Verifying long-term performance of stiffness-sensitive dish foundations

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ABSTRACT: The SKA Observatory (SKAO) is an intergovernmental organisation bringing together nations from around the world. It is one observatory, operating two telescopes, on three sites; a next-generation radio astronomy-driven big data facility that will revolutionise our understanding of the universe and the laws of fundamental physics. The SKAO's mid-frequency telescope, known as SKA-Mid, is being constructed in the Northern Cape of South Africa, with the core nestled between the towns of Carnarvon, Williston and Brandvlei and three spiral arms extending beyond. 133 parabolic radio dishes are being constructed which will ultimately integrate with the existing 64 dishes of the MeerKAT radio telescope, together forming the SKA-Mid array. The foundations for the SKA-Mid dishes are governed by strict client requirements relating to stiffness and movement tolerances. Verification of the designs was a continuous process throughout the phases of the project and spans from 2013 to 2024, encompassing the detailed design, procurement and now construction phases. This paper presents a holistic view on the methodology and findings of the verification process followed. It includes in-situ testing of the foundations, relating the results back to finite element modelling and the verification of the modelling approach adopted for the design.

1 INTRODUCTION

The design of the SKA-Mid dish foundations was governed by strict foundation stiffness and movement tolerances to ensure that the dishes are able to point accurately into space. One of the stringent requirements is compliance to the vertical, horizontal, torsional and rotational operational stiffness after a survival event has occurred. Due to the large project area footprint, stretching between the towns of Carnarvon, Williston and Brandvlei, varying geological profiles were encountered necessitating a robust methodology to effectively include each foundation's verification.

Based on the stiffness-strain relationship of soils, the soil stiffness will change with changes in resultant strains. If, in the instance of a survival event, strains in the soil exceed a certain threshold, soil stiffness degradation could occur, resulting in foundation stiffness and displacements not complying to the requirements. To mitigate stiffness degradation, the stresses induced on the soil adjacent to the foundation after a survival event are therefore needed to be kept in the elastic (small-strain) regime as discussed by Díaz-Rodríguez J.A. & López-Molina J.A. (2008), where no strength degradation occurs, and the soil behaviour is fully recoverable.

Finite element modelling with the aid of Plaxis 3D software was used in the design and verification processes of the project. These models were related back to a series of in-situ tests including full-scale pile load tests and a foundation prototype test. The loading set-up and sequences specified to the instrumented piles and prototype foundation were to assess the foundation response to different scenarios of operational and survival loading events as well as ground conditions.

2 REGIONAL GEOLOGY

According to the published 1:250 000 scale geological series of the area (3020 Sakrivier; 3022 Britstown and 3120 Williston), the site is underlain by a geological sequence ranging in age from the Permian (+ 300 Ma) to the recent Quaternary period. The Tierberg Formation, belonging to the Ecca Group of the Karoo Supergroup, form the oldest rocks in the project area and the alluvial deposits are the youngest. The Tierberg Formation comprises a predominantly argillaceous succession with the bulk being well laminated shales / mudstones. Calcareous concretions are common towards the upper formation. The Waterford Formation conformably overlies the Tierberg Formation and is limited to the south-eastern portion of the site. The formation comprises fine- to mediumgrained alternating sandstone, siltstone and shale (Johnson et al. 2006). Both the Tierberg and Waterford Formations are intruded by dykes and sills of Karoo dolerite. The Quaternary aged, red, aeolian sand forms dunes and patches of sandy areas while alluvial soils occur along streams, river channels and floodplains. With the installation of the foundations indicated in Figure 2, it was observed that the rock depths are rather uniform in the core area, with sudden changes in rock depth being the exception in localised spots. With the amount of drilling that took place in the core area the deeper rock sections could be delineated to form a line coinciding with a surface drainage channel and suggest the presence of paleo-river channels filled with deeper horizons of transported alluvial sands, gravels and, to a lesser degree, boulders. Rock depths across the site ranged from close to surface to around 15m depth.

