Assessing the Stability of Road Cuttings on the Zuurberg Pass using Romana’s Slope Mass Rating

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Abstract

Road cuts excavated in rock often exhibit geological hazards such as rock slope failures (wedge, planar and toppling). An assessment of the rock cuts along a 20 km section of the R335 Zuurberg Road was done. Stereographic projections, Slope Mass Rating (SMR) and Falling Rock Hazard Index (FRHI) were used to determine slope stability and compare results. Most of the 30 identified cuts contained small blocks of rock that could loosen but would not require extensive removal or reinforcement. The proposed rock slide catchments consisted of engineering benches, wider shoulders, steel barriers, net fences and periodic assessment. For the flatter slopes where rock falls tend to bounce and roll, a barrier would be needed to catch / trap the rocks away from the road, or alternatively a wider inside shoulder especially on tight corners. Stereographic projections and SMR correlated well, but FRHI seemed to give a very broad classification of the cuts.

Keywords: Zuurberg Pass, SMR, FRHI, Stereonets, rock slope

1 Introduction

Road cuts excavated in rock often exhibit geological hazards such as rock slope failures (wedge, planar and toppling). Traffic movement can be affected by these phenomena and may even cause accidents (Tomás, 1989). Slope stability can be evaluated using Slope Mass Rating (SMR, Romana, 1985), which is derived from Rock Mass Rating (RMR, Bieniawski, 1979). SMR is a very useful geomechanical classification used in rock slope characterization (Tomás, 1989). RMR became a standard for use in tunnels and many professionals apply it to describe any rock mass (Romana et al., 2003). The original RMR (Bieniawski, 1976) included an adjustment for the joint discontinuity rating based on the discontinuity orientation, namely: very favourable – 0, favourable – 5, fair – 25, unfavourable – 50, very unfavourable – 60. If the orientation adjustment is incorrect this value can override any careful evaluation of the rock mass, and classification becomes both difficult and arbitrary (Romana et al., 2003). Romana (1985, 1993, 2003) proposed an adjustment to the RMR concept, especially suited to slopes, which has been endorsed by Bieniawski (1989). More specific calculations for joint condition and discontinuity was included in Bieniawski (1989). The SMR is obtained from RMR by adding a factorial
adjustment factor depending on the relative orientation of joints and of the slope as well as an adjustment factor depending on the method of excavation.

Falling Rock Hazard Index (FRHI) was developed by Singh (2004), based on work done earlier at the Oregon and Washington Department of Transportation of United States. The FRHI is used to determine the degree of dangerous situations in the immediate vicinity of a rock slope excavation site (Koleini, 2013).

2 Project Overview

An assessment of the 20 km R335 Zuurberg Pass Road as shown in Figure 1, revealed a number of areas where geotechnical inputs were required. The 30 proposed cut slopes in the pass area included both existing cut slopes where the slope stability parameters had to be defined, and the stability confirmed, and new cut faces where stability of these cut slopes would need to be analysed to ensure safe slope designs are maintained. A study was done to assess the typical conditions of the cut slopes. The current conditions were assessed as these further reflect potential issues that might be expected during the upgrading. For the respective cut slopes, the field characterization required detailed engineering geological mapping of the affected cut slopes, including detailed description of the geological profile including the soil horizons if present and the rock mass. Joint line survey measurements included attitude and continuity of the discontinuity sets as well as current slope angles. Assessment and analysis of the discontinuity data was done using stereographic projections, in order to identify the potential failure modes, taking into account the envisaged geometry of the existing slopes and of new road cuttings. The geotechnical parameters that would be required for the design analyses had to be verified, including confirmation of the rock material strengths, and estimated shear strengths of the rock mass as well as of major discontinuities. The road cuttings are located mainly in micaceous shale and siltstone and subordinate sandstone of the Weltevrede Formation, and brownish weathered quartzitic sandstone and subordinate shale of the Witpoort Formation, Witteberg Group, Cape Supergroup.

![Figure 1. Route layout](image-url)
3 Methodology

The cuttings were investigated to determine the material strength and continuity. Joint line surveys were conducted in the rock cuttings and measurements were taken of the joints and beds in the rock faces, recording the dip direction and dip.

