Design of square foundations for lattice-type wind turbines in Madagascar

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ABSTRACT: This paper presents a case study of the design of wind turbine foundations for a wind energy farm located on the southeast coast of Madagascar, near Port Dauphin. The location of the site presented a number of unique challenges that influenced the development, design and construction of the wind energy farm and in particular the wind turbine foundations. These challenges included having limited geotechnical information, the remoteness of the site, limited supporting infrastructure (roads, ports, etc) and a shallow water table. The remote location of the site influenced the solutions that were developed. Originally, the wind farm comprised of three 120 m hub height wind turbine generators. However, this was changed to smaller repurposed 49 m hub height lattice type wind turbine structures, which could be shipped in containers and assembled on site. The square foundations for these structures are different to the more typical circular foundations and the conventional design approach typically applied for wind turbine foundations was adjusted. This paper presents the design methodology of the turbine foundations, the interpretation of the ground conditions for design based on limited available information and details the design and construction considerations.

1 INTRODUCTION

A new wind farm located on the southeast coast of Madagascar, near Port Dauphin, is being developed by CrossBoundary Energy, as part of a hybrid energy project, to supply power to Rio Tinto's QIT Madagascar Minerals (QMM) ilmenite mine. The wind energy facility is estimated to have a generation capacity of 16 MW. The project initially considered a wind farm layout comprising of three wind turbines, each with a hub height of 67 m, total height of 124.74 m and a generation capacity of 2.99 to 4.20 MW.

Following the geotechnical investigation, an alternative turbine model was selected, namely the Vestas V52 850 kW HH50m Wind Class IEC 1A turbine, with a hub height of 49 m and 52 m rotor diameter, that could be shipped in conventional shipping containers and constructed on site with a smaller crane. These were re-purposed lattice-type wind turbines decommissioned from another wind farm located in Europe. This turbine model offered a more feasible method of supply for the project. The number of turbines was increased from the original three to nineteen due to their smaller generation capacity. The lattice-type wind turbine is depicted in Figure 1.

The typical foundation for these structures comprised of a square shaped foundation, acting either as a shallow spread foundation or as a piled foundation. This differs from the more widely used, larger conventional steel or concrete wind turbine structures where a mass circular concrete foundation is commonly applied. In addition, as the turbine models were older, the standard loading document for the model was different to current practice where load cases and codes have since been updated. Due to the unconventional geometry and available loading information, the design approach was adapted to take this geometry into consideration, while satisfying the limit state design criteria recognised by international standards.

Figure 1. Lattice type structure

This paper presents the design methodology for the wind turbine foundations. It also presents the interpretation of the ground conditions that was based on limited information, the design and construction considerations, as well as the approach to how ground risk was managed in design and construction.

2 THE SITE

The project site is located approximately 5 km southwest of Taoloagnaro, as shown in Figure 2.

