# PILE SETTLEMENT DESIGN USING CONTINUOUS SURFACE WAVE TESTING AND MODULUS DEGRADATION

Hennie F. T. Barnard, Aurecon, Polokwane, Limpopo, South Africa
Theodore E. B. Vorster, Aurecon, Pretoria, Gauteng, South Africa

Where geotechnical designs are governed by settlement requirements, soil and rock moduli are important parameters for design. This paper reports on a method that was used to utilize dynamic modulus measurements in the field toward pile settlement designs for the proposed Square Kilometer Array (SKA) Project to be located near Carnarvon, South Africa. The project will comprise of over 3000 No. movement sensitive satellite dish structures. The method described in the paper utilizes strain-related modulus degradation across the complete ground profile, starting from measurements of peak particle velocity (and inference to dynamic modulus at small strain) as measured using Continuous Surface Wave (CSW) testing at individual dish positions. This is required due to the very small allowable foundation rotation of 5 arc seconds under the dishes' operational conditions. The strategy of analysis was to conduct an iterative process starting from the measured small strain modulus profile of the ground. The likely maximum pile force for a specific piled foundation option was estimated and a single pile finite element analysis was performed using Plaxis 2D (Version 2010) to estimate the expected ground strain developing along the length of the pile. By assuming the shape of the shear modulus degradation curve with increasing shear strain, the iteration process then converged on a likely degraded modulus profile associated with the single pile and maximum load assumed. This process enabled identifying the pile cap size, pile configuration, pile sizes and pile lengths that would meet the operational movement limits. The chosen pile group configuration was then analyzed using the Repute 1.5 pile group analysis software. The design process and typical results are discussed in this paper.

## 1 Introduction

Geotechnical engineers are continuously searching for better methods to predict foundation settlements. Ground shear modulus is dependent on the magnitude of soil shear strain incurred during loading. This paper reports on the analytical method that was used to take advantage of small strain shear modulus conditions and appropriate shear modulus degradation to optimize pile group designs for movement sensitive satellite dishes, to be located in an arid environment in the South African Karoo near Carnarvon.

The geotechnical investigation utilized Continuous Surface Wave (CSW) testing, to define the dynamic small strain shear modulus characteristics of the ground profile at individual dish positions (Matthews *et.al*, 1996; Heymann, 1998; and Matthews, 2000). Due to variable hardness ground layers, which would make core drilling difficult as well as severe time constraints, percussion drilling was performed at individual dish positions to enable describing the underlying ground profile. In addition, test pits were dug to visually classify the upper ground profile. Large diameter (600 mm) vertical plate load testing was done at selected dish positions to classify the dry and saturated modulus parameters (at higher strain levels) of the upper ground profile. Furthermore, laboratory testing was conducted for engineering classification purposes.

To arrive at appropriate founding solutions for individual dish foundations and meet the very stringent rotational limit of 5 arc seconds of rotation under the operational condition, a design philosophy was needed that could take advantage of the 'true' strain condition of the ground within the loading zone. Due to the high variability of ground conditions across the site, a representative ground profile was defined as follows:

| | |
|---|---|
| 0 – 6.0 m: (0 – 19.7 ft.) | Fine and medium sub-rounded platy mudstone and sandstone gravel with minor calcrete and occasional silty sand. Cemented Alluvium. |
| 6.0 – 10.0 m: (19.7 – 32.8 ft.) | Highly weathered mudstone with minor calcrete. Residual Mudstone. |
| >10.0 m: (> 32.8 ft.) | Moderately to slightly weathered, medium hard to hard rock, Mudstone (the mudstone occurs from 5m (16.4 ft.), depending on the position). |

Two possible founding solutions were anticipated, namely shallow pad foundations onto very soft rock (or better) mudstone or calcrete , or piled foundations where the depth of the overlying sandy alluvium and calcretised gravels exceeded 3 m (9.8 ft.). Pad foundations were soon discarded since it could not meet the ground movement limits. Optimization of pile lengths provided the distinction between foundations located on shallower more competent ground conditions and deeper ground conditions.

