Foundation analysis for anchored wind turbine foundations

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ABSTRACT

This paper will describe the method of analysis adopted for wind turbines within a wind farm in South Australia. It will be shown that the anchored mat foundation can be considered as a piled raft for the purposes of analysis, and that the anchors are very effective in enhancing the performance of the foundation system. A comparison is made between the performance of this form of foundation, an unanchored mat foundation, a more conventional fully piled foundation and a piled raft foundation. It is demonstrated that the anchored mat foundation behaves in a comparable manner to the piled raft foundation.

1 INTRODUCTION

Community demands for sustainable energy sources have resulted in the development of wind farms in which wind power is harnessed via large turbines. These turbines generally consist of a steel tubular tower with three blades attached to a nacelle, which contains the generator, gearbox and other generating equipment. The towers are typically very tall (60-90m in height) with large rotors having a diameter of a similar order to the height. Consequently, large overturning moments and lateral loads are generated during the operation of the turbines, and the turbine foundations must therefore be designed to cater for these loadings. A common form of foundation is a large concrete "gravity" footing which resists these moments and loads through its mass. Less usually, a significantly smaller mat or raft foundation which is anchored in order to resist uplift forces generated by the imposed moments, is adopted.

This paper will describe the method of analysis adopted for wind turbines within a wind farm in South Australia. It will first set out the principles of the analysis and the means by which the foundation elements and the applied loadings were modelled. The design principles will then be described. The results for the wind farm foundation system will then be presented, and the computed behaviour for the anchored foundation system will be compared with the results obtained for alternative foundation systems.

2 ANALYSIS AND DESIGN PROCEDURE

Figure 1 shows a series of alternative foundation systems that can be used to support a wind farm structure. This paper will focus on the anchored pad system, but consideration will also be given to an unanchored pad and a piled raft/pad system.

A key aspect to be considered in the design of wind farm foundations is their performance when subjected to the large moments imposed on them. In examining this performance, a useful tool is an analysis of the behaviour of a combined pile and raft system. Such an analysis has been described previously by Poulos (1994), Ta and Small (1996) and Small and Poulos (2007), among many others. The last of these papers describes the evolution of the computer program GARP (Geotechnical Analysis of Raft with Piles). The key features of this analysis are as follows:

a. The soil is modelled as an elastic layered continuum, via the finite layer method.
b. The raft is represented by 8-noded isoparametric shell elements, and limiting values of the soil-raft pressure for the raft elements are input into the analysis, to allow (approximately) for non-linear behaviour of the raft.
c. The piles are modelled as non-linear interacting springs whose stiffness can be computed via a boundary element analysis, or via the approximate closed-form solutions of Randolph and Wroth (1978) or else input into the analysis.

d. Limiting values of compressive and tensile capacity are input for the piles and the pile loads are not allowed to exceed these values.
e. Non-linear behaviour of the piles can be modelled by reducing the stiffness of the pile in an incremental analysis as load is increased. This is done approximately by using a hyperbolic function to fit the pile load-deflection behaviour up to failure.
f. Vertical loadings may be applied as concentrated point loads at nodes or as "patches" of uniformly distributed load over specified elements.
g. Applied moments can be imposed in each horizontal direction.

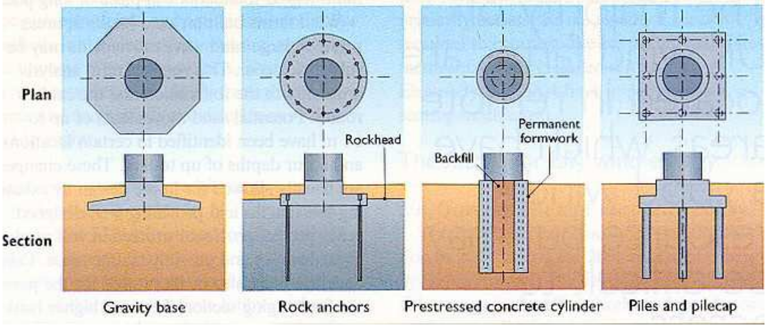

Figure 1: Alternative foundation systems

When applying the GARP analysis to design, the following approach is adopted:

1) For serviceability limit state (SLS) loadings, the soil and pile properties are the "best-estimate" values and are unfactored. The analysis then provides values of the displacements and rotations of the piled raft system which can be checked against the allowable values.

