Causes and remediation of slope failures on the N2 national route

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ABSTRACT: In April 2022, KwaZulu Natal in South Africa experienced extreme periods of heavy rainfall, resulting in catastrophic flooding and damage to infrastructure across the province. Three slope failures occurred on a section of a key national route. This paper presents a study into the cause of the slope failures. The underlying ground conditions and water regime of the site were complex and influenced the resultant failure mechanisms observed. The failures differed in extent, depth and form. A back analysis was undertaken of each failure utilising available geotechnical information, a drone survey and a 3D geological model created to understand the variability of ground across the greater site. The understanding of the factors that influenced the failures observed at the site and their extent was important in designing robust remediation solutions and assessing stability of existing slopes. Slope failures are becoming more prevalent with the impact of climate change. Understanding the factors impacting slope stability allows for the development of resilient solutions.

1 INTRODUCTION

In April 2022, KwaZulu Natal in South Africa experienced extreme periods of heavy and prolonged rainfall, resulting in catastrophic flooding and damage to infrastructure across the province. Zutari had been appointed in 2021 for settlement repairs to a section of the N2 national route where the slope had been creeping for a number of years. However, in April 2022 following heavy rainfall, severe cracking was observed on this stretch of road. An instruction was issued to close the road section and a slope failure occurred later that night. Upon inspection, major cracking and slope failure was also observed on another section of the road a short distance away and an additional failure was identified further north within close proximity.

The northbound carriageway was severely impacted, affecting this important national route. The scope of works comprised the investigation of the cause of the embankment slip failures, as well as the design solution and construction supervision required for the rehabilitation of the road sections.

The underlying ground conditions and water regime of the site were complex and influenced the failure mechanisms observed. A drone survey of the site was undertaken and a 3D geological model was created using Leapfrog (Bentley software) to assist in understanding the topography and ground conditions across the site. This ensured a robust and resilient solution was developed for the road reconstruction. This paper presents findings of the back analysis of the failures, the considerations applied and how the geology influenced the development of each failure zone.

Various remediation options that were considered are presented along with the final solution, comprising of a lateral support solution of the temporary cut, with reconstruction utilising rock fill to ensure a well-drained fill solution. Verification of conditions during construction was critical. Slope failures are becoming more prevalent with the impact of climate change and more extreme weather events. This paper presents the factors that impacted slope stability, as well as how these were considered in developing a resilient solution.

2 THE SITE

The project stretches over a length of 1.44 km on the National Road N2 (northbound carriageway) in the KwaZulu Natal Province, approximately 44 kilometres south of Durban.

The topography of the area is characterised by steep rolling hills. Homesteads with subsistence agriculture are located on the hill slopes and lower lying areas throughout the area. The embankment slip site has a well-developed drainage valley at the toe of the slope with a river running at the base of the slopes.

The road occurs on cut and fill where the failures appeared to occur in fill sections.

The regional geology comprises of Berea Red Formation sands and residual clayey sands / silty clay overlying the Vryheid Formation sandstone / siltstone and Pietermaritzburg Formation shale (Council for Geoscience 1988). According to Brink (1983), the Pietermaritzburg Formation consists mainly of shales which generally dip about 3° to 15° in an easterly direction. The rock strength is generally soft to very soft rock.

3 THE SLOPE FAILURES

The three slope failure sites along the northbound carriageway are shown in Figure 1. Site A was the original creeping slope embankment with Site B and C the additional failure sites. Prolonged rainfall would have saturated the constructed fill and underlying insitu soil horizons. The slope at Site A before failure is shown in Figure 2 with the resultant failure on the road surface extending into the fast (far-right) lane of the northbound carriageway shown in Figure 3. Figure 4 shows the road surface at Site A before failure. Repair works had been undertaken due to cracking from the creeping slope over a few years as shown in Figure 4.

This slip was understood to have a history in excess of 15 years of periodic movements, particularly after heavy rains (Drennan & Maud 2019). After the period of prolonged heavy rain, cracks opened up and an instruction was issued to close the carriageway. The failure occurred overnight and extended over a 100m length where the road surface dropped by over 1m. The failed material was observed down the slope showing a deep-seated failure.

The observed failure at Site B is shown in Figure 5. The slope was flatter than at Site A and the cracking occurred in the road shoulder over two 20 m stretches, with settlements measured to be over 50 mm.