To utilize the advantage of higher moduli occurring at smaller strains in design, an iterative process was followed. The initial shear modulus conditions for each of the shallower and deeper ground conditions mentioned earlier were taken as a lower-bound small strain ground modulus profile, derived from shear wave velocity profiles at each dish position measured using CSW testing. Preliminary estimates of the occurrence of shallow and deeper ground conditions (per foundation position for each of 44 No. positions) were identified based on penetration rates measured during the percussion drilling. For each of the two founding scenarios a lower bound dynamic shear modulus profile was generated by plotting individual CSW test results with depth and generating a lower bound shear modulus profile for each scenario. Equation 1-1 was used to calculate $G_0$ values from measured CSW results. Once the $G_0$ values were calculated, $E_{max}$ values could be inferred using equation 1-2 and assuming elasticity at small strains:

$$G_0 = \rho \cdot V_s^{\,2} \qquad (1\text{-}1)$$

$$E_{\max} = 2G_0(1+v) \qquad (1\text{-}2)$$

Where,
$G_0$ = Dynamic small strain shear modulus (MPa)
$V_s$ = Shear wave velocity (m/s)
$\rho$ = Soil density (kg/m$^3$)
$E_{max}$ = Small strain Young's modulus (MPa)
$v$ = Poison's ratio

The assumption of elasticity is justified by the low ground disturbance incurred during CSW testing (Matthews *et al.*, 1996). The combined modulus profiles shown as small strain shear modulus ($G_0$), are shown from foundation level (1.5 m (4.9 ft.) depth) to the maximum CSW measurement depth in Figure 1.

The maximum pile force in an envisaged pile group was estimated by analyzing the pile group in Repute 1.5. A single pile was then modeled with the maximum load applied in an axisymmetric finite element analysis using Plaxis 2D (version 2010). This gave an estimate of shear strain distribution due to pile loading along the length of the pile. An assumption is made regarding the relationship between $G/G_0$ ($G$ = Shear Modulus) and engineering shear strain, $\gamma$. Following each Plaxis 2D analysis, the shear modulus distribution with depth is updated until shear moduli for each layer differ by less than 0.1 MPa between iterations. The process in time converges on a ground modulus profile optimized in relation to pile-soil interaction. This happens typically within 4 No. iterations.

The new ground modulus profile for the single pile load case (with maximum load) is then used in a pile group analysis using Repute 1.5 (ensuring that piles are further than 2.5 pile diameters apart) to optimize the pile group and determine the individual pile loads more accurately.

