Square kilometre array pile foundation design methodology

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ABSTRACT: The Square Kilometre Array (SKA) project, spanning locations in the Northern Cape, South Africa, and Western Australia, is an international effort to build the world's largest radio telescopes. The primary scientific objective of the SKA project is to explore the universe. The scale and sensitivity of this project requires extremely precise design of foundations for its telescopes, as even minor settlements could result in significant misalignments, impacting data collection. This paper details the foundation design methodology for the SKA telescopes and the unique site conditions. Limited access, communication restrictions, and the need to minimize radio frequency interference all played a role in design decisions. A geological model was developed due to the lack of geotechnical investigations at all telescope positions, leading to the creation of five distinct ground profiles. The design required rigorous three-dimensional finite element modelling, considering small shear strain stiffness behaviour, to predict vertical and horizontal displacements, ensuring compliance with stringent tolerances for operational and survival loads. The design methodology demonstrates how the design effectively meets the required stiffness limits, with the most critical factor being the control of shear strain along the pile shafts.

1 INTRODUCTION

This paper presents the design methodology of piled foundations for telescopes in the remote Meerkat National Park, Northern Cape, South Africa. The site, characterised by limited access and communication, posed unique challenges, including the need to minimize radio frequency interference to protect telescope data integrity. Geotechnical investigations at all telescopes positions were not feasible and therefore a geological model was created. Five distinct ground profiles were developed by Zutari based on this model and assigned to each telescope.

The piled foundations were required to meet strict design criteria to avoid misalignments and ensure precise telescope operation. Key design requirements include:

1. limiting permanent plastic displacements to 20% for operational loads, and
2. ensuring that the pile cap does not exceed the allowable displacement during axial, lateral, moment and torsional loading for both operational and survival loading, and
3. to ensure that the stiffness response does not break down after a survival condition (i.e. decrease in stiffness response with an increase in unload reload cycles).

To illustrate the requirements, during operational loading, the maximum loaded pile was only allowed to settle by roughly 0.75 mm to adhere to the stiffness requirements. This number also includes elastic compression of the pile shaft.

Finite element modelling and hand calculations were used to predict vertical and horizontal displacements, with design checks to verify compliance with geotechnical capacity and structural requirements. The methodology discussed in this paper was developed to streamline the design of over a hundred telescopes grouped into five ground models.

Figure 1. Existing telescopes on the Meerkat National Park

This study emphasises the importance of collaboration between geotechnical and structural engineers to accurately model soil-structure interactions and ensure the success of the project in challenging site conditions. A photo of existing telescopes is provided in Figure 1.

2 LOCATION AND GEOTECHNICAL INVESTIGATION

The site is located within the Meerkat National Park in the remote Northern Cape Province, an area characterised by significant limitations in resources, access, and communication. This remoteness is one of the key reasons this site was chosen for this significant international effort. The Northern Cape is semi-arid, with sparse vegetation predominantly consisting of Karoo shrubs and grasslands. The site covers an area of over 135,000 hectares. Historically, the land has been primarily used for sheep farming.

A further complicating factor at the site is the restriction on radio frequency interference (RFI). The presence of radio frequencies disrupts the operation of the telescopes, potentially corrupting the data collected and undermining the integrity of the research. Consequently, the site strictly prohibits devices transmitting frequency, including mobile phones and Bluetooth devices. RFI checks are conducted upon entry to the site to ensure compliance with these regulations.

Due to the large area and the number of telescopes involved in the design, geotechnical investigations were not conducted at each telescope position. Instead, rotary core boreholes, percussion holes, test pits, and Continuous Surface Wave (CSW) tests were carried out at various points across the site. Zutari developed comprehensive ground models based on these investigations. From the overall site model, five distinct ground profiles, called Ground Models, were created. Each profile was designed to represent a specific rock interception depth, ranging from 1.5 m to 10.5 m below the pile cap. The lowest shear strain modulus profile from any investigation point within a specific ground profile was also selected as the governing stiffness profile.

Figure 2 shows the site's central core and the three spiral arms extending up to 100 kilometres.

