

Wind Turbine Foundation Behavior and Design Considerations

Kirk Morgan, Eric Ntambakwa
Garrad Hassan America, Inc.

ABSTRACT

Despite certain aspects of wind turbine foundations which are common industry-wide, there is marked diversity in the understanding of the foundation behavior and application of design codes. This has led to different foundation designs for identical turbines with similar geotechnical conditions. The wind industry continues to grow using larger multi-megawatt turbines. An understanding of the driving considerations for the associated growth of the foundations will permit safe and economical designs while giving due consideration to design codes that may not explicitly address wind turbine foundations. This paper discusses the key design criteria and components inherent to wind turbine foundation design, specifically the spread footing. Potential impacts to these areas from larger turbines and how the engineering/construction community may address them are explored.

1 INTRODUCTION

1.1 Foundation types

Wind turbines are founded on a variety of foundation types that for engineering purposes are primarily selected depending on geotechnical conditions and the foundation method of developing resistance to overturning forces.

The simplest form of foundation is the spread footing which is essentially a gravity foundation that relies upon soil overburden and concrete to provide sufficient weight to resist overturning of the foundation at extreme wind loads. This type of design is applicable in a broad range of subgrade strengths from soils to rock.

Rock socketed and short piers rely primarily upon end bearing and secondarily upon side wall friction and lateral earth bearing pressures. Their strength is derived from subgrades of significant strength at relatively shallow depth.

Pile and cap foundations are often found in regions where competent soil or rock is found at much greater depths. Piles transmit loads from the cap to the subgrade via a combination of friction and end bearing and resist lateral loads through lateral earth pressure on the pile.

Rock anchor and cap foundations are typically encountered in regions where strong bedrock is reachable at shallow depth. The foundation resists loads through a combination of bearing pressure beneath the cap at the bearing layer and tension in steel bars grouted into boreholes and post-tensioned (rock anchors).

Other hybrid types of foundations exist and their mechanisms for resisting overturning forces can be seen to be a combination of the mechanisms employed in the above foundation types.

This paper focuses on the spread footing.

1.2 Purpose

The purpose of this paper is to discuss certain areas of on-shore spread footing design and how they are expected to be impacted with the advent of larger turbines. Broad design criteria are detailed. Key design components are detailed and reference is given to anticipated changes and challenges facing foundation designers and contractors.

1.3 Impetus

From a distance, while turbines may have similarities across a range of rated power values, identical turbines in similar ground conditions can have very different spread footing foundation designs. Some of these differences appear in the footing width, thickness, amount of reinforcing steel, anchor bolt size and post-tension force, and construction requirements.

At stake are expected increases in construction costs as machine rated power increases. Three figures are provided with respect to rated power increases. Figure 1 depicts rising footing concrete volume. Figure 2 shows rising unfactored extreme overturning moment (footing structural demand). Figure 3 depicts rising tower base flange factored bearing pressure.

