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Lateral support for upmarket Lynnwood Junction project

INTRODUCTION

Lynnwood Junction is a multi-million rand investment which comprises an upmarket shopping complex, office park and hotel. The new development is located on the north-eastern side of the Lynnwood Road offramp off the N1 highway in Pretoria. Many challenges were encountered during the design and construction of this development. These challenges included problems posed by the construction of the new Lynnwood Road interchange, as well as Meiring Naude Road running along the western side of the site. This project required an excavation of about 12,5 m from original ground level in some sections of the site. In addition to this, Meiring Naude Road had to be raised, which required the construction of a SANRAL retaining wall along this section of the excavation. Figure 1 shows the site during the excavation phase of the project.

1 The site during the excavation phase

LATERAL SUPPORT

The depth of the excavation ranged from 12,5 m in the south-western corner to 7,6 m in the north-western corner of the site. Both the western and southern faces required geotechnical input to ensure that slope failure did not occur. For the southern slope there were very few constraints, as far as available space was concerned, and therefore it was possible to batter the slopes back to a safe slope angle. A slope angle of 60˚ was used to ensure that a factor of safety of approximately 1.3 for a temporary slope was achieved.

Battering of the western slope was not possible due to the position of the SANRAL retaining wall and the lack of space at the bottom of the excavation. The SANRAL retaining wall was located some 2,5 m from the face of the excavation in some sections and imposed loads of between 201 and 136 kPa. A further problem was the depth at which the retaining wall was founded below the top of the excavation (approximately 2 m). This meant that no (or very short) soil nails could be used in the first 2 m of the excavation. Figure 2 shows a typical section that was analysed.

The shear strength parameters of the in situ materials were back-calculated from information obtained during the initial field investigation and then used in the analysis of the sections. The problem was analysed using both Prokon's RockPF and Rocscience Phase2 finite element software. The following parameters were used in the analyses:

- $c = 18$ kPa
- $\phi = 25˚$
- $\gamma = 17$ kN/m^3

Critical sections of the slope were identified and analysed taking into consideration the magnitude and position of the loads applied by the SANRAL retaining wall. The force required to prevent a failure occurring was compared with the maximum force that could be obtained with the use of Y25 soil nails. The force

2 Typical cross section
3 Schematic drawing of anchor
4 Phase2 finite element model
5 Output of finite element model

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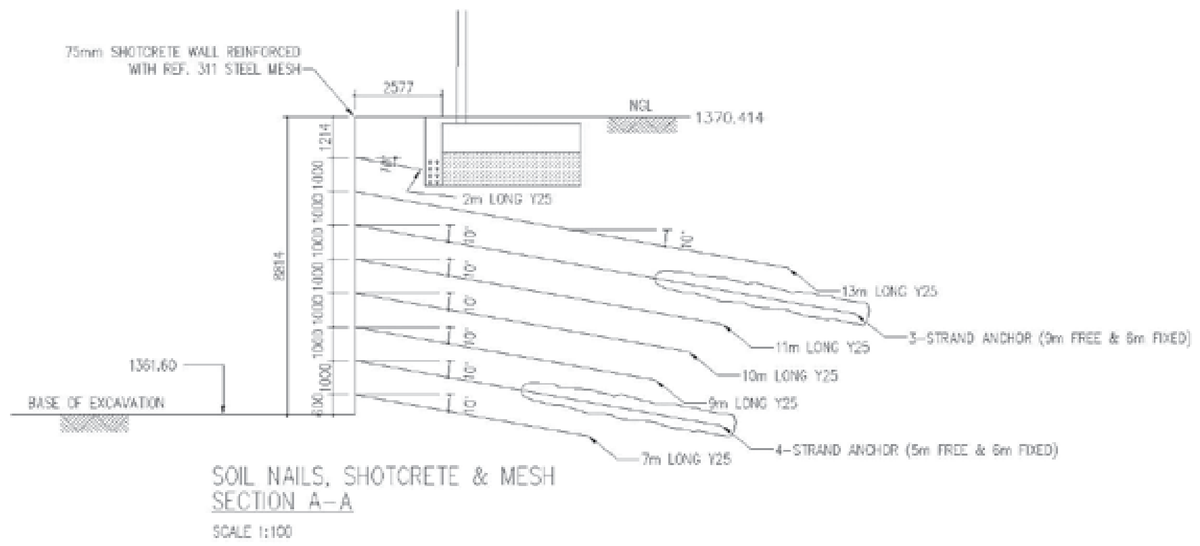