Figure 1. Locations of three slope failures

The third failure (Site C) (over a 60 m section) manifested as a circular failure surface in the soil horizons of the embankment fill with a near vertical failure scarp in close proximity to the road with bulging of the toe of the slope. Longitudinal cracks were also noted in the northern carriageway adjacent to the failure zone. Unlike the failures at Sites A and B which can be described as deep-seated failures, the failure at Site C had a high and very steep failure scarp and can be described as sloughing failure as shown in Figure 6.

Figure 2. Embankment slope before failure at Site A

Figure 3. Slope failure at Site A

Figure 4. Site A before failure showing previous repair works due to movements

4 APPROACH

Understanding the mechanism causing instability and slope failures at each site was important to be able to design a resilient rehabilitation solution. This required an understanding of the greater area and the topography; historical changes to the topography due to the original construction of the road (location of cut and fill sections); any features that could influence drainage and slope stability such as location of culverts, signs of seepage etc; understanding of water flow on the surface and subsurface of the greater site; and geology and ground conditions.

Figure 5. Site B observed failure

Figure 6. Slope failure at Site C

The approach comprised of a number of site walkovers observing the topography, failure features and extent of material movement, evidence of water and seepage, road drainage infrastructure and exposed ground conditions. A review of literature and published information on geology and slope failures in KwaZulu Natal and the surrounding area was undertaken. A geotechnical investigation (GI) was conducted comprising test pitting, borehole drilling, Dynamic Probe Superheavy (DPSH) testing, geophysical testing (seismic refraction surveys), installation and monitoring of piezometers and laboratory testing. The GI was scoped to understand subsurface conditions, depths of bedrock and water level and changes over the area where the failures occurred.

This information was used to understand the ground conditions across the site and develop design ground models. A drone survey was undertaken of the greater site area. A 3D geological model using Leapfrog software was created to understand the geology and variation in ground conditions and rock level across the site and at each failure zone.

The available geotechnical information was applied along with the drone survey to understand the ground conditions at each slip site and for the whole area to assess the likelihood of failure outside of the slip areas. A back analysis was conducted for each slip to assess the failure mechanism and causes of failure for each slip zone. This information was used to design the required remediation. Three remediation options for the project were assessed and the best workable solution was selected and developed for detailed design. Verification of ground conditions during construction was critical to verify applied design assumptions and to ensure safety on site.

5 REVIEW OF PREVIOUS SLOPE FAILURES IN THE AREA

In September 2021, an embankment failure occurred at the Ilfracombe/ Bonnyrigg interchange, which is located about 1km southwest of the site. The information of this historic embankment failure gave insight into the potential mechanisms of failure. In addition, published literature on embankment slips in KwaZulu Natal or similar geology were utilised.

5.1 *Historic Slip failure*

The Bonnyrigg interchange slip noted cracks developing after heavy prolonged rains. Rapid opening of the cracks and then failure occurred about 24 hours later in the form of a circular slip pattern in the road. The road surface had subsided by approximately 1 m. This was similar to the failure observed at Site A.

The investigation into the failure (Moore Spence Jones 2012) reported that the slip failure occurred as a result of the combination of a number of factors: Prolonged rains had raised the moisture content in the fill and underlying soils, decreasing the strength of these materials; The presence of clay gouge lenses in the weathered shale bedrock dipping down slope; The application of the fill resulting in a surcharge load being applied to these sensitive clays that is beyond their allowable strength tolerances and possible further weakening of the clays due to vibrations from bulk earthwork machinery during the fill construction;

Lack of deep slot type subsoil drainage measures prior to placement of the fill.

5.2 *Site A history*

As shown in Figure 4, Site A had a history of periodic movements after heavy rains. An assessment into the movements in 2019 reported that "the embankment is likely experiencing on-going creep type movement each time there is pore pressure build-up or phreatic surface elevation on the underlying sliding surface, resulting in further displacements occurring." Due to previous episodes of movement, the shear strength along the sliding surface had surpassed its peak strength and was only relying on residual strength. Catastrophic failure was stated as being a probable occurrence in the event of a severe storm. The regional geology shows that the historic slip at Ilfracombe / Bonnyrigg Interchange and the Site A slip occur on the same underlying regional geology.