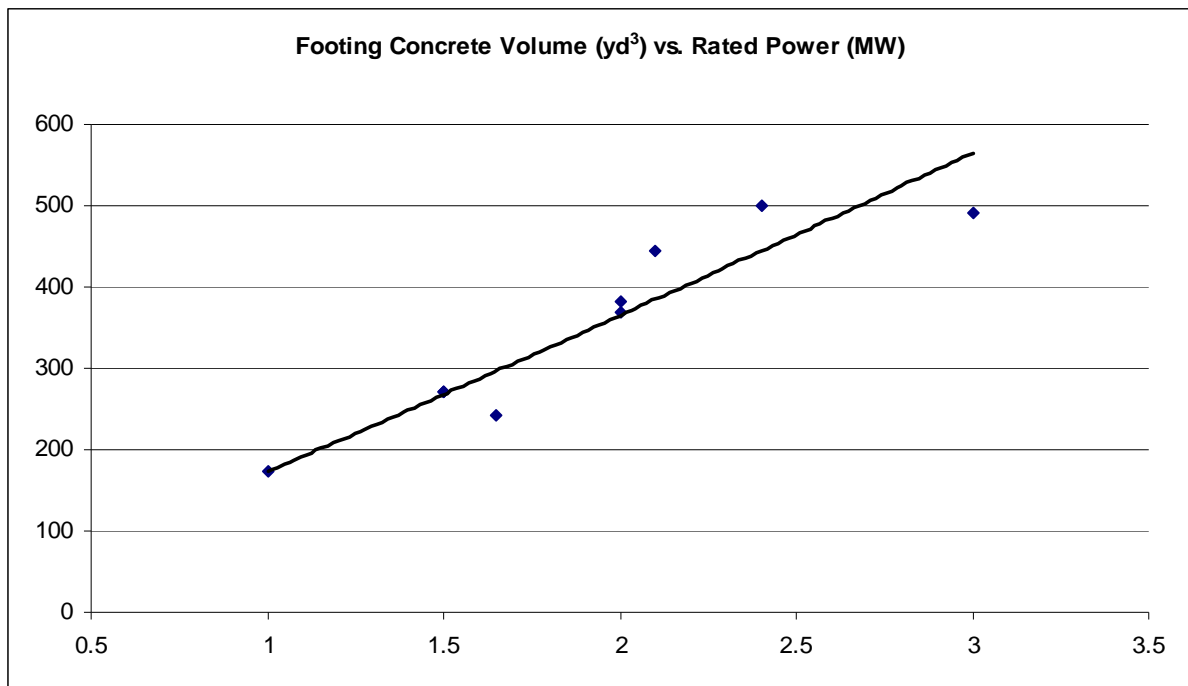


Figure 1

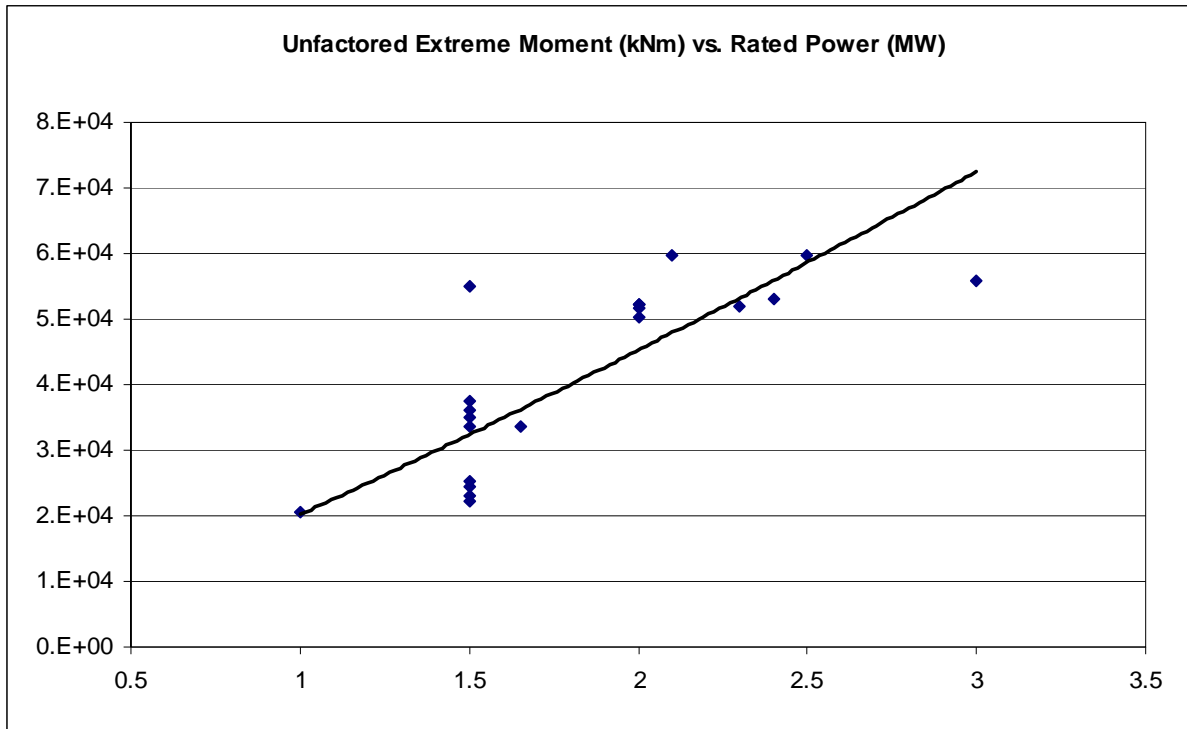


Figure 2

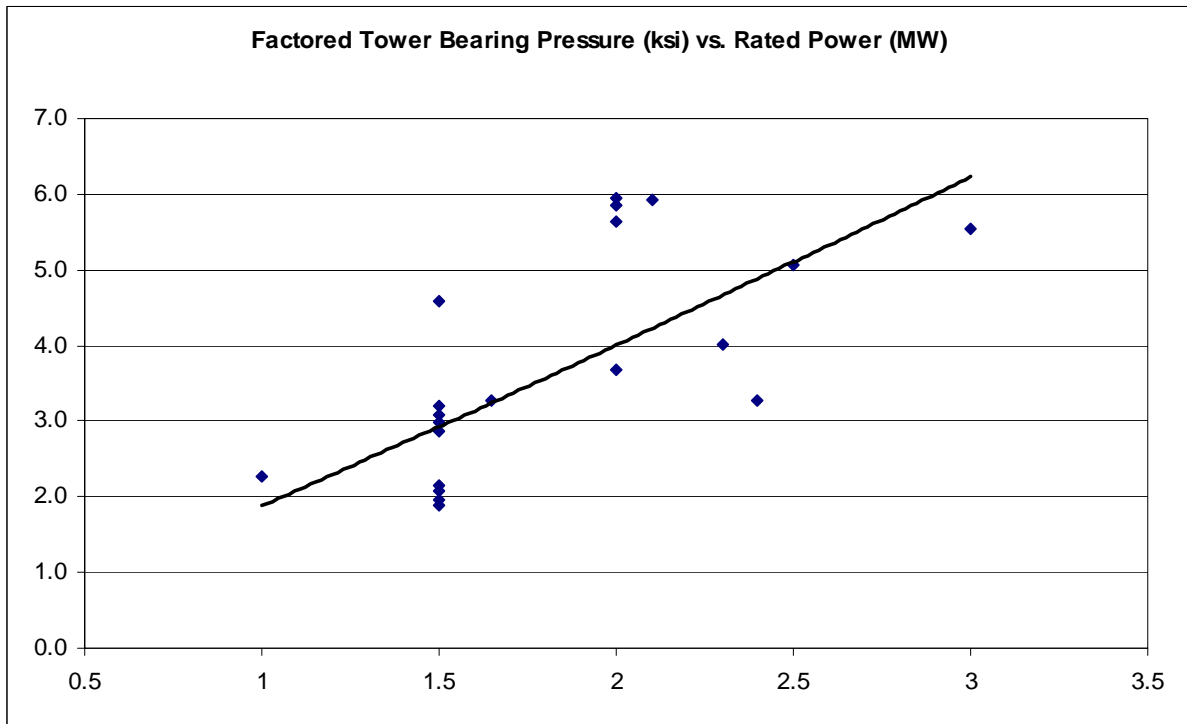


Figure 3

As turbine rated power increases, improved understanding of foundation behavior during design will help minimize the cost increases associated with the above trends. In addition, consensus within the industry on how foundation behavior drives design decisions will ensure consistent performance in the key areas of code provisions, safety margins, and predicted life.

2 CRITERIA

The following is an overview of foundation design criteria. It provides some relevant detail but is not meant to be exhaustive.

The essential criteria for wind turbine foundation design are:

- Stiffness
- Strength
- Stability
- Differential Settlement
- Durability
- Economy

Foundation designers are charged with ensuring that the various criteria are thoroughly addressed. All of this is undertaken with due consideration for the balance between technical ideals such as design efficiency and safety level and certain practical realities such as component shipping requirements and construction techniques.

The following segments will expand on the basic criteria listed above.

2.1 Stiffness

The foremost serviceability criterion and the most common foundation design specification provided by turbine manufacturers other than loads, is the minimum foundation rotational stiffness.

Foundation rotational stiffness is dependent on the soil-structure interaction of the flexible concrete footing with the flexible subgrade. Turbine manufacturers, in general, specify the required minimum foundation rotational stiffness instead of other dynamic metrics such as peak displacement or velocity at certain operating states. Lack of the more precise specifications tends to limit the adjustable parameters at the designer's disposal to subgrade stiffness; i.e. soil damping and mobilized mass are generally not considered though arguments can be made that, with more detailed performance criteria from the turbine manufacturers, consideration of these additional foundation dynamic characteristics could lead to more economical foundation designs.

Given the specified target of minimum rotational stiffness and the uncertainties associated with subgrade stiffness measurements as well as footing structural stiffness, spread footings are commonly designed assuming rigid footing behavior with an elastic half-space or a multilayer space representing the subgrade. There are various appropriate

references (Arya, et al. and DNV). For spread footings proportioned properly employing a rigid foundation analysis, margins over the required minimum stiffness are generally such that when the flexibility of the foundation is included the total foundation stiffness can still be adequate.

