations for Prestressed Rock and Soil Anchors".

Design manuals from the US Departments of Defense and Transportation and supplemented by peer reviewed technical journal articles from the American Society of Civil Engineers (ASCE) provide design guidance for the analyses as cited in the references. The "*Guidelines for Design of Wind Turbines*" published by DNV and Risф National Laboratory in Denmark provides general design guidance.

The following codes, standards and industry recommended practices are the main references used in the calculations and this document:

1. American Concrete Institute Building Code Requirements for Reinforced Concrete (ACI 318)

2. American Concrete Institute Suggested Design and Analysis and Design Procedures for Combined Footings and Mats (ACI 336.2)

3. Federal Highway Administration Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems (FHWA-IF-99-015)

4. International Building Code, 2003 edition

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5. Post-Tensioning Institute Recommendations for Prestressed Rock and Soil Anchors (PTI)

In addition to the codes and geotechnical references noted above, the design of wind turbine foundations for a particular site should also be in accordance with a site-specific geotechnical report. The wind loads applied to each particular wind turbine foundation should be in accordance with the site-specific wind load data provided by the wind turbine manufacturer.

Design Criteria

Design Criteria: The P&H rock anchor foundation is designed for use with a GE Wind 1.5sls MTS 1.5 megawatt ("MW") wind turbine generator ("WTG") with an 77-meter ("m") rotor diameter and a 80-m hub height. This WTG is designed by Gamesa in accordance with the International Electrotechnical Commission ("IEC") requirements for the IEC Class II_A wind load regime indicated in Table 1. Based upon these parameters, GE Energy provided the wind loads in a document entitled "*Foundation Data for Wind Turbine Generator Systems, GE 1.5sls MTS, 61.4-85m HH, IEC TCIIa with reduced gust (V_{e50} = 55m/s), LM 37.3P2, 50&60 Hz, GE 37c 60 Hz, dated March 23, 2006*. In this document, GE Energy provided foundation loads for the extreme wind for seven different cases, as shown in Table 2 (Earthquake loading not shown, does not govern for this site). Additionally, GE Energy provided minimum stiffness requirements for rotation and translation, as shown in Table 3.

Table 1
IEC IIA Wind Load Parameters

Annual Average Wind Speed (m/s)	8.5	
Annual Average Air Density (kg/m^3)	1.225	
Reduced Extreme Gust (3 sec, m/s)	55	(per GE)
Average Turbulence Intensity (I15)	0.16	

Design Loads: The most relevant design loads provided in the load document for the GE Wind 1.5 MW turbines for the project are summarized in the table below.

Table 2
GE Wind 1.5sle 1.5 MW, 77-m rotor on 80 m HH tower, Foundation Loads

Load Case Description	Blade Type	Design Load Case DLC	Load Factor LF	Vertical Fz KN	Horizontal Fr KN	Moment Mr KN-m	% M extreme
4.1 Extreme Load	GE37c	6.1j	1.00	-1844.5	523.4	34907.7	100%
4.2 Lift-off	LM37.3p2	1.0	1.00	-1958.9	219	17596.2	50%
4.3 Tilting	GE37c	6.1j	1.00	-1844.5	522.6	34907.7	100%
4.4 Sliding	GE37c	6.1j	1.00	-1843.4	523.4	34863.1	100%
4.5 Shear Failure	GE37c	6.1j	1.35	-2490	705.6	47125.1	135%
4.6Tension Loading	LM37.3p2	1.1	1.00	-1955.4	327.4	24238.1	69%

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Table 3
GE Wind Minimum Stiffness Requirements

Description	
Minimum stiffness for rotation about a horizontal axis	30 GN-m/radian

Geotechnical Parameters

The geotechnical borings by SW Cole and the geophysical investigation by Hager Richter Geosciences, Inc. are the basis for the geotechnical properties of the rock selected for the analyses. Representative rock core borings have been made to characterize each of the 38 turbines sites. The subsurface profile generally consists of dense glacial till of sands and gravels to variable depths to up to about 21.6 foot depth, but typically less than 6 foot depth. Beneath the soil overburden, generally lies variably weathered, fractured, meta siltstone (pelite) and volcanic rock (musdstone). The compressive strength of intact rock cores ranges from 2360 to 28090 psi, but generally above 3120 psi, and averaging about 6200 psi.

Design Methodology

Serviceability limit state (SLS) design is based on normal and fatigue wind loading events. Ultimate limit state (ULS) design is based on extreme wind loading events.

Load Cases

Based upon the information in ACI 318 and IEC 61400-1, the following load cases are considered in the design of the P&H rock anchor foundation:

Case 1 - Foundation Stability

Overturning and bearing strength for the foundation under unfactored Extreme wind loads

Case 2 - Foundation Strength

Strength of the foundation under factored Extreme wind loads

Partial Load Factors

The design of the foundation should consider the applicable partial load factors for ultimate limit state and serviceability design. For ultimate limit state design using Extreme and Abnormal conditions, the partial load factors in Table 4 are used. For the serviceability limit state design, the partial load factors in Table 5 are used.

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Table 4
Load Factors for Ultimate Limit State Design
Extreme Wind Loading - DLC 6.2

Parameter	Case 1 Stability	Case 2 Strength
Wind Turbine Load[(1)]	1.0	1.35
Dead Load (unfavorable)[(2)]	1.0	1.1/1.35
Dead Load (favorable)	0.9	0.9
Hydrostatic Pressure (unfavorable)	1.0	1.5
Hydrostatic Pressure (favorable)	0.0	0.0
Minimum Permissible Factor of Safety	1.5	n/a

(1) For seismic loading, the "wind turbine load" consists of the required seismic load applied to the wind turbine structure combined with the Normal Operating load condition. Both sets of loads are factored by the same partial load factor shown in the table.
(2) For manufactured wind turbine components, and materials that exhibit ductile behavior (e.g. anchor bolts), a partial safety factor of 1.1 may be used. For all other elements of the foundation system, a partial safety factor of 1.35 is used.

Table 5
Load Factors for Serviceability and Fatigue Limit State Design
Normal Operating Loads - DLC 1.1

Parameter	Serviceability Checks	Case 3 Soil/Rock Stability	Fatigue Life Assessment
Annual Turbine Load	1.0	1.0	n/a
Equivalent (20-year) Fatigue Load	n/a	n/a	1.0
Dead Load (unfavorable)	1.0	1.0	1.0
Dead Load (favorable)	0.9	0.9	1.0
Hydrostatic Pressure (unfavorable)	1.0	0.0	1.0
Hydrostatic Pressure (favorable)	0.0	0.0	0.0
Limiting Design Value	Per Table 6	3.0	0.33

Serviceability Criteria

The design of foundations considers the action of the foundation under the applicable serviceability loads and estimated foundation movements and settlements. Additionally, the structural calculations demonstrate that the foundation will remain serviceable after the application of the Extreme and Abnormal loadings. Allowable movements and settlements under serviceability loadings are indicated below.

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Table 6
Serviceability Criteria

Average Settlement (max mm)	Rotation (max radians)	Minimum Rotational Stiffness (GN-m/radian)
25	(0.001) Operational load (0.002) E-stop load (0.004) Extreme load	30

Rock anchor Foundation Components

The P&H rock anchor foundation consists of a circular reinforced concrete cap supported by post-tensioned (also called prestressed) rock anchors. The cap measures 24-feet in diameter, with a thickness of five feet. The 14 rock anchors that support the cap are equally spaced on a 20-foot diameter circle. Each rock anchor consists of a 40-foot deep, 5-inch diameter grouted hole that is post-tensioned using 2½-inch diameter, 150 kips per square inch (ksi) anchor rods that extend full depth.

Table 7 summarizes that key design parameters and specification shown on the drawings and the calculations.

Table 7
Summary – Design and Specification

Item & Parameter	Specification	No. & Dimensions
Cap - Concrete Strength	6,000 psi	24 foot diameter x 5 feet thick
Rock Anchor - Grout Strength	3,000 psi	14 -5 inch diameter x 40 feet long (mimimum)
Tower Anchor Bolt - Strength	Grade 150	112 - 1.375 inch diameter x 66 inch long
Tower Anchor Bolt - Prestress	120 kips	
Rock Anchor Bolt - Strength	Grade 150	
Rock Anchor Bolt - Prestress	435 kips	(minimum required 319 kips)
Rock Anchor Base Plate - Strength	50 ksi	14 - 12 inch square x 2.5 inches thick
Embedment Material	36 ksi	
Grout beneath Flange - Strength	10,000 psi	
CMP Material	A-929 or A-444	28 foot diameter x 5 feet long

Rock anchor - Free Stress Length = 10 feet
Rock anchor - Bonded Length = 28.5 feet (minimum in rock)

QA/QC Load Testing

A Patrick & Henderson, Inc. representative shall be present during anchor installation to verify, and to document via the checklists provided by P&H, that the conditions encountered are consistent with the conditions as indicated in the design calculations. All anchors will be tested in accordance with PTI recommendations for prestressed anchors. All anchors will be proof-tested to 133% of the design post-tensioned load before lockoff. At least one anchor per foundation will be performance tested with load and reload cycles.

The apparent free length of the anchor, La, may be calculated using the following equation:

$$La = \delta\, E\, At / P$$

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Where, At is the cross sectional area of the prestressing steel, Es is the Young's modulus of the prestressing steel, δ is the elastic movement at the test load, and P is equal to the test load minus the alignment load.

The minimum apparent free length is defined as the jack length plus 80 percent of the design unbonded length. The maximum value of apparent free length is restricted to elastic movements of 100 percent of the free length plus 50 percent of the bond length plus the jack length.

DESIGN METHODS

Part A - Geotechnical Design and Analysis (external force analysis)

Unless otherwise noted, the geotechnical design follows the recommendations of the site-specific geotechnical report. Unless otherwise noted, bearing capacity and lateral resistance calculations are in accordance with the parameters given with the recommendations of the site-specific geotechnical report.

Moment Capacity: The analysis must fundamentally demonstrate the rock anchor foundation is reasonably safe against complete upset by overturning. A global safety factor of at least 1.5 should be used in the ultimate limit-state (ULS) analysis for extreme wind loads as discussed in detail below.

Dynamic Stiffness: Dynamic analyses of the foundation are conducted to verify that the stiffness of the combined foundation-soil system is such that resonance and excessive vibrations (that potentially could lead to premature fatigue of the tower) are minimized. The stiffness of the foundation must avoid the critical first and third vibrational mode frequencies to limit dynamic amplification or resonance.

Rotational stiffness plays an important role in the foundation performance. Rotational stiffness typically expressed in units of GN-m/radian is defined as Moment (M) divided by rotation (θ) expressed in radians. To verify that the foundation-soil system remains outside of the critical frequency range, the rotational stiffness is evaluated and checked to whether that it meets the rotational stiffness requirements of the turbine manufacturer. It must be sufficiently large so as not to significantly alter the total system frequency of tower/nacelle/foundation to change the turbine and tower manufacturers fatigue load assumptions. Evaluation of the stiffness of the P&H anchor foundation system is based upon the vertical dead loads (including sustained post-tension force) and the maximum mean overturning moment due to operational loads specified by the wind turbine manufacturer's fatigue load analysis.

Design Post-Tension Load: The cap is structurally connected at the top bearing plate to the anchors. These high strength steel anchors that are post tensioned to maintain contact between the cap and anchor during most of the loads. The post-tension load is selected so that the cap and the anchors remain in contact for the normally encountered operational loads, and to minimize the deflections during a design basis event. It is not required for structural adequacy, to fully tighten the anchors to the value of tension that would occur due to the extreme design load. An anchor tension somewhat less than the maximum tension may be used.

After the post-tension load is determined, the maximum tension in the anchor is estimated. The anchor capacity is checked against pull-out with respect to the estimated maximum tension load. The mode in which the rock anchor resists the tension or compression load is via bond stress. Bond stress is the skin friction between the grout and the surrounding rock. The foundation is anchored (bonded) at depths of 10 to 44 feet. The foundation is checked against the design overturning moment similar to a spread foundation on rock except the rock anchors induced a tremendous vertical restraining force that keeps the foundation cap clamped down.

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Computations

Anchor Structural Capacity: The structural capacity of the rock anchor is limited by both the allowable tension load on the anchor taken as 60% of ultimate strength of the anchor rod, 0.6Fu.

Anchor Pullout (Geotechnical) Capacity: The anchor pullout capacity is a function of bond stress between the rock and the grout. Bond stress is dependent on the soil/rock type, condition, and the method of grouting. The ultimate geotechnical uplift capacity of the rock anchors for is given by the summation of perimeter skin bond stress:

$$Qp = \sum_{z=0}^{z=l} fs * \pi * d * \Delta z$$

where,

fs =ultimate bond stress, see discussion below

d = diameter of anchor or anchor group as shown below, used nominal diameter.

Δz = increment of depth along anchor bonded length

l = bonded length of anchor below cap

Based on the in-situ condition of the rock, the following ultimate bond stress value was assigned to the rock:

Rock Type & Condition	Ultimate Bond Stress
Fractured weathered rock	150 psi (1 MPa)

This value is based on PTI recommendations for rock type, and our experience with the successfully completed Mars Hill Maine project with similar geologic conditions and is only about 5% of the lower bound of of intact compressive rock core strengths.

The length of anchor within the soil that will resist the design tension load is called the bond length. After the bond length is determined, a second calculation is performed to check that there is enough soil/rock mass above the bond length to resist the design loads. The rock mass that is considered for resistance of uplift is a cylinder and cone extending from the perimeter of the anchor group.

Moment Capacity and Overturning Stability

P&H Design Methodology: The preliminary design of the P&H rock anchor foundation is based on a simple premise to specify the anchor rod size, number of anchors, and anchor circle diameter so that the post-tensioning force (when set near the allowable structural capacity of the rod) is greater than the force couple created by the characteristic (unfactored), extreme wind overturning moment. The force couple is given as:

M = nPD/4,

Where,

P = maximum tension or compressive force in the direction of loading,

M = moment,

n = number of anchors = 14, D = effective diameter of anchor circle = 20 feet.

Hence the post-tension load, P is set near or above $4M_{(extreme)}/nD - 0.9DL = 319$ kips,

where DL = dead weight of foundation cap and turbine

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Typically the specified Ps near 0.6Fu = 435 kips, to allow for at least 10% creep losses.

In that all anchors are proof-tested to 133% of the post-tensioned, they are in effect tested to the factored extreme wind load.

Overturning Stability with Post-Tension Load on Anchors: The overturning stability is evaluated similar to a spread foundation, whereby the load eccentricity (e) is defined by e = M/V, where M is the net overturning moment at the base of the cap and V is the sum of vertical forces (including the post-tensioned anchor force). In accordance with ACI 336.2-R88, the Stability Ratio (SR) (aka safety factor, FS) is evaluated to verify a SR of least 1.5 using the unfactored extreme wind load considering the R_{vmin} defined as the least resultant of all forces (including preload) acting perpendicular to base area under any condition of loading simultaneous with the overturning moment. The stability ratio (SR) or factor of safety (FS) against overturning is taken as FS = e_{max} /e. The net driving moment M_D is the sum of driving moments at the base (computed as Fx*(ez + h) – (any reliable passive and vertical side friction moment along the cap, generally neglected). The resisting (restoring) moments M_R is the base (or anchor group) reaction (R) at maximum allowable eccentricity, e_{max}, taken at e_{max} = 0.5B for solid rock (at point of over toppling). The factor of safety becomes There are two stability checks considered as follows:

Serviceability Limit State (SLS)

The GL Rules for wind turbine foundation design state that for spread and piled foundations no uplift should exist beneath the foundation base or on the anchors for maximum normal production loading. This implies that the relative eccentricity ratio, e/B < 1/6, so that base reaction are in compression across the full cap area and e/Da < 1/6 indicates that no net uplift occurs on the anchors at the anchor circle diameter, Da. This is easily satisfied as the foundation is designed

Foundation Stiffness Analysis

The stiffness of the rock anchor foundation system is dependent on the combined compressibility of the subgrade beneath the cap and the anchor system and a minor component of the rigidity and compressibility of the cap itself. To compute the stiffness of the anchor system requires the consideration of the stiffness of the various components of the foundation and how they interact with each other. From basic engineering mechanics, the following relationships are developed for springs in series or in parallel to each other.