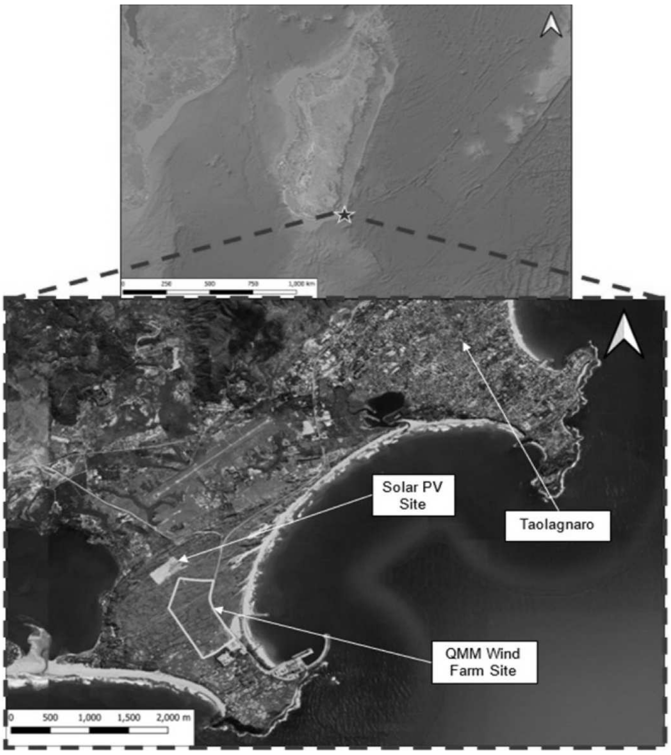

Figure 2. Project site location

The regional geology (Africa Geological Atlas, modified after Besaire, 1964; Ashwal 1997) indicates that the site is underlain by gneisses, leptinites, pyroxenites, cipolins, chamockites and werneritites of the Androyan Sequence (Archean). The geology in the area was characterised by dolerite dykes, dunes and estuarine sands and quaternary interstratified sandstones. The site is located on a peninsula, where relatively thick deposits of wind / alluvial sand overlying bedrock were expected. The 1:20 000 geological map indicated the site is underlain by "Dunes vives". The term translated as "vivid dunes", was described by Ashwaal (1997) as "the prograded sequences of wave-built deposits, called beach ridge systems". These "vivid dunes" were assumed to be potentially active sand dunes consisting of coarse sands which would be vulnerable to wind action / erosion when bush is cleared.

The topography of the site comprised of three ridges striking in a NW-SE direction on which the wind turbines were located.

Due to the location of the site, there were specific considerations that drove the project development, design and construction. Limited availability of materials in the area such as aggregate and layer works for roads was a key consideration. In addition, availability of plant on the island, such as piling rigs as well as cranes, influenced design decisions such as foundation types, as well as the wind turbine model. Transportation to the site was a major factor as the wind turbines needed to be shipped to Madagascar and transported to the site, where materials such as steel and cement were required, causing a cost implication. The design solutions needed to provide workable solutions for the constraints encountered while optimising material quantities and limiting costs.

3 INTERPRETATION OF GROUND CONDITIONS

The available geotechnical information consisted of two geotechnical interpretive reports, one report for the investigation conducted on the wind farm for the original three large wind turbines and another report consisting of eleven test pits and Dynamic Cone Penetration tests (DCP) conducted at the neighbouring site, located approximately 300 m north from the site. The geotechnical investigation for the wind farm site comprised of three boreholes to 25 m depth with piezometer and Standard Penetration Test (SPT) readings, as well as nine hand-dug test pits to a depth of 4 m. The borehole and test pit logs were not profiled in accordance with international best practice, which brought uncertainty to the interpretation of the ground conditions. There was no footprint specific geotechnical data to inform the foundation design of the nineteen wind turbines.

The ground profile was interpreted to comprise of sandy material up to a depth of 25 m. The consistency with depth was derived from the SPT data and transitioned from a loose to medium dense consistency in the upper 5 m, transitioning to a dense / very dense consistency at depths between 15 to 25 m. No bedrock was encountered.

Considering that the SPT N data was collected from only three boreholes across the site and the material type was uniform with depth across the boreholes, differing only in consistency, one representative design ground model with geotechnical parameters was derived. Due to the quality of information being considered to be poor in accordance with international best practice, a level of uncertainty arose within the interpretation of the ground conditions. Therefore, the design line considered a conservative lower-bound fit to the SPT data with depth, as depicted in Figure 3.

Ground water levels were measured from piezometers in the three boreholes, as well as in the test-pits, where the recorded ground water levels ranged between 1.5 and 24.6 m below natural ground level across the site. The data indicated that the ground water level may be linked to the topography of the site, where a shallower ground water table is likely to occur in the lower-lying southern section of the site. Due to this uncertainty, the design ground water level was conservatively taken as 1.5 m below natural ground level. This design assumption choice impacted the buoyancy that was applied to the foundation during design.