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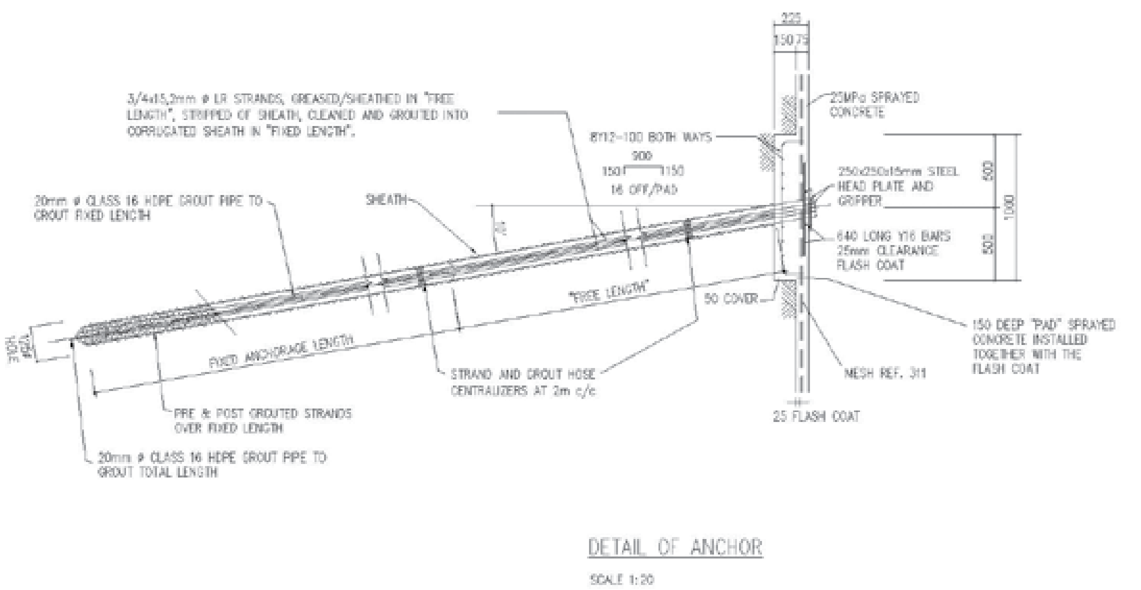