	Series	Parallel (two springs)
Deflections (and Rotation)	$\delta_T = \delta_1 + \delta_2 + \ldots \delta_n$	$\delta_T = \delta_1 = \delta_2$ or $\theta_T = \theta_1 = \theta_2$
Forces (and Moment)	$P_T = P_1 = P_2 = \ldots P_n$	$P_T = P_1 + P_2$ or $M_T = M_1 + M_2$
Spring Stiffness	$K_T = 1/(1/K_1 + 1/K_2 + \ldots 1/K_n)$	$K_T = K_1/F_1 = K_2/(1-F_1) = K1 + K2$ where $F_1 = K_1/(K_1 + K_2)$

And in general, K = AE/L, and δ = PL/AE, δ = P/K where,

P = Force (load), M = Moment, L = length of member under stress, A = Cross sectional area, E = elastic modulus, K = spring stiffness, δ = deflection, θ = rotation, subscript T is total, and subscripts 1 through n are individual elements.

The post-tensioned anchor rod (a) and the Prestress Zone of the subgrade (pz) acts as two springs in parallel to each other when external loads (P_T) are applied ($P_T = P_a + P_{pz}$). After post-tensioning the rod, only a portion of the load fluctuations are transmitted to the rod with the remaining load distributed through the bearing area, since the rod stiffness is far less the bearing stiffness (conversely rod elongation is far more than the bearing compression).

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So long as the preload is not exceeded, the rod stress remains relatively constant, hence little concern for fatigue and cyclic degradation.

During post-tensioning, several elements are in series to each other, each contributing to the compression deflection as a tension load is elongating the rod. These include:

Bearing Compression of the Top Anchor Plate (minor),

Local Bearing Compression of the Concrete beneath the Anchor Plate (minor),

Bearing compression of the Cap base (minor), and in parallel to each other:

Bearing compression of the Subgrade beneath the Cap (major component)

Compression of the Grout around the Rock anchor.

Anchor Rod tension causes mobilization of the total anchor perimeter skin friction, elongation of the free stress (unbonded) rod length, and elongation of an effective length along the bonded portion of the anchor. The skin friction and rod elongation are in series to each other.

Local compression (settlement) of the concrete bearing and subgrade compression (settlement) can be computed from the following classical equation:

$\delta = qBIp(1-\nu^2)/E$

Where, B = bearing width, q = contact pressure = P/A, ν = Poisson ratio

Ip = vertical strain influence factor that is dependent on the variation of modulus with depth.

Ip = $\pi/4$ and 1 for circular and square disks, respectively on elastic half space

E = elastic modulus, for soils and rock.

<u>Combined Cap Subgrade and Rock anchor System Stiffness:</u> The cap bearing on subgrade and the rock anchors act in parallel to each other with applied overturning moments. So from the preceding relationships, the proportion of moment capacity carried by the anchors can be derived if the anchor and the subgrade are assumed to be independent of each other:

Total moment = M_T =

Moment resisted by anchors (M_a) + moment resisted by cap bearing on subgrade (M_{pz})

Rotation, $\theta_{pz} = M_a/K_{\theta a} = M_c/K_{\theta pz}$,

Where, $K_{\theta a}$ and $K_{\theta pz}$ are the rotational stiffness of the anchor group & prestress zone of cap subgrade, respectively

Contribution of anchor group factor, $F = M_a/ M_T = K_{\theta a} /(K_{\theta a} + K_{\theta c})$

However, there remains additional rotation (θ_{lz}) below the prestressed zone (the lower zone, subscript lz) in series so that the total rotational stiffness of the foundation systems is:

$K_{\theta T} = [(K_{\theta a} + K_{\theta pz})^{-1} + K_{\theta lz}^{-1}]^{-1}$

Rotational Stiffness of Anchor Group – Simplified Approach

A general simplified approach to compute rotational stiffness of an anchor group is given Mokwa and Duncan (2003) by:

$$K_{M\theta} = \frac{\Delta M}{\Delta \theta} = \sum_{i=1}^{n} [K_{\Delta c}(x_i - a)^2] + \sum_{i=1}^{n} [K_{\Delta t}(x_i - a)^2]$$

But for a circular anchor group, the rotational stiffness of the anchor group is simpler still and may be computed as:

$K_{M\theta} = nDaKa^2/8$

Where n = number of anchors, Da = diameter of anchor circle, and Ka = axial stiffness of the anchor. The maximum tension or compression load to the anchor assuming a neutral axis at the centerline is P = 4M/nDa, where M = moment carried by rock anchor group. The maximum deflection of the anchor δa may be computed by θDa/2

The elastic elongation of the anchor may be computed from:

δ = P/Ka,
where K_a = Ka = axial stiffness of anchor = ArEr/La
Ar = cross sectional area of anchor
Er = Modulus of Elasticity of Steel
La = active length of anchor

The active length of the anchor, La may be variable, dependent of the nonlinear distribution of transfer of bond stress and skin friction along the length of the anchor. At minimum it is the unbonded length of the anchor, Lu, but as maximum it may be a portion of the unbonded length plus one half of the mobilized bonded length, Lb. We have used a 15 foot active length, that is the unbonded length plus one half of the estimated mobilized bonded length with a safety factor of a least two against ultimate bond stress. For prestressed anchors, the axial stiffness of the anchor is the sum of the rod stiffness and subgrade stiffness, so only a portion of the load induced from applied moments is carried by the anchor rod, with the remainder carried by change in compression of the subgrade and bearing plate.

Other minor, but quantifiable, component of deflection is associated with the eccentric loading from the transfer of loads from tower bolt circle to the anchor circle (similar to cantilever action in a very deep beam).

Stiffness of Cap Subgrade by Elastic Half Space (EHS) Method

Dynamic spring stiffness of the cap may be derived from the classical, closed form equations for an embedded rigid block within an elastic half space (EHS) theory. This method is used commonly for design of machine foundations and spread footings for wind turbines. The dynamic spring stiffness values may be derived using the following published equations from the *Guidelines for Design of Wind Turbines* (DNV and Risϕ National Laboratory, 2002). These basic equations may be modified by depth to stiffer (rigid) layer of embedment factors subject to the limitations of validity.

Vertical Stiffness at base: $K_z = 4Gr/(1-\nu)$
Horizontal Stiffness at base: $K_x = 8Gr/(2-\nu)$
Rocking Stiffness at base: $K_\theta = 8Gr^3/[3(1-\nu)]$

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Dynamic Soil or Rock Properties

The dynamic soil properties are derived for estimated average values of the subsurface profile beneath the foundation. The maximum shear modulus, G_o is the most relevant property for dynamic analyses. The maximum (low-strain) shear modulus is related to the shear wave velocity, Vs, and mass density, ρ by the equation:

$$G_o = \rho V_S^2$$

But, also $G = E/[2(1+\nu)]$

Stiffness Influence Factors for Layered Media

By extension of elastic theory we compute the moment/rotation influence factors (ΔI_θ and I_θ) for layered media, analogous to the vertical strain influence factors. In the case of rotations and rotational stiffness we have interchanged elastic modulus, applied pressure and incremental displacements for shear modulus, applied moment and incremental rotations, derived from I_θ, respectively. Rotations within each layer are:

$$\theta = M^*[8Gr^3/3(1-\nu)]^{-1} {}^* I_\theta$$

<u>Derivation of Moment/Rotation Influence factors (ΔI_θ and I_θ):</u> To ESSW knowledge, there are no published, closed form solutions for the complete layer influence factors at shallow depths for moment/rotation as there are for vertical pressures and strain. However, a reasonable relationship can be developed from what is published and understanding of basic elastic theory.

DNV/Risϕ (2002) cite the stiff (rigid) layer factor for rotational stiffness as $N_\theta = 1+R/6H$, where H is depth to a stiff (rigid) layer beneath of the foundation, and R is the radius of the foundation, but with a range of validity of $H/R > 0.75$, and cited elsewhere as $H/R > 2$ (DOD, 1997). Note that this is significantly less than the corresponding stiff (rigid) layer factor for vertical stiffness as $N_V = 1+1.28R/H$. This indicates that the majority of the stiffness and conversely rotations occurs nearer to the base of the foundations than vertical displacements.

The cumulative displacement or rotational influence factors (Ih or I_θ) are merely the inverse of the layer stiffness factors, that is to say, $Ih = 1/N_V$ and $I_\theta = 1/N_\theta$. These layer stiffness and influence factors are plotted in the attached results. Note however, that $I_\theta = [1+R/6H]^{-1}$ becomes invalid at about h/d = 0.9 as that the maximum I_θ factor can not exceed a value of [h/d], as this implies no attenuation of moment with depth and is the maximum possible influence factor for shallow depth in the range of invalidity for the equation. We have derive a reasonable equation that smoothly transitions from I_θ = [h/d] in the shallow zone to $[1+R/6H]^{-1}$ in the deep zone. We observe the Boussineq equation for vertical stress, but exchanging exponents will result in a ΔI_θ such that the cumulative I_θ relationship will provide such a reasonable and smooth transition.

Hence we derive: $\Delta I_\theta = 1 - [1 + (h/r)^{-4}]^{-7}$

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Part B - Structural Design and Analysis

Design Methods

The structural design analysis for the reinforced concrete pier and the anchor bolts consists of Microsoft Excel spreadsheet calculations that employ traditional design methods of the American Concrete Institute (ACI). The calculations include an input page that records the inputs for the remaining analyses.

The design analysis for the reinforced concrete cap, the anchor bolts, and the rock anchors employed traditional design methods. The concrete design for the anchors was based upon the criterion of the American Concrete Institute (ACI) and the Post--Tensioning Institute (PTI) Recommendations for Prestressed Rock and Soil Anchors.

The rock anchor foundation stiffness and deflections are calculated considering a combination of elastic compression/elongation of the anchors. The stiffness of the foundation system is ultimately calculated using the combined stiffness contributed by the cap on the subgrade based on the elastic half space method, and the calculated stiffness of the anchor group.

Design Assumptions

The calculations make the following assumptions with regard to the behavior of the cap/rock anchor system:

1. Vertical loads are transferred through the base plate to the concrete cap. The cap, in turn, transfers these axial loads to the subgrade and the circumferential rock anchors. The axial loads are then transferred to the soil from the rock anchors via skin friction.
2. Post-tensioning of the Rock anchors creates a compression stress in the subgrade that creates a clamping effect. This clamping effect is used to resist overturning moments in the foundation.
3. Due to the clamping force of the prestressed rock anchors, lateral loads on the cap are resisted by shear friction (or cohesion) between the base of the cap.
4. Overturning moments are resisted by skin friction along the length of the rock anchors.
5. The forces required to post-tension the WTG anchor bolts and the rock anchor bolts are calculated to exceed the load imposed by the maximum wind forces specified by the WTG manufacturer. It is therefore assumed that the normal operational loads of the WTG will not produce stress reversals at either the soil/concrete interface, in the WTG anchor bolts or in the Rock anchors. Further, no cyclic degradation of the soil is assumed to occur.

Structural Analysis

Cap Analysis and Design

The basic structural design calculations were performed by P&H. The basic design calculations cover all of the concrete design associated with the cap and the appurtenances required to anchor the wind turbine generator to the cap. The calculations produced by P&H consist of structural calculations that are based on the American Concrete Institute (ACI) design requirements for reinforced concrete. The analyses are all based upon linear elastic theory.

The calculations include analyses of the following items:

(i) analysis of the cap for flexural and shear capacity and reirnforcement,

(ii) anchor bolt analysis,

(iii) bearing on grout and concrete, and

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(iv) shear analysis.

All of the calculations prepared by P&H use the extreme wind load moment as the basis of design. For design based on the extreme wind load, the IEC specifies a partial load factor of 1.1 or 1.35 to be applied to the design load.

The cap analysis is based on the assumption that the cap will be required to resist bending that is induced by the transfer of forces between the tower anchor bolts and the Rock anchors. In this analysis, P&H assumes that the cap acts as a cantilever that is fixed at the location of the tower anchor bolts; the load is assumed to act at the location of the rock anchors. Further, the analysis considers a beam width that is equal to the tributary width of the tower anchor bolts calculated using the average of the anchor bolt and rock anchor bolt circles. Based on the use of the specified reinforcement, the P&H calculation determines the capacity of the concrete section considering the cases of a single layer of reinforcement and also as a doubly reinforced section. For both cases, the ultimate strength of the concrete section is calculated to exceed the applied loading.

The analysis of the tower anchor bolts is accomplished via an analysis that factors the extreme wind loads in generally in accordance with the provisions of the ACI code; for this analysis, the dead load factor is increased above that specified in the reference equation cited by P&H. The force in the anchor bolts is calculated by conservatively combining (absolutely) the tensile load induced by overturning of the tower with the compressive load associated with dead load of the tower and nacelle. The resulting bolt “tension” was compared to the yield strength of the anchor bolts and shown not to exceed said yield strength.

P&H uses the calculations of the bearing stresses on the grout and on the concrete as its basis for the required grout and concrete design strengths. The calculations of the compressive loads under the tower base flange are calculated using the combined compressive loads induced by the dead load and extreme wind load overturning moment. This analysis is based upon the use of factored extreme wind loads; the loads are factored using the same method as discussed in the tower anchor bolt analysis. Additionally, the prescribed post-tension load (unfactored) for the tower anchor bolts was also applied to determine the maximum amount of compression under the tower base flange.

To further confirm the required design strength of the grout beneath the tower base flange and the required concrete design strength, P&H performed an analysis using the serviceability requirements for prestressed concrete of the ACI code. For these calculations, the unfactored service loads (normal operating loads) were used as the basis of the analysis. In this analysis, both the concrete and grout were checked for resistance to forces caused by compression due to prestress plus sustained load and by compression due to prestress plus total load..

The base shear analysis shows that the lateral loads to due the extreme wind can easily be resisted by the anchor bolts themselves, without consideration of the additional capacity provided by the friction beneath the base plate.

The lateral loads are transferred from the cap to the soil and countered by the lateral sliding resistance of the cap due to the effects of dead load (turbine, tower and cap) plus the post-tensioned anchorage loads. In this case, the the cap and anchors are clamped together with the soil by the prestessing action. This set of conditions makes it highly unlikely that the soil at the base of the cap would separate from the cap. As a result, P&H assumes that the soil will remain in contact with the bottom face of the foundation. Thus, lateral loads can be transferred to the soil via the mechanism of skin friction/cohesion between the base of the cap and the soil mass below.