2) For the ultimate limit state (ULS) case, the soil modulus and pile stiffness values are still the "best-estimate" values, but the ultimate raft-soil pressures, and the ultimate compressive and tensile capacities of the pile, are factored by a geotechnical reduction factor ϕ_g. The objective of this analysis is to assess whether the foundation system will be stable under the applied ULS loadings.

3 MODELLING OF LOADINGS FOR WIND FARM FOUNDATIONS

The vertical load can be represented adequately by a uniformly distributed loading, but there are various means by which the applied moment can be represented. It has been found that the consideration of a single concentrated moment at the centre of the tower produces unrealistically large moments in the raft and should therefore not be employed. The most convenient approach appears to be to consider the applied moment as a line moment applied at the axis of the tower. In GARP, this is represented by a series of equal moments at nodes across the tower axis. The moments in the pad computed by this approach have been found to be similar to those obtained when the moment is represented by strips of uniform loading of varying magnitude. Thus, the line moment representation has been employed in the analyses described in this paper.

4 MODELLING OF FOUNDATION ELEMENTS

The GARP analysis models the raft (or in this case the pad) as a plate, using a finite element representation. Piles are modelled as non-linear interacting springs, but when applying the pile model to an anchor, the tensile capacity can be set to that of the anchor, while the compressive capacity can be set to a very small (almost zero) value. The stiffness of the anchor can be computed as for a pile, using pile load-settlement theory to compute the stiffness of the bonded length of the anchor and taking into account also the axial stiffness of the unbonded length. It is assumed that interaction between the bonded lengths of the anchors can be ignored.

5 APPLICATION TO HALLETT WIND FARM

5.1 Geotechnical Conditions

The north-south trending ridgeline of the Brown Hill Range, on which the Hallett wind farm is situated, is typically formed of siltsone and sandstone of the Precambrian Umberatana and Burra Groups. At the turbine locations bedrock is mantled by up to 1.5m of residual soil. The upper surfaces of the rock profile are typically moderately weathered and of high to very high strength. At least three major defect sets at dip angles varying from 40^o to subvertical are evident with sufficiently close spacing such that the rock mass breaks into fragments varying from 20mm to 300mm nominal size in excavation. As depth increases, the rockmass generally grades to slightly weathered, becomes stronger, and defect spacing widens.

A number of pressuremeter tests conducted within the rock mass immediately beneath the founding zone of some turbines indicated shear modulus values varying from 0.6 GPa to 3.8 GPa. The lower values measured generally represented localized zones of low or medium strength rock.

5.2 Design Requirements

The key design requirement was that the foundation system, under serviceability loading conditions, should have a rotational stiffness of at least 20 GNm/rad. It was also desired that, under ultimate limit state loadings, the maximum computed bearing pressure below the pad would not exceed the estimated ultimate bearing capacity of 10 MPa.

In addition, it was implicit that, under ultimate limit state loadings, the foundation system should not collapse.

5.3 Proposed Foundation System

The proposed foundation system consisted of an octagonal-shaped reinforced concrete mat enclosed by a 7m square, and 2.7m thick. A series of 8 anchors were located symmetrically around the outer portion of the foundation, with the bonded section of each anchor having a diameter of 0.2 m and a length of 7 m, with a total length of 14 m.

5.4 Loadings

The loadings set out in Table 1 were employed in the calculations

Table 1: Loadings used in analyses

Condition	Vertical Loading MN	Resultant Moment Loading MNm
Serviceability	2.936	20.408
Ultimate Limit State	2.898	58.945

5.5 Analysis Procedure

The following process was adopted for the evaluation of the proposed foundation system:

1. A uniform and deep rock profile was assumed.
2. As a preliminary estimate of foundation rotational stiffness, elastic solutions for the rotation of a rigid circular foundation (representing the octagonal foundation shape) were used.
3. For more detailed estimates of foundation rotational stiffness under serviceability conditions, and foundation stability under ultimate limit state conditions, GARP was used.

The following assumptions were made in the analyses employing the program GARP:

1. The ultimate bearing pressure on the rock was assumed to be 10 MPa, and the Young's modulus was taken as 500 MPa.
2. Young's modulus of the foundation concrete was 30000 MPa.
3. The anchors were represented by piles with tensile capacity only. Their axial stiffness was computed, via elastic theory for an axially loaded pile (Randolph and Wroth, 1978), to be 131 MN/m.
4. The ultimate axial capacity of the anchors was 4.5 MN (this was governed by structural strength rather than geotechnical capacity).