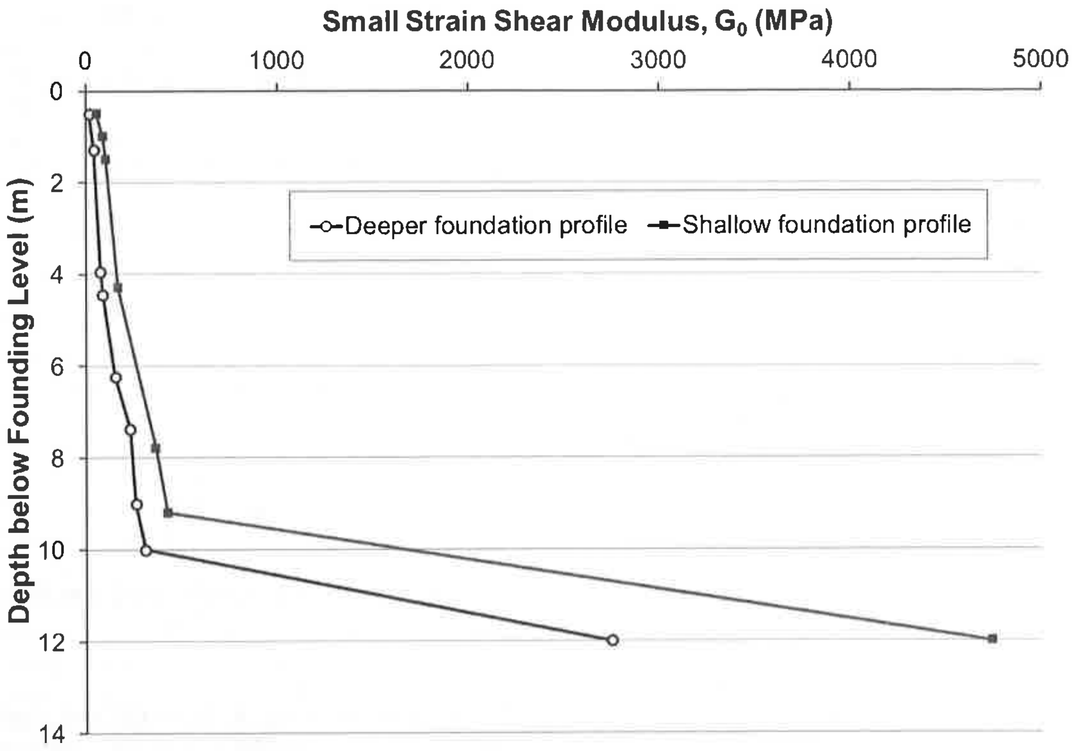


Figure 1: Small strain shear modulus profiles used in the design

Given that the maximum pile load in this analysis remains similar and lower than what was used in the Plaxis 2D iteration, the pile group would be safely designed, whilst utilizing the advantages of more accurate degradation of shear moduli with depth. The final dish foundation solution envisages groups of 600 mm (1.96 ft.) diameter bored piles installed to 10 m (32.8 ft.) depth or 5 m (16.4 ft.) depth for the deeper and shallow ground conditions, respectively. To illustrate the methodology described, a 10 m (32.8 ft.) piled solution is discussed in more detail.

## 2 The Design Process

The design process to utilize small strain modulus and modulus degradation is discussed in this section. Furthermore, an example which illustrates the application of the described method is discussed.

**Step 1: CSW testing and lower bound ground modulus profile**

CSW testing was performed at each dish position. A lower bound small strain shear modulus profile is generated by plotting all the relevant CSW test results with depth and utilizing the minimum values of the complete sample of measurements with depth as shown in Figure 1. For SKA the ground profile was subdivided into 9 No. modulus layers. Table 1 shows the proposed depth distribution, the estimated $G_0$ and $E_{max}$ values for each modulus layer, with Poison's ratio, $\nu$, taken as 0.25. Layer 9 was replaced with a 1 MPa modulus layer in the calculations (located only at the tip of the pile and not across the entire model), to simulate frictional pile behavior and to eliminate end bearing loads at the toe of the piles.

This was necessary to account for the risk of not being able to properly clean the pile bases. The dynamic small strain modulus profile for the deep founding condition described in Table 1 is used in the example.

Table 1: Small strain modulus profile for the deeper ground condition

| Layer | Depth (m) | Thickness (m,(ft.)) | $G_0$ (MPa) | $E_{max}$ (MPa) |
|---|---|---|---|---|
| 1 | 0.0 to 1.0 | 1.0 (3.28) | 17 | 42.5 |
| 2 | 1.0 to 1.8 | 0.8 (2.62) | 41 | 102.5 |
| 3 | 1.8 to 4.5 | 2.7 (8.86) | 74 | 185 |
| 4 | 4.5 to 5.0 | 0.5 (1.64) | 87 | 217.5 |
| 5 | 5.0 to 6.7 | 1.7 (5.58) | 152 | 380 |
| 6 | 6.7 to 7.9 | 1.2 (3.94) | 230 | 575 |
| 7 | 7.9 to 9.5 | 1.6 (5.25) | 259 | 647.5 |
| 8 | 9.5 to 10.0 | 0.5 (1.64) | 310 | 775 |
| 9 | 10.0 to 20.0 | 10 (32.80) | 2749 | 6872.5 |

**Step 2: Choosing a pile group configuration**

For each different load case an initial estimate is made to estimate the number of piles and layout spacing. The two main factors that play a role in choosing initial sizes are the maximum load on a single pile and the allowable rotational limits for each load case (see Table 2 which shows the load cases that determined the design).

