# GEOTECHNICAL DESIGN OF THE TE APITI WIND FARM

*Alexei Murashev, Robert Davey, Greg Cook*

Мурашов Алексей Константинович
Доктор философии (D.Ph.),
Новая Зеландия

**ABSTRACT**

Recently built Te Apiti Wind Farm, the largest in the southern hemisphere and New Zealand's newest, feeds up to 90 MW into the national power grid from the 55 1.65 MW turbines (each 106 m high) built on the north side of the Manawatu Gorge, near Palmerston North City. Te Apiti is located on 1200 hectares of undulating hill terrain. Steep eroded gullies and numerous active landslide scars expose an uplifted and tilted marine sequence of Tertiary rocks. Site investigations were undertaken to assess the stability of the wind turbine platforms and to provide information for the design of the access roads and the turbine foundations. The turbines were founded on 16 m to 18 m wide octagonal shallow pad foundations. Geotechnical investigation and design issues are discussed.

## 1. INTRODUCTION

Opus International Consultants Ltd was commissioned by Meridian Energy Ltd to undertake geotechnical investigations and detailed design of the proposed Te Apiti Wind Farm at the southern end of the Ruahjne Range above the Manawatu Gorge, east of Palmerston North City, New Zealand.

Te Apiti is located on 1200 hectares of undulating hill terrain (Figure 1). Steep eroded gullies and numerous active landslide scars expose an uplifted and tilted marine sequence of Tertiary rocks. The wind turbines were 1.65 MW capacity, with 70 m hub height and 72 m blade span giving the total height from the top of the foundation to the top of the blades of 106 m.

Detailed geotechnical investigations were undertaken to provide a sound basis for the design of the wind farm. The geotechnical investigations were scoped to address the key issues associated with construction of the proposed wind farm, which included:

Large scale earthworks to provide 21 km of access tracks to the turbine sites.

Control of permanent stormwater runoff, erosion and sedimentation during earth works.

Creation of 55 flat turbine platforms on the hill tops at each turbine site (the platforms were large enough to build the turbine foundation, to lay down the turbine shaft sections, the blades, nacelle and to park the crane prior to assembling the turbine).

Construction of 55 turbine foundations and turbines at each turbine platform.

Construction of the substation and operation/maintenance building.

Trenching and placement of 11KV cables along internal access tracks.

Disposal of surplus cut materials at nominated disposal sites.

Assessment of the risk of instability of the turbine platforms, access tracks and cable routes.

The design and construction period was short requiring careful staging of geotechnical investigations, testing, design and construction. The geotechnical work was undertaken in 2003 - 2004 y. Construction of the wind farm began on 10 No ember 2003, while the last stages of geotechnical investigations were still under ay in some parts of the site. Approximately 1,000,000 $m^3$ of earth was exca ated to construct 21 km of access roads and for the turbine foundations. The latter took around 21,000 $m^3$ of concrete to construct. The wind farm generated first power on 26 July 2004, with all 55 turbines being fully commissioned by 25 October 2004.

## 2. SCOPE OF GEOTECHNICAL WORK

Geotechnical work for the project was undertaken in a number of stages (Murashev, 2005). Initial geotechnical work included desk studies, geological and geotechnical inspections and mapping. Seven boreholes, 44 trial pits, and numerous laboratory tests including, consolidation, unconfined compressive, grading and compaction tests, were undertaken as part of the geotechnical investigations for the preliminary design. Construction phase geotechnical investigations comprised 59 boreholes, dynamic plate load tests, downhole wave velocity measurements, spectral


**Figure 1. General view of the Te Apiti wind farm site prior to construction**

analysis of surface waves, triaxial and shear box test, as ell as further unconfined compressive and consolidation tests to confirm strength and stiffness of the site materials. To reflect the actual loading conditions, the site materials in dynamic plate load tests (cyclic loading), triaxial and shear box tests were loaded at a rate similar to the rate of loading of soils under the wind turbines. Downhole wave velocity measurements and spectral analysis of surface waves were undertaken at three turbine sites to refine the soil and rock profile. Thermal resistivity tests on the site materials were undertaken to assist the cable designers. Up to five drilling rigs operated on the steep hillsides during the investigation and construction phases. Geotechnical inspections and testing for certification of the turbine foundations was undertaken as the construction progressed and required full time presence of an engineering geologist on site. Assessment of geotechnical risks for the design life of the wind farm was undertaken on the completion of the construction period.

