Unsaturated modelling, testing and monitoring towards the rehabilitation of a slope on National Route 3, KwaZulu-Natal, South Africa

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Abstract. The stability of over-steep road embankments often worsens with time. In many cases the slope deterioration can be ascribed to continual downslope creep of a surficial layer of soil that has been affected by infiltration of rainfall. Such infiltration reduces the suction in the surficial layer interacting with the environment. With every season the surficial layer moves further downslope, often resulting in cracks in the slow lane and placing structures constructed at the crest of fill slopes in potential jeopardy. A slope on the National Route 3 (N3), in the South African Province of KwaZulu-Natal, constructed at roughly 40° from the horizontal showed signs of distress in the slow lane many years after construction. The rehabilitation of this slope required stabilisation design using unsaturated soil testing, modelling, and in-situ suction measurement. The slope was stabilised using soil nails, a high-tensile steel mesh, and a suction measurement system was installed. An overview of the emergency works design, subsequent investigation, and monitoring is provided in this paper. It was found that the suction profile determined by transient finite element analysis matched well with the measured suction profile and provided a good basis for the emergency design works prior to full investigation.

1 Introduction

The National Route 3 (N3) is the busiest long-distance freight route in South Africa and a critical link between port infrastructure in the coastal city of Durban, and Gauteng, the economic hub of the country. It follows that the health of slopes on the route are assessed on a continual basis to prevent major socio-economic impacts that would occur if slope problems caused long-term lane closures.

The slope detailed in the current paper is 18m high with an angle of 40° and was earmarked for annual supervision along with some adjacent sections where stabilisation interventions had already been carried out. During the 2018 annual slope inspection, critical slopes were inspected along N3 between Durban and Johannesburg (Figure 1), it was noted that cracks developed in the shoulder and slow lane along a slope travelling towards Durban, north of Pietermaritzburg, near the town of Howick.

The cracks and edge settlement were progressing further by time of inspection and emergency stabilisation works were required before any subsurface investigation or soil testing could be undertaken. As part of these emergency works analysis was done to understand the underlying reason a previously stable section of slope now experienced problems.

The cracks were predominantly parallel to the slope's crest and tapered down at both ends. The road surfacing had settled between the crest and the cracks, indicating the edge was displacing relative to the remainder of the road. It is considered that this cracking is an example of translational slope failure movement due to creep.

Downslope creep movement occurs in slopes placed close to their angle of repose (i.e. FoS = 1.0 for a completely dry condition). This creep normally occurs during rainfall events as the effective stress reduces at the wetting front near the crest of the slope. This reduces the forces counteracting translational slope failure through frictional resistance. Additionally, any capillary cohesion that is present will be broken down upon saturation. A surficial layer of the slope, over which rainfall will infiltrate, will therefore be prone to continual downslope creep during successive intense rainfall events.

Once enough surficial creep movement has occurred, a tension crack will develop at the crest. Waterfilled tension cracks will mobilise slope creep deeper and result in progressive failure more rapidly than otherwise possible. These tension cracks will eventually result in linear cracks developing in the black top (roadway).

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Fig. 1: Location of slope movement on National Route 3.

Some of the fills along the route were constructed using shale and residual shale materials that are prone to slaking upon wetting and drying and associated disintegration. This fill material may have reduced the shear strength properties of the fill with time resulting in reduced stability.

In the 1980's consultants working on another section of the route undertook shear box tests on recompacted fill materials, at OMC +/-2%, to calculate an apparent cohesion. These apparent shear parameters were then used in slope stability analysis to establish the safety factor. It is unclear if this slope was designed using similar assumptions. This approach would however, result in un-conservatively low Factors of Safety (FoS) if the degree of rainfall infiltration, water table fluctuation, or drying of the edge are not well understood.

2 Climate

The embankment is located in a subtropical area of South Africa characterised by wet summers (average 174 mm/month) and dry winters (average 15 mm/month), with overall mean temperatures of 20°C in winter and 24°C in summer [1].
The site is located in a water surplus area [2] located on the edge of an area demarcated with a Thorntwaite Moisture Index (TMI) greater than 20 and an area with TMI of 0 -20. The depth of design suction change in such area is estimated as 3.0m as given in AS2870 [3].