Shear Analysis

The base shear analysis shows that the lateral loads to due the extreme wind can easily be resisted by the anchor bolts themselves, without consideration of the additional capacity provided by the friction beneath the base plate. The result of this analysis generally indicates that the nominal shear strength of the concrete exceeds applied ultimate shear loading.

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Anchorage Analysis

The anchorage analysis uses the bolt forces from the concrete analysis to estimate the compressive forces in the concrete under the base flange. The Anchorage Analysis calculates the total load on the base flange as the force in the bolts (due to the sum of the prestress and the shortening described in the Concrete Analysis), that is, the force that is "clamping" the base flange to the underlying grout and concrete. By subtracting the area of the bolts from the overall area of the flange, the compression stress in the concrete under the base flange is calculated and favorably compared to the stress to the design strength of the concrete.

The Anchorage Analysis also considers the design of the lower end of the anchor bolt where it bears on the embedment ring to create the compression in the concrete. Since the embedment ring has the same area as the base flange, and since the tension in the anchor bolt is the same at the top and at the bottom, the compressive stress in the lower concrete is the same as it is in the top under the base flange. The concrete stress check is equally acceptable.

Axial Bearing Analysis

The calculations include an analysis of the axial concrete strength that includes the appropriate dead overturning and post-tensioning loads. The analysis of the axial loads is based on the requirements of Section 18.4.2 of the ACI Code. In this analysis, the required strength is calculated using the service load level (rather than the extreme wind); additionally, the analysis considers that for the transient loading case, the allowable stress level is adjusted with a one-third increase in the allowed stress.

Conclusions

The Patrick and Henderson rock anchor foundation is a relatively simple and innovative design. As designed, it has adequate capacity against upset from the high overturning moment imposed by the wind turbine structures and provides sufficient rigidity (rocking stiffness). ***For turbine sites with rock at depths of 10 feet or less from top of the foundation, 40 foot long rock anchors are adequate and the mud mat should extend to the top of rock. For Turbine Sites T4 and T33 with the depth to rock at about 12.9 and 9.8 feet below expected mud mat elevations, respectively, the anchor lengths should be extended to 50 feet and the leveling course (mud mat need only to be minimum thickness of 6 inches. The required preload for these sites may need to be increased to 435 kips to allow for overconsolidation and subsequent relaxation of the glacial till soil***

EARTH SYSTEMS SOUTHWEST

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Attachments: References
Appendix A: Geotechnical Analysis Calculations
Appendix B: Structural Analysis Calculations

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APPENDIX A
Geotechnical Analysis Calculations

Earth Systems Southwest

Project: **Stetson Mountain Wind Project** Danforth, ME
14 - 40-foot Anchors on 20-foot Ring with 24-foot Diameter Cap
using: Williams 2-1/2-in diameter Grade 150 ksi Anchors or Equal
Wind Turbine: **GEWE 1.5sle 1.5 MW on 80-m Hub Height Tower**

ESSW Job No.: 09824-55 Date: 09/19/07

GEOTECHNICAL FOUNDATION ANALYSIS OF P&H ROCK ANCHOR FOUNDATION
Analysis and Microsoft Excel Spreadsheet Developed by Shelton L. Stringer, PE, GE

FOUNDATION DIMENSIONS AND PROPERTIES								
Outside Cap (CMP) Diameter	***Do***	=	**24.00**	**feet**	=	7.315	m	
Cap Thickness	***Tc***	=	**5.00**	**feet**	=	1.524	m	
Embedded Cap Depth	***Lc***	=	**4.0**	**feet**	=	1.219	m	
Anchor Diameter	***Da***	=	**5.0**	**inches**	=	0.127	m	
Anchor Pile Length below Cap	***La***	=	**34.5**	**feet**	=	10.516	m	
Anchor Circle Diameter	***D***	=	**20.00**	**feet**	=	6.096	m	
Number of Anchors	***n***		**14**					
Concrete Compressive Strength	*f'c*	=	5,000	psi	=	34.5	MPa	
Density of Concrete	γ_c	=	150	pcf	=	23.6	kN/m^3	<u>Equations:</u>
Concrete Modulus of Elasticity	*Ec*	=	4,030,000	psi	=	27,800	MPa	$57,000*f'c^{0.5}$
Base Area of Footing	*Af*	=	452.39	sf	=	42.03	m^2	$\pi D^2/4$
Weight Concrete	*Wc*	=	339.3	kips	=	1,509	kN	$Af*L*\gamma c$
Volume Concrete	*Vc*	=	83.8	cy	=	57.31	m^3	*Af*L/27 (cy)*

GEOTECHNICAL DATA										
		Description	Estimated GSI	Unit Weight γ_T (pcf)	kN/m^3	Friction Angle ϕ (deg)	Mass Strength q_m (ksf)	(MPa)	Compressive Strength *qu* (psi)	(MPa)
		Till		130	20	40				
		Weathered Bedock	35	150	24		25	1.2	1450	10
GWT:		GWT - Design Groundwater Table								

FOUNDATION LOADS								
Extreme Loading	Based on: Foundation Data for GW 1.5sle, IEC IIA 80 m Hub Height, DLC 6.1j							
Horizontal Load	*Fxy*	=	**117.665**	kips	=	523	kN	exclusive of
Axial Dead Load	*Fz*	=	**439.637**	kips	=	1,956	kN	partial safety
Bending Moment at Base	*Mxy*	=	**25,746.608**	ft-kips	=	34,908	kN-m	factors
Height of Tower	*h*	=	262.47	feet	=	80.00	m	
Effective Application Height	*ez*	=	218.81	feet	=	66.69	m	

CALCULATION OF ROCK ANCHOR PULLOUT CAPACITY

Project: Stetson Mountain Wind Project
14 - 40-foot Anchors on 20-foot Ring with 24-foot Diameter Cap
using: Williams 2-1/2-in diameter Grade 150 ksi Anchors or Equal
Calculated by: Shelton L, Stringer, PE, GE — Date: 9/19/2007

GIVEN:		Foundation Data for GW 1.5sle, IEC IIA 80 m Hub Height, DLC 6.1j				
Horizontal Load	**Fx**	**118**	**kips**	**523.4**	**kN**	
Vertical Load	**Fz**	**440**	**kips**	**1955.6**	**kN**	
Design Overturning Moment	**M_D**	**25747**	**ft-kips**	**34907.7**	**kN-m**	
Check Bonded Length Pullout Capacity			Reference:	PTI (1996)		Equations
Anchor Body Diameter	db	5.0	in	0.42	ft	
Bonded Length in Rock	**Lb**	**28.5**	**feet**			
Ultimate Bond Strength	**Tw**	**150**	**psi**	Fractured, Volcanic or Pelite rock		
Anchor Pullout Capacity	Pa	**806**	**kips**			
Design Load per anchor	**Fp**	**319**	**kips**			
Factor of Safety against Pullout	**FS**	**2.5**				

Check Free Stress Stress Length & Pullout Capacity of rock mass (individual & group)

Input			References:	FHWA-IF-99-915 & USCOE EM 1110-1-2906		
Anchor Tilt	α	90	degrees	from horizontal		
Total Unbonded Length	**Lu**	**10.0**	**feet**			
Anchor Length above cap	La	1.5	feet			
Total Length of Anchor	**Lt**	**40.0**	**feet**			Lu + Lb + La
Anchor Circle Dia	Da	20.00	ft	ra	10.21	feet = Da/2 + db/2
No. of Anchors	n	14				
Spacing between anchors	x	4.49	ft			πDa/n
Potential Failure Starts at		0.5	Lb, middle of bonded length for individual anchor			
Soil/Rock Info:	Till		over	Rock		
Half Cone Shear Angle	ϕ	40	deg	60	deg	
Cohesion/Shear Strength	c	0	psf	0	psf =	rock mass cohesion intercept
Moist Unit Weight	γ	130	pcf	170	pcf	
Depth to Top of Rock or free stress	zR	10.0	feet			
Groundwater Depth	GWT	100.0	feet			

	Depth	Unit Wt	Pressure		
Vertical Soil Pressure	z (feet)	γ (pcf)	σ'$_v$ (ksf)		qrm = factored rock mass compressive strength
at Cap Base	4.0	130.0	0.52		
at bottom of 1st layer or GWT	10.0	130.0	1.30		
at top of rock, zR	10.0	130.0	1.30		
Effective Pressure @ top of rock	σ_v'		1.30	9.03 psi	

Calculations		Individual	Group	Equations
Effective Length	Leff	24.3 feet	38.5 feet	Lu + (0.5Lb- individual, Lb - group)
Total Overburden Depth	Zt	24.3 feet	38.5 feet	Leff*sin(α)
Eff. Embedment Depth into Rock	Z	14.3 feet	28.5 feet	Zt - zR
Radius of Cone of Rock	R	24.7 feet	49.4 feet	Z*tanϕ
Area @ top of Cone (individual)	Ac	1913.8 sf		πR^2
Area @ top of Cone (group)	Acg		796.3 sf/anchor	$\pi(R+ra)^2/n$
Volume of Rock Cone (individual)	Vc	9090.6 cf		$1/3\pi R^2 Z$
Volume of Rock Cone group)	Vcg		6491.1 cf/anchor	$1/3\pi Z(R^2+ra^2+raR)/n$
Weight of Rock Cone (individual)	Wc	1545.4 kips		$Vc*\gamma_R$
Weight of Cone (group)	Wcg		1103.5 kips/anchor	$Vcg*\gamma_R$
Surface Area of Cone (individual)	As	2209.9 sf		πRZ/cosϕ
Surface Area of Cone (group)	Asg		762.0 sf/anchor	π(R+ra)(Z/cosϕ)/n
Shearing Resistance (individual)	S	0.0 kips		As*fa
Shearing Resistance (group)	Sg		0.0 kips/anchor	Asg*fa
Pullout Capacity (individual)	**P**	**4033 kips**		Wc + S +σ_v'*Ac
Pullout Capacity (group)	**Pg**	**2139 kips/anchor**		Wcg + Sg +σ_v'*Acg
Preload Load per anchor	**Fp**	**319 kips**		
Factor of Safety against Pullout	**FS**	**12.64 individual**		
Factor of Safety against Pullout	**FS**	**6.70 group**		

CALCULATION OF ROCK ANCHOR PULLOUT CAPACITY

Project: Stetson Mountain Wind Project
14 - 40-foot Anchors on 20-foot Ring with 24-foot Diameter Cap
using: Williams 2-1/2-in diameter Grade 150 ksi Anchors or Equal
Calculated by: Shelton L, Stringer, PE, GE — Date: 9/19/2007

GIVEN:		Foundation Data for GW 1.5sle, IEC IIA 80 m Hub Height, DLC 6.1j				
Horizontal Load	**Fx**	**118**	**kips**	**523.4**	**kN**	
Vertical Load	**Fz**	**440**	**kips**	**1955.6**	**kN**	
Design Overturning Moment	**M_D**	**25747**	**ft-kips**	**34907.7**	**kN-m**	
Check Bonded Length Pullout Capacity			Reference:	PTI (1996)		Equations
Anchor Body Diameter	db	5.0	in	0.42 ft		
Bonded Length in Rock	**Lb**	**30.1**	**feet**			
Ultimate Bond Strength	**Tw**	**150**	**psi**	Fractured, Volcanic or Pelite rock		
Anchor Pullout Capacity	Pa	**852**	**kips**			
Design Load per anchor	**Fp**	**435**	**kips**			
Factor of Safety against Pullout	**FS**	**2.0**		For T-4 & T-33		

Check Free Stress Stress Length & Pullout Capacity of rock mass (individual & group)

Input			References:	FHWA-IF-99-915 & USCOE EM 1110-1-2906		
Anchor Tilt	α	90	degrees	from horizontal		
Total Unbonded Length	**Lu**	**18.4**	**feet**	Maximum at T-4		
Anchor Length above cap	La	1.5	feet			
Total Length of Anchor	**Lt**	**50.0**	**feet**			Lu + Lb + La
Anchor Circle Dia	Da	20.00	ft	ra	10.21	feet = Da/2 + db/2
No. of Anchors	n	14				
Spacing between anchors	x	4.49	ft			πDa/n
Potential Failure Starts at		0.5	Lb, middle of bonded length for individual anchor			
Soil/Rock Info:		Till	over	Rock		
Half Cone Shear Angle	φ	40	deg	60	deg	
Cohesion/Shear Strength	c	0	psf	0	psf =	rock mass cohesion intercept
Moist Unit Weight	γ	130	pcf	170	pcf	
Depth to Top of Rock or free stress	zR	18.4	feet			
Groundwater Depth	GWT	100.0	feet			

Vertical Soil Pressure	Depth z (feet)	Unit Wt γ (pcf)	Pressure σ'_v (ksf)		qrm = factored rock mass compressive strength
at Cap Base	4.0	130.0	0.52		
at bottom of 1st layer or GWT	18.4	130.0	2.39		
at top of rock, zR	18.4	130.0	2.39		
Effective Pressure @ top of rock	σ_v'		2.39	16.58 psi	

Calculations		Individual	Group	
Effective Length	Leff	33.4 feet	48.5 feet	Lu + (0.5Lb- individual, Lb - group)
Total Overburden Depth	Zt	33.4 feet	48.5 feet	Leff*sin(α)
Eff. Embedment Depth into Rock	Z	15.1 feet	30.1 feet	Zt - zR
Radius of Cone of Rock	R	26.1 feet	52.2 feet	Z*tanφ
Area @ top of Cone (individual)	Ac	2140.4 sf		πR^2
Area @ top of Cone (group)	Acg		874.1 sf/anchor	$\pi(R+ra)^2/n$
Volume of Rock Cone (individual)	Vc	10752.0 cf		$1/3\pi R^2 Z$
Volume of Rock Cone group)	Vcg		7580.4 cf/anchor	$1/3\pi Z(R^2+ra^2+raR)/n$
Weight of Rock Cone (individual)	Wc	1827.8 kips		$Vc*\gamma_R$
Weight of Cone (group)	Wcg		1288.7 kips/anchor	$Vcg*\gamma_R$
Surface Area of Cone (individual)	As	2471.5 sf		πRZ/cosφ
Surface Area of Cone (group)	Asg		844.2 sf/anchor	π(R+ra)(Z/cosφ)/n
Shearing Resistance (individual)	S	0.0 kips		As*fa
Shearing Resistance (group)	Sg		0.0 kips/anchor	Asg*fa
Pullout Capacity (individual)	**P**	**6937 kips**		$Wc + S + \sigma_v'*Ac$
Pullout Capacity (group)	**Pg**	**3375 kips/anchor**		$Wcg + Sg + \sigma_v'*Acg$
Preload Load per anchor	**Fp**	**435 kips**		
Factor of Safety against Pullout	**FS**	**15.95 individual**		
Factor of Safety against Pullout	**FS**	**7.76 group**		

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PATRICK & HENDERSON TENSIONLESS PIER FOUNDATION

Project: **Stetson Mountain Wind Project**
Wind Turbine: **GEWE 1.5sle 1.5 MW on 80-m Hub Height Tower**
ESSW Job No.: 09824-55
Date: 09/19/07