## 3. SITE CONDITIONS

Te Apiti Wind Farm is located on the northern side of the Manawatu Gorge at the southern end of the Ruahine Range, North Island, New Zealand. The site lies approximately 15 km northeast of Palmerston North City and access to the site is via Saddle Road that links the t o rural to ns of Ashhurst (west) and Woodville (east). Te Apiti is situated between 250 m and 360 m above mean sea le el, covering a total area of about 20 square km. The general topography is hilly terrain comprising moderately steep undulating ridge tops flanked by steep gullies that lie at the foot of the Ruahine Range. Numerous shallow landslides scar the surface of oversteepened slopes. Remnants of native bush exist in isolated gullies around the site and at the southern boundary with the Manawatu Gorge. Figure 2 is an engineering geological map of Te Apiti that indicates the extent and distribution of geological units identified at the site and includes active landslide traces. The map was initially de eloped using available data sources and early geotechnical site investigations (walkover survey, trial pits, and boreholes). During construction, the map was continually updated, as more exposures were uncovered from excavation activities. T o major active faults, parallel to one another, lie within the Te Apiti area: the northern portion of the Wellington Fault and the adjacent Ruahine Fault. The Ruahine Fault is closest to Te Apiti lying approximately 500 m southeast of the nearest wind turbines and the Wellington Fault lies within 3 km of the eastern most wind turbine site. The Wellington Fault is considered to be more active than the Ruahine Fault, but both ha e estimated magnitudes greater than 7.5 around the Te Apiti area.

## 4. GROUND CONDITIONS

The ground conditions comprise tertiary soft rocks (siltstone, conglomerate, mudstone, sandstone) and soft to firm soils (loess, alluvial silts and clays). The alluvial soils had some organic content, their undrained shear strength aried from 40 kPa to 80 kPa, unconfined compressive strength from 80 kPa to 160 kPa, cohesion from 3 kPa to 15 kPa, angle of internal friction from 20 to 26 degrees and shear a evelocities from 80 m/s to 150 m/s. Tertiary soft rocks had undrained shear strength higher than 100 kPa, unconfined compressive strength between 170 kPa and 1500 kPa, cohesion between 20 kPa and 60 kPa, angle of internal friction between 27 and 35 degrees and shear ware velocities from 200 m/s to 600 m/s. Thermal resistivity tests for a range of the site materials indi-

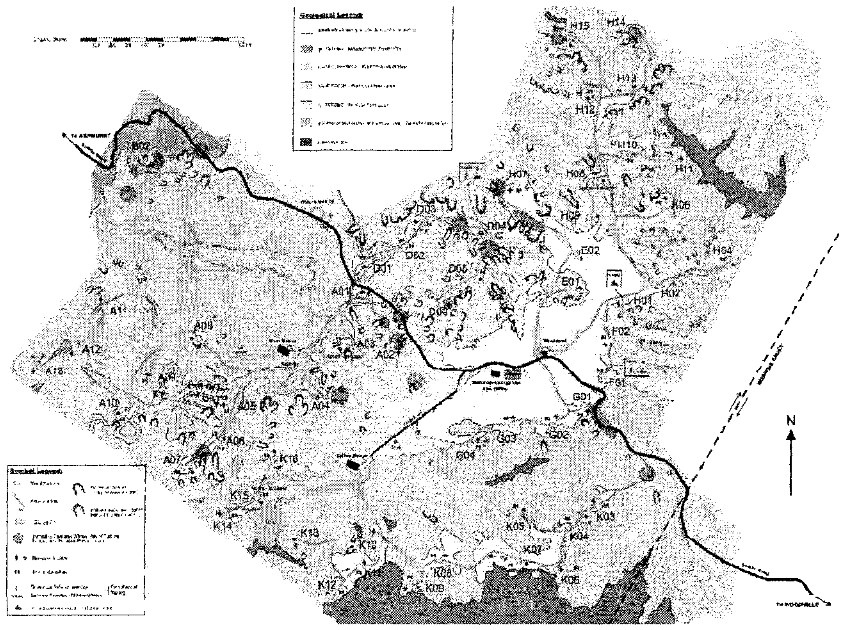


**Figure 2. Geological map of the Te Apti wind farm site**

cated that thermal resistivity varied from 0.51 m K/W to 1.62 m K/W depending on material nature and water content. Analysis of water contents and typical material types for the site, indicated that it is appropriate to base cable ratings on a maximum thermal resistivity of 1.2m K/W.