Insofar as operational loads contribute the vast majority of fatigue-related stresses on turbine components, foundation stiffness has historically been required for a range of operational loads only. Not all manufacturers are clear in specifying the upper load limit for which the foundation minimum rotational stiffness must be in effect, however. When turbine manufacturers do specify this load, in the case of spread footings, this upper operational load is referred to as the “gapping” load. The check on minimum rotational stiffness at a certain upper operational load is thereby, in the case of a spread footing, converted to a check on contact pressures on the upwind side of the footing (point of “gap” equal to zero contact pressure). This gapping limit duly acknowledges that in the case of a spread footing the rotational stiffness decreases non-linearly after foundation uplift/zero pressure. In certain soil conditions, a limit on gapping will also ensure that soils subject to cyclic degradation are prevented from experiencing multiple instances of zero pressure which, in the presence of water, could lead to breakdown of the in-situ soil structure and subsequent related serviceability problems.

Horizontal stiffness as well as coupled stiffness (off-diagonal terms in stiffness matrices) are sometimes specified in addition to rotational stiffness because they are necessary for design of other (deep) foundation types besides the spread footing.

2.2 Strength

The wind turbine foundation must have sufficient strength to withstand turbine extreme loads with a margin of safety (factored loads). In addition, the foundation must be designed to withstand fatigue damage due to operational loads and certain functions such as nacelle yawing and start-ups and shut-downs.

Common wind turbines for the utility market have loads specified per IEC standards. Turbine manufacturers provide these loads at the tower base in specifying foundation design requirements. Foundations designers consider these in addition to soil and concrete weights, as well as buoyant and seismic forces to establish the structural demands on foundation elements.

The computed structural loads (serviceability limit state or ultimate limit state) are compared to subgrade and structural capacity to determine whether sufficient strength exists.

A variety of approaches to determination of subgrade strength is available that are outlined in reference texts, DNV Guidelines, Eurocode 7, and FHWA and USACE documents. US guidelines tend to employ allowable stress design approaches to subgrade strength evaluations and European codes tend to employ ultimate limit state methods.

Various codes are used to assess structural capacity in the US, with the predominant code being that of the American Concrete Institute (ACI) as, by reference in most jurisdiction building codes, it is the most frequently required code for reinforced concrete structures. The ACI code has certain limitations which, e.g. in the case of fatigue design, lead to frequent use of certain European codes better suited to address fatigue-related design issues (DNV, Eurocodes). All codes used to evaluate structural strength incorporate load and resistance factors or partial material factors for both serviceability limit state and ultimate limit state design.

2.3 Stability

Spread footing stability refers to the foundation's resistance to excessive movement (rotation or translation) under extreme loads. For geotechnical work in the US the Factor of Safety (FoS) approach is employed in this exercise. In most cases the spread footing is first proportioned for overturning stability (and bearing capacity) and then horizontal (sliding) stability is evaluated and is typically determined to be acceptable due to the large frictional contact area.

2.4 Differential Settlement

Foundations must remain level within limits specified by the turbine manufacturer but not all turbine manufacturers specify these limits. Due consideration should be provided for both short-term (recoverable elastic) and long-term (plastic, consolidation) differential settlements. A variety of approaches with respect to geotechnical practice are applicable. It should be noted, however, that careful consideration must be given to the combination of dead loads with turbine operating states for the various checks on foundation differential settlement.

2.5 Durability

Turbine foundations should be constructed to resist the deleterious effects of the environment in which they are placed in order to maintain serviceability for the design life of the turbine. Issues of reinforcing corrosion, concrete chemical attack, and freeze / thaw cycling are addressed in various codes. The American Concrete Institute provisions for durable concrete are most widely used in the US.

2.6 Economy

The incentive for wind turbine foundations to be designed economically is self-evident for all foundation types, not just the spread footing.

Trade-offs between excavation and concrete volume and steel tonnage arise as the incremental costs of labor and each material vary. Certain aspects of the footing geometry can be optimized depending on the pricing of materials and labor.

An inherent conflict emerges when large projects face steel and concrete supply / cost issues, which would prompt the need for very efficient structural designs, but geotechnical uncertainties create a need for a very robust and possibly inefficient design.

In some cases cost savings in geotechnical investigations that may be attractive for a large site may not translate well to cost savings in steel and concrete. In some instances cost savings on design-phase geotechnical investigation can lead to construction-phase cost overruns due to unanticipated turbine moves or ground improvement requirements owing to unforeseen geotechnical conditions. Thus a reduced cost for geotechnical investigations can be seen to detract from the overall foundation design value given certain circumstances.

3 KEY COMPONENTS

3.1 Stability

Design for stability compares unfactored overturning moments and horizontal forces with resisting moments and sliding resistance. For both sliding and overturning stability, convention among geotechnical designers is that the FoS must be greater than or equal to 1.5. This minimum is established to preserve safety while allowing for minimal amounts of soil backfill and concrete dead weight (resistance mechanism).

For overturning stability the convention also is to consider moments about an axis at the toe (downwind edge) of the foundation.