DYNAMIC FOUNDATION ANALYSIS **by Shelton L. Stringer, PE, GE**

Reference: DNV/Risϕ (2002) Guidelines for Design of Wind Turbines

Foundation Dimensions & Weights

Outer Radius	r_o =	12.0	feet =	144 in.	=	3.66 m		
Rock Properties		**Bearing on Rock**						
Density	γ =	170	pcf		=	26.7 kN/m^3		
Mass Density	ρ = γ/g		=	0.000255 lb-sec^2/in^4	=	2,720 kg/m^3		
Intact Compressive Strength	qu =	6200	psi		=	42.7 MPa		
Geological Strength Index	GSI =	35		*Em = (qu/100)$^{0.5}$ *10$^{[(GSI-10)/40]}$ (GPa)*			D =	0
Young's Rock Mass Modulus	Em =		=	406,000 psi	=	2,800 MPa		
Avg. Shear Modulus	G = Es/2(1+ν)		=	156,000 psi	=	1,080 MPa		
Avg. Shear Wave Velocity	Vso = $(G/\rho)^{0.5}$		=	2060 ft/sec	=	628 m/sec		
Strain Reduction Ratio	G/Go =	0.80	for rock subgrade or very dense soil					
Shear Modulus	G = Go*G/Go Ratio		=	124,800 psi	=	860 MPa		
Poisson Ratio	ν =	0.30						
Dynamic Spring Constants								
Vertical	Kz = $4G*r_o/(1-\nu)$		=	102,693 kip/in	=	18.0 GN/m		
Horizontal	Kx = $8G*r_o/(2-\nu)$		=	84,570 kip/in	=	14.8 GN/m		
Rocking	Kθ = $8G*r_o^3/[3(1-\nu)]$		=	1.42E+12 lb-in/rad.	=	**160 GN-m/rad.**		

CALCULATION OF ROTATIONAL STIFFNESS CONSIDERING PRESTRESS EFFECTS OF ANCHORS

Project: **Stetson Mountain Wind Project**

Reference: DNV/Risϕ (2002) Guidelines for Design of Wind Turbines

Foundation and Layer Dimesions		**Assume T4 conditions -13 feet till beneath foundation**				
Diameter of Foundation	d	24.00	feet	7.315	m	
Thickness of Foundation	t	5.00	feet	1.524	m	
Base Area	A	452.4	sf			
Radius of Foundation	a (r)	12.00	feet	3.658	m	
Weight of Fouindation	WT	339	kips	1509	kN	
Embedment Depth to Subgrade	z_E	5.00	feet	1.524	m	
Prestress Depth Zone	z_{PZ}	20.00	feet	6.096	m	
Stiff Layer Depth	h	50.0	feet	h/r:	3.75	
Anchor Properties & Preload						
Anchor Circle Diameter	Da	20	feet	6.096	m	
Deisign Prestress per Anchor	Pta	368	kips	1637	kN	
Number of Anchors	n	14				
Prestress Load to Foundation	Ppt	5152	kips	22917	kN	n*Pta
External Applied Load	Pext	440	kips	1958.9	kN	Fz
Total Load to Base of Foundation	Pt	5932	kips	26385	kN	Ppt+Pext+WT
Anchor Hole Diameter	d	5.0	inches	127	mm	
Anchor Rod Diameter	da	2.57	inches	65	mm	
Area of Anchor	Ar	5.19	in^2	3347	mm^2	
Modulus of Anchor	Er	29000000	psi	200	GPa	
Active Length of Anchor	La	20.00	feet	6.096	m	
Moment Resistance Developed by Anchors:	Mr	25760	ft-kips	34926	kN-m	nD*Pta/4
Anchor Axial Stiffness	K_a	627	kip/in			ArEr/La
Prestressed Zone (PZ) Stresses						
Base Pressure	q ($\Delta\sigma_v$)	13.11	ksf	91	psi	Pt/A
Unit Weight in Prestressed Zone	γ	130	pcf			
Insitu Vertical Stress at Prestress Zone	σ_v	11.3	psi	78	kPa	γ*avg(z_E, z_{PZ})
Operational Modulus Factor in PZ	Io	3.0		Input or $[(\sigma_v + \Delta\sigma v)/\sigma_{v]}^{0.5}$		

Layer Properties		G/Gmax	0.80				G' =G*G/Gmax*Io
I_θ (pz) = (z_{PZ}- z_E)/d	Influence	Stiffness	Elastic	Poisson's	Shear	Operational	
I_θ (lz) = I_θ (h) - I_θ (pz)	Factor	Factor	Modulus	Ratio	Modulus	Shear Modulus	
I_θ (h) = 1/(DNV stiff layer factor)	I_θ	$N_\theta = 1/I_\theta$	E (ksi)	ν	G (ksi)	G' (psi)	G,(MPa)
Prestress Zone (pz)	**0.900**	1.111	50	0.30	19	46	319
Below Prestress Zone (lz)	**0.091**	10.932	400	0.30	154	123	848
at Stiiff Layer (h):	**0.991**	1.009	400	0.30	154	154	1060

Anchor Group Rotational Stiffness	$K_{\theta a}$	5267850	kip-ft/rad		$nDa^2Ka/8$	
		7.1	GN-m/rad			
Prestressed Subgrade Rotational Stiffness	$K_{\theta ps}$	66.1	GN-m/rad		$8/3G'_{pz}r^3/(1-\nu)*N\theta$	
Prestressed Zone Rotational Stiffness	$K_{\theta pz}$	73.3	GN-m/rad		$K_{\theta a} + K_{\theta pz}$	
Lower Subgrade Rotational Stiffness	$K_{\theta lz}$	1728.1	GN-m/rad		$8/3G_{lz}r^3/(1-\nu)*N\theta$	
Total Rotational Stiffness	$K_{\theta T}$	**70.3**	**GN-m/rad**		$[K_{\theta pz}^{-1}+K_{\theta lz}^{-1}]^{-1}$	
Operational Moment	**Mxy**	**12978**	**ft-kips**	**17596**	**KN-m**	
Total Rotation	θ	0.00025	radians			$Mxy/K\theta_T$
Average Shear Strain	γ	0.013%				$\theta/2$
Relative Rotation of Anchor Group	θ	0.00024	radians			$Mxy/K\theta_{pz}$
Elongation of Anchor	δa	0.029	inches	0.73	mm	$\theta Da/2$
Change in Rod Force	ΔFa	18	kips	80	kN	$\delta a*Ka$
Factor of Increase in Anchor		**1.05**				$[Pta+\Delta Fa]/Pta$

Earth Systems Southwest

OVERTURINING STABILITY ANALYSIS OF P&H ANCHOR FOUNDATION

Project: ***Stetson Mountain Wind Project***
14 - 40-foot Anchors on 20-foot Ring with 24-foot Diameter Cap
using: *Williams 2-1/2-in diameter Grade 150 ksi Anchors or Equal*
Turbine: GEWE 1.5sle 1.5 MW on 80-m Hub Height Tower

Anchor: Williams		2-1/2	
fy	120 ksi		
fu	150 ksi		
d	2.50 inch	64 mm	
Ar	5.19 in^2	3348 mm^2	
Fu	**778 kips**	**2769 kN**	
Fy	622 kips	3461 kN	
0.6Fu	467 kips		
P-allow.:	**467 kips**		

Outer Anchor Diameter	Dc-o	5.00 inches	127 mm		
Outer Rod Diameter	Dro	2.56 inches	65 mm		
Concrete/Grout Strength	f'c	3000 psi	20.7 MPa		
Concrete/Grout Modulus	Ec	3120000 psi	21,500 MPa		
Steel Modulus	Es	29000000 psi	199,900 MPa		
Rod Area	Ar	5.19 sq-in	3348 mm^2		
Anchor Circle Diameter	Da	20.00 feet	6.096 m		
Number of Anchors	n	14			
Min. PreLoad per Anchor	**Pta**	**319 kips**	**1419 kN**		
Prestress Load to Foundation	Ppt	4466 kips	19866 kN	n*Pta	
Vertical Load	**Fz**	**414.7 kips**	**1844.5 kN**	Fz	
Foundation Weight	W	339.3 kips	1509.2 kN	$\pi B^2/4*T*\gamma c$	
Total Load to Base of Foundation	Pt	5220 kips	23219 kN	Ppt+Fz+W	
Active Length of Anchor	La	15.00 feet	4.572 m		
Resistance Developed by Anchors:	Mr	22330 ft-kips	30275 kN-m	nD*Pta/4	
Anchor Axial Stiffness	K_a	836 kip/in	146435 kN/m	ArEr/La	
Preload Elongation of Anchor	δp	0.382 inches	9.69 mm	Pta/Ka	
Anchor Group Rotational Stiffness	$K_{\theta a}$	7.02E+06 kip-ft/rad	9.5 GN-m/rad	$nDa^2Ka/8$	7.38E+05 kip-ft/rad to 1 GNm/rad
Subgrade Rotational Stiffness in PZ	$K_{\theta ps}$	5.16E+07 kip-ft/rad	70.0 GN-m/rad	$8/3G'_{pz}r^3/(1-\nu)*N\theta$	from Rotational Stiffness Worksheet
Prestressed Zone Rotational Stiffness	$K_{\theta pz}$	5.87E+07 kip-ft/rad	79.5 GN-m/rad	$K_{\theta a} + K_{\theta pz}$	
Diameter (width) of Foundation	B	24.00 feet	7.315 m		
Thickness of Foundation	T	5.00 feet	1.524 m		
Moment	**Mxy**	**25746.6 ft-kips**	**34907.7 KN-m**		100% M-extreme
Horizontal Load	**Fxy**	**117.7 kips**	**523.4 KN**		
Moment at Base	Mb	26334.9 ft-kips	35705.4 KN-m	Mxy + Fxy(T)	
Rotation at Base	θ	0.00044 radians		$Mb/K\theta_{pz}$	
Max Force to Anchor (e=0)	Fa	376.2 kips	1673.5 kN	4M/nD	

Load Case:	**4.1**	**Extreme Load**				1
Blade	DLC	LF	Fz KN	Fr KN	Mr KN-m	% Mext
GE37c	6.1j	1.00	-1844.5	523.4	34907.7	100%

Sliding Stability Check					
Friction ϕ	Coeff. μ	Ff kips	**FS**		FS = Ff/Fxy Ff = μ*Pt
40	0.76	3942.1	**33.5**	Okay	μ = 0.9tan ϕ

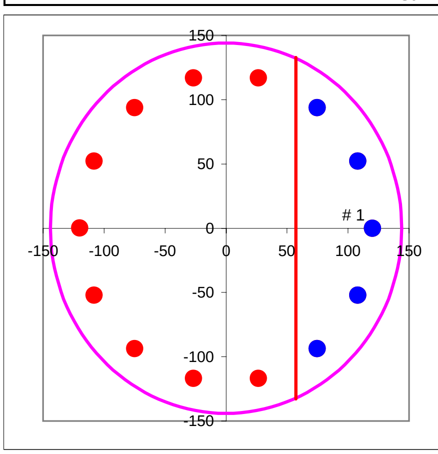
Moment Capacity and Stability of Anchor System

Eccentricity e 4.78 feet 1.456 m Mb/Vt $Vt = \Sigma(T+DL)$ **Relative Eccentricity** **e/B** **0.199** < 0.3 Okay

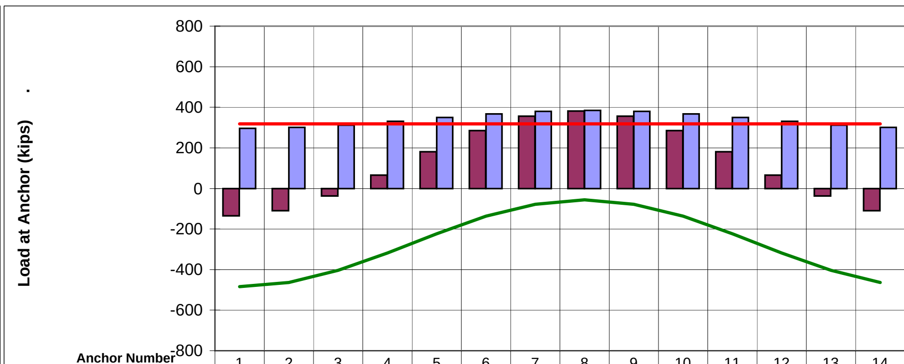
Stability Ratio (Factor of Safety): SR (FS) **2.51** e_{max}/e $e_{max} = 0.5B$

No of Rows: 1

No. of Anchors	Offset Distance (feet)	Offset Position	Position	x Distance (in)	y Distance (in)	x Distance (feet)	$\theta Da/2$ δa Anchor Deflection (inches)	$\delta a*Ka$ ΔFa Change in Rod Force (kips)	P External Load @ Anchor (kips)	T Tension in Rod (kips)	C Tributary Subgrade Compression (kips)	(Fz+W)/n DL Dead Load (kips)	Pt Preload (kips)	[Pta+ΔFa]/Pta Factor of Increase or Decrease	P*x External Moment (ft-kips)	T*x Anchor Moment (ft kips)	DL*x Dead Load Moment (ft kips)
1	0	0	0	120.00	0.00	5.22	-0.0275	-23.0	-135.0	296.0	-484.8	53.9	319.0	0.928	-705	-1546	-281.4
1	0	0	1	108.12	52.07	4.23	-0.0223	-18.6	-109.4	300.4	-463.6	53.9	319.0	0.942	-463	-1272	-228.0
1	0	0	2	74.82	93.82	1.46	-0.0077	-6.4	-37.7	312.6	-404.1	53.9	319.0	0.980	-55	-456	-78.6
1	0	0	3	26.70	116.99	-2.55	0.0134	11.2	65.9	330.2	-318.2	53.9	319.0	1.035	-168	842	137.3
1	0	0	4	-26.70	116.99	-7.00	0.0369	30.8	180.9	349.8	-222.8	53.9	319.0	1.097	-1266	2449	377.0
1	0	0	5	-74.82	93.82	-11.01	0.0580	48.5	284.5	367.5	-136.9	53.9	319.0	1.152	-3132	4046	593.0
1	0	0	6	-108.12	52.07	-13.79	0.0726	60.7	356.2	379.7	-77.4	53.9	319.0	1.190	-4910	5235	742.4
1	0	0	7	-120.00	0.00	-14.78	0.0778	65.1	381.8	**384.1**	-56.2	53.9	319.0	**1.204**	-5641	5675	795.7
1	0	0	8	-108.12	-52.07	-13.79	0.0726	60.7	356.2	379.7	-77.4	53.9	319.0	1.190	-4910	5235	742.4
1	0	0	9	-74.82	-93.82	-11.01	0.0580	48.5	284.5	367.5	-136.9	53.9	319.0	1.152	-3132	4046	593.0
1	0	0	10	-26.70	-116.99	-7.00	0.0369	30.8	180.9	349.8	-222.8	53.9	319.0	1.097	-1266	2449	377.0
1	0	0	11	26.70	-116.99	-2.55	0.0134	11.2	65.9	330.2	-318.2	53.9	319.0	1.035	-168	842	137.3
1	0	0	12	74.82	-93.82	1.46	-0.0077	-6.4	-37.7	312.6	-404.1	53.9	319.0	0.980	-55	-456	-78.6
1	0	0	13	108.12	-52.07	4.23	-0.0223	-18.6	-109.4	300.4	-463.6	53.9	319.0	0.942	-463	-1272	-228.0
14					Sum:			294	1727	4760	-3787	754	4466	14.923	-26335	25818	3601