3 Emergency Works Design

As the slope required emergency stabilisation, no subsurface investigation could be undertaken before stabilisation works commenced. Some information was however available from the nearby slopes that were stabilised between 2012 and 2018. However, no unsaturated soil tests were undertaken at that time.

The fill material was assumed to classify as a silt of low plasticity (ML) in accordance with the Unified Soil Classification system (USCS) with a assumed liquid limit <30%. A Swiss code was used to derive anticipated effective shear parameters [4]. The assumed shear parameters used were $\phi' = 30°$ and $c' = 0$ kPa at an 84% confidence index.

Rocscience's Slide 2D was used to model the 18m high slope with a water table at 17m below the crest of the slope. The simple unsaturated hydraulic permeability function for a silt (provided in Slide 2D) was chosen to simulate the permeability over a range of suctions. An air-entry value (AEV) of 30 kPa was assumed and SWRC derived using the Zapata estimation method [5], based on the assumed grading. Using the Fredlund and Rahardjo method for increased unsaturated shear strength [6], a value of 15° was conservatively chosen for the unsaturated shear strength parameter (ϕ'_b). The ϕ'_b parameter is assumed lower than ϕ' when the suction exceeds the AEV, based on the relatively low AEV, a suitably low ϕ'_b was chosen.

A steady state analysis was undertaken using Slide 2D with negative pore-water pressures developing from the groundwater table up. A rainfall event of 400mm over four days, simulating a rainfall event similar to the rainfall that occurred during Domoina tropical storm in KwaZulu Natal during 1984 was used in a transient analysis, assuming zero net flux at the surface before such event. The initial suctions at the crest should be relatively low, as the slope is in a water surplus area and some water infiltration would have already taken place.

The slope had a FoS of 1.3 prior to the simulated rainfall event due to increased strength from matric suction. After the fourth day of rainfall the FoS over the crest reduced to below unity for all the failure planes showed in Figure 2. The negative pore pressures varied between 0 and -30kPa over the entire depth above the groundwater table. The model showed that the rainfall event had weakened the surficial layers which had previously been stable in their original conditions. While no such exceptional rainfall event occurred before the slope in question experienced translational failure, the net rainfall surplus area the slope is located in means that successive rainfall events will cause water to accumulate in the surficial areas of the slope without enough evaporation to remove infiltrated water. Over time this will lead to the same low suction conditions found by the model.

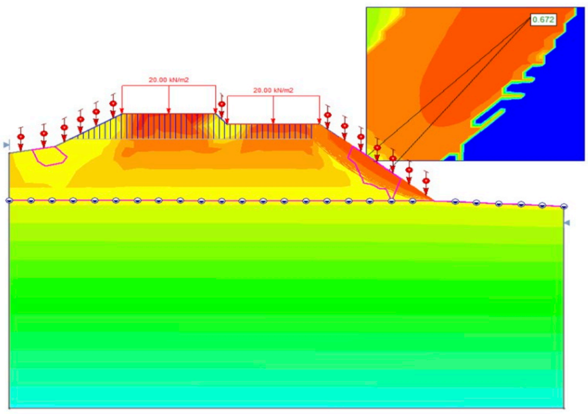

Fig. 2: Transient slope stability analysis simulating rainfall infiltration

The inclusion of passive soil nails to intercept the failure plane enhanced the sliding resistance. The model was therefore adjusted to include for passive soil nails until an FoS>1.5 was achieved after the simulated rainfall event. This is shown in Figure 3 below.

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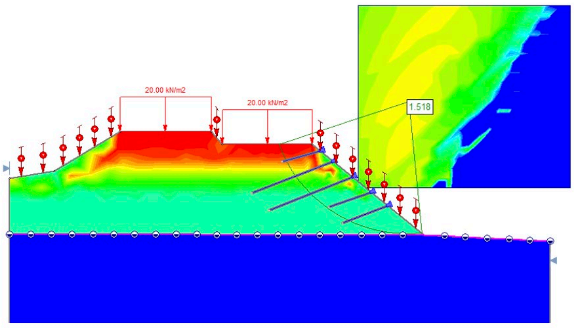

Fig. 3: Transient slope stability analysis simulating rainfall infiltration with slope stabilisation measures

The model was then transferred into Rocscience's two-dimensional finite element (FE) suite to review structural forces in both the soil nails and the high tensile steel mesh that transferred load between adjacent soil nails.