In the area of stability Germanischer Lloyd and certain turbine manufacturers also employ a specification that limits foundation uplift at unfactored extreme loads to 50% of the footing, in order to prevent overturning of the foundation during an extreme wind event.

3.1.1 Stability – Impact of Larger Turbines

It can be anticipated that as turbines become larger and foundation loads increase that wider, deeper, heavier foundations will be required as long as the criteria and methods for evaluating stability remain the same.

To combat this, enhanced analysis of the footing interaction with the surrounding excavation side walls may allow for realization of more overturning resistance through consideration of a soil wedge on the perimeter of the footing. This can only be undertaken with due consideration of the lower bound properties for backfill unit weights and strength properties in addition to construction earthwork specifications which will ensure that these properties are obtained and can be verified.

Although less foreseeable for the immediate future, with reference to some of the ductile failure modes achievable with reinforced concrete, as turbines (and therefore footings) get bigger it may be reasonable to expect that upper bound limits on total footing rotation may be specified by turbine manufacturers as a stability criterion to prevent so much rotation of the footing that tower buckling becomes a concern. A short study suggests that this rotation amount might be as much as six degrees (a seemingly large number) but this would need to be understood with due consideration for load combinations and

factored loads as needed as well as all sources of movement from pedestal anchorage yielding, to flexural cracking of the footing base, to elasto-plastic soil deformation.

3.2 Bearing Capacity

Bearing capacity is traditionally evaluated with allowable stress design approaches according to various reference texts including DNV Guidelines, FHWA, and USACE documents. The bearing capacity factors per Meyerhof (N_q , N_c) are the most widely accepted. N_γ may be per DNV, Meyerhof, Hansen, or Vesic with engineering judgment applied as to which is appropriate for a given project based on the results of the geotechnical investigation and perhaps the standard of practice within a give area.

Because the predominant loading from the turbine is in the form of overturning moment, the eccentricity of the bearing pressure drives the design for bearing capacity. For extreme loads at and above the point of footing uplift, adequate bearing capacity can be difficult to achieve due to the strong nonlinearity between the applied moment and the foundation contact pressure.

Bearing capacity is often evaluated at both service and extreme loads with appropriate FoS and triangular/trapezoidal contact pressure distributions as follows:

- Capacity at service / fatigue loads, $FoS \geq 3$
- Capacity at unfactored extreme loads, $FoS \geq 2.26$

This approach can lead to difficulties when ultimate limit state design is employed for structural elements as detailed in a later section herein. In addition, given that the rest of the turbine and tower system is designed with partial or global load factors, the design for subgrade capacity should follow with design for factored loads to obtain good economy and consistent safety level.

In the context of factored loads for consideration of bearing capacity, there are two major issues at play. The first is that the footing load eccentricity is high a priori at unfactored loads with resultants at the edge of or well-outside the central third of the footing (See figure 4). Almost by definition this leads to difficulty in applying separate dead and wind load factors to the contributions to bearing pressures from each source (as is often done in normal building structures) for purposes of computing structural element demands. The second issue is that the major load factor (1.35) is applied to the wind load (overturning). When this load factor is applied to the location where the turbine load is introduced to the footing (and not instead applied to the unfactored bearing response) this amplifies the eccentricity already in place and drives the bearing pressures higher and further away from the centerline of the footing (See figure 5). To demonstrate the strong nonlinearity referred to above, comparing Figure 5 to Figure 4, note the contact pressure has increased by nearly 50% when the wind overturning load has increased by only 35% with the appropriate IEC load factor for normal extreme events. In these two graphics it is noteworthy to mention that the pressure areas depicted are the reduced effective contact

pressures per the Meyerhof method for eccentrically loaded foundations which can be applied at both factored and unfactored loads for appropriate structures.