Anchors - Red Applied Tension, Blue- Compression



	1	2	3	4	5	6	7	8	9	10	11	12	13	14
External Load	-135.0	-109.4	-37.7	65.9	180.9	284.5	356.2	381.8	356.2	284.5	180.9	65.9	-37.7	-109.4
Anchor Tension	296.0	300.4	312.6	330.2	349.8	367.5	379.7	384.1	379.7	367.5	349.8	330.2	312.6	300.4
Subgrade Compression	-484.8	-463.6	-404.1	-318.2	-222.8	-136.9	-77.4	-56.2	-77.4	-136.9	-222.8	-318.2	-404.1	-463.6
Preload	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0

Earth Systems Southwest

OVERTURINING STABILITY ANALYSIS OF P&H ANCHOR FOUNDATION

Project: ***Stetson Mountain Wind Project***

14 - 40-foot Anchors on 20-foot Ring with 24-foot Diameter Cap

using: *Williams 2-1/2-in diameter Grade 150 ksi Anchors or Equal*

Turbine: GEWE 1.5sle 1.5 MW on 80-m Hub Height Tower

Anchor: Williams		2-1/2	
fy	120 ksi		
fu	150 ksi		
d	2.50 inch	64 mm	
Ar	5.19 in^2	3348 mm^2	
Fu	**778 kips**	**2769 kN**	
Fy	622 kips	3461 kN	
0.6Fu	467 kips		
P-allow.:	**467 kips**		

Outer Anchor Diameter	Dc-o	5.00 inches	127 mm			
Outer Rod Diameter	Dro	2.56 inches	65 mm			
Concrete/Grout Strength	f'c	3000 psi	20.7 MPa			
Concrete/Grout Modulus	Ec	3120000 psi	21,500 MPa			
Steel Modulus	Es	29000000 psi	199,900 MPa			
Rod Area	Ar	5.19 sq-in	3348 mm^2			
Anchor Circle Diameter	Da	20.00 feet	6.096 m			
Number of Anchors	n	14				
Min. PreLoad per Anchor	**Pta**	**319 kips**	**1419 kN**			
Prestress Load to Foundation	Ppt	4466 kips	19866 kN	n*Pta		
Vertical Load	**Fz**	**440.4 kips**	**1958.9 kN**	Fz		
Foundation Weight	W	339.3 kips	1509.2 kN	$\pi B^2/4*T*\gamma c$		
Total Load to Base of Foundation	Pt	5246 kips	23334 kN	Ppt+Fz+W		
Active Length of Anchor	La	15.00 feet	4.572 m			
Resistance Developed by Anchors:	Mr	22330 ft-kips	30275 kN-m	nD*Pta/4		
Anchor Axial Stiffness	K_a	836 kip/in	146435 kN/m	ArEr/La		
Preload Elongation of Anchor	δp	0.382 inches	9.69 mm	Pta/Ka		
Anchor Group Rotational Stiffness	$K_{\theta a}$	7.02E+06 kip-ft/rad	9.5 GN-m/rad	$nDa^2Ka/8$	7.38E+05 kip-ft/rad to 1 GNm/rad	
Subgrade Rotational Stiffness in PZ	$K_{\theta ps}$	5.16E+07 kip-ft/rad	70.0 GN-m/rad	$8/3G'_{pz}r^3/(1-\nu)*N\theta$	from Rotational Stiffness Worksheet	
Prestressed Zone Rotational Stiffness	$K_{\theta pz}$	5.87E+07 kip-ft/rad	79.5 GN-m/rad	$K_{\theta a} + K_{\theta pz}$		
Diameter (width) of Foundation	B	24.00 feet	7.315 m			
Thickness of Foundation	T	5.00 feet	1.524 m			
Moment	**Mxy**	**12978.3 ft-kips**	**17596.2 KN-m**		50% M-extreme	
Horizontal Load	**Fxy**	**49.2 kips**	**219.0 KN**			
Moment at Base	Mb	13224.5 ft-kips	17930.0 KN-m	Mxy + Fxy(T)		
Rotation at Base	θ	0.00022 radians		$Mb/K\theta_{pz}$		
Max Force to Anchor (e=0)	Fa	188.9 kips	840.4 kN	4M/nD		

Load Case:	**4.2**	**Lift-off**				2
Blade	DLC	LF	Fz KN	Fr KN	Mr KN-m	% Mext
LM37.3p2	1	1.00	-1958.9	219	17596.2	50%

Sliding Stability Check

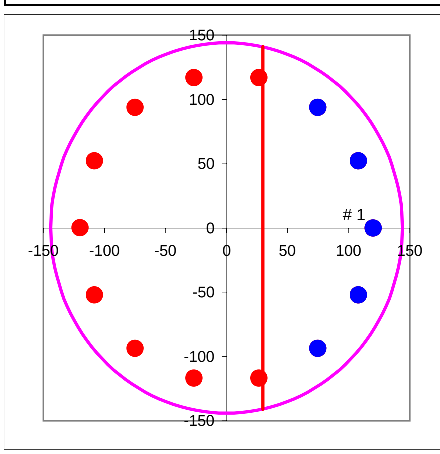
Friction ϕ	Coeff. μ	Ff kips	FS		
					FS = Ff/Fxy
					Ff = μ*Pt
40	0.76	3961.5	**80.5**	Okay	$\mu = 0.9\tan\phi$

Moment Capacity and Stability of Anchor System

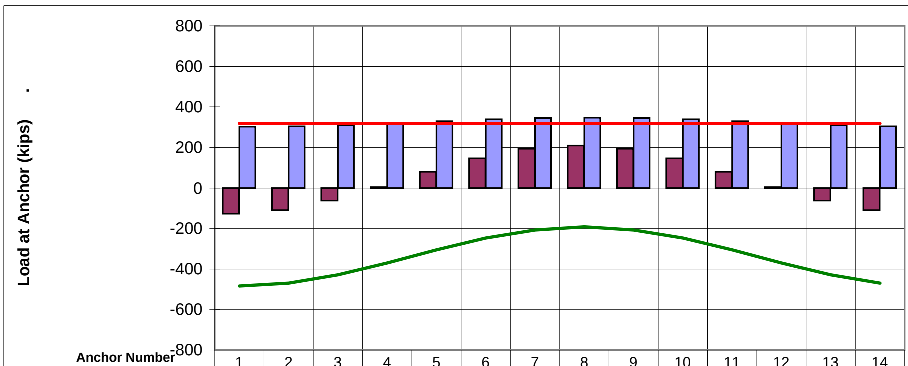
Eccentricity	e	2.48 feet	0.757 m	Mb/Vt	$Vt = \Sigma(T+DL)$	**Relative Eccentricity**	**e/B**	**0.104** < 0.167 Okay
Stability Ratio (Factor of Safety):	SR (FS)	**4.83**		e_{max}/e	$e_{max} = 0.5B$			

No of Rows: 1 $\theta Da/2$ $\delta a*Ka$ (Fz+W)/n [Pta+ΔFa]/Pta

No. of Anchors	Offset Distance (feet)	Offset Position	Position	x Distance (in)	y Distance (in)	x Distance (feet)	δa Anchor Deflection (inches)	ΔFa Change in Rod Force (kips)	P External Load @ Anchor (kips)	T Tension in Rod (kips)	C Tributary Subgrade Compression (kips)	DL Dead Load (kips)	Pt Preload (kips)	Factor of Increase or Decrease	P*x External Moment (ft-kips)	T*x Anchor Moment (ft kips)	DL*x Dead Load Moment (ft kips)
1	0	0	0	120.00	0.00	7.52	-0.0200	-16.7	-126.4	302.3	-484.4	55.7	319.0	0.948	-950	-2272	-418.5
1	0	0	1	108.12	52.07	6.53	-0.0173	-14.5	-109.7	304.5	-469.9	55.7	319.0	0.955	-716	-1987	-363.4
1	0	0	2	74.82	93.82	3.75	-0.0100	-8.3	-63.1	310.7	-429.4	55.7	319.0	0.974	-237	-1165	-208.9
1	0	0	3	26.70	116.99	-0.26	0.0007	0.6	4.4	319.6	-370.9	55.7	319.0	1.002	-1	83	14.4
1	0	0	4	-26.70	116.99	-4.71	0.0125	10.5	79.2	329.5	-305.9	55.7	319.0	1.033	-373	1552	262.3
1	0	0	5	-74.82	93.82	-8.72	0.0232	19.4	146.6	338.4	-247.4	55.7	319.0	1.061	-1278	2950	485.6
1	0	0	6	-108.12	52.07	-11.49	0.0305	25.5	193.3	344.5	-206.9	55.7	319.0	1.080	-2222	3960	640.1
1	0	0	7	-120.00	0.00	-12.48	0.0331	27.7	209.9	**346.7**	-192.5	55.7	319.0	**1.087**	-2621	4329	695.3
1	0	0	8	-108.12	-52.07	-11.49	0.0305	25.5	193.3	344.5	-206.9	55.7	319.0	1.080	-2222	3960	640.1
1	0	0	9	-74.82	-93.82	-8.72	0.0232	19.4	146.6	338.4	-247.4	55.7	319.0	1.061	-1278	2950	485.6
1	0	0	10	-26.70	-116.99	-4.71	0.0125	10.5	79.2	329.5	-305.9	55.7	319.0	1.033	-373	1552	262.3
1	0	0	11	26.70	-116.99	-0.26	0.0007	0.6	4.4	319.6	-370.9	55.7	319.0	1.002	-1	83	14.4
1	0	0	12	74.82	-93.82	3.75	-0.0100	-8.3	-63.1	310.7	-429.4	55.7	319.0	0.974	-237	-1165	-208.9
1	0	0	13	108.12	-52.07	6.53	-0.0173	-14.5	-109.7	304.5	-469.9	55.7	319.0	0.955	-716	-1987	-363.4
14					Sum:			77	585	4543	-4738	780	4466	14.242	-13224	12842	1937



Anchors - Red Applied Tension, Blue- Compression



	1	2	3	4	5	6	7	8	9	10	11	12	13	14
External Load	-126.4	-109.7	-63.1	4.4	79.2	146.6	193.3	209.9	193.3	146.6	79.2	4.4	-63.1	-109.7
Anchor Tension	302.3	304.5	310.7	319.6	329.5	338.4	344.5	346.7	344.5	338.4	329.5	319.6	310.7	304.5
Subgrade Compression	-484.4	-469.9	-429.4	-370.9	-305.9	-247.4	-206.9	-192.5	-206.9	-247.4	-305.9	-370.9	-429.4	-469.9
Preload	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0

Earth Systems Southwest

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fu	150 ksi		
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Ar	5.19 in^2	3348 mm^2	
Fu	**778 kips**	**2769 kN**	
Fy	622 kips	3461 kN	
0.6Fu	467 kips		
P-allow.:	**467 kips**		

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Outer Rod Diameter	Dro	2.56 inches	65 mm			
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Concrete/Grout Modulus	Ec	3120000 psi	21,500 MPa			
Steel Modulus	Es	29000000 psi	199,900 MPa			
Rod Area	Ar	5.19 sq-in	3348 mm^2			
Anchor Circle Diameter	Da	20.00 feet	6.096 m			
Number of Anchors	n	14				
Min. PreLoad per Anchor	**Pta**	**319 kips**	**1419 kN**			
Prestress Load to Foundation	Ppt	4466 kips	19866 kN	n*Pta		
Vertical Load	**Fz**	**414.7 kips**	**1844.5 kN**	Fz		
Foundation Weight	W	339.3 kips	1509.2 kN	$\pi B^2/4*T*\gamma c$		
Total Load to Base of Foundation	Pt	5220 kips	23219 kN	Ppt+Fz+W		
Active Length of Anchor	La	15.00 feet	4.572 m			
Resistance Developed by Anchors:	Mr	22330 ft-kips	30275 kN-m	nD*Pta/4		
Anchor Axial Stiffness	K_a	836 kip/in	146435 kN/m	ArEr/La		
Preload Elongation of Anchor	δp	0.382 inches	9.69 mm	Pta/Ka		
Anchor Group Rotational Stiffness	$K_{\theta a}$	7.02E+06 kip-ft/rad	9.5 GN-m/rad	$nDa^2Ka/8$	7.38E+05 kip-ft/rad to 1 GNm/rad	
Subgrade Rotational Stiffness in PZ	$K_{\theta ps}$	5.16E+07 kip-ft/rad	70.0 GN-m/rad	$8/3G'_{pz}r^3/(1-\nu)*N\theta$	from Rotational Stiffness Worksheet	
Prestressed Zone Rotational Stiffness	$K_{\theta pz}$	5.87E+07 kip-ft/rad	79.5 GN-m/rad	$K_{\theta a} + K_{\theta pz}$		
Diameter (width) of Foundation	B	24.00 feet	7.315 m			
Thickness of Foundation	T	5.00 feet	1.524 m			
Moment	**Mxy**	**25746.6 ft-kips**	**34907.7 KN-m**		100% M-extreme	
Horizontal Load	**Fxy**	**117.5 kips**	**522.6 KN**			
Moment at Base	Mb	26334.0 ft-kips	35704.1 KN-m	Mxy + Fxy(T)		
Rotation at Base	θ	0.00044 radians		$Mb/K\theta_{pz}$		
Max Force to Anchor (e=0)	Fa	376.2 kips	1673.4 kN	4M/nD		

Load Case:	**4.3**	**Tilting**				**3**
			Fz	Fr	Mr	% Mext
Blade	DLC	LF	KN	KN	KN-m	
GE37c	6.1j	1.00	-1844.5	522.6	34907.7	100%

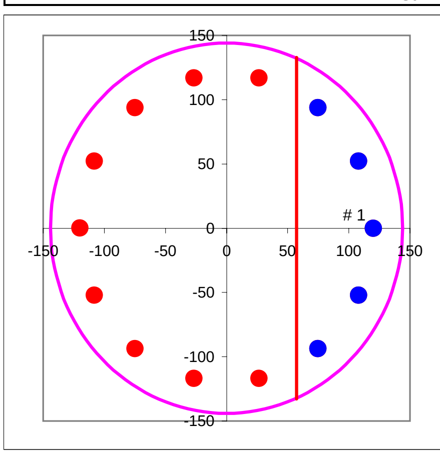
Sliding Stability Check					
Friction	Coeff.	Ff	**FS**		FS = Ff/Fxy
ϕ	μ	kips			Ff = μ*Pt
40	0.76	3942.1	**33.6**	Okay	μ = 0.9tan ϕ

Moment Capacity and Stability of Anchor System

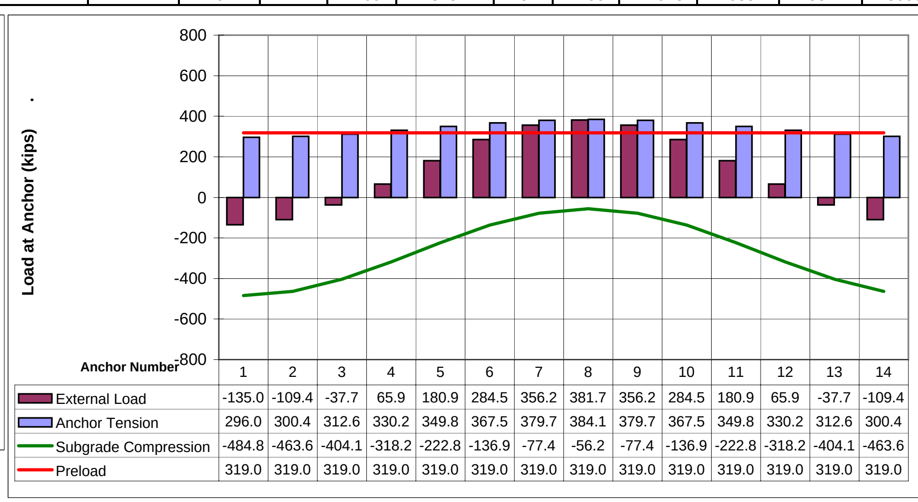
Eccentricity	e	4.78 feet	1.456 m	Mb/Vt	$Vt = \Sigma(T+DL)$	**Relative Eccentricity** **e/B**	**0.199** < 0.25 Okay
Stability Ratio (Factor of Safety):	SR (FS)	**2.51**		e_{max}/e	$e_{max} = 0.5B$		