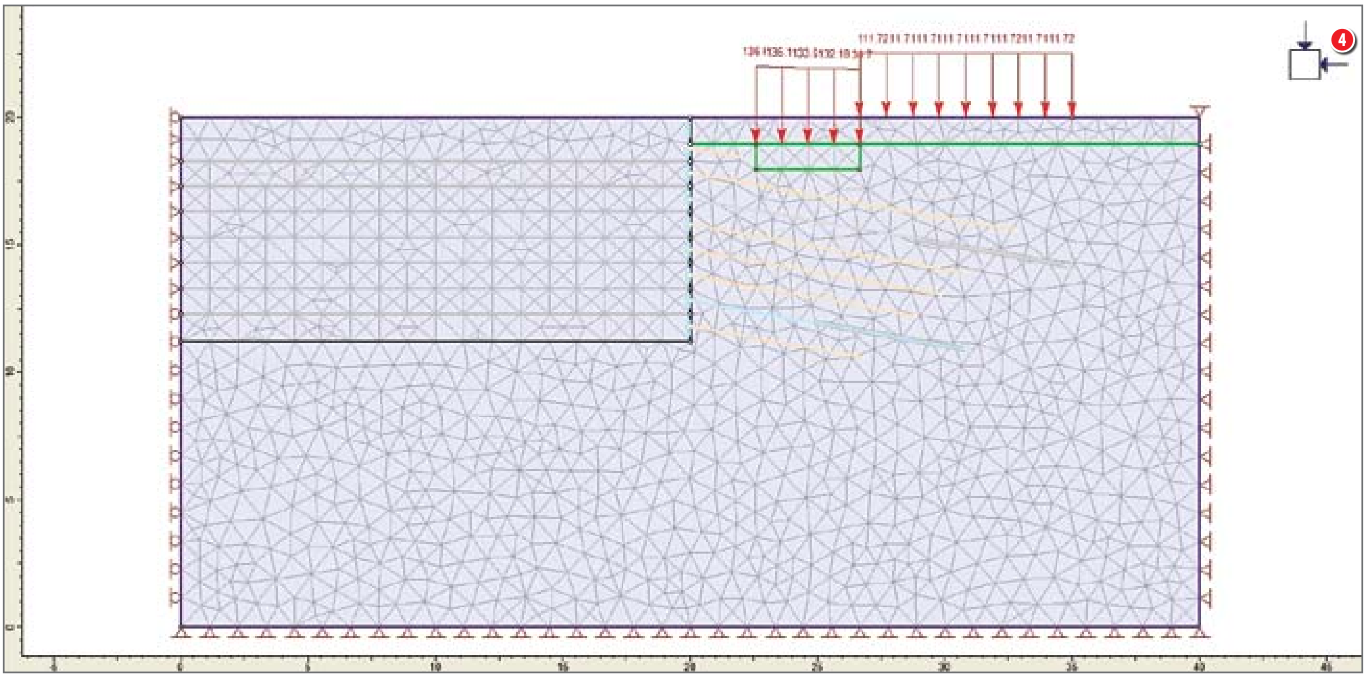
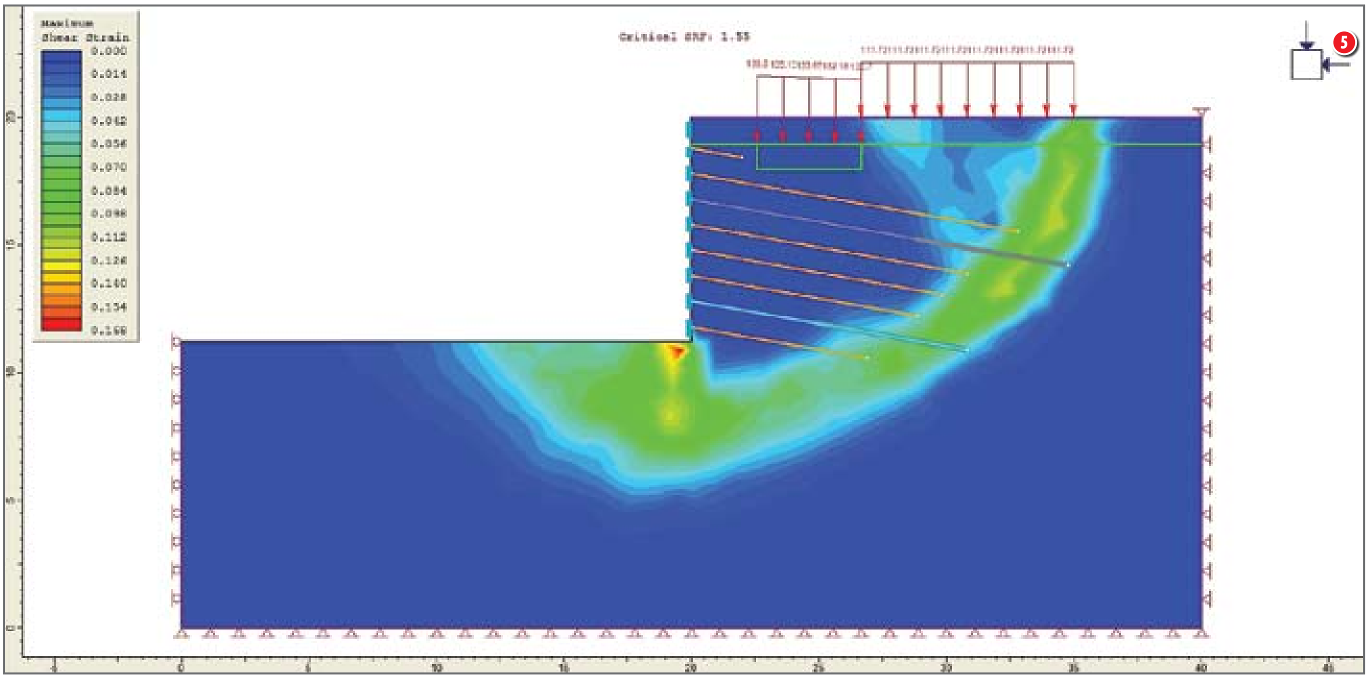


