Geotechnical Engineering Division: partner in Civilution

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Geotechnical aspects of an **Eastern Cape wind farm**

INTRODUCTION

Various wind farms are now under consideration by the South African power authorities, as part of the renewable energy drive to lower the reliance on coal. This article describes the geotechnical aspects of one at present under construction in the Eastern Cape.

Comprising 9 x 95 m hub-height, 3 MW turbines, the foundations are hexadecagon (16-sided) reinforced concrete structures 18.1 m across flats some 3 m thick, as detailed in Figure 1.

Wind loads induce some 85 MNm of moment which, when coupled with a total vertical force of some 20 MN, lead to high eccentricities in the base. Under serviceability conditions dead load stress is limited to some 75 kPa, but when coupled with the live load moment, a bearing stress of some 300 kPa at the edges is generated.

The design criteria limit differential deflections to some 3 mm per metre of

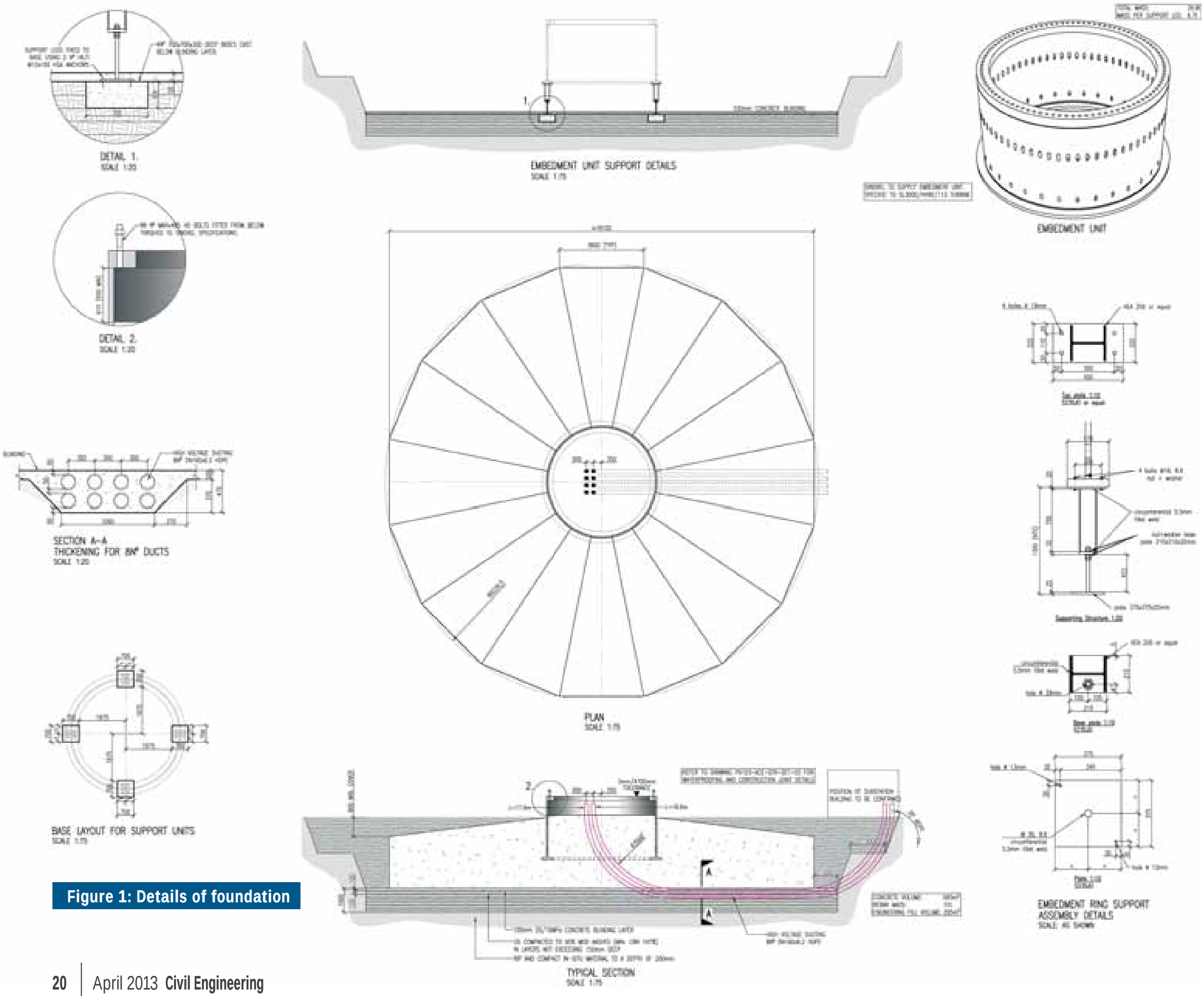


Figure 1: Details of foundation