3.1 Stereographic projections

Stereographic projections are used to represent and analyse three-dimensional orientation data in two dimensions. This allows lines or points to represent planes, and points to represent lines. However, these projections consider only angular relationships between lines and planes, and cannot represent the position or size of the feature (Wyllie et al., 2004). The orientation of a plane can be represented on a stereographic projection using the pole of the plane. A radial line in a direction normal to the plane would pierce the surface of the reference sphere at a point called the pole. The strike and dip measurements taken in the field are used to plot poles based on the orientation of the discontinuity (Wyllie et al., 2004). The strike and dip readings were inserted into stereographic projections on Dips (RocScience), creating a stereonet per identified cut orientation. Figure 2 shows an example of the bedding plane and joints from Cut 1 represented as poles and planes on a stereonet.

![Stereogram](image)

**Figure 2. Bedding plane and joints from cut 1 represented as poles and planes**

Planar failures were determined using a daylight envelope of the cut face and a 30° friction angle cone. Any poles visible in the area outside of the friction angle cone, but within the daylight envelope was then marked as a possible failure surface. Wedge failures were determined using a 30° friction angle cone and the cut face plane. Any line of intersection of the two planes within the friction angle cone was marked as a possible failure as these will exceed the frictional resistance and the wedge will slip. Finally toppling failures were determined using a 30° friction angle as well as a slip limit determined from the cut face (Phillips and Phillips, 1971). Any poles within the friction angle but outside of the slip limit was identified as a possible failure area as indicated in Figure 3.
3.2 SMR rating

Slope stability can be determined using the Slope Mass Rating classification system (SMR, Romana, 1985, 1993, 2003), which is derived from the well-known Rock Mass Rating (RMR, Bieniawski, 1989). SMR is determined using the uniaxial compressive strength (UCS), rock quality designation (RQD), joint spacing, joint and groundwater conditions of the rock together with the method of failure with its dip and strike. The method of excavation and cut slope and direction is also used. The SMR is derived from the following equation:

\[ \text{SMR} = \text{RMR}_b + (F_1 \times F_2 \times F_3)+F_4. \]  

(1)

Where (Romana et al., 2015):
- \( \text{RMR}_b \) is the basic RMR index resulting from Bieniawski’s rock mass classification;
- \( F_1 \) depends on the parallelism (A in Table 1) between discontinuity dip direction, \( \alpha_j \), and slope dip, \( \alpha_s \), (Table 1);
- \( F_2 \) is related to the probability of discontinuity shear strength (Romana, 1993) and depends on the discontinuity dip, \( B=\beta_j \), in the case of planar failure (Table 1). For toppling failure, this parameter adopts the value 1.0.
- \( F_3 \) depends on the relationship (C in Table 1) between slope, \( \beta_s \), and discontinuity, \( \beta_j \), dips (Table 1). This parameter is the original Bieniawski adjustment factor (from 0 to -60 points) and expresses the probability of the discontinuity to outcrop on the slope face (Romana, 1993) for planar failure.
- \( F_4 \) is a correction factor that depends on the excavation method (Table 2).
Table 2. Values corresponding to the factor F4 (Romana et al., 2015)

<table>
<thead>
<tr>
<th>Excavation method (F4)</th>
<th>Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Presplitting</td>
<td>+10</td>
</tr>
<tr>
<td>Smooth blasting</td>
<td>+8</td>
</tr>
<tr>
<td>Natural slope</td>
<td>+15</td>
</tr>
<tr>
<td>Blasting or mechanical</td>
<td>0</td>
</tr>
</tbody>
</table>

Each parameter is compared to a set standard, which is then used to classify the slope using a total calculated score. This number is used to divide a slope into five classes, namely: very bad, bad, normal, good or very good. Each class has a corresponding stability and structurally controlled failure designated as shown in Figure 4. Based on these results, mitigation measures are then suggested accordingly. Figure 4 indicates the SMR frequency distribution and the class of Cut 01-3 with possible mitigation measures. The approach used combined Romana’s SMR method with a Monte-Carlo analysis, which allowed evaluation of the probability (%) of a particular slope condition occurring for every cut.