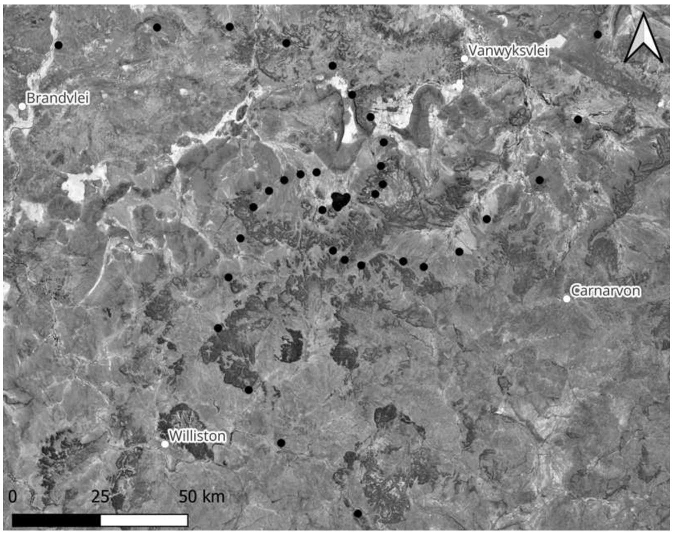

Figure 2. Area layout of the SKA-Mid telescope

3 VERIFICATION PROCEDURE

3.1 *Introduction*

The foundation design methodology included the creation of a total engineering geological model as proposed by Fookes et al. (2001, 2005) and was used as being flexible in the sense that as new relevant information was revealed, it could be added to the model. This involved the creation of a 3D geological model using Leapfrog (Bentley Software) (Fig. 2) which incorporated all the available information such as topographical surveys and available geotechnical data for the site. Using all the data and a total engineering geology approach gave an understanding of geological and geotechnical variability across the large expansive site and expected ground conditions at dish locations. This provided value especially at locations where drilling could not be undertaken due to constraints such as access.

Representative ground models were then established from the total engineering geological model used in Plaxis 3D, decreasing the number of design ground models to assess the foundations from 133 No. to 5 No. The design ground models were derived to represent conditions on site driving the design such as rock levels, overlying soil consistency, rock strength etc. Soil-structure finite-element analyses using Plaxis 3D software were conducted on the delineated ground models to assess stiffness and deflection compliance to the operational, survival and earthquake load cases. Verification of the finite element modelling, in-situ foundation behaviour and the potential of stiffness degradation and residual movements were done by a series of tests including a prototype test and full-scale pile load tests.

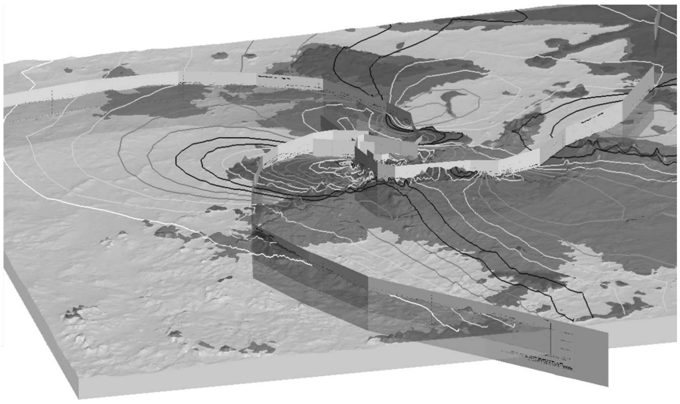

Figure 2. 3D geological model created with Leapfrog Works (Bentley) software

3.2 *Onsite verification of ground conditions*

The foundation design comprised of a piled foundation and a shallow raft foundation founded on rock. The piled foundation consisted of a 7m diameter pile cap with 8 No. 750mm or 1050mm piles. To achieve the stringent stiffness requirements the piled foundation design was driven by the ground model, applied stiffness parameters and in particularly the depth of rock. A minimum rock socket was required dependent on the rock depth and rock strength, and it was critical that this condition during installation was achieved.

The expected ground conditions, rock socket and rock strength per dish foundation were detailed on the construction drawings where the construction specification document included installation rules guiding decision making of rock socket lengths required for the different ground conditions encountered. The verification process during construction comprised of a geotechnical engineer on site supervising the drilling works. Rock depth levels were recorded, and the rock strength were verified by drilling rates and spoil. This data was then utilised to verify against expected conditions and against the construction rules to define the required rock socket for each dish.