**Step 3: Repute 1.5 calculations to estimate the maximum pile load**

Once the initial sizing is fixed, the pile group is analyzed (for SKA Repute 1.5 pile group analysis software was used) to estimate the maximum vertical load on any particular pile in the group.

For SKA, the pile foundation configuration options consisted of 4 No., 8 No. and 12 No. pile group layouts with pile spacing of either 1.5 m or 3.0 m (see Figure 2). This resulted always in at least 2.5 $D_p$ spacing ($D_p$ is the pile diameter) and justifies the modulus degradation calculation to be done as a single pile analysis.

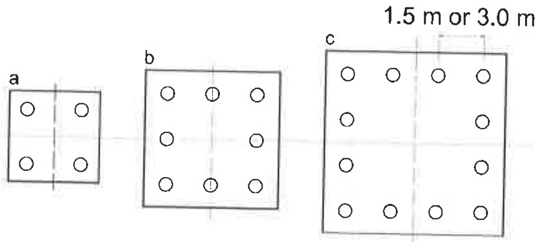


Figure 2: Proposed pile layouts, (a) 4-pile group; (b) 8-pile group; (c) 12-pile group

In the example an 8 No. x 10 m (32.8 ft.) long, 600 mm (1.97 ft.) diameter pile group, analyzed for load case 5 was used. The initial analysis is done with both the small strain shear modulus profile as well as a "softer" profile (i.e. with modulus degradation) in order to develop an envelope of possible ground shear moduli within which the pile group is expected to behave.

Table 2: Loads and performance requirements

| Load Case No. | Load case | Loads | Performance requirement |
|---|---|---|---|
| 1 | 36 km/h wind load | $M_y$ (Bending) = 553 kN.m<br>$M_z$ (Torsion) = 62.6 kN.m<br>$F_z$ (Axial) = 426.8 kN<br>$F_x$ (Radial) = 27.8 kN | Max allowable deflection = 5 arc seconds |
| 2 | 60 km/h wind load | $M_y$ (Bending) = 994.4 kN.m<br>$M_z$ (Torsion) = 168.0 kN.m<br>$F_z$ (Axial) = 497.4 kN<br>$F_x$ (Radial) = 70.9 kN | No permanent settlement |
| 5 | Operations | $M_y$ (Bending) = 903.0 kN.m<br>$M_z$ (Torsion) = 50.0 kN.m<br>$F_z$ (Axial) = 195.0 kN<br>$F_x$ (Radial) = 62.0 kN | Max allowable deflection = 5 arc seconds |

For SKA the "softer" profile was conservatively represented by ground shear moduli values taken as 20% of the small strain shear moduli to come to a situation representing the largest possible movement envisaged. If it is found that even with the small strain modulus profile, the foundation movements exceed the design limits, a new foundation layout needs to be assumed and the process from Step 2 needs to be repeated. By using the proposed envelope of shear moduli revisiting of pile groups were not done for the SKA project as the initial pile configurations could be chosen sufficiently well.

For the example the initial analysis was done with both the small strain modulus profile as well as a "softer" profile at 80% modulus degradation. Results from the two modulus profiles, as described above, allowed establishing an envelope range of expected maximum pile loads within which the pile group was expected to behave. Values of 79.9 kN (small strain modulus profile) and 82.6 kN ("softer" modulus profile) were estimated using Repute 1.5. The estimated maximum pile force, calculated as 82.6 kN was then used in a single pile finite element analysis using Plaxis 2D (Version 2010) to allow for an 'upper bound' modulus degradation profile (in terms of magnitude of modulus degradation).