Figure 4. CSMR classification and mitigation options for Cut 01-03

3.3 Falling Rock Hazard Index (FRHI)

Falling rock hazard index is used to determine the degree of dangerous situations in the immediate vicinity of a rock slope excavation. FRHI is determined using the face height, face inclination, face irregularity, rock condition, equivalent RQD, spacing of discontinuity, block size and volume of falling rocks, excavation methods, time factor without remedy and the rockfall frequency. These parameters are then used in a rating system to calculate a final score as shown in Table 3, which then results in a rockfall hazard classification (Table 4). This classification produces the fall hazard and mitigation measures based on the score range achieved (Koleini, 2013).
Table 3. FRHI worksheet (after Singh, 2004)

<table>
<thead>
<tr>
<th>Face height</th>
<th>Scoring breakdown</th>
<th>1.5 m - 4.5 m</th>
<th>1.5 m - 2 m = 3</th>
<th>2 m - 3 m = 3</th>
<th>3 m - 4 m = 3</th>
<th>4 m - 4.5 m = 6</th>
<th>4.5 m - 7.5 m</th>
<th>7.5 m - 9 m = 10</th>
<th>9 m = 12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Face inclination</td>
<td>Scoring breakdown</td>
<td>30° - 75° or 90°</td>
<td>30° - 35°</td>
<td>90° - 80° = 2</td>
<td>80° - 75° = 3</td>
<td>30° - 35° = 4</td>
<td>35° - 60°</td>
<td>60° - 75°</td>
<td>60° - 65° = 8</td>
</tr>
<tr>
<td>Face irregularity</td>
<td>Scoring breakdown</td>
<td>Few, clear cut = -1</td>
<td>Occasional irregularities = 3</td>
<td>Many irregularities = 8</td>
<td>Major launching features = 11</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock condition</td>
<td>Scoring breakdown</td>
<td>Hard and intact</td>
<td>No joints or cracks = -1</td>
<td>Few joints and cracks</td>
<td>Firm interlock of blocks between joints = 3</td>
<td>Very blocky, many fractures</td>
<td>Imperfect interlock of intact rock fragments; Many fractures = 7</td>
<td>Highly fractured</td>
<td>Completely Crushed = 10</td>
</tr>
<tr>
<td>Equivalent RQD, %</td>
<td></td>
<td>100 - 90</td>
<td>90 - 50</td>
<td>50 - 25</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing of discontinuity</td>
<td>Scoring breakdown</td>
<td>Very wide</td>
<td>0.9 m - 0.2 m</td>
<td>0.9 m - 0.6 m = 2</td>
<td>0.6 m - 0.3 m = 3</td>
<td>0.3 m - 0.2 m = 4</td>
<td>Close</td>
<td>0.2 m - 0.05 m = 6</td>
<td>Very close</td>
</tr>
<tr>
<td>Block size of falling rock</td>
<td>Scoring breakdown</td>
<td>&lt; 0.05 m</td>
<td>0.05 m - 0.1 m</td>
<td>0.05 - 0.076 m = 2</td>
<td>0.076 - 0.1 m = 3</td>
<td>0.1 m - 0.2 m</td>
<td>0.1 m - 0.127 m = 4</td>
<td>0.127 m - 0.13 m = 3</td>
<td>0.13 m - 0.17 m = 6</td>
</tr>
<tr>
<td>Volume of rockfall</td>
<td>Scoring breakdown</td>
<td>&lt; 4.54 kg</td>
<td>4.54 kg - 13.6 kg</td>
<td>4.54 kg - 6.8 kg = 3</td>
<td>6.8 kg - 9.1 kg = 5</td>
<td>9.1 kg - 13.6 kg = 7</td>
<td>13.6 kg - 22.7 kg = 9</td>
<td>13.6 kg - 15.88 kg = 9</td>
<td>15.88 kg - 18.14 kg = 10</td>
</tr>
<tr>
<td>Excavation method</td>
<td>Scoring breakdown</td>
<td>Control blasting</td>
<td>None to few fractures = 1</td>
<td>Mechanical excavation</td>
<td>Smooth exc. = 1</td>
<td>Regular cut; some fractures = 3</td>
<td>Manual cut = 4</td>
<td>Regular blasting</td>
<td>Fractures; some irregularities = 1</td>
</tr>
<tr>
<td>Time factor w/o remedy</td>
<td>Scoring breakdown</td>
<td>&lt; 1 day</td>
<td>1 day - 1 month</td>
<td>1 day - 5 days = 2</td>
<td>5 days - 10 days = 3</td>
<td>10 days - 1 month = 4</td>
<td>4 years or &gt; 1 year = 5</td>
<td>1 month - 2 months = 5</td>
<td>2 months - 4 months = 6</td>
</tr>
<tr>
<td>Rockfall frequency</td>
<td>Scoring breakdown</td>
<td>No rockfall</td>
<td>Rare rockfall</td>
<td>No rockfall in natural condition; rockfall when disturbed = 3</td>
<td>Occasional rockfall</td>
<td>Rockfall in natural condition; Much falls with disturbance = 8</td>
<td>Frequent rockfall</td>
<td>Rockfalls without disturbance; high frequency = 8</td>
<td>Total Score</td>
</tr>
</tbody>
</table>