No of Rows: 1 θDa/2 δa*Ka (Fz+W)/n [Pta+ΔFa]/Pta

No. of Anchors	Offset Distance (feet)	Offset Position	Position	x Distance (in)	y Distance (in)	x Distance (feet)	δa Anchor Deflection (inches)	ΔFa Change in Rod Force (kips)	P External Load @ Anchor (kips)	T Tension in Rod (kips)	C Tributary Subgrade Compression (kips)	DL Dead Load (kips)	Pt Preload (kips)	Factor of Increase or Decrease	P*x External Moment (ft-kips)	T*x Anchor Moment (ft kips)	DL*x Dead Load Moment (ft kips)
1	0	0	0	120.00	0.00	5.22	-0.0275	-23.0	-135.0	296.0	-484.8	53.9	319.0	0.928	-705	-1546	-281.4
1	0	0	1	108.12	52.07	4.23	-0.0223	-18.6	-109.4	300.4	-463.6	53.9	319.0	0.942	-463	-1272	-228.0
1	0	0	2	74.82	93.82	1.46	-0.0077	-6.4	-37.7	312.6	-404.1	53.9	319.0	0.980	-55	-456	-78.6
1	0	0	3	26.70	116.99	-2.55	0.0134	11.2	65.9	330.2	-318.2	53.9	319.0	1.035	-168	842	137.3
1	0	0	4	-26.70	116.99	-7.00	0.0369	30.8	180.9	349.8	-222.8	53.9	319.0	1.097	-1266	2449	377.0
1	0	0	5	-74.82	93.82	-11.01	0.0580	48.5	284.5	367.5	-136.9	53.9	319.0	1.152	-3132	4046	592.9
1	0	0	6	-108.12	52.07	-13.79	0.0726	60.7	356.2	379.7	-77.4	53.9	319.0	1.190	-4910	5234	742.4
1	0	0	7	-120.00	0.00	-14.78	0.0778	65.1	381.7	**384.1**	-56.2	53.9	319.0	**1.204**	-5640	5675	795.7
1	0	0	8	-108.12	-52.07	-13.79	0.0726	60.7	356.2	379.7	-77.4	53.9	319.0	1.190	-4910	5234	742.4
1	0	0	9	-74.82	-93.82	-11.01	0.0580	48.5	284.5	367.5	-136.9	53.9	319.0	1.152	-3132	4046	592.9
1	0	0	10	-26.70	-116.99	-7.00	0.0369	30.8	180.9	349.8	-222.8	53.9	319.0	1.097	-1266	2449	377.0
1	0	0	11	26.70	-116.99	-2.55	0.0134	11.2	65.9	330.2	-318.2	53.9	319.0	1.035	-168	842	137.3
1	0	0	12	74.82	-93.82	1.46	-0.0077	-6.4	-37.7	312.6	-404.1	53.9	319.0	0.980	-55	-456	-78.6
1	0	0	13	108.12	-52.07	4.23	-0.0223	-18.6	-109.4	300.4	-463.6	53.9	319.0	0.942	-463	-1272	-228.0
14					Sum:			294	1727	4760	-3787	754	4466	14.923	-26334	25817	3600



Anchors - Red Applied Tension, Blue- Compression



Earth Systems Southwest

OVERTURINING STABILITY ANALYSIS OF P&H ANCHOR FOUNDATION

Project: ***Stetson Mountain Wind Project***

14 - 40-foot Anchors on 20-foot Ring with 24-foot Diameter Cap

using: *Williams 2-1/2-in diameter Grade 150 ksi Anchors or Equal*

Turbine: GEWE 1.5sle 1.5 MW on 80-m Hub Height Tower

Anchor:	Williams	2-1/2	
fy	120 ksi		
fu	150 ksi		
d	2.50 inch	64 mm	
Ar	5.19 in^2	3348 mm^2	
Fu	**778 kips**	**2769 kN**	
Fy	622 kips	3461 kN	
0.6Fu	467 kips		
P-allow.:	**467 kips**		

Outer Anchor Diameter	Dc-o	5.00 inches	127 mm	
Outer Rod Diameter	Dro	2.56 inches	65 mm	
Concrete/Grout Strength	f'c	3000 psi	20.7 MPa	
Concrete/Grout Modulus	Ec	3120000 psi	21,500 MPa	
Steel Modulus	Es	29000000 psi	199,900 MPa	
Rod Area	Ar	5.19 sq-in	3348 mm^2	
Anchor Circle Diameter	Da	20.00 feet	6.096 m	
Number of Anchors	n	14		
Min. PreLoad per Anchor	**Pta**	**319 kips**	**1419 kN**	
Prestress Load to Foundation	Ppt	4466 kips	19866 kN	n*Pta
Vertical Load	**Fz**	**414.4 kips**	**1843.4 kN**	Fz
Foundation Weight	W	339.3 kips	1509.2 kN	$\pi B^2/4*T*\gamma c$
Total Load to Base of Foundation	Pt	5220 kips	23218 kN	Ppt+Fz+W
Active Length of Anchor	La	15.00 feet	4.572 m	
Resistance Developed by Anchors:	Mr	22330 ft-kips	30275 kN-m	nD*Pta/4
Anchor Axial Stiffness	K_a	836 kip/in	146435 kN/m	ArEr/La
Preload Elongation of Anchor	δp	0.382 inches	9.69 mm	Pta/Ka
Anchor Group Rotational Stiffness	$K_{\theta a}$	7.02E+06 kip-ft/rad	9.5 GN-m/rad	$nDa^2Ka/8$
Subgrade Rotational Stiffness in PZ	$K_{\theta ps}$	5.16E+07 kip-ft/rad	70.0 GN-m/rad	$8/3G'_{pz}r^3/(1-\nu)*N\theta$
Prestressed Zone Rotational Stiffness	$K_{\theta pz}$	5.87E+07 kip-ft/rad	79.5 GN-m/rad	$K_{\theta a} + K_{\theta pz}$
Diameter (width) of Foundation	B	24.00 feet	7.315 m	
Thickness of Foundation	T	5.00 feet	1.524 m	
Moment	**Mxy**	**25713.7 ft-kips**	**34863.1 KN-m**	100% M-extreme
Horizontal Load	**Fxy**	**117.7 kips**	**523.4 KN**	
Moment at Base	Mb	26302.0 ft-kips	35660.8 KN-m	Mxy + Fxy(T)
Rotation at Base	θ	0.00044 radians		$Mb/K\theta_{pz}$
Max Force to Anchor (e=0)	Fa	375.7 kips	1671.4 kN	4M/nD

7.38E+05 kip-ft/rad to 1 GNm/rad

from Rotational Stiffness Worksheet

Load Case:	**4.4**	**Sliding**				4
Blade	DLC	LF	Fz KN	Fr KN	Mr KN-m	% Mext
GE37c	6.1j	1.00	-1843.4	523.4	34863.1	100%

Sliding Stability Check

Friction ϕ	Coeff. μ	Ff kips	**FS**		
					FS = Ff/Fxy
					Ff = μ*Pt
40	0.76	3941.9	**33.5**	Okay	μ = 0.9tan ϕ

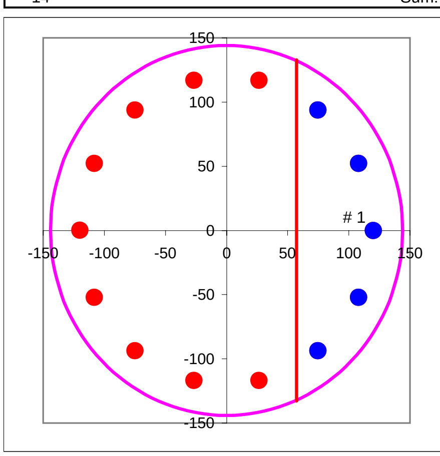
Moment Capacity and Stability of Anchor System

Eccentricity e 4.77 feet 1.454 m Mb/Vt Vt =Σ(T+DL) **Relative Eccentricity e/B 0.199** < 0.3 Okay

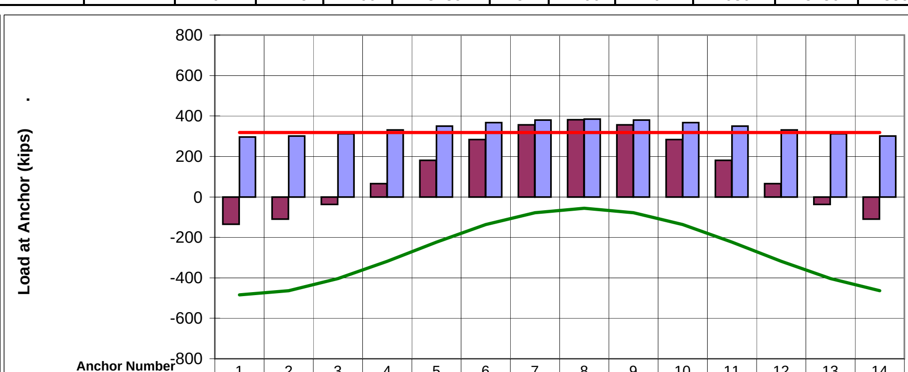
Stability Ratio (Factor of Safety): SR (FS) **2.52** e_{max}/e e_{max} = 0.5B

No of Rows: 1 θDa/2 δa*Ka (Fz+W)/n [Pta+ΔFa]/Pta

No. of Anchors	Offset Distance (feet)	Offset Position	Position	x Distance (in)	y Distance (in)	x Distance (feet)	δa Anchor Deflection (inches)	ΔFa Change in Rod Force (kips)	P External Load @ Anchor (kips)	T Tension in Rod (kips)	C Tributuary Subgrade Compression (kips)	DL Dead Load (kips)	Pt Preload (kips)	Factor of Increase or Decrease	P*x External Moment (ft-kips)	T*x Anchor Moment (ft kips)	DL*x Dead Load Moment (ft kips)
1	0	0	0	120.00	0.00	5.23	-0.0275	-23.0	-135.0	296.0	-484.9	53.8	319.0	0.928	-706	-1548	-281.5
1	0	0	1	108.12	52.07	4.24	-0.0223	-18.6	-109.5	300.4	-463.7	53.8	319.0	0.942	-464	-1273	-228.2
1	0	0	2	74.82	93.82	1.46	-0.0077	-6.4	-37.8	312.6	-404.2	53.8	319.0	0.980	-55	-458	-78.8
1	0	0	3	26.70	116.99	-2.55	0.0134	11.2	65.7	330.2	-318.3	53.8	319.0	1.035	-167	840	137.0
1	0	0	4	-26.70	116.99	-7.00	0.0368	30.8	180.6	349.8	-223.0	53.8	319.0	1.096	-1264	2447	376.6
1	0	0	5	-74.82	93.82	-11.01	0.0579	48.4	284.2	367.4	-137.1	53.8	319.0	1.152	-3127	4044	592.5
1	0	0	6	-108.12	52.07	-13.78	0.0725	60.6	355.8	379.6	-77.6	53.8	319.0	1.190	-4903	5231	741.9
1	0	0	7	-120.00	0.00	-14.77	0.0777	65.0	381.4	**384.0**	-56.4	53.8	319.0	**1.204**	-5633	5671	795.2
1	0	0	8	-108.12	-52.07	-13.78	0.0725	60.6	355.8	379.6	-77.6	53.8	319.0	1.190	-4903	5231	741.9
1	0	0	9	-74.82	-93.82	-11.01	0.0579	48.4	284.2	367.4	-137.1	53.8	319.0	1.152	-3127	4044	592.5
1	0	0	10	-26.70	-116.99	-7.00	0.0368	30.8	180.6	349.8	-223.0	53.8	319.0	1.096	-1264	2447	376.6
1	0	0	11	26.70	-116.99	-2.55	0.0134	11.2	65.7	330.2	-318.3	53.8	319.0	1.035	-167	840	137.0
1	0	0	12	74.82	-93.82	1.46	-0.0077	-6.4	-37.8	312.6	-404.2	53.8	319.0	0.980	-55	-458	-78.8
1	0	0	13	108.12	-52.07	4.24	-0.0223	-18.6	-109.5	300.4	-463.7	53.8	319.0	0.942	-464	-1273	-228.2
14					Sum:			294	1725	4760	-3789	754	4466	14.921	-26302	25786	3596



Anchors - Red Applied Tension, Blue- Compression



	1	2	3	4	5	6	7	8	9	10	11	12	13	14
External Load	-135.0	-109.5	-37.8	65.7	180.6	284.2	355.8	381.4	355.8	284.2	180.6	65.7	-37.8	-109.5
Anchor Tension	296.0	300.4	312.6	330.2	349.8	367.4	379.6	384.0	379.6	367.4	349.8	330.2	312.6	300.4
Subgrade Compression	-484.9	-463.7	-404.2	-318.3	-223.0	-137.1	-77.6	-56.4	-77.6	-137.1	-223.0	-318.3	-404.2	-463.7
Preload	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0

Earth Systems Southwest

OVERTURINING STABILITY ANALYSIS OF P&H ANCHOR FOUNDATION

Project: ***Stetson Mountain Wind Project***

14 - 40-foot Anchors on 20-foot Ring with 24-foot Diameter Cap

using: *Williams 2-1/2-in diameter Grade 150 ksi Anchors or Equal*

Turbine: GEWE 1.5sle 1.5 MW on 80-m Hub Height Tower

Anchor: Williams		2-1/2	
fy	120 ksi		
fu	150 ksi		
d	2.50 inch	64 mm	
Ar	5.19 in^2	3348 mm^2	
Fu	**778 kips**	**2769 kN**	
Fy	622 kips	3461 kN	
0.6Fu	467 kips		
P-allow.:	**467 kips**		