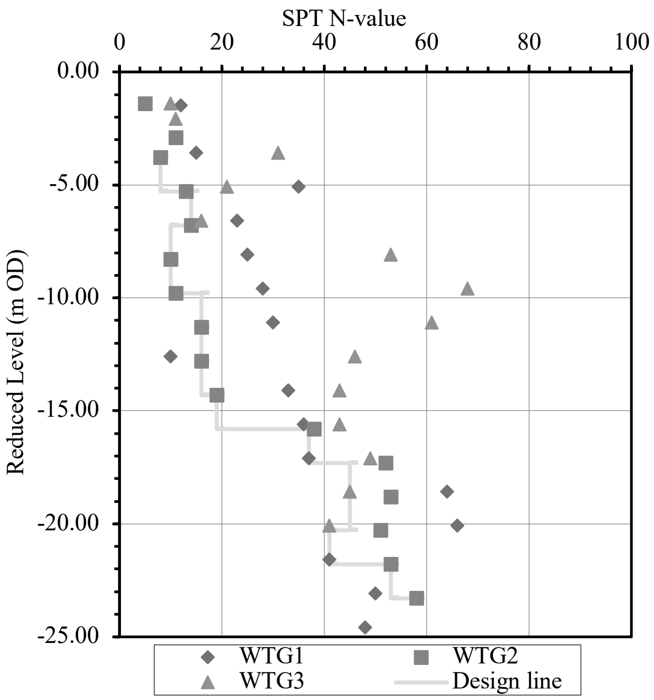

Figure 3. SPT-N plot with depth

4 SELECTION OF FOUNDATION TYPE AND GEOMETRY

The turbine has a lattice-type tower which is joined to steel connections cast into the concrete of the foundation at the four corners. The generic foundation applied for this structure is typically either a spread (shallow) foundation or a piled (deep) foundation.

The interpretation of the ground conditions indicated that a piled foundation would be best suited. However, considering the remoteness of the site, limited availability of piling rigs and the cost to mobilise rigs to the site, a piled foundation was deemed unfeasible. Therefore, the spread foundation was the foundation solution developed for the site.

The standard foundation geometry for this specific turbine model comprised of a square 2-tier reinforced concrete structure, where the centre component of the top tier is backfilled with a granular material. The dimensions of the top tier were fixed to allow for the structural connection to the tower. The top tier was 9.38 m x 9.38 m with a depth of 2.7 m. The bottom tier was a solid concrete slab with a fixed thickness of 0.8 m to accommodate the structural connection at the four steel posts cast into the foundation. Therefore, only the bottom tier side lengths could be adjusted to achieve the required design criteria. The foundation geometry is shown in Figure 4. The founding level was 3.8 m below ground level. The outer perimeter of the bottom tier was overlain by backfilled material that added a stabilising force to the structure. Due to the nature of the site conditions and the vulnerability to wind erosion, the sensitivity of the design to a reduced thickness of backfill was also considered. This impacted the resultant size of the foundations, and it was thus decided to put measures in place to control erosion and ensure this stabilising force was in place for the lifetime of the wind farm.

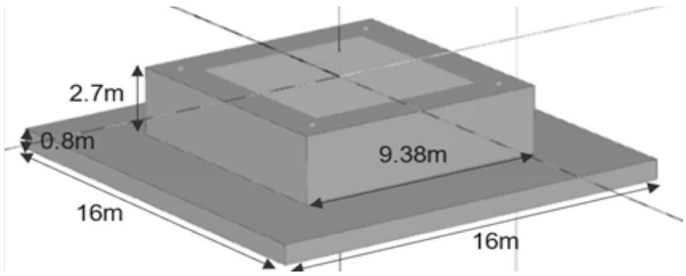

Figure 4. Foundation Geometry

5 CONSIDERATIONS FOR DESIGN AND CONSTRUCTION

Based on the available information, geotechnical risks for design and construction for the site were identified.

Due to the change in turbine model that subsequently changed the turbine layout across the site, none of the proposed turbine locations had geotechnical data within the footprint of the proposed locations. This introduced risk associated with uncertainty in founding ground conditions whether the base design assumption of sand to depth with limited variability in materials across the site was representative. This would influence wind turbine foundation size and type that is applied across the site. An additional geotechnical investigation was undertaken to verify the design and to potentially optimise the proposed foundation size. This investigation consisted of eighteen Cone Penetration Tests with pore pressure monitoring (CPTu) as well as six Mostap samples.

Due to the loose to medium dense consistency of the ground profile and the ground consisting mainly of sandy material, as well as the presence of a shallow water table, a liquefaction assessment was performed utilising the SPT N data and the approach by Seed & Idriss (1971) to ensure the proposed foundations were not at risk of liquefaction.