The profile for Ground Model 1 is provided in Figure 3 as an example.

3 DESIGN REQUIREMENTS

Strict design criteria were required for the foundations of the telescopes to ensure that no displacements at the base would result in significant misalignments of the telescope's projection.

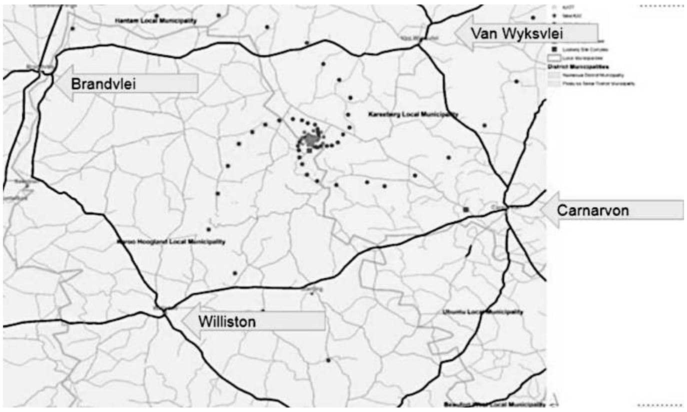

Figure 2. Core and three spiral arms of the site

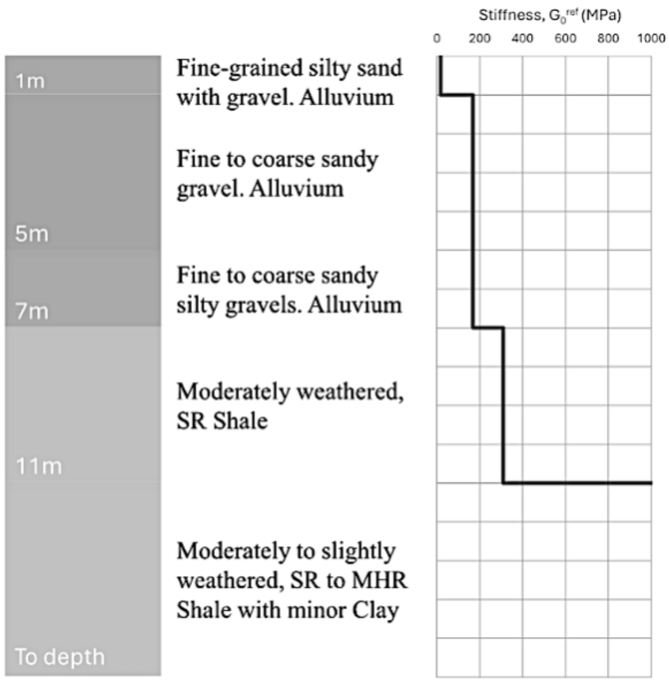

Figure 3. GM1 Profile

Stiffness limits (load/displacement) were placed on axial, lateral, tilting, and torsional loading. Numerous design load cases were considered, including Operational, Survival non-stow, Survival stow and Earthquake combinations. These were factored using partial factors provided in SANS10160-1, to determine the design structural loads within the piles.

Each load case was required to meet a Factor of Compliance (FoC) exceeding unity. Permanent plastic deformations under operational loads were limited to 20% of the total displacement experienced under full operational load. Breakdown of the stiffness response after survival load cases had to be prevented.

4 FINITE ELEMENT MODELLING

To ensure the design requirements were met, rigorous three-dimensional finite element (FE) analyses were undertaken using Plaxis 3D. These models included the pile cap, and all stiffness requirements were related to the top of the cap where the telescope was connected to the foundation system.

4.1 *Piles as volumetric vs embedded beams*

Both volumetric elements and embedded beams were considered in the FE model to represent the piles below the base. An embedded beam simulates the behaviour of a pile in a simplified way, significantly reducing modelling time. The volumetric elements produced less conservative results when compared to a model with the same loading but with embedded beam elements. It was therefore decided to proceed using embedded beams. Figure 4 provides FE extracts for the volumetric elements a) and embedded beams b).