4 Implemented Stabilisation Works

As a result of the transient slope stability analysis three rows of conventional soil nails, and two rows of self-drilling anchors were installed. Sub-horizontal drains were included at the toe of the slope to provide dewatering, and tension cracks at the crest were sealed with an impermeable clayey infill to prevent further ingress of water. A schematic of the stabilisation works is presented in Figure 4.

Ground investigation was undertaken during the installation process to verify design assumptions. In-situ suction measurements were taken after the main installation over a period of 5 months using suction sensors at various depths until moisture damaged the data logger [7].

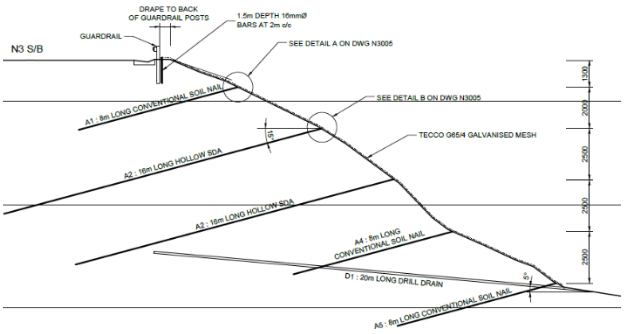

Fig. 4: Cross-section of implemented stabilisation works

5 Subsurface Investigation

The subsurface investigation for this slope consisted of a rotary-core borehole drilled to 22m. The water table was intercepted at 15m below the crest of the slope. The subsurface profile is detailed in Table 1.

Table 1. Subsurface profile

Depth (m)	Soil Description
0 - 2.2	Clayey Silt, Fill
2.2 – 4.5	Clayey Sand, Fill
4.5 – 16.0	Silty Clay, Fill
16.0 – 22.0+	Soft to Medium Hard Rock, Dolerite

6 Soil Testing

The fill soils tested classified as clays and silts of low plasticity (CL and ML) and silt of high plasticity (MH) with plasticity index (PI) ranging from 6 - 19% with a low potential expansiveness. A triaxial test was performed on soil retrieved from 10 - 10.5m showed the clayey soil to have effective shear properties of φ' =28° and c' = 24 kPa.

A saturated shear box test showed the same effective friction but with an effective cohesion of 38 kPa. The saturated shear box test is notorious for higher cohesion values due to difficulty in preventing development of excess pore-water pressure during shearing. For this reason, the triaxial effective cohesion result was preferred. The effective cohesion of 24 kPa from the triaxial test is however still high when compared to the 0 kPa originally assumed.

Unsaturated shear box tests were also undertaken at quick shear rates of 1 mm/min, in accordance with guidance provided in [8], to minimize changes in moisture content during shearing of the sample, at various degrees of saturation.

The derived capillary cohesion values (effective cohesion subtracted) are provided in Figure 5.

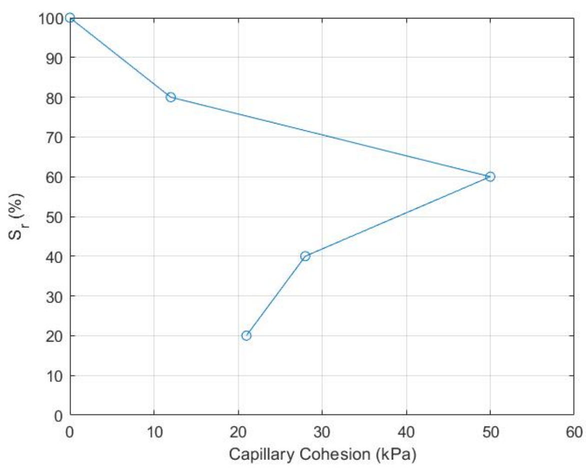

Fig. 5: Capillary cohesion from unsaturated shearbox test

Two samples retrieved by Shelby sampling tube were sent to the University of Pretoria for soil-water retention curves (SWRC) measurement in accordance with the test procedure detailed in [9]. These samples were retrieved between 4.95 and 5.45m, in silty clayey fill material. The results of these SWRC, after curve fitting using the Fredlund and Xing method [10], are provided in Figure 6.