A shallow ground water level results in buoyancy effects on the foundations. An accurate understanding of the ground water levels was necessary to design for uplift pressures. Uplift pressures can result in the need for significant foundation sizes to resist these pressures. In addition, the ground water level would need to be understood during the construction stage to ensure efficient constructability of the foundations and achieving the required excavation depth where dewatering would be challenging.

In the event that ground conditions differed from the design ground model, with a weaker ground, there would be a need for improvement within the founding profile. This had a material cost risk to the contractor which was identified and allowed for in the Bill of Quantities (BoQ).

6 DESIGN METHODOLOGY

The foundation design followed the approach outlined in DNV guideline (2018) and Vestas (2011) in accordance with IEC 61400-1 and 6 and Eurocode 7 (BS EN 1997), where the design needed to comply with the following design requirements:

- No excessive foundation overturning
- No sliding of the foundation
- No soil shear failure underneath the foundation (bearing)
- The effects of buoyancy must be considered
- The foundation must provide the minimum required foundation dynamic (rocking) stiffness
- No foundation lift-off / gap formation is considered beneath the foundation.
- Differential settlement is limited to 3mm/m

The design loads were provided by the manufacturer (Vestas 2005) in the turbine model loading document for extreme load cases according to design wind speeds.

The design considered an initial sizing, followed by a limit state design consisting of ultimate (ULS) and serviceability limit state (SLS) design checks, where partial factors were applied to design loads and material parameters in accordance with BS EN 1997-1 UK National Annex.

6.1 *Design approach*

The wind turbine foundation was analysed by means of two assessments, namely a geotechnical and a structural design. The geotechnical design comprised of:

- interpretation of the ground conditions and their variability across the site with depth and spatially
- derivation of representative design ground model and parameters
- design analyses of ULS cases and rocking stiffness
- static soil-structure assessment utilising 3D finite element software (Plaxis 3D) to assess the foundation response to applied load cases.

The structural design comprised of structural analysis utilising 3D finite element software (DIANA) to assess the reinforcement requirements. The design interface between the two analyses to ensure one cohesive, safe foundation design was assessed through a convergence check, as shown in Figure 5.

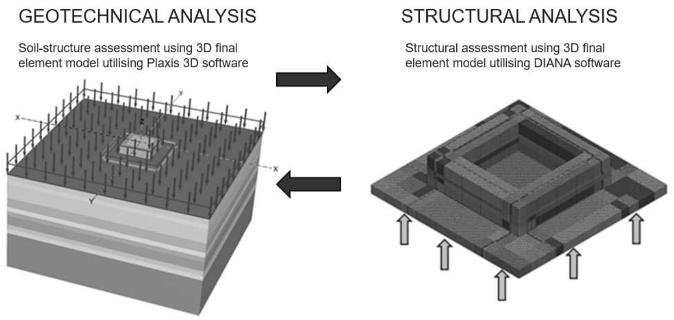

Figure 5. Foundation design interface between geotechnical and structural models

6.2 *Initial sizing of foundation*

Wind turbines are dynamically sensitive structures. To avoid resonance of the foundation-tower-turbine system, the frequency assumed for design of the overall system must avoid the frequency content of applied loads. Each wind turbine model has a specified minimum rocking stiffness that must be achieved. The manufacturer's loading document specified the minimum acceptable rocking stiffness of the foundation to be greater than 18 GNm/rad. An initial size of the foundation was calculated by carrying out a first principle rocking stiffness calculation as stated in Vestas (2011). The rocking stiffness is calculated for a quadratic footing on a half-space where no gapping of the foundation occurs, as detailed in Equation 1.

$$K_R = \frac{G.B_{\varphi}.B^3}{(1-v)} \quad (1)$$

where K_R = rocking stiffness, required to be greater than 18 GNm/rad; G = mass shear modulus; B_ϕ = factor accuracy angle; v = Poisson's ratio and B = foundation base width.

The rocking stiffness is based on a mass modulus for the ground profile representing an overall mass response for the founding profile. The founding material typically comprises of different layers of soil and/or rock, each with their own modulus values and properties. Using the elastic moduli determined for each layer in the design ground profile, the representative mass ground modulus was assessed using the empirical method proposed by Fraser & Wardle (1976).