## 5. SLOPE STABILITY

Aerial photographs from 1942, 1979 and 1995 indicated numerous landslides existed over the site, however very few had changed in dimension over this 53-year period. Aerial photograph examination and field inspection of several landslide features indicated that most are water induced regolith failures that occur m areas of excess ground water seepage. Most slope instability occurs in the siltstone terrain of the Mangatarata and Te Aute Formations (Figure 2). Hillsides formed in these geological units slope between 15° to 40°, but generally slopes steeper than 30° are likely to show some instability. The risk of slope instability at Te Apiti was assessed prior to construction for both the access tracks and the proposed turbine sites. The turbine platforms were located on hill tops and the access tracks followed the ndgelines wherever possible to minimise the volume of cut (and hence cost and land disturbance) and to reduce risk of erosion and their ulnerability to landsliding. Only two access tracks were considered to ha e a potential for scarp failure, which could impact upon construction activity and compromise the safety of the site. The overall risk of instability at the site was assessed to be medium and the critical areas were monitored during construction by a site engineer. Only one turbine site was considered to be at risk from slope instability and therefore during detailed design it was re-positioned a ay from the hazardous slope. All other turbine sites were initially not considered to be at risk. However, slope instability became a major problem at Te Apiti during the construction period in February 2004 when a 100 year return period storm triggered a large number of major landsliding events in the Manawatu region. Heavy rainfall (over 240 mm in 48 hours) caused numerous shallow regolith failures on slopes steeper than 25° and major soil and rock failures occurred in several road cuts on slopes steeper than 35° within the Teritary rock formations and greywacke rock masses. Water infiltration along pre-existing weakness planes (generally joints) induced plane- and edge-type failures in these excavated slopes. Several access tracks were compromised by landslides and became unsafe. The Pohangina Bridge at the Ashhurst end of Saddle Road, was washed away effectively closing Saddle Road and preventing access from Palmerston North. To get to the other side of the range and access up Saddle Road was via the difficult Pahiatua Track, as the Manawatu Gorge Road was also closed due to large dropouts. A temporary crossing and later a Bailey bridge, was built across the Pohangina River to continue construction activities. Short-term stability measures, such as battering back of road cut slopes and diverting surface runoff away from slope edges via cut trenches were implemented. None of the turbine foundation sites were directly affected by slope instability, however some sites were re-positioned (generally less than 10 m) to a void any potential risk associated with nearby shallow regolith failures. E en though construction was delayed by sixweeks due to the storm, it was completed within 12 months as per the original program. On the completion of construction, a detailed geological mapping and risk assessment was undertaken to estimate the risks to the turbine foundations, access roads and buried cable routes from landslides, and a detailed monitoring program was put in place for the design life of the wind farm.

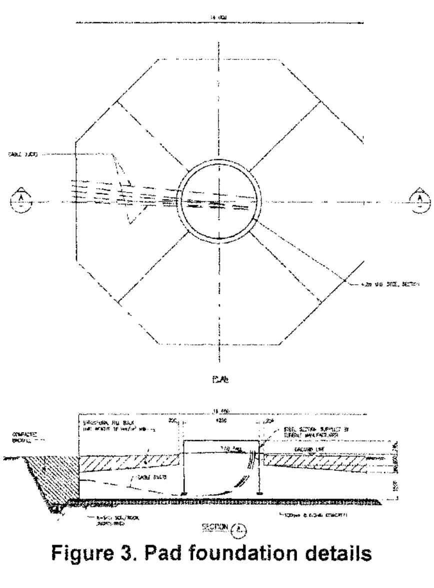

Figure 3. Pad foundation details

## 6. TURBINE FOUNDATIONS

Foundations for wind turbines are low-frequency machine-loaded structures subjected to coupled horizontal-rocking vibrations. The turbine-specific load spectra were calculated by NEG M1CON using an aero-elastic computer model of the turbine. The load spectrum for the Te Apiti turbines was based on a 20 year turbine life. Seismic loads were calculated by Opus in accordance with the New Zealand loading standard. Seismic loads, based on fully elastic re-

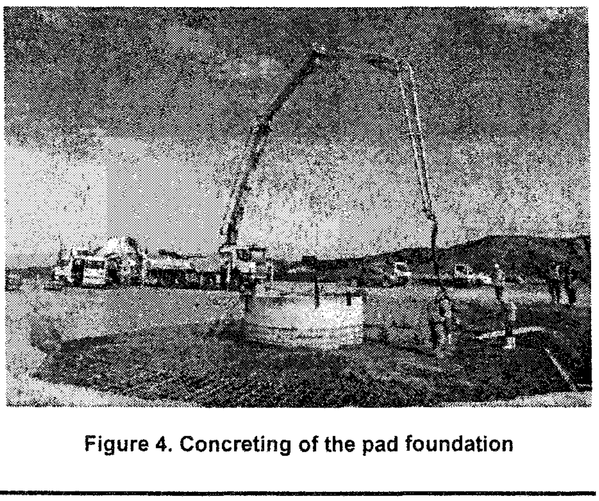

Figure 4. Concreting of the pad foundation

sponse, appeared to be less than the extreme wind loads. Considered foundation options included shallow pad, rock anchored pad, "mono"-pile and "multi"-pile foundations. A shallow 16 m size octagonal pad foundation (Figures 3 and 4) was adopted for most of the turbine sites based on the comparison of the foundation options. The geotechmcal and structural design of the pad foundation considered the following issues:

- Bearing capacity failure of the soil
- Drainage measures were put in place to keep the ground water table below the underside of the pad, so that buoyancy forces could be neglected in the design
- Size of the zone of plastic deformation in the soil base beneath the pad foundation
- Degradation of strength and stiffness of soils under cyclic loading from the turbine
- Rocking stiffness of the pad foundation - soil system (refer to Section 7)
- Overturning of the tower and pad foundation under seismic and wind loads
- Shear failure of the pad foundation
- Flexural behavior of the reinforced concrete pad
- Loss of anchorage of the tower embedment cylinder within the pad
- Fatigue analysis of the turbine-pad system to ensure that the foundation structure has a design life of not less than $10^9$ load cycles

In France and Russia, geotechnical design codes include a requirement for no loss of contact pressure beneath the foundation under a frequently occurring load combination, and for no more than 25% of the base area to uplift for an infrequent load combination. EN 1997 Eurocode 7 - Geotechnical Design and New Zealand Building Code (verification document B1/VM4) have similar requirements with respect to the maximum acceptable load eccentricity. These requirements are aimed at limiting the size of the plastic zones in the soil beneath the pad and achieving adequate serviceability behavior of conventional structure foundations. The loading on the turbine foundations is substantially different from that for conventional structure foundations (e.g. lower gravity load due to light eight of the turbine, high load eccentricity due to light eight of the turbine and high extreme wind loads). The load eccentricity under high wind loads can be substantial, resulting m stress concentration under the edge of the pad. The width of the equivalent footing changes from one loading case to another as a result of changes in the eccentricity associated with different loading cases. The loss of basal contact and stress concentration under the edge of the pad were carefully considered. Under the production load, the hole pad remained in contact with the soil. For infrequent seismic and ultimate wind loads, loss of basal contact was considered to be acceptable. The maximum bearing pressure under the pad as a function of the bending moment applied to the top of the pad is shown on Figure 5. For soft sites here either ultimate bearing capacity requirement could not be satisfied or degradation of soil strength and stiffness could potentially result in a shorter design life of the turbine, a larger 18 m size pad was used. On a number of sites with intermediate soil properties a solution comprising the 16 m size pad was retained, but 0.5 m to 1.5 m thickness of soft materials immediately beneath the pad were undercut and replaced with compacted granular fill. The 16 m or 18 m size pad with undercut, resulted in a reduced plastic zone and a reduced stress le el in the soft soils beneath the pad (or beneath granular material), to meet the 20 year design life requirement.

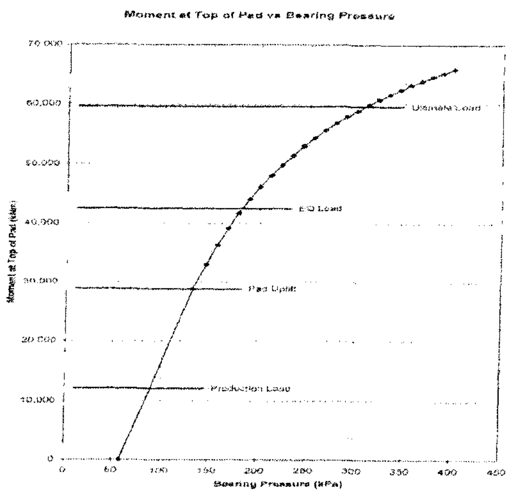


**Figure 5. Maximum bearing pressure as a function of the bending moment**

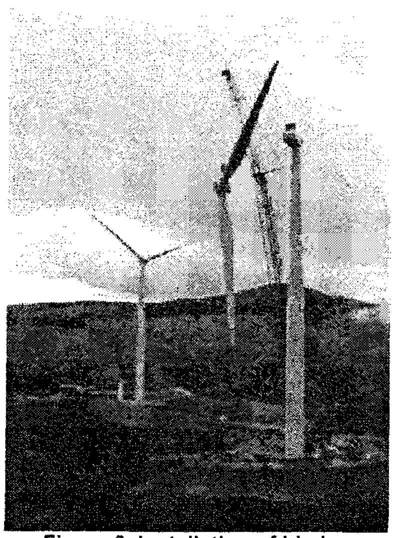

**Figure 6. Installation of blades**

The concrete for the pads was supplied from an on-site batching plant. Each 16 m size pad contained 375 $m^3$ of 30 MPa concrete and 28 Te of reinforcing steel. A more detailed discussion of the structural design issues for the turbine foundations is given in Davey and Green (2006). On the completion of the pad foundations, turbine shaft sections, nacelle and blades were installed to form a complete wind turbine (Figure 6).