3.3 *Prototype pull test*

To verify the modelling approach and tilting stiffness of the SKA-Mid dish foundations, a prototype piled foundation had been undertaken as a prior stage of the project which exerted a moment force as indicated in Figure 3. The loading sequence was staged in four cycles with the first three cycles exerting equivalent loads for performance verification whereas the last cycle's load was increased to simulate the effect of an earthquake event. The ground model associated with the prototype foundation was characterised by medium dense alluvial material in the upper 3.4m, underlain by partially calcretised alluvium up to 5m below ground level (b.g.l.), which in turn is underlain by mudstone/shale bedrock varying in strength from soft to very hard rock. The piles were terminated at a depth of between 11.8m and 12m which gave a rock socket length of 6.8m to 7m into mudstone/shale of the Tierberg Formation.

Figure 3. Prototype pull test setup

3.4 *Back-analysis of prototype foundation*

A back analysis of the prototype foundation was undertaken utilising the finite element software Plaxis 3D (Connect Edition Version 20.03.00.60) to model the soil-structure interaction and assess the foundation behaviour. This was applied to assess the resultant strains induced in the ground for the ground conditions and applied loading.

Two modelling approaches were assessed which included a Mohr-Coulomb model (linear elastic, perfectly plastic) and a Hardening soil model with small strain stiffness (HS small). The Mohr Coulomb model approach is an iterative approach whereby the soil strains in the output are used in conjunction with the modulus reduction curve for gravely materials by Rollins et al. (1998) to iterate the material stiffness relative to induced strain until convergence is achieved. The hardening soil model with small-strain stiffness (HS small) automatically iterates the soil stiffness based on the induced strains with the addition of stress-dependent stiffness.

3.5 *Full-scale pile load testing*

Implementation of the full-scale pile load tests took place in 2013, 2016, 2021 and 2024 which included a total of 7 No. 750mm diameter augured piles as well as 1 No. 1050mm diameter augured pile. Pile head displacement in the test piles (TP) was monitored with load variance displacement transducers (LVDT), whereas test piles installed in 2021 and 2024 (TP6, TP7 and TP8) had in addition strain gauges down the length of the pile as well as inclinometers fitted on the compression side of the pile. These sets of results were cross referenced to validate the design and used as a contingency measure when anomalies are recorded with measurements from certain instruments. Figure 4 shows the pile load test setup as conducted in September 2024.

As the test piles were installed near operating radio telescopes, there were restrictions on equipment and instruments that could be utilised due to radio frequency interference requirements. This influenced the test setup design.

Figure 4. Pile load test setup, 2024

Table 1 summarises the installation details of the pile load tests with their corresponding rock sockets.

Table 1. Summary of pile load testing installation details

Test Pile	Type of test	Date under-taken	Pile length below NGL (m)	Pile diameter (mm)	Socket length in SR mudstone (m)	Socket length in MHR mudstone (m)
TP1	C	2013	8.7	750	2.6	4.7
TP2	C	2013	8.7	750	4.7	1.7
	H	2016				
TP3	C	2013	10.2	750	7.7	2.3
TP4	C	2013	6.5	750	-	0.4
	H	2016				
TP5	C	2013	11.2	750	-	0.6
TP6	C&H	2021	9.6	750	-	-
TP7	C&H	2024	10.35	750	3.5	-
TP8	C&H	2024	10.3	1050	1.1	-

Notes: C = Compression, H = Horizontal, SR = soft rock 3-10 MPa, MHR = medium hard rock 10-25MPa.

3.6 *Back-analysis of the pile load tests*

Finite element back-analyses in Plaxis 3D were performed on pile load tests TP6, TP7 and TP8 to verify the stiffness performance and in particular the residual stiffness performance of the piles after extreme loading events occurred. Compliance was achieved by proving that the finite element model with the design ground models either gave accurate or conservative soil strain, stiffness and deflection results when compared to the results from the pile load tests.

4 DISCUSSION

4.1 *Prototype pull test*

The tilt results of the prototype piled foundation measured in arcsecond is presented in Figure 5 and the testing configuration in Figure 3. The testing included three operational loading events and a survival event. The foundation rotations reverted back to the initial value after all four loading and unloading cycles, proving that at such small rotations the founding material was still within the elastic (small-strain) regime and hence it was inferred that no measurable residual deformation occurred. The reason for the higher rotations observed in the first loading application compared to the second loading application is attributed to gusty wind conditions. It was observed that the rotation started to decrease prior to the unloading event coinciding with the die down of the wind conditions.