6.3 *Ultimate Limit State Checks*

The ultimate limit state (ULS) checks consisted of overturning, sliding resistance and bearing capacity checks, where the appropriate partial factors were applied to the loads and material parameters, as outlined in the Vestas foundation design document (Vestas 2011).

6.4 *Serviceability Limit State Checks*

The serviceability limit state (SLS) checks consisted of assessing the rocking stiffness and gapping using finite element software. Bentley's finite element software Plaxis 3D (version 21.00.01.07) was used as shown in Figure 6.

A static analysis was undertaken utilising static Young's Moduli for the ground layers. This is in accordance with the DNV guideline (2002) where "for wind loading of wind turbine foundations, onshore as well as offshore, the induced vibrations will be of such a nature that the static stiffnesses will be representative for the dynamic stiffnesses that are required in structural analyses." The structure of the foundation was modelled as a volume element, with the linear elastic and non-porous constitutive model. Soil and backfill materials were modelled with the Mohr-Coulomb constitutive model. Interfaces were added for all areas where the foundation came into contact with the soil. A friction reduction factor of 0.7 was applied for the interface.

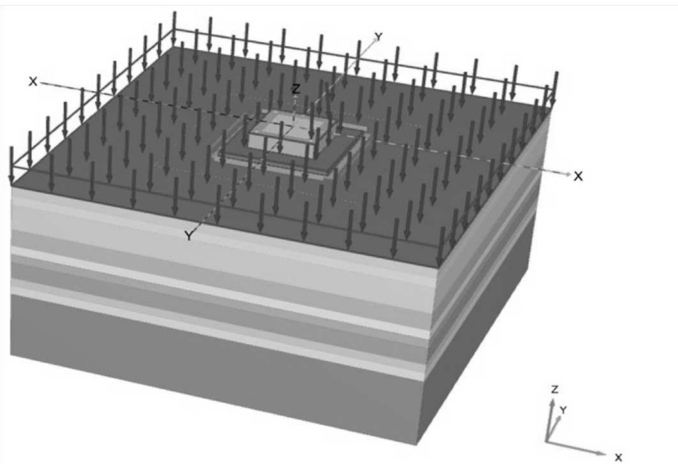

Figure 6. Plaxis 3D model of the foundation

The effect of buoyancy was applied to the bottom of the foundation as a uniformly distributed load in the upward direction with a load magnitude of 20 kPa (2 m x 10 kPa). Wind turbine structural loads were applied as resultant point loads at each corner at the post locations in the x, y and z directions. The extent of the model was 60 m x 60 m x 30 m. Where backfill material was considered, this was applied as a surface load onto the top surface of the bottom tier slab which incorporated a reduction in weight for the water table at 1.5 m bgl. The non-buoyant unit weight of the backfill was assumed to be 16 kN/m³

6.4.1 *Rocking stiffness*

The rocking stiffness was determined utilising two approaches:

a) Approach 1 (empirical method) utilising Equation 1 defined above and the mass ground modulus.
b) Approach 2 (numerical method) using a 3D finite element model. The rocking stiffness of the foundation was calculated using the resultant movement to the applied loads on the foundation in Plaxis 3D. This check was the driver of the foundation design.

6.4.2 *Gapping*

Gapping refers to the situation where the foundation loses contact with the ground and effectively bears the applied load on a reduced foundation footprint. The degree of gapping allowed on a wind turbine foundation can be interpreted differently based on a review of available design documents. The Germanscher Lloyd guideline (GL 2010) specifies two design load cases under which no gapping is permitted to occur. It further specifies forces according to particular design load cases under which gapping is allowed to occur up to the centre of gravity of the bottom area of the foundation (GL 2010).

The degree of gapping a designer adopts for a particular foundation design needs to be selected on account of the underlying ground conditions and their potential to soften or degrade in stiffness over time. Degradation of the stiffness of the founding ground profile due to cyclic loading may result in an increased gapping over time (Wojtowitz & Vorster 2014). At present, it is not clearly understood to what extent this increase in gapping will occur.

A degree of gapping can generally be allowed for on a very stiff founding ground profile, such as competent rock. For wind turbines founded on soil, gapping is generally not allowed due to the possibility of the degree of gapping increasing over time caused by stiffness degradation. For this reason, no gapping was allowed for in the wind turbine foundation design at QMM.