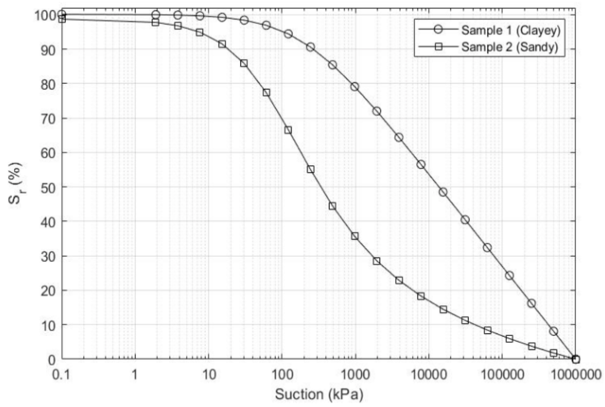

Fig. 6: SWRC's between 4.95 – 5.45m [7]

The difference in the two measured curves is likely due to the fact that the Shelby sample was retrieved at an interface between two different grades of fill (clayey sand above and silty clay below). The two samples tested were taken from the top and bottom of the Shelby sample respectively.

7 Review of test results

The capillary cohesion values determined from the unsaturated shear box testing is compared to the predicted capillary cohesion [$(u_a - u_w)\tan\phi_b$] from the Fredlund & Rahardjo method. The predicted values are based on suctions taken from the Sample 2 SWRC in Figure 6. The results are presented in Table 2. Here it can be seen that the measured and predicted capillary cohesion values compare favourably, except when suction values exceed 200 kPa. The modelled pore pressures are typically <140 kPa and therefore the estimation is considered suitable over the expected suction ranges.

Table 2. Comparison of measured and predicted capillary cohesion

Suction (kPa)	Estimated Capillary Cohesion (kPa)	Measured Capillary Cohesion (kPa)
50	13	11
200	48	50
600	160	28

Prior experience with capillary cohesion methods have shown that overestimations of capillary cohesion become more likely in the higher suction ranges, and as a result should be used in these ranges only when both the method and soil in question is well understood by the practitioner.

The effective friction angle assumed from the triaxial test results are similar to those measured, although the effective cohesion greater than initially (conservatively) assumed. If further infiltration of rainfall into the slope occurs and the existing suctions are reduced further, the loss of capillary cohesion lower than that modelled in the FE model will be offset by the in-situ effective cohesion that was not initially used in the model but only later confirmed by testing.

From a parameter point of view the analysis undertaken is therefore considered to err on the conservative side, which is considered suitable for emergency works that had to be undertaken prior to full investigation.

8 Review of in-situ measurements

The suction values measured in-situ are provided in Figure 7 alongside the predicted suction profile from Slide 2D's transient FE model (after four days and 400mm rainfall). The measured suction data was retrieved over the period 1 October 2019 to 1 January 2020. It should be noted that this slope is located in a summer rainfall area, October therefore falls under the drier part of the year, while January is well into the rainy season.

The estimated suction profiles match the measured data well, validating the use of the transient FE model in the emergency works design. In terms of infiltration influence with depth, the following observations can be made from Figure 7:

- The measured data shows seasonal suction fluctuations were mostly limited to the top 3m of profile, with some minor changes down to 6m.
- Minor suctions (<20 kPa) are present over the top 6m of the profile. This differs from the AS2870 [3] design suction depth of 3m.
- The net rainfall surplus has influenced suctions down to approximately 10m below the crest, after which the suctions from the water table's capillary zone become dominant.