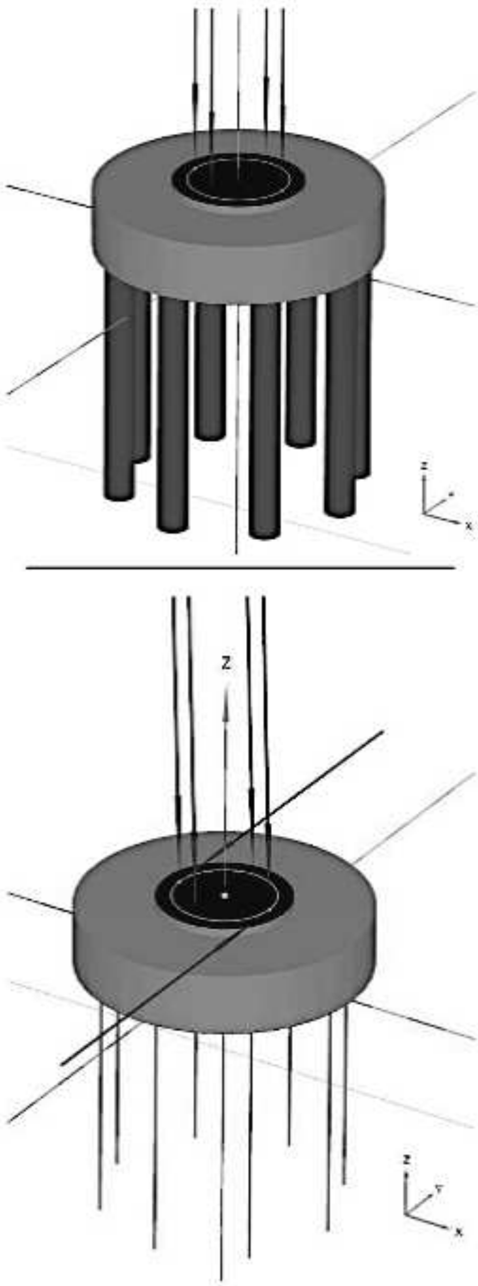

Figure 4. a) piles as volumetric elements and b) piles as embedded beams.

4.2 *Constitutive Model*

Due to the strict displacement tolerances, stiffness degradation with an increase in shear strain had to be considered in the FE modelling. This was done using the Hardening Soil model with small strain stiffness (HSS). The rock was modelled using Mohr-Coulomb.

4.3 *Sensitivity checks*

To establish which input parameters had the greatest effect on the model output, in particular the displacements, numerous input parameters were changed within the model to determine the effect they have on the results. It was found that the displacements were predominantly governed by the soil stiffness, particularly the small strain shear stiffness (G_0^{ref}), as only very small strains could develop to ensure compliance. Figure 3 provides the G_0^{ref} values vs depth for Ground Model 1.

5 DESIGN

5.1 *Hand Calculations*

It was imperative that two methods be used to predict the displacement to ensure that the FE modelling considering numerous input parameters was within reason. Therefore, hand calculations (HC) were performed in conjunction with FE analyses to predict the vertical and horizontal displacements.

5.1.1 *Vertical displacement*

Two equations from Zhang (2005) were used to predict elastic shaft compression and socket displacement for piles that derive most of their capacity by socketing into rock. The predictions from these two equations can be summed to determine the total predicted displacement of the pile. Equations 1 and 2 predict elastic shaft compression and socket displacement, respectively:

$$\delta_E = \frac{P\,L}{A\,E} \qquad (1)$$

where δ_E is the shaft displacement; P is the load; L is the length of the pile in soil; A is the area of the pile; and E is the pile`s Young`s Modulus.

$$\delta_s = \frac{P_t\,I}{E\,D} \qquad (2)$$

where δ_s is the socket displacement; P_t is the load at the top of the socket; I is the influence factor; E is the rock mass modulus; and D is the diameter.