The loading document (Vestas 2005) only detailed extreme loads and fatigue loads. The load case defined for the gapping assessment in accordance with current best practice was not included in the loading document. As a result for the gapping assessment, the extreme loads were applied with no partial factors. Due to the rectangular shape of the foundation, the empirical gapping assessment detailed in Vestas guideline (2011) could not be applied. Therefore, the gapping assessment was undertaken utilising the soil-structure analysis in Plaxis 3D.

6.4.3 *Differential settlement*
An upper limit of permanent settlement (rotation) of the foundation of 3 mm/m or 0.17˚ was ensured in accordance with the DNV guideline (2018).

6.4.4 *Foundation design*
The resultant foundation solution comprised of a 16 m x 16 m bottom tier size as shown in Figure 7. There was a concern that founding the foundation structure directly onto the insitu sand materials (which were interpreted to be between loose to medium dense) could result in compaction and movement of the foundation overtime due to the cyclic loading effects of the wind turbine. In addition, this could result in localised overstressing of the insitu materials particularly at the edges of the foundation. To mitigate this, an improved (working) layer was specified to be placed underneath the foundation.

The design was governed by rocking stiffness and gapping of the foundation. The rocking stiffness determined from the Plaxis 3D analysis at the top of the foundation was 19.6 GNm/rad and met the minimum requirement of 18 GNm/rad. The backfill material weight was required to achieve the design requirements

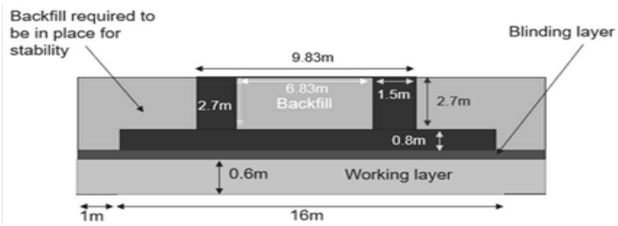

Figure 7. Founding solution

6.5 *Adoption of additional ground information*

Considering that none of the nineteen wind turbine locations had supporting ground information within footprint, CPT testing was carried out across the site. However, not all CPT tests were conducted within the final proposed turbine locations. This information was used to verify the initial ground model.

During this verification process, the ground water level was identified to be shallower than the design ground water level at turbine positions located in the southern section of the site. As a result, new ground models were derived based on the CPT results where the foundation design was checked with a shallower ground water level. For these foundations to satisfy the design criteria, the backfill applied within the centre of the foundation was required to be heavier than the initial design.

The additional ground information was also used to optimise the foundation size based on the following:

1. Only turbine foundations located in the northern section of the site were considered for optimisation due to no indication of shallow ground water.
2. Only turbine foundations where information was obtained within footprint were considered for optimisation.

As a result, the size of eight foundations were optimised from 16 x 16 m to 13 x 13 m sizes.

6.6 *Soil-structure convergence check*

Similar to the geotechnical model set-up in Plaxis 3D, a structural model was set up using DIANA FEA. Representation of the soil-structure response to the applied load in the structural assessment is critical in ensuring the system is modelling a representative reaction. The ground response was modelled with a subgrade reaction placed at the base of the DIANA model. Therefore, the resultant subgrade reaction was determined from the geotechnical model and applied in the structural model, where convergence of the models was achieved when the resultant movements of the two models were the same.

7 CONCLUSION

This paper presents a case study of the design of wind turbine foundations for a wind energy farm making use of re-purposed lattice-type structures requiring square foundations. Due to the challenges experienced on the site, the foundations consisted of spread foundations. The design methodology consisted of an initial sizing based on rocking stiffness requirements, followed by ultimate and serviceability limit state checks, where the latter made use of Plaxis 3D finite element software.

Following the interpretation of the available geotechnical information, various considerations were incorporated to guide the design and construction processes to manage ground risk. One of the considerations was to conduct an additional geotechnical investigation, which provided information to verify the design, as well as to optimise the foundation size for eight foundations.

8 ACKNOWLEDGEMENTS

The authors would like to acknowledge the project developers, CrossBoundary Energy, for their invaluable contribution to the development of this project. Their support and expertise have been essential in bringing this initiative to fruition.

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