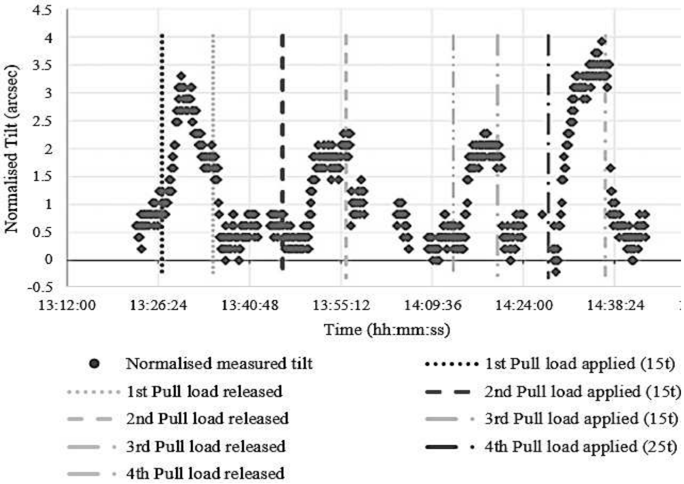

Figure 5. Results of the SKA prototype foundation tilt test

To aid in assessing the modelling approach for the design of dish foundations, three scenarios were assessed. Scenario A is characterised by the Mohr-Coulomb approach without modelling a gap between the pile head and ground level whereby Scenario B relates to the HS small model without a gap, refer to Figure 6. Scenario C assessed the HS small model with a gap ensuring that the applied loading is only transferred to the piles and no bearing resistance is mobilised from the pile cap. The actual tilting rotation measured from the prototype test compared best with the model outcome of Scenario A, with the measured prototype rotations being the lesser. Scenario A doesn’t account for the lateral resistance of the ground from the pile cap in the prototype test and it could be argued that it adds a degree of conservatism to the modelling. Between the three scenarios the HS model with a gap had the lowest factor of compliance (FoC) and thus Scenario C was selected as the preferred approach for the design of the dish foundations. The FoC checks refers to compliance to the client’s foundation stiffness requirements. This includes for example compliance to the vertical, horizontal, torsional and rotational operational stiffness after a survival event has occurred.

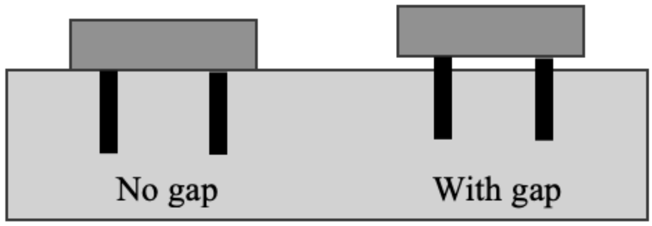

Figure 6. Scenario of 'no gap' and 'with gap'

The results from the back analysis were applied as a verification of: firstly, the design approach used utilising Plaxis 3D with a Hardening soil model with small strain stiffness (HS Small); and secondly of the design and behaviour of the foundation. This was undertaken by comparing the induced shear strain of the prototype under the moment loading in the back analysis to the induced shear strain in the ground under the same loading conditions in the design models. If the strain ranges are similar in the design to the prototype, the residual response will be similar and in all probability result in very small or negligible residual movement. On this basis the design was deemed to comply to the allowable residual tilt criteria. This was only relative to similar ground conditions.

4.2 *Pile load tests*

The 8 No. pile load tests included horizontal and compression tests on augured piles with installation details as presented in Table 1. Tests TP1 to TP5 assessed only pile head displacement whereas TP6 to TP8 included inclinometers and strain gauges down the length of the instrumented pile. Compression tests were undertaken on all the piles whereas the horizontal loading tests were either done at a later stage or only on certain piles. All the test piles were installed with a rock socket, except for TP6 which was terminated on bedrock. Care should be taken in comparing the pile test data with one another as the installation details and loadings differ.

4.2.1 *Discussion on TP1 – TP5*

The test results from the maximum load versus pile head displacement are tabulated in Table 2. The loading was staged in specific sequence to assess the effect of loading and unloading cycles on the stiffness behaviour of the pile.

In all the compression tests conducted, the vertical pile stiffness increased in the second load cycle (and third load cycle in TP2). This suggests that in compression the pile load displacement curve is on the unload-reload rebound curve which gives a stiffer response after the first load cycle.