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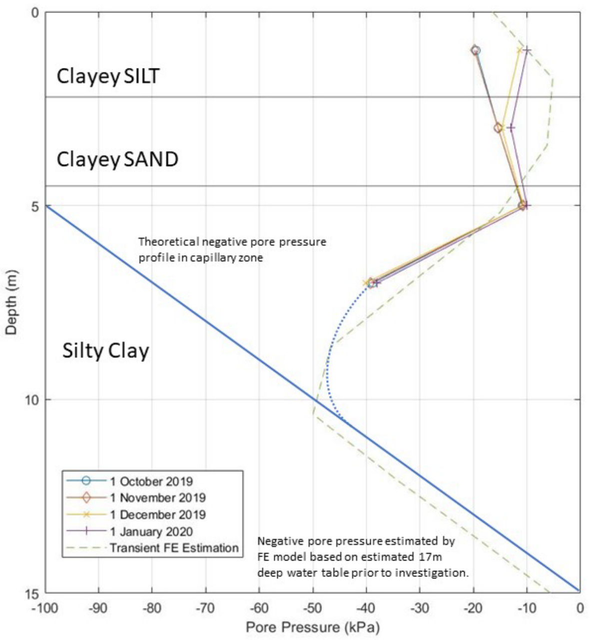

Fig. 7: Measured suction profile with depth against profile estimated by FE transient analysis (adapted from [7])

It should be noted that 2019/2020 were moderately wet years, it is assumed that the suction over the top 6 to 7m can reduce even further during a more extreme rainfall event. It is also clear that the net surplus rainfall has in the past already influenced the slope's negative pore pressure regime over the upper crust, as the existing low suctions reflect substantial water infiltration down to the 4.5m interface with the clayey material.

With the information above, it is worth returning to the question of why the slope experienced the shallow failure it did after many years of stability. The historic stability is believed to be as a result of higher suctions 'built-in' to the slope at construction due to the Optimum Moisture Content for compaction being between 60% < S_r < 80%. This range encompasses some of the peak capillary cohesion values as shown by the unsaturated shear box results in Figure 5. The net rainfall surplus environment slowly eroded the suctions present in the slope from the crest down, until the reduced shear strength near saturation was insufficient to support the steep slope angle and a shallow translational failure resulted.

If capillary cohesion reduces with further infiltration, the stabilising effect of the effective cohesion found in the triaxial test found after the initial emergency works design will maintain a FoS in its current state.

Any future deeper rainfall infiltration will reduce the suctions and shed load to the soil nails and result in minor further strain of the passive system. It is uncertain how likely the deeper infiltration is, due to the clay layer at 4.5m depth having a lower permeability. This lower permeability decreases the likelihood of water infiltrating deeper and is reflected in the increase in suctions below 4.5m.

9 Conclusions and Recommendations

A slope on the National Route 3 in South Africa had shown signs of distress in the slow lane. Slope stability analyses were undertaken using transient groundwater FE modelling and associated shear strength increase due to suction. A slope stabilisation measure using passive soil nails and high-tensile steel mesh were detailed to stabilise the slope as an emergency intervention.

The investigation, testing and in-situ suction measurements closely matches that derived by the analyses and estimation procedures. It would however be advantageous to reinstate damaged instrumentation to measure the suction profile and observe long term trends. No further slope or road cracking has been noted on this slope after the stabilisation works. The primary conclusions may be summarised as follows:

1. Despite the unfavourable geometry (slope angle and height), the slope was stable until recently likely due to suctions 'built-in' at construction. The net surplus rainfall in the area has over time degraded the suctions originally present near the crest which led to a shallow translational slope failure.
2. The stabilisation design implemented is safe even if remaining suctions in the upper layers were to disappear with further water ingress. This is due to the effective cohesion of the fill material being assumed as 0 kPa in the emergency works design, with later testing showing greater effective cohesion values which will offset possible future suction based capillary cohesion losses.
3. Capillary cohesion estimates used in the design compared well with capillary cohesions measured in unsaturated shear box testing in the ≤ 200kPa range. In the higher suction range measured values were far below those estimated. The estimation procedures are very valuable to a design engineer, but a user must be cognisant of what the method in question is sensitive to, as overestimations of capillary cohesion at higher suctions may result. Further work is required to evaluate the sensitivity of common capillary cohesion estimation procedures and develop protocols for their use.

10 Acknowledgement

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