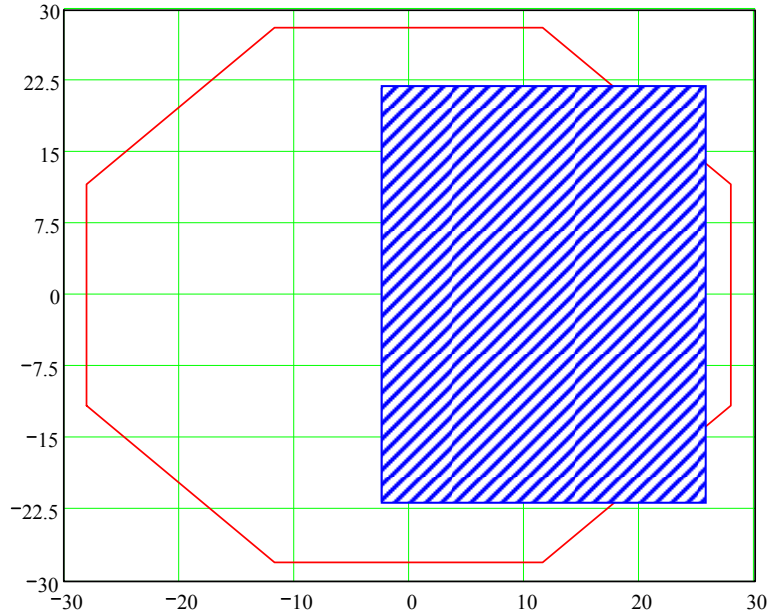


Figure 4 – 2600 ft² footing, Unfactored Loads, 1200 ft² contact area, 3280 psf contact pressure

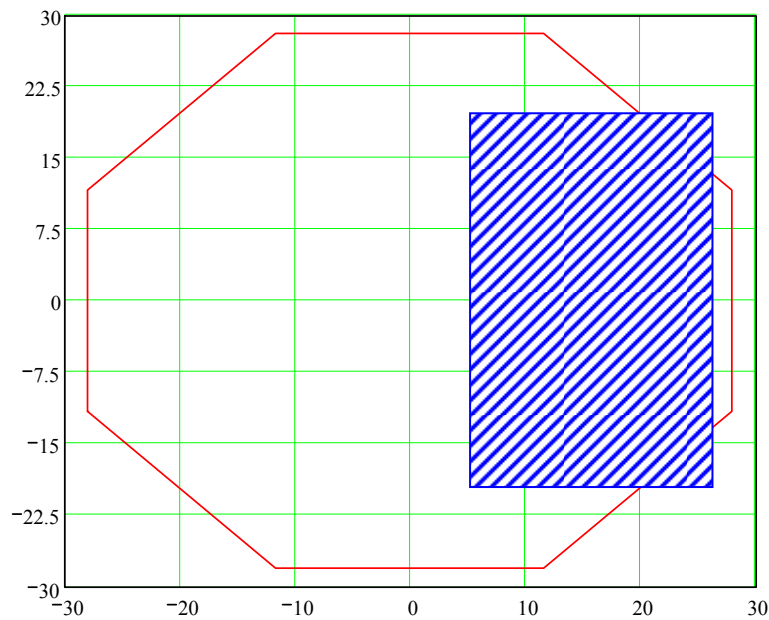


Figure 5 – 2600 ft² footing, Factored Loads, 800 ft² contact area, 4900 psf contact pressure

3.2.1 Bearing Capacity – Impact of Larger Turbines

In the area of soil bearing capacity, as turbines increase in size with corresponding higher demands, in order to limit the incremental costs of larger footings it may prove beneficial to begin to adopt ultimate limit state (ULS) checks over the allowable stress design (ASD) checks commonly employed. The most significant reason for this is the relative capacities (denominators in demand / capacity ratios). For a ULS check the capacity (after reduction with an appropriate material factor) may be as much as 73% greater than the ASD capacity. This would work well in the instance as per above when the ULS demand is only 50% greater than the ASD demand even when the eccentricity and bearing pressure is allowed to increase due to the contact nonlinearity of the footing.

In addition to the above, certain code bodies (AASHTO) are developing Load and Resistance Factor Design (LRFD) approaches for geotechnical design. This is similar if not identical to ULS approaches with the emphasis on reliability and strength.

Finally, given the expected increases in turbine loads and footing size, it is reasonable to expect that enhanced analysis will become more common. Such analysis will better model the soil/structure interaction in an attempt to economize on the bearing pressure computations which to date have relied on hand methods by-and-large.

3.3 Base Internal Forces

Determination of the internal forces in the base is a critical step in the structural analysis and design of the spread footing. The internal forces impact the chosen thickness of the base, the reinforcing used and, to a lesser degree, the concrete strength.

Of the base internal forces, in general, due to the nature of the spread footing and the turbine loading, there is a high proportion of force (moment or shear) that is due to the tower overturning load as compared to that portion of the internal force due to dead loads (resisting loads). Design for the structural elements must be undertaken with factored loads in keeping with modern codes, however. The load factors for dead loads such as the turbine, soil and concrete weights are usually different from the load factors for overturning loads. Thus the structural analysis approach must have a means to apply the dead and overturning load factors separately to their separate load sources. Under certain circumstances of linear structural systems, recognizing the principle of superposition, this can be done by factoring the corresponding fraction of internal forces. For the case of a geometric or material nonlinearity, however, this would not be possible and instead the load factors must be applied to the actions on the outside of the structure for computation of the correct “factored” internal forces.