Ground Model 5, with bedrock at 10.5m below the pile cap, relied predominantly on capacity derived from side shear. The vertical displacement for these piles was predicted using Everett (1991) as provided in Equation 3.

$$f = C_{ult}\left(1 + e^{-K_1 d_{ps}}\right) + F_{ult}\left(1 + e^{-K_2 d_{ps}}\right) \qquad (3)$$

where f is unit side shear transfer; C_{ult} is ultimate adhesion transfer; F_{ult} is ultimate intergranular transfer; d_{ps} is pile shaft movement; and K_1 and K_2 are constants.

5.2 *Horizontal displacement*

The horizontal displacement was predicted using Broms (1964) as provided in Equation 4.

$$\delta_0 = \frac{0.93\ H}{n_h^{3/5}\ (EI)^{2/5}} \tag{4}$$

where δ_0 is the displacement at the pile head; H is the horizontal load; n_h is the coefficient of modulus (derived from stiffness at appropriate shear modulus ratio); E is the concrete's Young`s Modulus; and I is the pile Moment of Inertia.

It is crucial to perform hand calculations to verify that the results from the FE models are within an acceptable range, as FE models involve many components and parameters, many of which can be misinterpreted and significantly influence the results. As an example, Table 1 presents a comparison between the HC displacements and those derived from FE modelling for Ground Model 1.

Table 1. GM1 HC and FE displacements

Load	HC displacement	FE displacement
410 kN	0.359 mm	0.412 mm
850 kN	0.838 mm	0.955 mm

The predictions for the vertical and horizontal displacements indicate that the displacements obtained from the FE models were reasonable.

5.3 *Design checks*

5.3.1 *Check 1: Pile length*

The aim was to take all piles to rock as this would have the most significant effect on limiting displacement. In determining the pile and socket length, the end bearing was ignored because the presence of any loose material left by the drilling operation at the socket toe could lead to significant displacement, even if the material is thin. The design was primarily governed by displacement and plastic/residual displacement, rather than by the ultimate pile capacity.

Loads experienced by each pile in the pile cap were calculated for the different load cases. Survival non-stow was the worst survival load case scenario.

Using the maximum calculated load, an FE model of a single pile was first considered for each ground model to derive various load-displacement curves for different pile lengths. Figure 5 shows the load-displacement curves for different socket lengths.

The shear strain adjacent to the pile shaft was assessed for Operational and Survival non-stow load cases for various socket lengths. This prevented the need to undertake iterative time-consuming modelling of the full model (all piles and pile cap) using different socket lengths and loads.

Fixity affects lateral load; therefore, the shear strain in the soil adjacent to the pile was determined from a model with all piles, only applying the lateral load component. This exercise aimed to ensure that the plastic displacement was limited after the Operational load case occurred and that the stiffness response did not significantly break down after a Survival non-stow load.

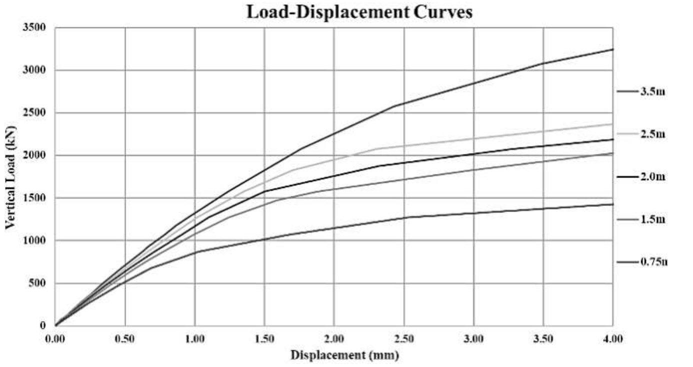

Figure 5. Load-displacement curves for different socket lengths.

Figure 6, from Díaz-Rodríguez (2009), shows that shear strain and displacement will be fully recoverable up to a shear modulus ratio of approximately 0.9 on the shear strain stiffness curve.