5. The vertical loading was applied as a uniformly distributed loading over the footprint of the tower (assumed to be 4m in diameter).
6. The resultant moment loading was applied as a uniformly distributed line moment acting along the axis of the tower structure.
7. Only the vertical and rotational responses were considered. No consideration was given to the effects of lateral loading, which was found to be catered for adequately by the embedded foundation pad.

5.6 Rotational Stiffness

From elastic theory, he rotational stiffness K_θ of a rigid circular foundation of radius a subjected to applied moment M is given by the following expression:

$$K_\theta = M/q = 4.E_s.a^3 / 3(1-\nu^2) \qquad (1)$$

where E_s = Soil or rock Young's modulus

ν = Soil or rock Poisson's ratio

Ignoring the effects of the anchors, and assuming the foundation to be circular with a radius a = 3.5m, and the rock properties E_s = 500 MPa and ν = 0.2, the computed value of K_θ was 29.8GNm/rad.

5.7 More Detailed Analysis Results Using GARP Program

The program GARP was used for both the serviceability and the ultimate load cases, using the loadings set out in Table 1. The octagonal shape of the raft has been modelled via finite elements, using a total of 656 elements and 2081 nodes. Figure 2 shows the details of the mesh used and the locations of the anchors.

Table 2 summarizes the results of the calculations. The following points are noted:

1. Under serviceability loading conditions, the computed rotational stiffness is 28.5GNm/rad, which is similar to the value computed via elastic theory.
2. Under serviceability conditions, no loads are induced in the anchors.
3. Under ultimate loading conditions, the computed rotational stiffness appears to have increased slightly to 30.0GNm/rad, probably due to the fact that tension is mobilized in two of the anchors.
4. Under limit state loading conditions, if no reduction factors are applied to the ultimate bearing capacity of the pad, the maximum contact pressures were found to be below the specified ultimate value of 10 MPa. Consequently, even under these conditions, the foundation behaves elastically.

Table 2: Summary of Results from GARP Analyses

Quantity	Serviceability Analysis	Ultimate Limit State Analysis (Ultimate Bearing pressure not factored)
Rotation rad	.000717	0.001965
Rotational Stiffness GNm/rad	28.5	30.0
Maximum Settlement mm	8.5	12.5
Minimum Settlement mm	4.3	1.0
Maximum Anchor Load MN	0	0.677
Minimum Anchor Load MN	0	0

5.8 Comparison with Alternative Foundation Systems

GARP analyses were carried out for two alternative foundation systems:

1. A pad without anchors;

2. A pad with 8 piles instead of 8 anchors. The piles were assumed to be 0.75m in diameter and 7m long. The computed ultimate capacities were 9.4MN in compression and 3.5MN in tension, while the axial stiffness of each pile was found to be 1850MN/m.

In addition, analyses were carried out in which the pad was ignored (by setting its bearing capacity to zero) and only the 8 piles were assumed to carry load.
In each case, calculations were carried out to compute the rotational stiffness of the foundation under the serviceability loadings and the stability of the foundation system under the ultimate limit state loadings in Table 1. For the ultimate limit state, a geotechnical reduction factor ϕ_g of 0.5 was applied to both the ultimate bearing pressure of the pad foundation and the ultimate resistance of the anchors and piles.