Building codes in the US such as the ACI code are focused on structures tending to have high gravity loads with slight eccentricities on the footings (overturning component) and seem to offer limited guidance on load factoring for structures having severe eccentricities. One note in the commentary of Chapter 15 of ACI 318-05, “In the case of

eccentric loading, load factors may cause eccentricities and reactions that are different from those obtained by unfactored loads.” This seems to recognize the issue but unfortunately does not establish that the movement of the eccentricity and changes in the reactions due to load factors must be considered for design. Other documents by ACI discuss the determination of the size of the footing using unfactored loads and then recommend factoring the resultant bearing pressures for computing design structural forces. The lack of clarity in this regard is problematic and this is manifest in different designers approaching this issue in different ways and determining differing amounts of reinforcing required.

Another dimension of base internal forces computation is that of the transverse distribution of stresses with respect to the pedestal and direction of overturning. Some foundation designers currently employ what is often referred to as a “yield line” approach where the full tensile strength (f_y) of each reinforcing bar across the width of the base is utilized in tension under load. Other designers suggest that the footing is unable to mobilize all reinforcing in the transverse direction to the same strain level as that reinforcing immediately in the area of the pedestal. These designers use finite element analysis (FEA) to evaluate reinforcing demands and place reinforcing accordingly.

3.3.1 Base Internal Forces – Impact of Larger Turbines

As turbines become larger, the design community should begin to achieve consensus on the above issues. Aside from this, it can be anticipated that with bigger turbines come higher internal forces and therefore bigger footings, with the two main elements of these being concrete and reinforcing.

As more concrete is required for spread footing foundations, certain limits due to heat of hydration and practical limits on concrete placement will be encountered. With regard to heat of hydration, thermal cracking is the primary concern. Larger aggregates, higher amounts of pozzolanic material, and special cements may be employed to address concrete heat in addition to certain methods to control mix constituent temperatures at batching. Given some foundations for 3 MW machines (approximately 500 cy concrete) can already take as long as 8 hours to place it is reasonable to expect that new concreting methods might emerge to either speed concrete placement or separate placements into multiple lifts.

With regard to reinforcing several possibilities are foreseeable. Better analysis such as soil/structure interaction and plastic design can both lower design internal forces and possibly take better advantage of the reinforcing through allowance for plastic load redistribution. Innovations in reinforcing placement such as pre-manufactured shear reinforcing or bundled horizontal bars may be employed. Finally, as has affected elevated slab construction and slab-on-grade construction in certain cases, the use of high strength steels and post-tensioning may prove to be viable.

In general as turbines become larger and contractors and designers are confronted with requirements for increasing quantities of material the incremental costs will have to be balanced with their incremental benefits to the footing structures. Global competition for

steel and cement and certain region-specific issues will continue to place a special emphasis on achievement of this balance.

3.4 Flange Bearing Stresses

Concrete compressive stresses are highest beneath the tower base flange. Typical spread footing foundations in the US have 5000 psi concrete whose strength requirement is dictated by the configuration (grout, pedestal edge distance, etc.) and loading at the tower base.

Shore-based turbine tower diameters are limited for shipping purposes which places a constraint on design for bearing.

With regard to bearing, the short-term effects of extreme loads from tower base flanges are not well documented. There is a general lack of empirical data on the nature of failure. In addition, the long-term effects are not well documented for large turbines either although some standards do address concrete fatigue.

Approaches taken by foundation designers vary but some codes used are:

- ACI Chapter 10 – simple column bearing
- ACI Chapter 18 – post-tensioned anchorage
- AASHTO 5.7.5, 5.10.9 – bearing / post-tension / reinforcement
- PTI anchorage zone design
- Eurocode 2 - 5.10, 6.5, 8.10.x
- DNV Offshore Standards - J101, C502 – concrete fatigue

Choice of code is driven by the assumed behavior. For example if simple column bearing behavior is assumed then meeting the basic compression stress limits of ACI Chapter 10 may be sufficient. However, if the simple Chapter 10 rule cannot be applied due to very high stresses, analysis of the location as a post-tensioned anchorage under some other set of provisions (ACI or another code), with all requirements for bursting reinforcing and service level stress maxima, may be required.

3.4.1 Flange Bearing Stresses – Impact of Larger Turbines

If the flanges and tower diameters stay at the same relative proportions as turbines become larger, the bearing stresses will increase. This may have impacts in the areas of construction and design.