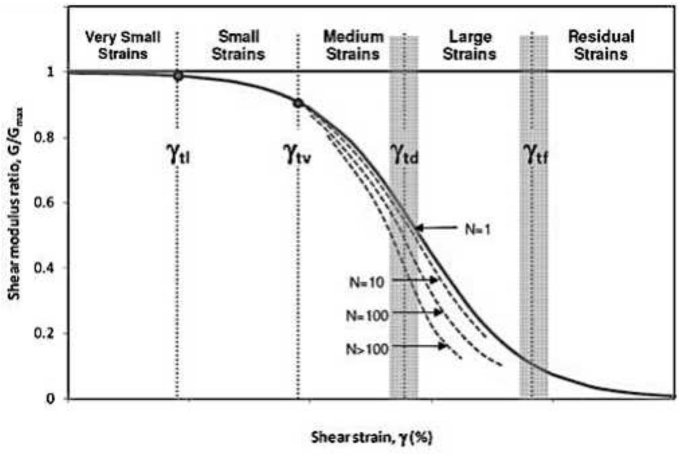

Figure 6. Shear Strain-Stiffness Curve with repetitive loads (Díaz-Rodríguez, 2009)

Equation 5 was used to estimate the minimum shear modulus ratio at a point below a ratio of 0.9, where plastic displacement would be kept below 20%, where δ_1 and δ_2 are the socket displacements; P_t is the load; I is the influence factor; E is the rock mass modulus; and D is the pile diameter.

$$\delta_1 - \delta_2 = \frac{P_t I}{x E_0 D} - \frac{P_t I}{0.9 E_0 D} < (0.2)\delta_1 \qquad (5)$$

$$\therefore \frac{P_t I}{x E_0 D} - \frac{P_t I}{0.9 E_0 D} < (0.2)\frac{P_t I}{x E_0 D}$$

$$\therefore x = 0.72$$

Therefore, if the shear modulus ratio exceeds 0.72, plastic deformation will remain below 20% of the total displacement for a given load.

To prevent stiffness breakdown after a survival event, Figure 6 shows that repetitive loads of up to 10 will not significantly reduce the stiffness response to a shear modulus ratio of 0.55 (curves for N=10 and N=1 are parallel up to this point).

The shear modulus ratio corresponding to the shear strain along the pile shaft was determined from FE models for vertical and horizontal loads in the operational and survival non-stow load cases. If the pile lengths modelled did not achieve a shear modulus ratio of 0.72 for operational loads and 0.55 for survival non-stow loads, they were increased until the shear modulus ratios achieved were acceptable. Figures 7 and 8 show the shear strains in the FE model output for a single pile with vertical load and the pile foundation with horizontal load respectively.

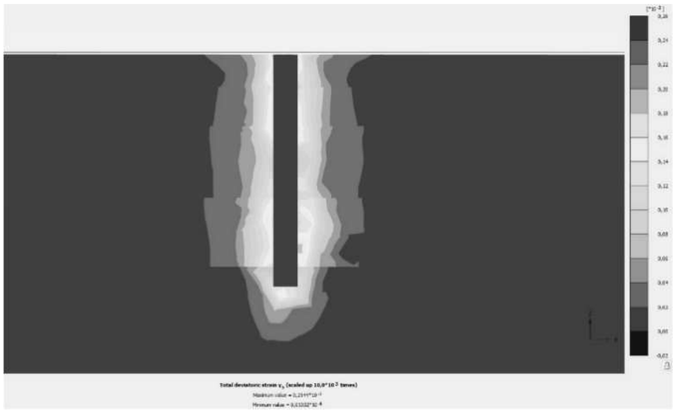

Figure 7. Shear strain output for vertical loading (single pile).

Figure 8. Shear strain output for horizontal loading (all piles and pile cap).

For instances where the horizontal shear check was not adhered to due to low-stiffness material directly below the pile cap, pile diameters had to be increased. Pile diameters were increased from 900 mm to 1050 mm to reduce the shear strains. This was the case for GM3 & GM5 where a deep rock profile was present.

In addition to this, the entire load-displacement curve for various pile lengths was derived as seen in Figure 5. If the load remained within the linear portion of the load-displacement curve, it was considered that minor plastic strain would occur.

5.3.2 *Check 2: Factor of compliance.*

After determining the required pile length in Check 1, the complete model with all piles and the pile cap was updated to reflect this pile length. Factor of compliance (FoC) checks were conducted on pile lengths that had sufficient shear modulus ratios. In most instances, the pile lengths were governed by Check 1, resulting in FoCs greater than unity, implying that Check 2 did not govern.