5.3 *Documented mechanisms of slope failure in Kwazulu Natal*

Brink (1983) details that in general, there are 3 No. failure mechanisms for slope instability documented to have occurred in KwaZulu Natal and the associated geology typical of the site. These comprise of:

- Failure within rock along clay joints.
- Failure within overlying residual clay.
- Failure along the intersection between layers such as the fill and Berea Sand materials or fill and residual clay.

Failure within rock (first mechanism) results from sliding along the clay infilling in the weathered rock. This occurs in the planes dipping at about 10°-20° due to water ingress resulting in pore water build up (Brink 1983). Clay infilling was observed in the shallow dipping joints within the boreholes, however signs of sliding within the clay infilling were not apparent. In addition, the shale bedrock dips in an easterly direction and not in the direction of the failure.

The latter two failure mechanisms were considered more likely due to the observed conditions within the boreholes and rock jointing direction relative to the slope directions. The back analyses undertaken provided insight into the resultant failure mechanism for each of the three sites.

6 GROUND CONDITIONS

A 3D digital geological model was created using Bentley Leapfrog (Version 2021.2) software, the topographical and drone surveys and borehole and test pit data. A geophysical survey was applied to correct rock levels generated in the model. The 3D geological model (shown in Fig. 7) assisted in visually and spatially understanding the complexity of the ground conditions across the site, the vertical and lateral variability of materials, and the relation of this variability of in-situ materials to topography.

In general, the lower lying areas at the base of the slope consist of an upper layer of transported material (described as clayey silty sand) underlain by Berea Red Formation sands. Below this, a clay layer of residual shale overlies shale rock. A fill of variable nature was used to form the road embankment. This fill material varied from a gravelly to silty sand to a sandy clay or clayey sand.

The ground conditions varied between the sites. This variability is seen in the change in residual clay layer thickness and its presence across the site. The residual clay layer was prominent across Site A with it being thickest near the drainage feature. This correlated to the rock levels where the rock dipped toward the drainage feature changing in depth from 2 m to 16.8 m at Site A. Rock comprised of highly weathered, very soft rock becoming soft rock shale, as well as moderately weathered, soft rock becoming medium hard rock shale.

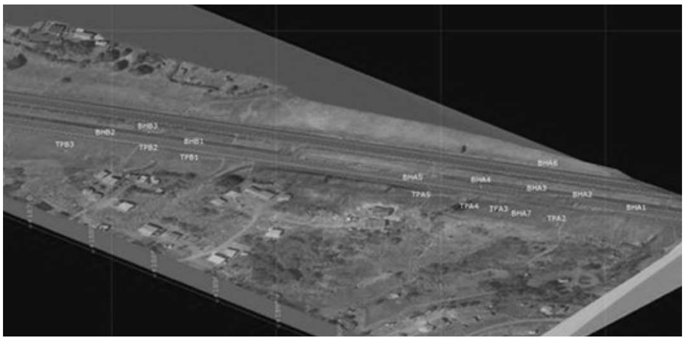

Figure 7. 3D model generated in Leapfrog showing surface and geological layers (drone survey image overlain over geological model)

The residual shale clay layer was not as prominent at Site B and Site C. In addition, where the clay layer was not present there was a boulder layer observed between the Berea Sands and the shale bedrock. The presence of the boulder layer and residual shale clay layer influences the observed resultant failure mechanism as observed in the back analysis. The geotechnical investigation information was applied to develop a representative ground model for each site.

7 BACK ANALYSIS

A back analysis was conducted for each slip. These back analyses were first conducted in limit equilibrium slope stability software Geostudio Slope/W (Version 2021 R2) and the Morgenstern-Price method was applied. In addition, to assess a likely probable failure mechanism, a Plaxis 2D finite element analysis (Connect Edition Version 20.03.00.60) was undertaken. This assessment determines the failure mechanism based on a phi-c reduction stage where the strength parameters are reduced until failure occurs determining a resultant FoS.

The geotechnical parameters and water table levels were adjusted and iterated until a FoS of less than 1 was achieved and the output of the models produced a failure zone of similar geometry to that which was observed in reality. These parameters provided insight into the conditions for failure. The back-analysis was applied to assess failure mechanism and causes of failure for each slip zone.

This information was used to design the required remediation for each slip zone. Initially, drained strength parameters were applied for the residual shale clay layer (based on Mohr-Coulomb strength parameters of friction angle and cohesion). This resulted in shallow slip surfaces, or slip surfaces that did not extend far enough into the carriageway which did not correlate to the observed failure.