width or 54 mm in total. This article details how these stresses and deflection limits were accommodated from a geotechnical point of view, and the dynamic analysis which was performed.

GEOTECHNICAL INVESTIGATION

A very comprehensive investigation was conducted by Terrastest in conjunction with Wilson and Pass, at each of the nine envisaged tower positions. This comprised:

a) Core drilling to 30 m
b) Standard penetration tests (SPTs) conducted at 1.5 m intervals
c) Disturbed and undisturbed samples retrieved from various depths
d) Laboratory testing, including gradings, Atterbergs, collapse potential, shear box, triaxials, point load index
e) Extensive trial pitting
f) Dynamic probe super heavy (DPSH) testing performed to 10 m depth
g) Dynamic cone penetrometer (DCP) tests to a depth of 2 m.

A summary of the results is contained in Table 1.

The pertinent aspects from the above data are:

1. The position of the calcrete layers within the profile
2. The strength of the calcrete layers
3. The stiffness of the in-situ materials below the envisaged founding depth.

FOUNDATION DESIGN

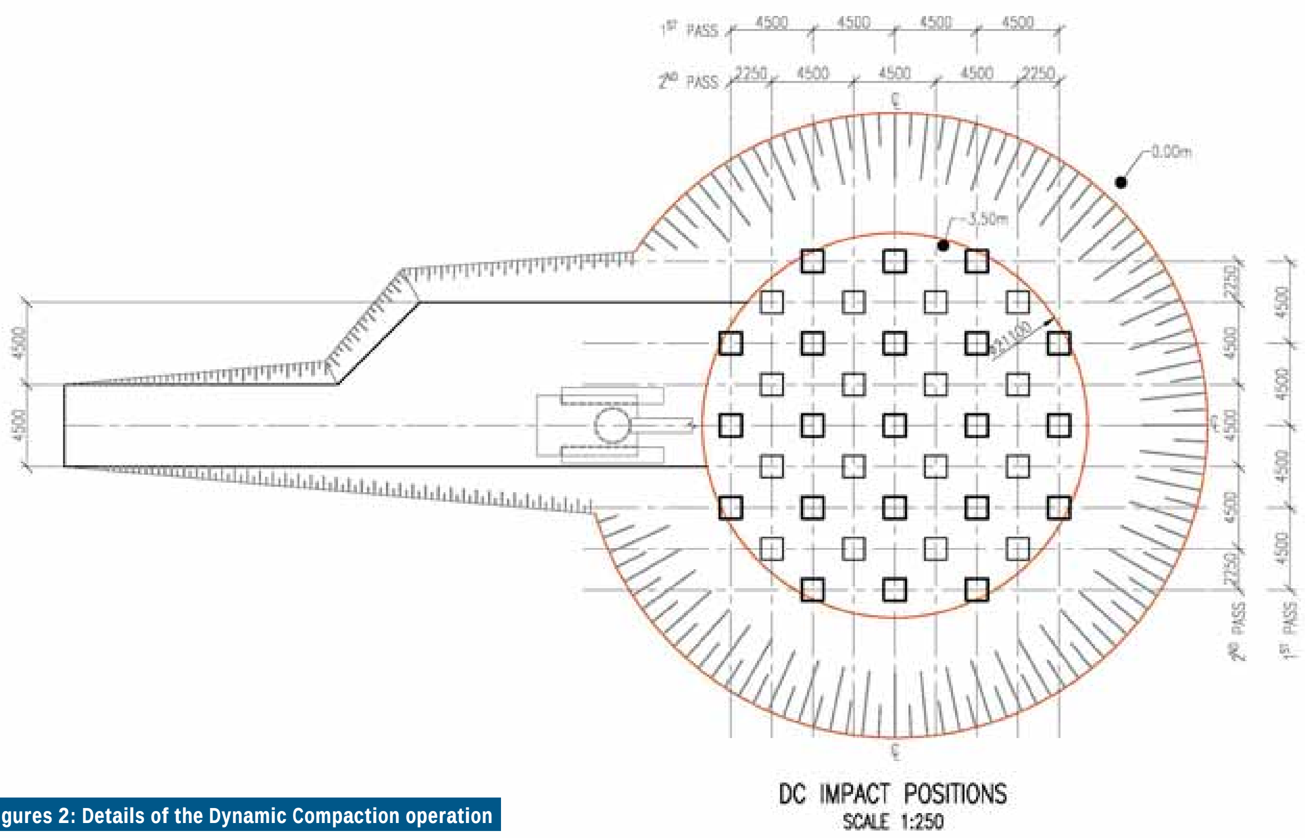
Three possible solutions were initially considered:

1. Rapid impact compaction (RIC) (9-12 tonne mass falling through 1.0-1.5 m
2. Dynamic compaction (DC) (12 tonne mass falling through 18 m)
3. Founding using piled foundations.

After much deliberation it was decided that DC would be used. The design, based on the formulations of Oshima & Takada (1997) called for 13 blows of a 12 tonne mass falling through 18 m on a 4.5 m grid, with one in the middle as detailed in Figures 2 and 3. This primary compaction was followed by an ironing operation.

Table 1 Geotechnical attributes of the site

Borehole no	Depths of calcrete (m)	SPT in calcrete zone	Average SPT from 3-8 m	Shear box		Triaxial	
				c	φ	c	φ
S01	10.69-12.00	25	24.5	4.2	29.1	20.0	34.0
S02	1.65-3.00	40	40.8				
S03	13.08-14.1	49	28.0	11.2	32.3		
S04	None	N/A	34.0			8.0	30.5
S05	11.71-12.20	29	23.5				
S06	1.29-1.50 and 12.72-13.46	20,29	24.7	4.0	32.6		
S07	None		39.3	4.1	33.8		
S08	17.34-18.76	13/23	25.0	5.7	35.8		
S09	4.08-7.50	35	35.0				



Figures 2: Details of the Dynamic Compaction operation

Figures 3: Details of the Dynamic Compaction operation

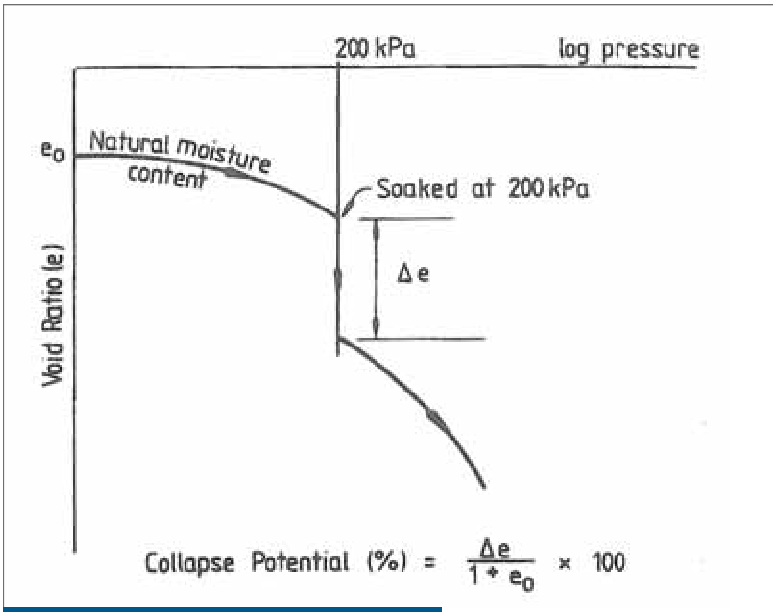


Figure 4: Typical collapse potential test result

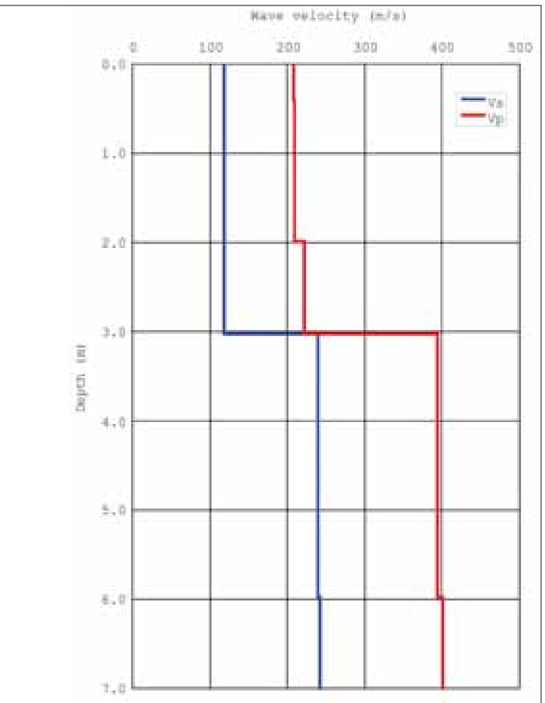


Figure 5: CSW test after DC

Figure 6: Completed initial foundation treatment and CSW testing

FOUNDATION VERIFICATION

In order to verify the efficacy of the DC process, continuous surface wave (CSW) testing was conducted before and after compaction. Examination of initial results revealed that the small strain stiffness of the 'after DC result' was less than that before. This initially caused much consternation, but on reflection the following was realised:

Collapse potential