required was found to be greater than the force that could be generated by Y25 soil nails at 1,5 m centres, and thus, to compensate, the spacing was reduced to 1,0 m. Even with the reduced vertical spacing, some sections still required marginally more resisting force than could be generated by Y25 soil nails. The use of multi-strand anchors was therefore required. In the sections where excessively high forces were needed, some of the rows of soil nails were replaced with multi-strand anchors. The anchors comprise fixed and free length – the fixed length is found at the end of the anchor and is grouted into the soil/rock such that tension in the strands can be transferred into the surrounding material, while the free length of the anchor is not grouted, which enables the tension of the anchors to be adjusted as required after installation. For this design, both three and four strand anchors were used. Each strand is made up of a 15,2 mm diameter, low-relaxation 7-wire strand, and each strand can be tensioned individually to the required tension up to a maximum of 175 kN. Figure 3 shows a schematic drawing of the anchor.

The results of the Prokon analysis revealed that a minimum force of 1 105 kN was required to generate an acceptable factor of safety of at least 1.3. Figure 4 shows a model that was analysed in the Phase2 finite element analysis program. Phase2 calculates a strength reduction factor which is the equivalent of a factor of safety. Figure 5 shows the results of the finite element analysis of a critical section which had been analysed. This layout generates a maximum resisting force of 1 865 kN and generates a predicted factor of safety of 1.55.

An advantage of a finite element model is its ability to predict movements that will occur. Not only were all sections checked for their respective factors of safety, but also for the expected horizontal movements. The finite element model in Phase2 predicted a maximum horizontal movement of 33,5 mm. The western wall of the lateral support was surveyed on a bi-weekly basis and the results were recorded. The maximum recorded movement that occurred on the western face of the excavation was





32 mm. Figure 6 shows the displacements obtained in the finite element model.

CONCLUSION

Both limiting equilibrium and finite element analyses were used in the design of this challenging project. Appropriate resisting forces using soil nails and a combination of soil nails and anchors, appear to have been correctly assigned, with monitoring results indicating near-perfect fit with predicted values.

ACKNOWLEDGEMENTS

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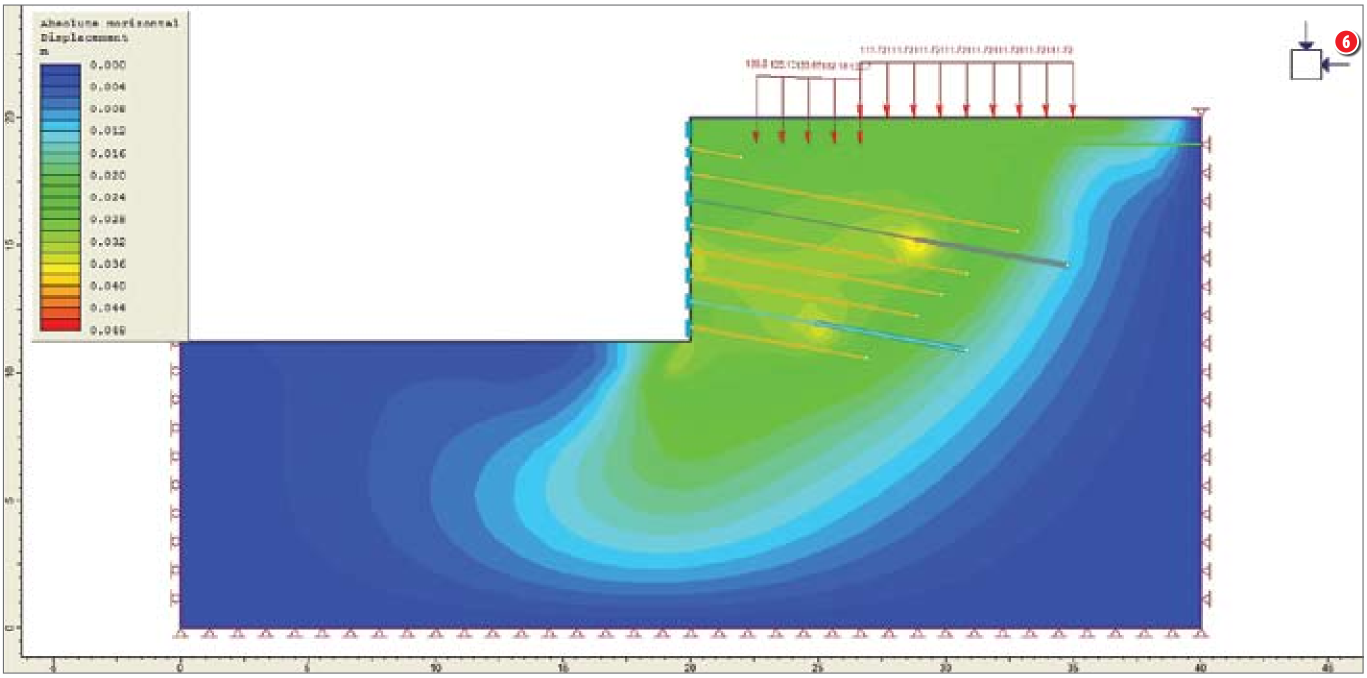
6 Displacements calculated in Phase2



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