Subsequently, undrained parameters (i.e. undrained shear strength, C_u) for the residual shale clay material were used in the analyses. This resulted in a deeper failure mechanism that extended further into the carriageway as observed in reality. In addition, the water table was changed, at first utilising the measured water levels as per the fieldwork and then a rise in the water table.

The analyses show that a reduction of undrained shear strength coupled with a rise in water table level was required to cause failure. The resultant undrained shear strength for each analysis was similar at around 23 kPa. These results correlated to the undrained shear strength results from the Ilfracombe/Bonnyrigg failure assessment where tested shear strengths varied from 35 kPa to 158 kPa for vane shear strength and 16 kPa to 51 kPa for remoulded undrained shear strength; as well as the published range of values for soft clays in the Durban area (Byrne et al. 2019). For sections analysed just outside of the slip zone, a higher shear strength of 50 kPa was used.

The resultant failure mechanism for the back analysis for Site A is shown in Figure 8 and Figure 9 for the Slope W and Plaxis 2D assessments. The slip at Site A had been moving over a number of years. This creep type movement resulted in the strength of the residual shale clay layer within the slip zone reducing over time reaching a remoulded residual undrained shear strength value. The heavy rains that occurred, resulted in a rise in the water table and saturated ground conditions. This further reduced the shear strength of the residual shale clay material and failure of this layer in an undrained manner, resulting in the deep-seated failure that was observed. This was shown by the back analysis.

Figure 10 shows the resultant failure mechanism for Site C. Site C has different ground conditions to those observed at Site A. This typically shows ground profiles for higher topographical elevations where there is no or limited presence of the residual shale clay layer. There was a clear boundary observed between the Berea Sands and the shale bedrock comprising of a boulder/gravel layer separating the overlying sands with the bedrock. The presence of this layer and no residual shale clay influenced the observed resultant failure mechanism.

The slope failure appeared to be constrained to the fill and soil horizons and its extent was limited by the presence of the gravel/boulder layer overlying the shale bedrock. This observation is important and indicates how the underlying ground conditions and geology influences the failure mechanisms and their extent especially in these geological conditions in KwaZulu Natal. This understanding guides remediation solutions applied and could also be used to assess the potential impact of slope failures in this type of geology.

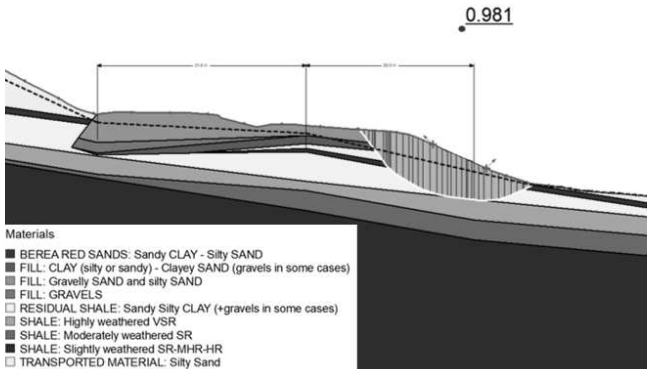

Figure 8. Slope W Failure mechanism for Site A

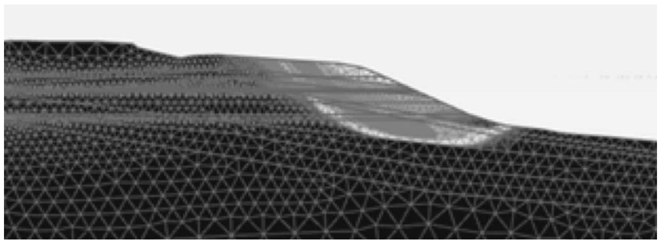

Figure 9. Plaxis 2D Failure mechanism for Site A

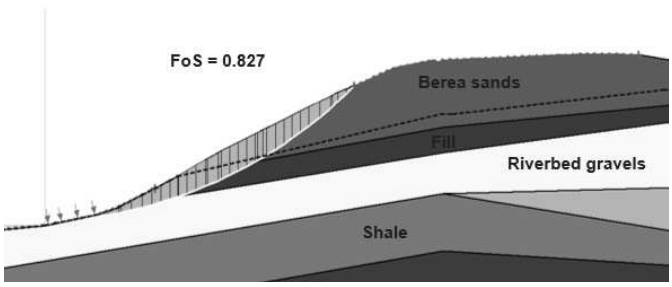

Figure 10. Slope W back analysis failure mechanism for Site C

The extent of the failure for Site A also appeared to be driven by the topography and rock level. The material movement appeared to have been parallel to the slope angle where the material moved downslope in relation to the rock slope level. Shallow rock constrained the failure from extending further into the road reserve. In addition, the failures occurred within the fill embankment sections of the road whereas cut sections seemed to constrain the extent of the failures.