The calcretised formations which are present on site are likely to be representative of potentially collapsible materials. This was in fact confirmed when examination of collapse potential (CP) tests, performed as part of the geotechnical investigation, generated values which, although low, did show signs of collapse. It is surmised that the DC process destroyed the cemented bonds which existed prior to the operation. Thus, in the initial state the material is cemented due to the pozzolanic action of the carbonates within the calcrete and exhibits high stiffness due to the rigidity of the cemented structure. However, after the application of the DC these bonds are broken and a less stiff structure results.

In this regard consider Figure 4 taken from the seminal paper by Jennings and Knight (1975).

This figure represents a typical collapse potential test. As may be observed, before collapse the material exhibits a fairly stiff structure, as evidenced by the shallow slope of the initial portion of the curve. However, after collapse the slope is very much steeper, representing a less stiff material.

Thus, although the DC had the effect of generating a less stiff material after compaction than before, the open grain structure due to the cementation would have been destroyed and a much denser material obtained. However, this is not to say that the densities achieved are sufficient. An examination of the measured shear wave velocities revealed the following (also see Figure 5):

Shear wave velocity (Vs-values) in excess of 160 m/sec are deemed acceptable. This is not so where values of around Vs=120 m/sec were obtained. Remedial action was deemed necessary. This action was, however, complicated by the fact that CSW testing was only conducted after base preparation works, which included a high-strength geosynthetic, 6 x 150 mm G5 layer, the blinding and the initial steel placement, as detailed in Figure 6 (which also shows the CSW testing being undertaken).

Further foundation treatment

Grouting

For this particular base a grouted solution was thought to be the most cost-effective. Very low stresses are present except on the outer ring under wind load conditions (see Figure 7).