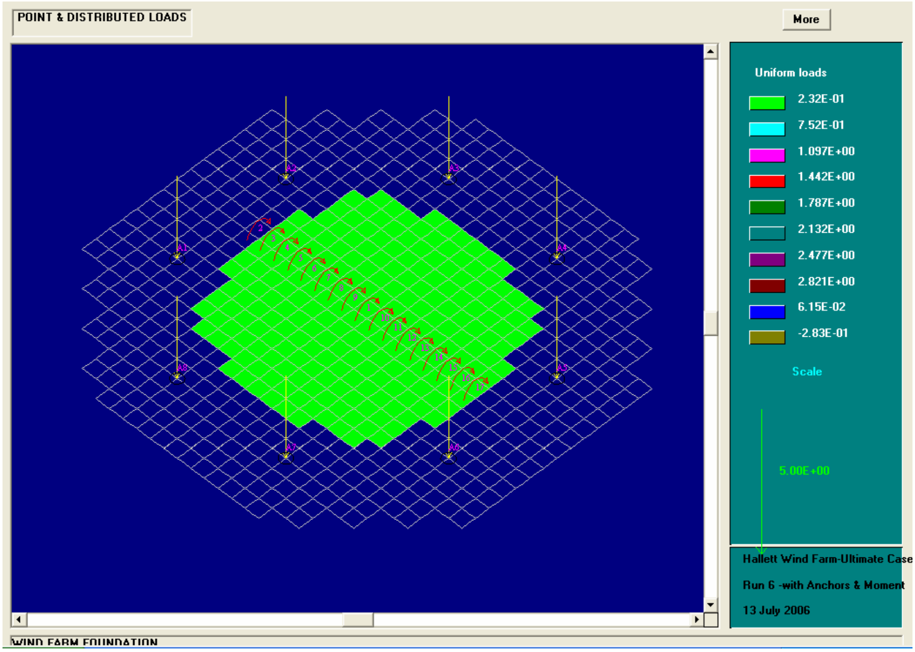

Figure 2: Finite element mesh for Hallett wind turbine analysis

Table 3 summarizes the results obtained. It can be seen that, under the serviceability loadings, the rotational stiffness of the pad without anchors is considerably less than that for the pad with anchors, because of “lift-off” of a portion of the pad. The stiffness of the pad with piles is somewhat larger than for the anchored pad. For the ultimate limit stated loadings, the pad alone was found to be unstable (i.e. it would fail if subjected to the ultimate limit state loadings and the bearing pressures were reduced by 50%). Both the anchored pad, and the pad with piles, were found to be stable.

It is interesting to note that, for the case of the pad with piles, if the presence of the pad were ignored and only the piles considered, the analysis indicates that:

1. The rotational stiffness would be reduced by about 5%;
2. The foundation would not be stable under the ultimate limit state loadings.

This result clearly indicates that it would be unduly conservative to ignore the contribution of the pad, as a larger number of piles would then be required for stability purposes.

For the serviceability limit state, it was found that the computed maximum moments in the pad for the various foundation systems. The maximum moments are similar for all three foundation systems.

Figure 3 compares the maximum settlement for the various foundation systems. The anchored pad system experiences the largest settlement, but this is because of the settlement due to the preloading of the anchors, which contributes almost 6mm to the settlement. The settlement due to the applied SLS loadings is very small (about 2mm).

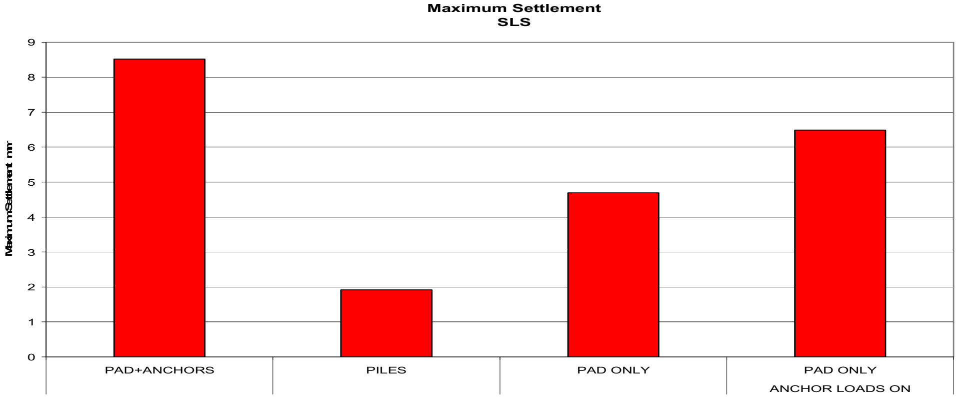

Figure 3: Comparison of computed maximum settlements for SLS case

Table 3: Comparison of Analysis Results for Alternative Foundation Systems

Foundation System	Rotational Stiffness Under SLS Loading MN/rad	Stability Under ULS Loading
Pad + 8 anchors	28.5	Stable
Pad alone	9.9	Unstable
Pad + 8 piles	41.1	Stable
8 Piles (pad ignored)	39.0	Unstable

6 CONCLUSIONS

The analyses carried out confirmed that, for the octagonal anchored pad system proposed for the Hallett wind farm, the computed rotational stiffness of the octagonal foundation was comfortably in excess of the required value of 20 GNm/rad. It was found that a pad with pile support would also provide satisfactory performance, although the costs would be greater. However, if piles were to be used and no account was taken of the effects of the pad, the foundation system would be computed to be unstable. It is therefore excessively conservative to ignore the contribution of the pad when designing a wind farm foundation supported by piles

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