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4 Results

The unstable sections in the studied area was divided into three methods of failure namely ravelling / erosion, planar or toppling failure. Ravelling of loose rock blocks and erosion of smaller rock pieces and talus were found to be potentially unstable along the pass at most of the proposed cuts with only a few exceptions. With FRHI analysis, all the ravelling areas had a classification of moderate risk (class III). The ravelling and erosion indicates unstable to stable classification according to their SMR rating. Table 5 lists all the cuts where a method of failure could be identified using stereographic projection, with their joint line survey parameter values, stereo net failure mechanism, SMR class and FRHI class.

<table>
<thead>
<tr>
<th>Cut No.</th>
<th>Cut Dip</th>
<th>Bedding Dip</th>
<th>Joint 1 Dip</th>
<th>Joint 1 Strike</th>
<th>Joint 2 Dip</th>
<th>Joint 2 Strike</th>
<th>Failure Mode</th>
<th>SMR Class</th>
<th>FRHI Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>01-1</td>
<td>56</td>
<td>108</td>
<td>55</td>
<td>045</td>
<td>83</td>
<td>301</td>
<td>40</td>
<td>238</td>
<td>III: 90%, IV: 10%</td>
</tr>
<tr>
<td>01-2</td>
<td>56</td>
<td>138</td>
<td>55</td>
<td>045</td>
<td>83</td>
<td>301</td>
<td>40</td>
<td>238</td>
<td>III: 96%, IV: 4%</td>
</tr>
<tr>
<td>01-3</td>
<td>56</td>
<td>161</td>
<td>55</td>
<td>045</td>
<td>83</td>
<td>301</td>
<td>40</td>
<td>238</td>
<td>II: 2%, III: 98%</td>
</tr>
<tr>
<td>02-1</td>
<td>56</td>
<td>282</td>
<td>72</td>
<td>53</td>
<td>37</td>
<td>235</td>
<td>N/A</td>
<td>N/A</td>
<td>III: 100%</td>
</tr>
<tr>
<td>02-2</td>
<td>56</td>
<td>264</td>
<td>72</td>
<td>53</td>
<td>37</td>
<td>235</td>
<td>N/A</td>
<td>N/A</td>
<td>III: 45%, IV: 55%</td>
</tr>
<tr>
<td>02-3</td>
<td>56</td>
<td>239</td>
<td>72</td>
<td>53</td>
<td>37</td>
<td>235</td>
<td>N/A</td>
<td>N/A</td>
<td>III: 17%, IV: 83%</td>
</tr>
<tr>
<td>03-2</td>
<td>56</td>
<td>41</td>
<td>68</td>
<td>059</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>P</td>
<td>II: 35%, III: 65%</td>
</tr>
<tr>
<td>05-1</td>
<td>56</td>
<td>285</td>
<td>23</td>
<td>057</td>
<td>68</td>
<td>287</td>
<td>N/A</td>
<td>N/A</td>
<td>III: 100%</td>
</tr>
<tr>
<td>05-2</td>
<td>56</td>
<td>253</td>
<td>23</td>
<td>057</td>
<td>68</td>
<td>287</td>
<td>N/A</td>
<td>N/A</td>
<td>III: 100%</td>
</tr>
<tr>
<td>05b-1</td>
<td>56</td>
<td>246</td>
<td>10</td>
<td>070</td>
<td>80</td>
<td>230</td>
<td>59</td>
<td>358</td>
<td>T, W: III: 100%</td>
</tr>
<tr>
<td>09-1</td>
<td>56</td>
<td>227</td>
<td>15</td>
<td>228</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>T</td>
<td>II: 4%, III: 96%</td>
</tr>
<tr>
<td>09-3</td>
<td>56</td>
<td>280</td>
<td>15</td>
<td>228</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>P</td>
<td>III: 100%</td>
</tr>
<tr>
<td>12-1</td>
<td>56</td>
<td>64</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>N/A</td>
<td>III: 96%, IV: 4%</td>
</tr>
</tbody>
</table>