Outer Anchor Diameter	Dc-o	5.00 inches	127 mm		
Outer Rod Diameter	Dro	2.56 inches	65 mm		
Concrete/Grout Strength	f'c	3000 psi	20.7 MPa		
Concrete/Grout Modulus	Ec	3120000 psi	21,500 MPa		
Steel Modulus	Es	29000000 psi	199,900 MPa		
Rod Area	Ar	5.19 sq-in	3348 mm^2		
Anchor Circle Diameter	Da	20.00 feet	6.096 m		
Number of Anchors	n	14			
Min. PreLoad per Anchor	**Pta**	**319 kips**	**1419 kN**		
Prestress Load to Foundation	Ppt	4466 kips	19866 kN	n*Pta	
Vertical Load	**Fz**	**559.8 kips**	**2490.0 kN**	Fz	
Foundation Weight	W	339.3 kips	1509.2 kN	$\pi B^2/4*T*\gamma c$	
Total Load to Base of Foundation	Pt	5365 kips	23865 kN	Ppt+Fz+W	
Active Length of Anchor	La	15.00 feet	4.572 m		
Resistance Developed by Anchors:	Mr	22330 ft-kips	30275 kN-m	nD*Pta/4	
Anchor Axial Stiffness	K_a	836 kip/in	146435 kN/m	ArEr/La	
Preload Elongation of Anchor	δp	0.382 inches	9.69 mm	Pta/Ka	
Anchor Group Rotational Stiffness	$K_{\theta a}$	7.02E+06 kip-ft/rad	9.5 GN-m/rad	$nDa^2Ka/8$	7.38E+05 kip-ft/rad to 1 GNm/rad
Subgrade Rotational Stiffness in PZ	$K_{\theta ps}$	5.16E+07 kip-ft/rad	70.0 GN-m/rad	$8/3G'_{pz}r^3/(1-\nu)*N\theta$	from Rotational Stiffness Worksheet
Prestressed Zone Rotational Stiffness	$K_{\theta pz}$	5.87E+07 kip-ft/rad	79.5 GN-m/rad	$K_{\theta a}+K_{\theta pz}$	
Diameter (width) of Foundation	B	24.00 feet	7.315 m		
Thickness of Foundation	T	5.00 feet	1.524 m		
Moment	**Mxy**	**34757.7 ft-kips**	**47125.1 KN-m**	135% M-extreme	
Horizontal Load	**Fxy**	**158.6 kips**	**705.6 KN**		
Moment at Base	Mb	35550.8 ft-kips	48200.4 KN-m	Mxy + Fxy(T)	
Rotation at Base	θ	0.00059 radians		$Mb/K\theta_{pz}$	
Max Force to Anchor (e=0)	Fa	507.9 kips	2259.1 kN	4M/nD	

Load Case:	**4.5**	**Shear Failure (Bearing)**				**5**
Blade	DLC	LF	Fz KN	Fr KN	Mr KN-m	% Mext
GE37c	6.1j	1.35	-2490	705.6	47125.1	135%

Sliding Stability Check					
Friction ϕ	Coeff. μ	Ff kips	**FS**		FS = Ff/Fxy; Ff = μ*Pt
40	0.76	4051.6	**25.5**	Okay	μ = 0.9tan ϕ

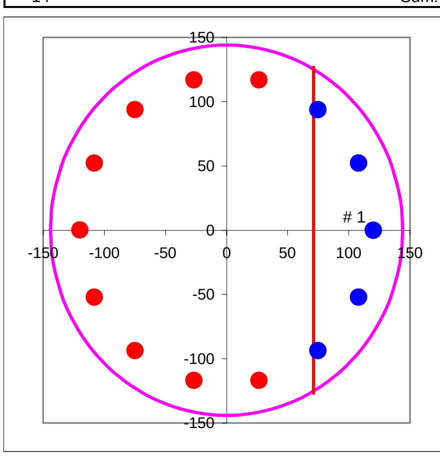
Moment Capacity and Stability of Anchor System

Eccentricity e 5.93 feet 1.807 m Mb/Vt Vt =Σ(T+DL) **Relative Eccentricity e/B 0.247** < 0.3 Okay

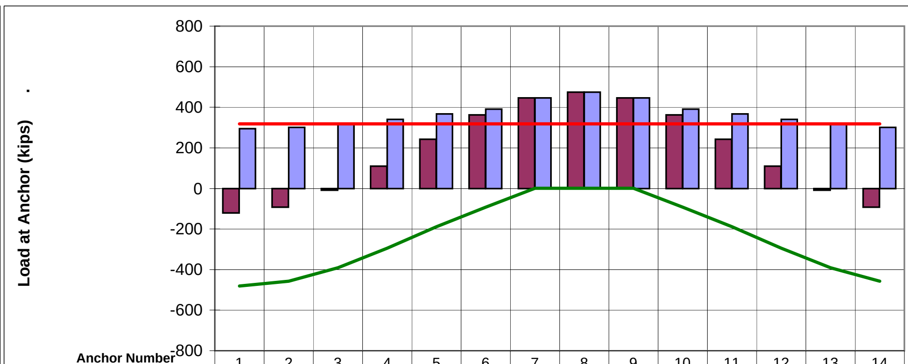
Stability Ratio (Factor of Safety): SR (FS) **2.02** e_{max}/e e_{max} = 0.5B

No of Rows: 1

No. of Anchors	Offset Distance (feet)	Offset Position	Position	x Distance (in)	y Distance (in)	x Distance (feet)	δa Anchor Deflection (inches)	ΔFa Change in Rod Force (kips)	P External Load @ Anchor (kips)	T Tension in Rod (kips)	C Tributary Subgrade Compression (kips)	DL Dead Load (kips)	Pt Preload (kips)	Factor of Increase or Decrease	P*x External Moment (ft-kips)	T*x Anchor Moment (ft kips)	DL*x Dead Load Moment (ft kips)
							θDa/2	δa*Ka				(Fz+W)/n		[Pta+ΔFa]/Pta			
1	0	0	0	120.00	0.00	4.07	-0.0289	-24.2	-121.4	294.8	-480.4	64.2	319.0	0.924	-494	-1200	-261.4
1	0	0	1	108.12	52.07	3.08	-0.0219	-18.3	-91.8	300.7	-456.7	64.2	319.0	0.943	-283	-926	-197.8
1	0	0	2	74.82	93.82	0.31	-0.0022	-1.8	-9.1	317.2	-390.5	64.2	319.0	0.994	-3	-97	-19.6
1	0	0	3	26.70	116.99	-3.70	0.0263	22.0	110.5	341.0	-294.8	64.2	319.0	1.069	-409	1263	237.9
1	0	0	4	-26.70	116.99	-8.16	0.0580	48.5	243.2	367.5	-188.5	64.2	319.0	1.152	-1983	2997	523.7
1	0	0	5	-74.82	93.82	-12.16	0.0865	72.3	362.7	391.3	-92.8	64.2	319.0	1.227	-4412	4760	781.2
1	0	0	6	-108.12	52.07	-14.94	0.1062	126.5	445.5	445.5	0.0	64.2	319.0	1.396	-6655	6655	959.4
1	0	0	7	-120.00	0.00	-15.93	0.1133	156.0	475.0	**475.0**	0.0	64.2	319.0	**1.489**	-7566	7566	1023.0
1	0	0	8	-108.12	-52.07	-14.94	0.1062	126.5	445.5	445.5	0.0	64.2	319.0	1.396	-6655	6655	959.4
1	0	0	9	-74.82	-93.82	-12.16	0.0865	72.3	362.7	391.3	-92.8	64.2	319.0	1.227	-4412	4760	781.2
1	0	0	10	-26.70	-116.99	-8.16	0.0580	48.5	243.2	367.5	-188.5	64.2	319.0	1.152	-1983	2997	523.7
1	0	0	11	26.70	-116.99	-3.70	0.0263	22.0	110.5	341.0	-294.8	64.2	319.0	1.069	-409	1263	237.9
1	0	0	12	74.82	-93.82	0.31	-0.0022	-1.8	-9.1	317.2	-390.5	64.2	319.0	0.994	-3	-97	-19.6
1	0	0	13	108.12	-52.07	3.08	-0.0219	-18.3	-91.8	300.7	-456.7	64.2	319.0	0.943	-283	-926	-197.8
14					Sum:			630	2475	5096	-3327	899	4466	15.975	-35551	35672	5331



Anchors - Red Applied Tension, Blue- Compression



	1	2	3	4	5	6	7	8	9	10	11	12	13	14
External Load	-121.4	-91.8	-9.1	110.5	243.2	362.7	445.5	475.0	445.5	362.7	243.2	110.5	-9.1	-91.8
Anchor Tension	294.8	300.7	317.2	341.0	367.5	391.3	445.5	475.0	445.5	391.3	367.5	341.0	317.2	300.7
Subgrade Compression	-480.4	-456.7	-390.5	-294.8	-188.5	-92.8	0.0	0.0	0.0	-92.8	-188.5	-294.8	-390.5	-456.7
Preload	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0	319.0

APPENDIX B
Structural Analysis Calculations

PATRICK & HENDERSON PILE ANCHOR FOUNDATION

Project: **Stetson Mountain Wind Project**
Wind Turbine: **GEWE 1.5sle 1.5 MW on 80-m Hub Height Tower**
Location: **Danforth, ME**
ESSW Job No.: 09824-55 P&H Job No.: 07-036
Date: 09/19/07

FOUNDATION DIMENSIONS							
Cap Outside Diameter	*Do*	=	24.0	**feet**	=	7.315 m	
Foundation Cap Thickness	*T*	=	5.0	**feet**	=	1.524 m	
Concrete Compressive Strength	*f'c*	=	**6,000**	psi	=	41 MPa	
Volume of Concrete Cap	*Vcap*	=	84	cy	=	64 m^3	
Pile Anchors							
Number of Anchors	*n*	=	14				
Outer Anchor Circle Diameter	*Do*	=	20.0	feet	=	6.096 m	
Inner Anchor Circle Diameter	*Di*	=	0.0	feet	=	0.000 m	
Pile Anchor Diameter	*Da*	=	24	inches	=	0.6 m	
Anchor Post Tension Force	*Pt*	=	**435**	kips	=	1935 kN	
Anchor Bolt - Strength	Fu	=	100,000	psi	=	689 MPa	
Anchor Bolt - Length	*La*	=	**45.0**	feet	=	13.7 m	630 LF total
Anchor Bolt - Nominal Size	d	=	2.5	inches	Williams Grade 150, 2.5"		
Anchor - Grout Strength	f'cg	=	3,000	psi	=	21 MPa	
Tower Anchor Bolts							
Specified Post Tension of Bolts	*Ps*	=	**76,200**	lbs	=	525 MPa	
Number of Tower Bolts	nb	=	140		Williams Grade 80, 1-1/4"		
Nominal Diameter of Tower Bolts	db	=	1.38	inches	=	35 mm	
Length of Bolts	Lb	=	67	inches	=	1.702 m	

Reinforcement Schedule	Bar Size	LF/ea	No. Pieces	LF	Weight/ea (lbs)	Weight (lbs)	Weight kN
Radial Steel							
Type A	10	16	35	560	69	2410	10.72
Type B	10	16	35	560	69	2410	10.72
Type C	10	15	35	525	65	2259	10.05
Type D	10	15	35	525	65	2259	10.05
Stirrups	10	7.50	70	525	32	2259	10.05
Circumferential Hoops	9	57.2	18	1030		3501	15.57
Center Mat							
Top	10	10.1	36	363.6		1565	6.96
Bottom	10	10.1	36	363.6		1565	6.96
Total				4452		18226	81.1

FOUNDATION LOADS Based on: Preliminary Loading Data for MWT95/2.4-80m with IEC Class IIA

At base of Tower				Axial	Horizontal	Moment						
Load Case	Description	DLC	Load Factor	Fz KN	Fxy KN	Mxy KN-m	Fz kips	Fxy kips	Mxy kip-ft	% Mext	Load Case	Blade
1	Extreme Load	6.1j	1.00	**-1845**	**523**	**34,908**	-414.7	117.7	25,747	100%	1	GE37c
2	Lift-off	1.0	1.00	**-1959**	**219**	**17,596**	-440.4	49.2	12,978	50%	2	LM37.3p2
3	Tilting	6.1j	1.00	**-1845**	**523**	**34,908**	-414.7	117.5	25,747	100%	3	GE37c
4	Sliding	6.1j	1.00	**-1843**	**523**	**34,863**	-414.4	117.7	25,714	100%	4	GE37c
5	Shear Failure (Beari	6.1j	1.35	**-2490**	**706**	**47,125**	-559.8	158.6	34,758	135%	5	GE37c
6	Tension Loadinç	1.1	1.00	**-1955**	**327**	**24,238**	-439.6	73.6	17,877	69%	6	LM37.3p2
7												

Patrick & Henderson Anchor Foundation
Stetson Mountain Wind Project GEWE 1.5sle 1.5 MW on 80-m Hub Height Tower
Code References based on 2006 IBC & ACI 318

1.0	**Design Input**	Equations or Notes	Symbol	Imperial Units		Metric Units	
Item	***Cap***			Value	Unit	Value	Unit
1	Cap Inside Diameter		Di	0.00	feet	0	mm
2	**Cap Outside Diameter**		Do	**24.00**	feet	7315	mm
3	**Thickness of Foundation Cap**		T	**5.00**	feet	1524	mm
4	Inside Bolt Diameter		Bi	13.5499	feet	4130	mm
5	Outside Bolt Diameter		Bo	14.5210	feet	4426	mm
6	Inside Base Flange Diameter		Id	13.1234	feet	4000	mm
7	Outside Base Flange Diameter		Od	14.9471	feet	4556	mm
8	Diameter of Tower Wall		Dwall	14.0715	feet	4289	mm
9	Width of Base Flange on Tower		bf	10.942	inch	278	mm
10	Inside Width		bfi	4.350	inch	111	mm
11	Base Flange Thickness		Bft	2.95	inch	75	mm
12	Embedment Plate Width		Pw	10.942	inch	278	mm
13	Diameter of Base Flange Bolt Hole		d	1.500	inch	39	mm
14	Nominal Diameter of Bolts	Williams Grade 80, 1-1/4"	db	1.375	inch	35	mm
15	Area of Bolts		ab	1.27	sq-in	819	
16	Number of Tower Bolts		nb	140		140	
17	Yield Stress of Bolts		fy	75,000	psi	517	MPa
18	Ultimate Stress of Bolts	Grade 75	fpu	**100,000**	psi	689	MPa
19	Ultimate Strength of Bolts		Fu	127,000	lbs	565	KN
20	**Specified Post Tension of Bolts**		Ps	**76,200**	lbs	525	MPa
21	Yield Strength Tower Base Plate	Grade 50	fyb	50,000	psi	345	MPa
22	Rebar Yield Strength	Grade 60	fy	60,000	psi	414	MPa
23	Grout beneath Flange - Strength		f'g	**8,500**	psi	59	MPa
24	**Cap Concrete Compressive Strength**		f'c	**6,000**	psi	41	MPa
25	Concrete Modulus of Elasticity	$Ec = 57{,}000*f'c^{0.5}$	Ec	4,415,000	psi	30,400	MPa
26	Steel Modulus of Elasticity		Es	29,000,000	psi	200	GPa
27	Density of Concrete		γ_c	150	pcf		
28	CMP (3 x 1 Helical)			12 Gauge			
29	CMP Uncoated Thickness		t	0.109	inch	2.8	mm
30	CMP Developed Width Factor		be	1.241			
31	***Pile Anchors***						
32	**Number of Anchors**		*na*	**14**			
33	Outer Anchors		*no*	**14**			
34	Inner Anchors		*ni*				
35	Outer Anchor Circle Diameter		*Di*	**20.00**	feet		
36	Inner Anchor Circle Diameter		*Do*		feet		
37	Effective Anchor Circle Diameter	Da = (no*Do+ni*Di*Di/Do)/na	*Da*	20.00	feet		
38	**Specified Anchor Post Tension Force**		*Pt*	**435**	kips		
39	Anchor Bolt - Strength	Williams Grade 150, 2.5"	Fu	150,000	psi		
40	Anchor Bolt Area		ab	5.19	sq-in		
40	Anchor - Grout Strength		f'cg	3,000	psi		