Another contributing factor to the failures was the poor drainage conditions in the area. The subsurface drainage conditions across the site appeared to be complex with high water tables present after rainfall events. From as-built information and evidence from the topography and walkovers, the fill embankment appeared to be situated directly over an ancient drainage valley. Culvert diversions indicated the attempt made to divert the water; however, it was not being sufficiently captured as sub-surface water was flowing into the fill embankment. This appeared to influence the sub-surface conditions across the site and the resultant failures, contributing to high moisture and water table levels. This needed to be accounted for in the applied remediation solutions to ensure long-term stability.

The understanding of the factors that influenced the three failures observed at the site and their extent is important in designing robust remediation solutions as well as assessing stability of existing slopes. The key factors are the influence of topography, underlying ground conditions such as residual clay layer thickness and rock levels and how these change over an area, occurrence of fill and cut sections of a road, groundwater levels as well as surface and sub-surface drainage.

8 FINAL SOLUTION

From the understanding gained from the back-analysis, it was not possible to reconstruct the embankment by building on the sensitive reworked clayey soils. The residual clay soils needed to be removed during the remediation construction of the embankment to ensure that no clays were left in the embankment that would allow similar failures to reoccur in the future. In addition, the drainage system needed to be redesigned and the embankment needed to be constructed of stable, free-flowing material.

Three remediation options for the project were considered where the existing road needed to be supported for remediation to occur. Option 1 comprised the removal of the failed material and replacement with rock fill where an excavation with lateral support was required. Option 2 considered the installation of an anchored sheet pile wall, after which the failed material could be removed to rock level and replaced with fill material. Option 3 considered the use of a piled wall with regrading of the slope.

The applicability of each was assessed considering a number of factors: Extent of required excavation cut from failed zone; potential interference with existing drainage infrastructure and services and relocation requirements; extent of remediation solution within road reserve and in relation to environmental boundaries; potential reuse of excavated material; impact on road user; cost, safety, constructability, long term functionality and environment.

The best workable solution was selected to develop for detailed design. The solution comprised of excavation and removal of the failed and problem material and stabilising the cut slope with lateral support (i.e. soil nailing and shotcrete) before reconstructing the embankment with rock fill. The design considered sensitivity of increased water levels and the effect on the stability of the temporary and long-term solution. This was important as with effects of climate change, extreme rainfall events could result in high subsurface water tables. Rock fill allowed the development of a robust solution as it is stronger and more permeable than typical soil type fill materials, considering the complex subsurface drainage observed at the site.

This solution was considered as having the most cost-effective temporary support of a soil nail wall, where the nails form part of the permanent solution. In terms of constructability, this solution was considered to be the easiest to construct with the requirement that the ground conditions on site during excavation and the cut slope conditions were verified to design assumptions. Verification that the residual shale clay materials were removed within the excavation was required. The solution also considered long term functionality as a result of the removal of the residual clay and the installation of effective drainage, designing for a potentially higher water table and thus managing the cause of failure for future flood events. This solution also allowed for the potential future expansion of the pavement.

9 CONCLUSIONS

Three slope failures occurred along a section of road after prolonged, heavy rainfall. The failures differed in extent, depth and form. A back analysis was undertaken of each failure utilising available geotechnical information, a drone survey and 3D geological model created to understand the variability of ground across the greater site.

The key factors are the influence of topography, underlying ground conditions such as residual clay layer thickness and rock levels and how these change over an area, occurrence of fill and cut sections of a road, groundwater level as well as surface and subsurface drainage. The failures occurred in fill sections of the road. Where thicker residual clay was present, deep seated failure occurred; whereas where the clay layer was not present, a high and very steep failure scarp occurred. The understanding of the factors that influenced the three failures and their extent was important in designing robust remediation solutions and assessing stability of existing slopes.

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