Table 4. Rock fall hazard classification (after Singh, 2004)
<table>
<thead>
<tr>
<th>Cut No.</th>
<th>Cut Dip</th>
<th>Bedding Dip</th>
<th>Strike</th>
<th>Joint 1 Dip</th>
<th>Strike</th>
<th>Joint 2 Dip</th>
<th>Strike</th>
<th>Failure Mode</th>
<th>SMR Class</th>
<th>FRHI Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>12-2</td>
<td>56</td>
<td>88</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>T</td>
<td>III: 96%, IV: 4%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>12-4</td>
<td>56</td>
<td>0</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>T</td>
<td>III: 98%, IV: 2%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>12-5</td>
<td>56</td>
<td>321</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>T, P</td>
<td>III: 98%, IV: 2%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>12-6</td>
<td>56</td>
<td>284</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>T, P</td>
<td>III: 96%, IV: 4%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>12-7</td>
<td>56</td>
<td>298</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>T</td>
<td>III: 98%, IV: 2%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>12-8</td>
<td>56</td>
<td>318</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>T, P</td>
<td>III: 26%, III: 74%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>12-9</td>
<td>56</td>
<td>333</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>T, P</td>
<td>III: 98%, IV: 2%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>12-10</td>
<td>56</td>
<td>351</td>
<td>71</td>
<td>340</td>
<td>15</td>
<td>150</td>
<td>N/A</td>
<td>P</td>
<td>III: 98%, IV: 2%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>13-1</td>
<td>56</td>
<td>355</td>
<td>56</td>
<td>049</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>P</td>
<td>II: 43%, III: 57%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>13-2</td>
<td>56</td>
<td>20</td>
<td>56</td>
<td>049</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>P</td>
<td>II: 33%, III: 67%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>13-3</td>
<td>56</td>
<td>35</td>
<td>56</td>
<td>049</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>P</td>
<td>III: 55%, IV: 45%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>13-4</td>
<td>56</td>
<td>45</td>
<td>56</td>
<td>049</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>P</td>
<td>III: 44%, IV: 27%, V: 28%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>13-5</td>
<td>56</td>
<td>317</td>
<td>56</td>
<td>049</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>P</td>
<td>II: 54%, III: 46%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>14-1</td>
<td>56</td>
<td>216</td>
<td>55</td>
<td>038</td>
<td>47</td>
<td>222</td>
<td>N/A</td>
<td>T, P</td>
<td>V: 87%, IV: 13%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>14-2</td>
<td>56</td>
<td>194</td>
<td>55</td>
<td>038</td>
<td>47</td>
<td>222</td>
<td>N/A</td>
<td>T, P</td>
<td>II: 78%, III: 22%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>14-3</td>
<td>56</td>
<td>180</td>
<td>55</td>
<td>038</td>
<td>47</td>
<td>222</td>
<td>N/A</td>
<td>T, P</td>
<td>II: 78%, III: 22%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>14-4</td>
<td>56</td>
<td>201</td>
<td>55</td>
<td>038</td>
<td>47</td>
<td>222</td>
<td>N/A</td>
<td>T, P</td>
<td>II: 73%, III: 27%</td>
<td>III: 100%</td>
</tr>
<tr>
<td>15-1</td>
<td>56</td>
<td>177</td>
<td>52</td>
<td>030</td>
<td>N/A</td>
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<td>N/A</td>
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</tr>
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<td>82</td>
<td>212</td>
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<td>49</td>
<td>050</td>
<td>N/A</td>
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<td>N/A</td>
<td>W</td>
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<td>N/A</td>
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<td>N/A</td>
<td>P</td>
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Notes: 1 - P = Planar Failure, T = Toppling, W = Wedge Failure

5 Recommendations

Small rock fragments/blocks are present in most of the 30 identified cuts and may mobilise in the future but do not require extensive removal or reinforcement. The proposed catchments consist of engineering benches, wider shoulders, steel barriers and net fences. The catchment will need to be accessed periodically to remove the loose debris. For the flatter slopes where rock falls tend to bounce and roll, a barrier is needed to deflect rocks away from the road, or alternatively a wider inside shoulder especially on tight corners.