2.0	**Loads**	Based on: Preliminary Loading Data for MWT95/2.4-80m with IEC Class IIA					
	Base Shear	*Characteristic Extreme loads*	Fxy	117,665	lbs	**523**	KN
	Dead Load	*no load factor*	Fz	414,660	lbs	**1,845**	KN
	Maximum Moment	*applied*	Mxy	25,746,608	ft-lbs	**34,908**	KN-m

				P (kips)	V (kips)	M (k-ft)	LF
	Dead Load		D	414.7			
	Extreme Wind		W		117.7	25,747	1.00
2.1	**Characteristic Loads**			**414.7**	**158.6**	**34,758**	
2.3	**Applicable ACI Load Combinations - Section 9.2.1**			P_u (kips)	V_u (kips)	M_u(k-ft)	
	U = 0.9D+1.3W	Eq. (9-6)	with exception (b) 1.3W	373.2	153.0	33,471	1.30
	U = 1.2D+1.3W	Eq. (9-5)		497.6	153.0	33,471	1.30
	U = 1.2D+1.0E	Eq. (9-7)		497.6	0.0	0	1.00
	IEC Load Case	IEC Load Factor	1.35	**414.7**	**214.1**	**46,923**	

3.0 Resistance To Base Shear (at Top of Foundation)

Allowable Shear	Fv = 0.17*fu*ab*nb	Fv =	3,023	kips
Factored Base Shear		Vu =	153	kips
Fv >> Vu ?			**OK**	

4.0 Required Thickness of Base Flange

Area of Base Flange	$A_F = \pi/4\ [144*(Od^2 - Id^2) - nb*d^2]$		5,542	sq-in.
Compression at Base Flange	$fcbf = (Uc+Fpu)*nb/A_F$	fcbf =	3,605	psi
Required Thickness	$tb = bfi*[3*fcbf/(0.75*fyb)]^{0.5}$	tb =	2.34	inches
Base Flange Thickness Design by Others		Bft =	2.95	inches
Bft > tb ?			**OK**	

ACI 318 Code Compliance Checks

5.0 Tower Anchor Bolt Analysis - ACI 318 Section 9.2.1

Dead Load	D = Fz/nb	D =	2,962	lbs
Wind Load	W = 4Mext/(nb*Dwall)	W =	52,277	lbs
Earthquake	E = 4Mxy/(nb*Dwall)	E =	0	lbs
Min. Required Post Tension Load	Set near W - 0.9D	P =	49,611	lbs
Specified Post Tension Load		Ps =	76,200	lbs
Effective Post Tension Load	Pse = Ps*0.9 with losses	Pse =	68,580	lbs
Factored Tension	Ut = -0.9 D + 1.35 W (or 1.0 E)	Ut =	67,908	lbs
Allowable Anchor Tension	$Fb = ab*0.7f_{pu}$ per 18.5.1c	Fb =	88,900	lbs
Fb > Ut ?			**OK**	

5.5 Pile Anchor Bolt Analysis - ACI 318 Section 9.2.1

Foundation Weight	$Wt\text{-}fnd = \pi Do^2/4*T*\gamma'_c$	Wt-fnd =	339	kips
Dead Load	D = (Fz+Wt-fnd)/na	D =	54	kips
Wind Load	W = 4Mext/(na*Da)	W =	368	kips
Earthquake	E = 4Mxy/(na*Da)	E =	0	kips
Min. Required Post Tension Load	Set near W - 0.9D	P =	319	kips
Specified Post Tension Load		Ps =	435	kips
Effective Post Tension Load	Pse = Ps*0.9 with losses	Pse =	392	kips
Factored Tension	Ut = -0.9 D + 1.35 W (or 1.0 E)	Ut =	448	kips
Allowable Anchor Tension	$Fb = ab*0.7f_{pu}$ per 18.5.1c	Fb =	545	kips
Fb > Ut ?			**OK**	

6.0 Compression on Grout Bearing Analysis - ACI 318 Section 9.3.2.5

Factored Compression per bolt	Uc = 1.2 D + 1.35 W (or 1.0 E)	Uc =	74,128	lbs
Area Base Flange	$A_F = \pi/4\ [144*(Od^2 - Id^2) - nb*d^2]$		5,542	sq-in
Compression by moment	$Cm = Uc*nb/A_F$	Cm =	1,872	psi
Compression by post tension	$Cps = Ps*nb/A_F$	Cps =	1,925	psi
Total Compression on Grout		Cgt =	3,797	psi
Strength reduction factor		ϕ =	0.85	
Grout Compressive Strength		f'g =	8,500	psi
Nominal Grout Strength	$Cgn = \phi*f'g$	Cgn =	7,225	psi
Cgn > Cct ?			**OK**	

7.0 Concrete Stress under Grout Analysis - ACI 318 Section 9.3.2.5

Grout Trough Depth		dg =	2.0	inches
Area under grout	$A_G = A_F + 2*dg*Dwall*12*\pi$	A_G =	7,664	sq-in
Compression	$Cct = (Uc + Ps)nb/A_G$	Cct =	2,746	psi
Strength reduction factor		ϕ =	0.85	
Concrete Compressive Strength		f'c =	6,000	psi
Nominal concrete strength	$Ccn = \phi*f'c$	Cn =	5,100	psi
Ccn > Cct ?			**OK**	

8.0	**Stresses In Concrete at Sustained Service Loads - ACI 318 - Section 18.4.2(a)**			
	Grout under Tower Base Flange			
	Stress from Pre-Stress Force	$Sp = nb*Ps/A_F$	*Sp =*	1,925 psi
	Stress from Sustained Tower Load	$Ss = Fz/A_F$	*Ss =*	75 psi
	Total Stress on Grout	St-s = Sp + Ss	*St-s =*	2,000 psi
	Nominal Grout Strength	Sg = ϕ*f'g, ϕ = 0.45	*Sg =*	3,825 psi
	Sg > St-s ?			**OK**
	Concrete under Grout			
	Stress from Pre-Stress Force	$Sp = nb*Ps/A_G$	*Sp =*	1,392 psi
	Stress from Sustained Tower Load	$Ss = Fz/A_G$	*Ss =*	54 psi
	Total Stress on Concrete	St-s = Sp + Ss	*St-s =*	1,446 psi
	Nominal Concrete Strength	Sc = ϕ*f'c, ϕ = 0.45	*Sc =*	2,700 psi
	Sc > St-s ?			**OK**

9.0	**Stresses In Concrete at Transient Service Loads - ACI 318 - Section 18.4.2(b)**				
	Grout under Tower Base Flange				
	Service Load Moment (Msl)	Use Max Operational	Msl =	12,873 ft-kips	17,454 KN-m
	Axial Load from Msl per bolt	$Usl = 4Msl/n_b/D_{wall}$	*Usl =*	26,139 lbs	
	Stress from Moment	$Sm = nb*Usl/A_F$	*Sm =*	660 psi	
	Stress from Pre-Stress Force	$Sp = nb*Ps/A_F$	*Sp =*	1,925 psi	
	Stress from Sustained Tower Load	$Ss = Fz/A_F$	*Ss =*	75 psi	
	Total Stress on Grout	St-t = Sm + Sp + Ss	*St-t =*	2,660 psi	
	Nominal Grout Strength	Sg = ϕ*f'g, ϕ = 0.60	*Sg =*	5,100 psi	
	Sg > St-t ?			**OK**	
	Concrete under Grout				
	Stress from Moment	$Sm = nb*Usl/A_G$	*Sm =*	477 psi	
	Stress from Pre-Stress Force	$Sp = nb*Ps/A_G$	*Sp =*	1,392 psi	
	Stress from Sustained Tower Load	$Ss = Fz/A_G$	*Ss =*	54 psi	
	Total Stress on Concrete	St-t = Sm +Sp + Ss	*St-t =*	1,923 psi	
	Nominal Concrete Strength	Sc = ϕ*f'c, ϕ = 0.60	*Sc =*	3,600 psi	
	Sc > St-t ?			**OK**	

11.5	**Shear CapacIty Analysis - between Tower and Anchor ACI 318 - Section 11.3.1**			
	Beam Width per bolt pair	bw = π*(Da+Dwall)/nb	bw =	9.17 inches
	Beam Depth	d = T - 6'	d =	54.00 inches
	Shear Force Across Section	$Vu = 2*4*Mu/n_b/D_a$	Vu =	134.1 kips
	Shear Stress	v = Vu/(bw*d)	v =	155 psi
	Shear Capacity			
	Equation (11-3)	$Vc = 2*f'c^{0.5}*bw*d$	Vc =	76.8 kips
	Area of Stirrup Bar	#10 bar	*As =*	1.27 in^2
	Stirrup Spacing		*s =*	24.0 inches
	Rebar Yield Strength		*fy =*	60,000 psi
		Vs = 0.85As*fy*d/s	Vs =	145.7 kips
	Nominal Shear Capacity	*Vn = ϕ(Vc+Vs), ϕ = 0.75*	Vn =	166.9 kips
	ϕVn > Vu?			**OK**
	Therefore use #10 bar @ 24-inches or equivalent			

11.7	**Uplift CapacIty Analysis of Embedment Ring**			
	Beam Width per bolt pair	bw = π*Dwall/(nb/2)	bw =	7.58 inches
	Eff. Depth for Embedment Ring	d = T - 6' - 8" -3"	d =	43.00 inches
	Uplift (Shear) Force per Bolt	Vu = Ut	Vu =	67.9 kips
	Shear Stress	v = Vu/(bw*d)	v =	208 psi
	Shear (Uplift) Capacity of Half Cone projecting at 45 degrees upward			
	Equation (11-3)	$Vc = 2*f'c^{0.5}*bw*d$	Vc =	50.5 kips
	Area of Stirrup Bar	#10 bar	*As =*	1.27 in^2
	Stirrup Spacing	Consider one bar through plane	*s =*	24.0 inches
	Rebar Yield Strength		*fy =*	60,000 psi
		Vs = 0.85As*fy*d/s	Vs =	116.0 kips
	Nominal Shear Capacity	*Vn = ϕ(Vc+Vs), ϕ = 0.75*	Vn =	124.9 kips
	ϕVn > Vu?			**OK**
	Therefore use #10 bar @ 24-inches or equivalent			

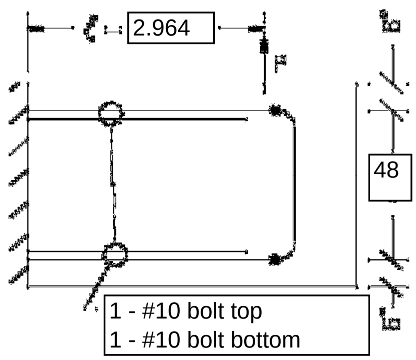
11.9 Check Embedment Ring Thickness				
Embedment Plate Width		b = bf =	10.94	inches
Embedment Plate Thickness		t = Pt =	**1.50**	inches
Required Thickness	tb = bfi*[3*fcbf/(f*fyb)]0.5	tb =	1.61	inches
			Marginally OK	
Radius of between bolt pairs	$l = r_{bp} = \pi$Dwall/nb*2	l =	7.58	inches
Consider embedment ring as cantilever beam between bolt pairs with compressive load set at Ut				
Compression at Base Flange	fcbf = Ut*nb/A_{BF}	fcbf =	1,715	psi
Compressive Load	w =fcbf*bf	w =	18,770	lb/in
Moment	M = w*l^2/12	M =	89,832	in-lbs
Section Modulus of Ring	S =bd^2/3	S =	8.21	in^3
Bending Stress	fb = M/S	fb =	10,946	psi
Yield Strength Tower Base Plate	ϕ = 0.75	ϕfyb =	37,500	psi
ϕfyb > fb ?			**OK**	

12.0 Flexural Capacity - Reinforcement Analysis

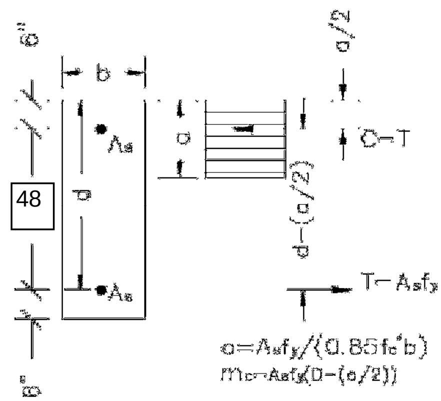
Moment Calculation and Uplift - consider width of foundation at each bolt pair:

Uplift Force Induced by Moment	Pu = Ut-anchor*2*na/nb	Pu =	89.6 kips
Beam Width	b = π*(Da+Dwall)/nb	b =	0.765 feet
Uplift Force Arm	L = (Da -Dwall)/2	*L* =	2.964 feet
Moment Induced by Uplift Force	Mp = Pu*L (tension side)	*Mp* =	**265.6 ft-kips**
Elastic Section Modulus	S =bd^2/6	*S* =	5,505 in^3
Elastic Bending Stress	fb = M/S	fb =	579 psi

Section Check:



Double Reinforcement Check:



Design Radial Reinforcement - Top Section @ each bolt pair

Area of Bar	1 - #10 bolt top	*As* =	**1.27** in^2
Bar Yield Strength		*fy* =	60,000 psi
Bar Yield Force	T = Asfy	T =	76.2 kips
Location of Netural Axis	a = Asfy/(0.85f'c*b)	a =	1.63 inches
Bar Tension Force Arm	d = T - 6"	d =	54.0 inches
Nominal Moment Capacity	$\phi(A_sf_y)(d - a/2)$, ϕ = 0.90	ϕMc =	304.0 ft-kips
ϕMc > Mp?			**OK**

Design Radial Reinforcement - Bottom Section @ each bolt pair

Bearing Stress	q = na*Pt/A + Pu/(b*L)	q =	53.0 ksf
Unit Load	w = q*b	w =	40.5 kips/ft
Moment Induced by Bearing	Mq = w*L^2/2	*Mq* =	**178.0 ft-kips**
Elastic Bending Stress	fb = M/S	fb =	388 psi
Area of Bar	1 - #10 bolt bottom	*As* =	**1.27** in^2
Bar Yield Strength		*fy* =	60,000 psi
Bar Yield Force	T = Asfy	T =	76.2 kips
Location of Netural Axis	a = Asfy/(0.85f'c*b)	a =	1.63 inches
Bar Tension Force Arm	d = T - 6"	d =	54.0 inches
Nominal Moment Capacity	$\phi(A_sf_y)(d - a/2)$, ϕ = 0.90	ϕMc =	304.0 ft-kips
ϕMc > Mq?			**OK**

Design Circumferential Hoops

Area of Rebar	1 - #9 rebar	*As* =	**1.00** in^2
Spacing (beam width)	@ 9 inches or equivalent	s = b =	0.75 feet
Rebar Yield Strength		*fy* =	60,000 psi
Rebar Yield Force	T = Asfy	T =	60.0 kips
Elastic Section Modulus	S =bd^2/6	*S* =	5,400 in^3
Elastic Bending Stress	fby = M/S take as 50% of fbx	fb =	290 psi
Max. Circumferential Bending Moment	Mc = fby *S	*Mc* =	130.3 ft-kips
Location of Netural Axis	a = Asfy/(0.85f'c*b)	a =	1.31 inches
Nominal Moment Capacity	$\phi(A_sf_y)(d - a/2)$, ϕ = 0.90	ϕMc =	240.1 ft-kips
ϕMc > Mp?			**OK**