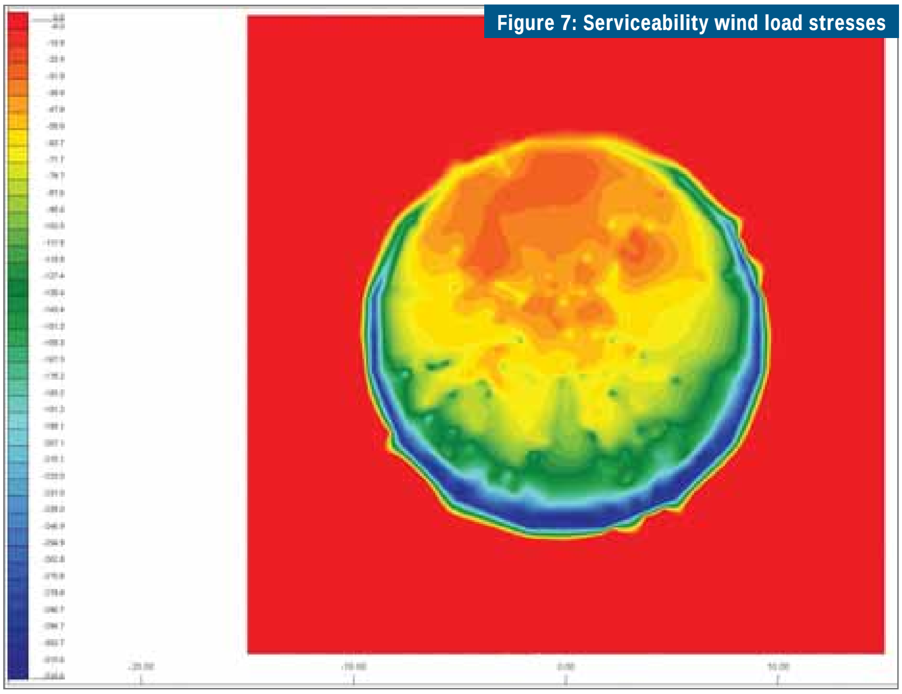


Figure 7: Serviceability wind load stresses

A system was detailed in which DPSH tests were conducted around the edge to a depth of 6 m at 600 mm centres. The 50 mm cone at the tip of the DPSH was welded to the rods, which were jack-extracted to enable re-use. The blow count per 300 mm penetration results were electronically recorded in a spreadsheet and sent to the designer on a daily basis for evaluation. This enabled a much more detailed assessment to be made of in-situ soil consistency. The holes were then pressure-grouted on a grout-one-skip-one basis under 100 kPa using an initial mix of 60 litres of water to 50 kg of cement. If grout takes exceeded 20 litres, the mix was progressively thickened to 40 litres of water, and then in a final step to 20 litres (see Figures 8 and 9).

Vibratory padfoot rolling

In many of the bases, the CSW testing only indicated non-compliance in the top 2 m or so. In this case it was reasoned that rolling would be sufficient to ensure compliance. Calculations revealed that 55 passes of a 380 kN centrifugal force, 1.9 mm amplitude, vibrating padfoot roller (VPR) would be needed for the required compaction standard. This was conducted as per Figure 10 with a typical CSW result, as detailed in Figure 11. What is obvious from these figures is the effect of the roller on the top 2 m of the profile.

Dynamic response

Due to the oscillatory nature of the turbine, a dynamic analysis of the soil response was undertaken utilising the formulations of El Naggar (2009). An analysis of the shear wave velocities at which the operating and natural frequencies correlate was conducted. Correlation occurs at shear wave velocities of less than 80 m/s, and also between 185 m/s and 260 m/s. Although the lower range is not critical, the upper range corresponds closely with the soil conditions present at the site after compaction. Since the natural and operating frequencies correlate for this range of shear wave velocities, resonant frequency is likely to occur, resulting in possibly significant, compounding forces being imparted into the foundation. Typical amplitudes calculated over this range of shear velocities were some 5 mm, as detailed in Figure 12.

This exceeds the allowable limit for harmonic machine foundations. The analysis was repeated to determine the minimum embedment depth required to minimise the amplitudes to within allowable limits. Soil embedment effects assist with damping and thereby minimising the effects of the various amplitudes. The minimum embedment depth required for the foundation was determined to be just greater than 1.5 m. Since the embedment depth detailed in the design is some 3.5 m, the amplitudes generated due to the resonant frequency would be reduced to insignificant magnitudes.