These instabilities are suggested to be mitigated with construction of a wider toe ditch to catch / retain rock mass ravelling and a crest drain to control run-off, to reduce long term erosion of the cut face. The ditch will require maintenance and clean out long term. A catch fence may need to be constructed for cuts higher than 6m to catch any falling rocks. An alternative is to widen the cut so that a spillage area is allowed between the rock face and the toe ditch. This is important around blind / tight bends. Netting can also be used to catch falling rocks and keep them away from the road. Planar failure of unstable to partially stable SMR rating was noted and a wider toe ditch is advised, with run-off control and maintenance. If the cuts cannot be cut back, then consideration should be given to systematic bolting of the most susceptible areas. Stable to unstable toppling was mitigated with a wider toe ditch with run-off control and maintenance.

6 Conclusion

The proposed cuts for Zuurberg Pass will need some stabilization for possible falling rocks, but generally they seem to be relatively stable in relation to major failure mechanisms. The stereographic method is a useful analysis that gave valuable information for the various cuts and their failure methods. These methods were confirmed using SMR, gaining additional information on the stability of the cuts and their possible failure modes. The mass failure classified using SMR was compared to FRHI analysis, which gave similar results. All the FRHI analyses were classified as moderate risk, compared to the range of stable to unstable as SMR indicated. FRHI gives a much broader classification over the entire area, whereas SMR is much more specific. FRHI suggests netting as mitigation for all classes of more than minimal risk.
SMR suggests a range of mitigation options depending on the stability class and the preferred support system, again giving more specific suggestions than the FRHI analysis.

7 Acknowledgements

I would like to thank Aurecon South Africa (Pty) Ltd for supporting the preparation of this technical paper. Gratitude is also due to the following colleagues at Aurecon: Gerhard Keyter, Doug Dorren and Gary Davis; as well as to Prof Louis van Rooy at the University of Pretoria, who provided expertise and assisted in reviewing this paper. Also note that the SMR Monte-Carlo simulation used for this project as presented in this paper was developed by Aurecon colleagues, namely Gerhard Keyter and Danie Snyders.

References

Koleini, M. 2013. Engineering geological assessment and rock mass characterization of the Asmari formation (Zagros Range) as large dam foundation rocks in southwestern Iran.
Successful Application of a Flexible Slope Stabilisation System as the Replacement of a Failed Hard Facing Installation on a Creeping Soil Slope

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Abstract

The alignment of a new highway in the North of Greece required several massive cuts in soil slopes. Mainly due to creep movements within the superficial layer of a cut soil slope, the shotcrete facing reached its limitations and failed. The friction forces as a result of interaction between shotcrete facing and soil surface caused an overstressing of the stiff facing and finally its collapse. The shotcrete was removed and replaced by a flexible slope stabilisation system consisting of a high-tensile steel wire mesh cover in combination with an adequate nailing. Flexible slope stabilisation systems are widely used to stabilise soil and rock slopes. They are economical and a good alternative solution to stiff measures with shotcrete or massive structures with the possibility of revegetating stabilised slopes. Special concepts have been developed for the dimensioning of flexible slope stabilisation systems considering superficial instabilities. Numerous implemented cases in soil as well as in rock with and without vegetated faces confirm that these measures are suitable for practical application.

Keywords: landslide, flexible slope stabilisation, shotcrete, high-tensile steel mesh, cut slope

1 Introduction

A modern closed motorway 680 kilometres long over the greatest part of its length following a new alignment and running across Epirus and Northern Greece from Igoumenitsa to Evros, the Egnatia Motorway is one of the largest road construction projects in Europe. Nine major vertical axes connect the motorway with Albania, Macedonia, Bulgaria and Turkey. Furthermore, 5 ports and 6 airports service the road. The Egnatia Motorway as the modern reincarnation of the great Roman highway was designed to the specifications of the Trans-European road network.