Table 2. Test results of pile load tests TP1 to TP5

Test pile	Load / Unload cycle	Type of test			
		Compression (2013)		Horizontal (2016)	
		Final cycle load (kN)	Max. displacement (mm)	Final cycle load (kN)	Max. displacement (mm)
TP1	1	1658	1.1	-	-
	2	0	0.16	-	-
	3	2487	1.38	-	-
	4	0	0.41	-	-
TP2	1	1658	0.82	100.6	0.28
	2	0	0.27	0	0.05
	3	2487	1.53	151.4	0.64
	4	0	1.14	0	0.07
	5	4151	2.84	-	-
	6	0	1.55	-	-
TP3	1	1658	0.47	-	-
	2	0	0.01	-	-
	3	2487	0.63	-	-
	4	0	0.28	-	-
TP4	1	1658	0.47	100.6	0.27
	2	0	0.01	0	0.01
	3	2487	0.63	151.4	0.69
	4	0	0.28	0	0.07
TP5	1	1658	0.67	-	-
	2	0	0.4	-	-
	3	2487	1.18	-	-
	4	0	0.63	-	-

Notes: Load cycles 1,3 and 5 are loading cycles and load cycles 2,4 and 6 are unloading cycles.

The horizontal tests were conducted in approximately 25kN load intervals up to the maximum loads indicated in Table 2. To assess the residual tilt deformation of the foundation, the pile response from the horizontal loading was assessed; it should be noted that the test piles have a “free” head whereas the heads of the piles in the foundation will be “fixed” to the pile cap providing additional horizontal stiffness. It was observed that in both TP2 and TP4 the horizontal pile spring stiffness remained approximately similar in the second loading cycle compared to the first loading cycle. To assess the residual deformation after a loading event, the horizontal displacement was measured directly after reaching 0kN in the unloading phase as well as 12h thereafter. In both cases the residual displacement was less at 12h than directly after the unloading event indicating the time dependent rebound stiffness response of the pile. The residual displacements of TP2 and TP4 at 12h was 10.9% and 1.4% respectively.

4.2.2 *Discussion on TP6 – TP8*

To specifically assess the effect of extreme loading events on the operational stiffness, the loading sequence was staged in 4 No. cycles, including an operational loading followed by two independent survival events followed by a second operational loading as presented in Table 3. The first operation loading was then compared with load cycle four’s operational loading to assess the impact of the extreme loading events. In TP 7 and TP8 identical loading procedures were followed as presented in Table 3, with the only difference being the increase in the operational loading from 300kN to 350kN.

Table 3. Test results of pile load test TP6

Test pile	Load cycle	Type of test			
		Compression		Horizontal	
		Max. load (kN)	Max. displacement (mm)	Max. load (kN)	Max. displacement (mm)
TP6	1	300	0.23	-	-
	2	-	-	100	1.37
	3	800	0.34	-	-
	4	300	0.18	-	-

Based on the load displacement curves from the compression test’s LVDT data, the vertical pile stiffness increased in TP6 after being exposed to the extreme loading events. This correlates with the findings from TP1 to TP5 discussed in Section 4.2.1. The LVDT data from TP7 and TP8, however, gave questionable results, atypical of the expected linear or hyperbolic nature of a load-settlement curve and possible seating errors were therefore noted. Emphasis was put on the strain measurements down the length of the pile and compared with the adjusted LVDT displacement to verify that the client’s operational stiffness requirements after the survival events are still met. The inclinometer results of TP6 were not assessed due to erroneous readings, however it proved to be sufficiently accurate in TP7 and TP8 where it was used in verifying elastic displacement conditions of a singular laterally loaded pile.

5 CONCLUSIONS

The design of the SKA-Mid dish foundations was governed by strict client requirements relating to foundation stiffness and movement tolerances. Verification of these requirements was done through a series of in-situ tests, including a prototype test and 8 No. pile load tests between 2013 and 2024. The results from the in-situ tests were related back to finite element modelling using Plaxis 3D and aided in the selection of an appropriate modelling approach for the design of the foundations as well as verification of residual performance of the foundation. One of the stringent requirements was compliance to the vertical, horizontal, torsional and rotational operational stiffness after a survival event has occurred ensuring no stiffness degradation occurs after an extreme loading event. This was achieved by staged load tests on instrumented piles assessing the difference in pile load response before and after survival loadings were exerted on the pile.

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