**Step 4: Single pile analysis using Plaxis 2D**

Following the verification that design limits are met under small strain conditions, the estimated maximum pile force determined in Step 3 is used in a single pile, axisymmetric, elastic finite element analysis using Plaxis 2D (Version 2010). The objective of the finite element analysis is to calculate the shear strain induced in the center of each of the demarcated ground modulus layers. The maximum engineering shear strain values arising from the pile-soil interaction are then used to calculate corresponding shear moduli used as input for the next iteration. Since the ground on the SKA site is primarily granular, it was assumed that the relationship of modulus degradation could be described by Rollins *et al.* (1998) as follows:

$$\frac{G}{G_0} = \frac{1}{\left[1 + 16\gamma\left(1.2 + 10^{(-20\gamma)}\right)\right]} \qquad (2\text{-}1)$$

Where,

$G$ = Shear modulus (MPa)

$G_0$ = Small strain shear modulus (MPa)

$\gamma$= Engineering shear strain (%)

*Note that the original Rollins et al. (1998) paper has an error in the equation, which is corrected in Eq. 2-1.*

Figure 3 shows the relationship between $G/G_0$ and $\gamma$. The relationship between $G$ and $E$ (Young's modulus) is maintained under the elastic relationship described in Eq. 1-2.

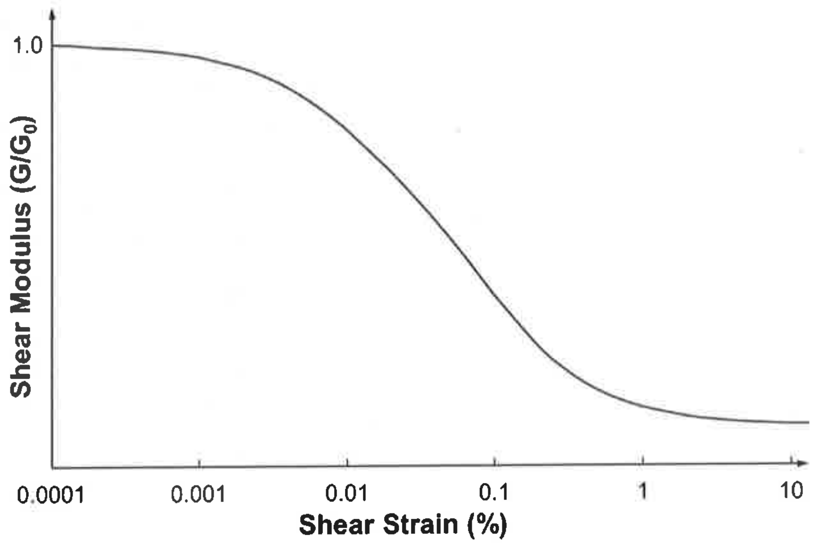


Figure 3: Shear modulus degradation curve (after Rollins *et al.*, 1998)

This is a theoretical shortcoming of the design process described here, but for the purpose of design was deemed sufficiently accurate. The axisymmetric finite element model (50 m wide and 50 m deep) consisted of 15-noded elements and in total the mesh generated 446 soil elements and 3681 nodes. Thus, with the 10 m pile analysis, the depth below the pile toe was 40 m. For each layer the corresponding value of $\gamma$is determined in the first iteration and was used to calculate degraded modulus values for the next iteration by using Equation 2-1.

**Step 5: Repeating the iterative process**

The process described in Steps 3 and 4 is repeated until the modulus difference between iterations for each layer converges to less than 0.1 MPa as shown in Table 3. The final degraded modulus profile that was used to estimate final foundation movements is shown as modulus $E_4$. This implies that the process converged within 4 No. iterations. This was generally the norm. Table 3 also shows the modulus degradation percentages that occurred during iterations. The benefit of utilizing small strain modulus is realized in the fact that below 5.0 m (16.4 ft.), more than 80% of the small strain modulus is used. This is hugely beneficial to the design.

**Step 6: Finalizing foundation movements**

The final estimated foundation movements are calculated using pile group analysis software such as Repute 1.5 and the final degraded ground modulus profile.