For construction of the tower base flange area there may be more use of high strength cementitious and epoxy grouts. This has already been observed with certain 2+ MW machines. Higher strength concrete in the pedestals may be required. Enhanced reinforcing details at the bearing area may also be required. This has already been observed with a 3 MW machine.

Designers may address higher flange bearing stresses by employing lower post-tensioning loads and looking more closely at fatigue requirements of the anchor bolts and the grout / concrete. In addition, designers may begin to evaluate the plastic capacity of the bearing area under factored loads when it is difficult to demonstrate that certain code provisions are met and provision for more sophisticated analysis is made (e.g. for the case of design against bursting).

3.5 Anchorage Pullout

The typical spread footing tower anchorage assembly is two concentric rings of anchor bolts held in place at the bottom by a steel ring plate that is embedded in the concrete. The tower base configuration in this instance is that of a tee-flange. Although common in Europe the embedded tower section form of anchorage to the footing is not common in the US.

Foundation designers face an issue with the design of the tower anchorage in that the structural element configuration is poorly handled by the reference US building code, ACI. The section of ACI dealing with embedded parts, Appendix D Anchoring to Concrete refers to steel elements that are both out of scale and of a very different geometry than that of the typical embedded tower anchorage.

Foundation designers in the industry employ a variety of rational methods to design the anchorage. Fundamentally at the beginning of design, the designer must assume a particular shape of the volume of concrete that will pull out from the foundation upon failure at factored load levels. The shape of this pullout volume has seen some variation among designers and independent reviewers. Notwithstanding this, in broad terms, two approaches to design for the anchorage exist.

The first approach can be referred to as a shear capacity approach where both the strength of concrete in shear and the benefit of vertical reinforcing in the pedestal are both included in capacity calculations.

The second approach used can be thought of as a deep beam approach whereby certain concepts applicable to the design of corbels and deep beams, as per ACI Appendix A Strut-and-Tie Models, are applied to ensure a load path from the embedment ring to the base reinforcing.

Limited or no empirical data exists to support the validity of either of the methods. However, few argue the point that insofar as the deep beam approach does not rely on concrete shear strength it is by definition more robust than the shear capacity approach. The deep beam approach also tends to result in greater quantities of reinforcing which lead to increases in comparative cost and potential problems with placement.

3.5.1 Anchorage Pullout – Impact of Larger Turbines

As turbines increase in size it can be anticipated that greater emphasis will be placed on the criticality of the assumed pullout volume (failure mode) as this impacts issues from

whether or not one can rely on concrete strength, to whether the connection has any deformation capacity (ductility) beyond initial fracture.

In conjunction with the above, there is likely to be a reassessment of the load path and reinforcing placement requirements. Some changes to reinforcing patterns of certain designers have already been observed.

To support the above changes it can be expected that enhanced structural analysis methods will be utilized to evaluate the anchorage. These might include nonlinear material models for concrete and steel with the aim to capture load effects in the plastic regime.

With reference to construction, it is expected that the conventional reinforcing cage, a circular curtain of vertical hooked bars near the face of the pedestal (sometimes inside the embedment ring diameter as well) and extending from the base into the pedestal, will change. The expected change is that more reinforcing will remain in the base than extends into the pedestal because continuity in tension between the base and the pedestal is, in most cases, not required for extreme events. The pedestal reinforcing, with reference to some of the issues cited earlier herein, may take on a different layout as well.

4 CONCLUSIONS

In summary, for shore-based turbines on spread footings, as machines reach higher rated power, the conventional design approaches will likely lead to bigger footings having more concrete and reinforcing. It is also likely that there will be increased stresses at the tower base flange and more emphasis on the tower anchorage and load transfer to the base.

Cost increases should be minimized and forestalled with enhanced soil/structure interaction analysis including a ULS approach to bearing capacity evaluations. In addition, enhanced structural analysis for both fatigue and ultimate limit state including certain plastic analyses will be helpful. Innovations in materials such as high strength grout and steels, and higher strength concrete may emerge. In broad terms, increased attention to balancing cost tradeoffs between items such as excavation depth, amount of concrete, and amount of steel may prove beneficial from project to project.

REFERENCES

1. American Concrete Institute Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)
2. DNV/Risø Guidelines for Design of Wind Turbines
3. DNV Offshore Standard DNV-OS-J101 Design of Offshore Wind Turbine Structures
4. DNV Offshore Standard DNV-OS-C502 Offshore Concrete Structures
5. IEC 61400-1 Ed.3: Wind Turbines – Part 1: Design Requirements
6. Eurocode 2: Design of Concrete Structures
7. AASHTO LRFD Bridge Design Specifications
8. Post-Tensioning Institute Post-Tensioning Manual