In addition to the specialised analysis conducted and described above, the following guidelines, based on past experience for sizing shallow foundations subject to reciprocating or centrifugal machines were generated:

1. The mass of the foundation should be 2-3 times the mass of the supported centrifugal machine, and 3-5 times the mass of the supported reciprocating machine. In this case, the foundation (16 363.1 kN) is 3.6 times the mass of the machine (4 525.0 kN) and is therefore acceptable.
2. The top of the block should be 0.3 m above the elevation of the finished floor (ground level in this case).


Figures 8 and 9: DPSH tests were conducted to assess in-situ soil consistency

Figure 10: VPR roller compacting surface

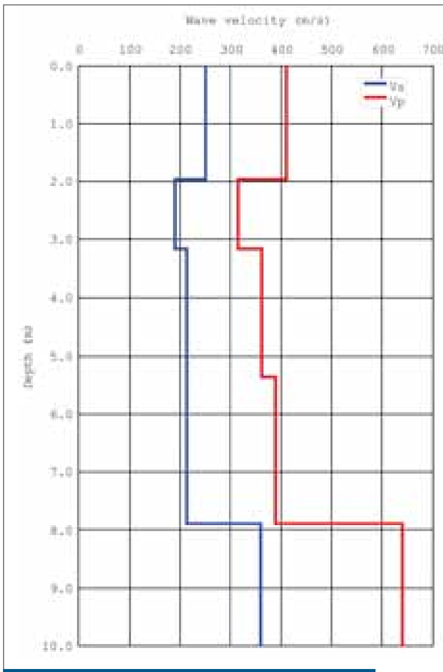


Figure 11: CSW result after DC and VPR

3. The thickness of the foundation should be the greatest of 0.6 m or one fifth of the least dimension of the footing (9.05/5 = 1.8 m). The foundation has a minimum thickness of 3.0 m and is therefore acceptable.
4. The width should be at least 1-1.5 times the vertical distance from the base to the machine centreline to increase damping in rocking mode. For this foundation, the ratio is 9.05/3.5 = 2.6 and is therefore acceptable.
5. It is desirable to increase the embedded depth of the foundation to increase the damping and provide lateral restraint as well. The provided embedment depth of 3.5 m is much greater than the required depth of 1.5 m, and is therefore acceptable. It is however important that the backfill is compacted to ensure a high-strength material. Hence a density of 93% Mod AASHTO compacted at 0 to +2% of the Mod AASHTO optimum is advised.

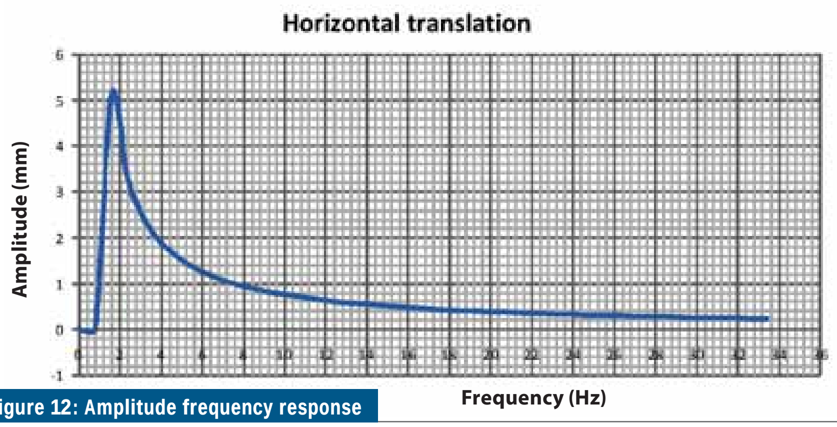


Figure 12: Amplitude frequency response

CONCLUSION

The above details the process followed to date to ensure a fit-for-purpose foundation for a large wind turbine. The cooperation from others on the projects and from the client, is acknowledged with thanks.

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