## 7. ROCKING STIFFNESS OF TURBINE FOUNDATIONS

The turbine manufacturers specified the minimum rocking stiffness of the pad foundation required to comply with the parameters used in their load modeling. The rocking stiffness of the pad foundation is governed by the size of the pad, soil stiffness, rate of loading, size of the zone of plastic deformation in the soil, soil potential for degradation of strength and stiffness. Dynamic plate loading tests in the field were undertaken to confirm the soil stiffness. The plate load test data indicated that the hard siltstone (Figure 7) demonstrated essentially elastic behavior under cyclic loading, while the soft silt (Figure 8) had a potential

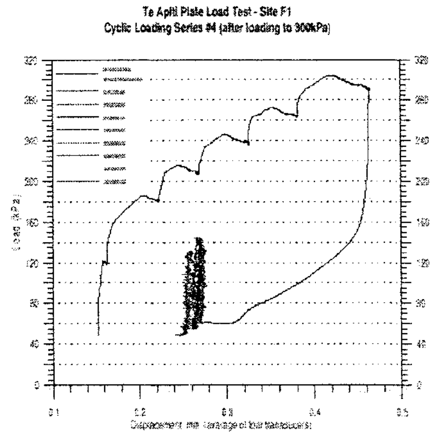


Figure 7. Plate load test data, hard siltstone

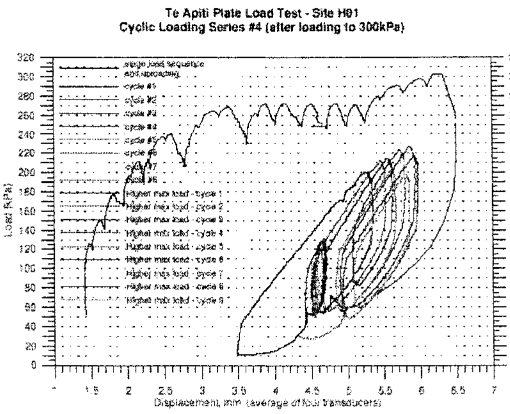


Figure 8. Plate load test data, soft silt

for degradation of stiffness. The rocking stiffness of the pads founded on soft silt during the initial load cycles would still be within the required limits, but accumulation of plastic deformation in the plastic zone under the pad foundation in further loading cycles could result in reduced rocking stiffness of the foundation-soil base system. Also, for some soft sites the ultimate bearing capacity condition could not be satisfied. It was therefore considered necessary to reduce the size of the zone of plastic deformation and the stress le el in the soft silt to ensure the elastic response of the soil governs the rocking stiffness, and ensuring the ultimate bearing capacity remains adequate. The methodology adopted for the soft sites is described in Section 6. Calculations of the rocking pad stiffnesses based on analytical solution for the rigid circular pad on elastic half-space indicated that the manufacturer's requirement of the minimum rocking stiffness of 20 GNm/rad could be satisfied.

## 8. CONCLUSIONS

Extensive geotechnical investigations were undertaken for the design and construction of the Te Apiti wind farm. The geotechnical design considered a number of issued associated with construction of wind farms in hilly terrain. In spite of some construction challenges, the construction of the wind farm was successfully completed within 12 months (Figure 9).

## 9. ACKNOWLEDGEMENTS

The authors wish to thank Meridian Energy Ltd for allowing the information contained within this paper to be published and Meridian's Te Apiti project manager David Green ho was fully supportive of the extensive and thorough geotechnical investigation program for the project. Opus Central Laboratories Ltd is thanked for the assistance with laboratory and field soil testing.

## REFERENCES

Davey, R. and Green, R. (2006). Te Apiti wind farm turbine foundations: design and construction. Proceedings of the 2006 NZ Concrete Conference.

Murashev, A. (2005). Te Apiti wind farm: megawatt - class machines aided by geotechnical expertise. New Zealand Geomechanics New, Issue 69, June 2005.