If, during this step (or any prior step) it transpires that the estimated foundation movements exceed the allowable design limits, the iteration process is repeated. Where the allowable movement criterion is met, spring stiffness response per pile is calculated in order to complete the remainder of the structural design.

For the example the Repute 1.5 analysis was done with the final degraded soil modulus profile determined in Step 5. It was found that with the final degraded profile, the maximum pile load is 80,2 kN, and in effect would have changed the modulus profile if the iteration process was repeated with the new calculated maximum load. However, because the difference compared to the estimated 82,6 kN maximum load estimated during the high level process in Step 3 is only 3%, and due to the fact that smaller loads were calculated for other piles in the group (i.e. even stiffer movement responses were generally expected than the analysis implied) it was deemed that the movements corresponding with the 82,6 kN pile load condition would be sufficiently conservative to determine the final foundation response.

Table 3: Modulus values and engineering strains for the 82.6 kN load iterations

| Depth (m) | $E_0$ (MPa) | $\gamma_{xy}$ [$10^{-6}$] | $E_1$ (MPa) | $E_1 / E_{max}$ (%) | $\gamma_{xy}$ [$10^{-6}$] | $E_4$ (MPa) | $E_4 / E_{max}$ (%) |
|---|---|---|---|---|---|---|---|
| 1.0 | 42.5 | *200.4* | 28.1 | 66.1 | *222.0* | 27.3 | 64.3 |
| 1.8 | 102.5 | *155.5* | 72.2 | 70.4 | *175.7* | 70.1 | 68.4 |
| 4.5 | 185 | *106.8* | 141.3 | 76.4 | *124.6* | 136.9 | 74.0 |
| 5.0 | 217.5 | *79.4* | 175.3 | 80.6 | *95.9* | 169.6 | 78.0 |
| 6.7 | 380 | *57.1* | 322.1 | 84.8 | *70.1* | 312.6 | 82.3 |
| 7.9 | 575 | *39.7* | 509.3 | 88.6 | *49.9* | 496.1 | 86.3 |
| 9.5 | 647.5 | *33.7* | 582.8 | 90.0 | *43.1* | 568.3 | 87.8 |
| 10.0 | 775 | *36.0* | 693.3 | 89.5 | *46.3* | 674.6 | 87.0 |
| 20.0 | 6872.5 | *0.0* | 6872.5 | 100.0 | *0.0* | 6872.5 | 100.0 |

**Step 7: Determining spring stiffness**

To complete the structural design, spring stiffness values for individual piles are required to model the proposed piled foundations. The final observed foundation rotations are converted to couple reactions pivoting about the central axis of the foundation. The individual pile loads estimated in Repute 1.5 together with the total vertical displacement (initial vertical displacement of pile group and calculated rotational displacement) are used to calculate the spring stiffness values for each pile in units of [kN/mm].

For the example the estimated foundation movements did not exceed the allowable limit and therefore the pile configuration could be fixed and spring stiffness values could be calculated. Table 4 shows the final movements for load case 5.

Table 4: Settlement and rotation movements (From Repute analysis)

| Settlement and rotation | Value |
|---|---|
| Vertical settlement | 0.05 mm |
| Horizontal settlement | 0.16 mm |
| Actual Rotation | 4.7 < 5 arcsec.→O.K. |

## 3 Summary and Conclusion

This paper shows a novel approach of estimating pile foundation movement response by utilizing dynamic small strain modulus data measured using Continuous Surface Wave (CSW) testing. A method was devised whereby the small strain modulus is degraded in relation to estimated ground shear strain response during loading of a single pile. By using a combination of Repute 1.5 pile group analyses to estimate pile load response, Plaxis 2D finite element analyses to estimate shear strain during each iteration and an assumption of the shape of the modulus degradation curve, the final pile group response could be determined. This process allowed for analyzing dish foundation solutions for the SKA project in South Africa to meet a very stringent rotational limit of 5 arc seconds under operational condition.

The iteration process generally converged within 4 No. iterations, making it a relatively quick and useful method to take account of soil modulus degradation and maintain the benefit of small strain modulus along portions of the pile.

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