The FoCs were calculated by determining the stiffness at the top of the pile cap and dividing it by the required stiffness. The stiffness at the top of the foundation was calculated using the displacement and rotational angle of a plate on top of the pile cap. An FoC of 1.25 was desired to account for additional movements within the bolt cage connection to the telescope.

FoC values were calculated for the operational load case by applying loads individually. Four possible load scenarios were considered for each of the ultimate load cases. The FoC was calculated from the final stage after applying three of the four load components in one phase and the last load component in the final phase. The effect of only the last load component on displacement was considered by resetting the displacement from the previous phase but not the shear strain already developed in the model.

5.3.3 *Check 3: Geotechnical capacity.*

The geotechnical capacity of the piles was determined from the shaft resistance in the soil and side shear resistance in the socket, ignoring base resistance. The required pile length from Checks 1 and 2, for geotechnical capacity, was assessed using hand calculation methods. The Factor of safety (FoS) and utilisation factors for the designed pile length from Checks 1 and 2 were calculated. The utilisation factor was always lower than unity, highlighting that shear strain, rather than the geotechnical capacity, governed the design. It can, therefore, be inferred that the utilisation factor achieved aligns with a limited shear strain design and a reduced reliance on geotechnical capacity.

Due to the scale of the project and the large site area, different rock depths and strengths were anticipated in the same ground model, requiring an on-site procedure for pile length changes depending on rock depth and hardness. Although the design was based on a particular rock strength (e.g. soft rock), a pile length for very soft and medium hard rock was derived from achieving the same utilisation factor which would ensure that Checks 1 and 2 are met if the rock strength is different.

Therefore, recommended pile lengths for different rock strengths can be predicted by correlating the utilisation factor for the rock strength modelled in FE to a weaker or stronger rock strength where a different pile length has the same utilisation factor. This allows the design to be adaptable to various site conditions as they arise and eliminates the need for further design review and delays when a different rock hardness profile is encountered on a specific ground model. With the site's communication limitations, the adaptable design was essential to the project's success.

5.4 *Springs*

Spring values were calculated at the head of each pile in the base for the effect the vertical, horizontal, moment and torsional loads had individually, in the operational and lowest FoC survival load case. Vertical springs were calculated using Equation 6 for the vertical and moment loads, and horizontal springs were computed using Equation 7 for the horizontal and torsional loads.

$$\text{Vertical Spring} = \frac{\Delta P}{\Delta \delta_V} \tag{6}$$

$$\text{Horizontal Spring} = \frac{\Delta H}{\Delta \delta_H} \tag{7}$$

where: P and H are the changes in load; δ_V and δ_H are the displacements from only the fourth load component applied.

The springs determined at the head of the piles for operational, survival, and factored survival load cases were supplied to HHO's structural engineers to incorporate into their FE models to evaluate the base connection. Constant interaction between the geotechnical and structural engineers was crucial to ensure the models converged.

6 CONCLUSIONS

This paper presents the design methodology of pile foundations for telescopes in Meerkat National Park, addressing the unique challenges posed by the remote site conditions and strict performance criteria. The study predicted vertical and horizontal displacements through a combination of finite element modelling and hand calculations, ensuring that the pile foundations met the required stiffness and displacement limits. The use of five ground models, with the potential for the socket length to change due to rock hardness, based on all geotechnical investigations, allowed for accurate modelling despite the lack of site-specific data at some telescope locations.

The design methodology followed, ensures that the piles can support the telescopes without significant deformation, even under survival loading conditions. Limiting plastic deformation and preventing stiffness breakdown after survival loading, by reviewing the shear strain that develops along the shaft, was found to govern the design. The stiffness requirements and geotechnical capacity were generally met if the shear strain was appropriately limited.

The design methodology also emphasises the importance of collaboration between geotechnical and structural engineers to model soil-structure interaction problems accurately. The adaptable design approach ensures that the project can accommodate varying site conditions, minimise delays and facilitating successful implementation in this challenging environment.

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