In the area of Metsovo / Peristeri, a section of the Egnatia Motorway was constructed but not finished in 1992. Thereby, several natural slopes needed to be cut and were stabilised with soil
nailing in combination with a shotcrete facing. This was a tremendous natural impact without considering natural aspects.

Seven years after cutting numerous natural slopes and stabilising them with soil nailing in combination with a shotcrete facing, EGNATIA ODOS A.E. worked out a proposal to remove the partly failed and unnatural looking shotcrete coverage in regard to visual aspects and static requirements. The main goal was to renaturate and successfully stabilise the slope cuttings. A cost effective and reliable solution for greening and recultivation with local plants, to obtain a natural state and prevent the slopes from further erosion and any instabilities, was demanded. Finally, one specific section was replaced by a flexible slope stabilisation system consisting of a high-tensile steel wire mesh in combination with a nailing to date.

![Figure 1. Egnatia Motorway illustrated by the red line, location of the project in the area of Metsovo/Peristeri (black dot).](image1)

**2 Project**

The renaturated slope cutting is situated near Peristeri, Greece, above a tunnel open pit constructed on an altitude of about 720 m a.s.l. (location A) The inclination of the stabilised 420 m long slope with a height of 40 m in the entrance area as well as 18 m above the covered tunnel amounts to 40 – 60 degrees.

The climatical conditions are comparable with the inner-alpine dry zones of mountain and subalpine altitudes. The annual precipitation amounts to approximately 920 mm. The seasonal distribution of precipitation is not uniform, longer dry periods in summer and autumn are changing with rainfall-periods in winter. The project area is exposed to the north, the incidence of sun-rays is moderate.

The project area is surrounded by undisturbed slopes covered by a continuous vegetation layer established during along process over some centuries. This ecological system is in a labile balance and therefore reacts very sensitive to man made cuttings and impacts.
3 Geology

The area of interest is location A (ch. 4+511 – ch. 4+616) where sandstones are prevalent.

Geologically, the area is structured by flysch of Pindos zone. Tectonically, the flysch is intensely folded and sheared. On the surface, the flysch is weathered and shattered down to a 6 to 7 m depth. Seasonal perched water tables are developed in the surface loose and shattered flysch zone. The erosion caused by Metsovitiokos river at the base of the landslide plays an important role in the landslide of area A.

Failures were observed in the embankment fill, the weathered part of the flysch and the deeper layers within the flysch.

The failures observed can be separated into creep and landslides. The biggest part of A area is an active deep landslide with local creep. Creep causes instability at shallow depth.

![Figure 2. Overview looking south to Egnatia](image)

![Figure 3. Shotcrete facing at location D (left) and at neighboring slopes](image)
4 Shotcrete facing with soil nailing

In the surveyed section, the slope surfaces were completely covered with a shotcrete layer of varying thickness (5–25 cm) and inhomogeneous quality. The overall stability of the slope was guaranteed by steel bar anchors of type GEWI $d = 28$ mm with 12 m length, applied in a grid of approximately 3.0 x 3.0 m. In steeper slope sections, the shotcrete was generally reinforced by a steel mesh with an opening size of 100 mm and wire diameter of 3–4 mm, connected to the anchor heads with quadratic spherical steel plates. In flater areas, the shotcrete was reinforced with steel fibers approx. 50 mm long.

![Failed and superficial cracked shotcrete in the Flysch zone (left) and throughout the neighboring slopes (right).](image)

In the period between 1992 and 1999, no major problems concerning the overall stability were observed, excluding two small and shallow landslide areas resulting in a complete destruction of the shotcrete coverage.

In general, the applied shotcrete is of poor quality. Evidently, the fraction of cement added was partially too low visible due to the darker color of the shotcrete. In those areas, up to 20 – 25 mm of the top layer the shotcrete facing is strongly weathered and mellow with the result of cracks within the shotcrete facing. The areas lighter emerging are of better quality less weathered and in better condition. Next to the bad quality of the shotcrete facing, the main problem was the erosion of the subsoil just behind the shotcrete due to insufficient or not adequate drainage measures, respectively. Locally, there was no contact anymore between the subsoil surface and the shotcrete facing.

5 Application of the flexible slope stabilisation system TECCO®

In a first step, the shotcrete facing was completely removed. Instead of a cover layer impermeable with a stiff behavior, a flexible slope stabilisation system was installed consisting of the TECCO® high-tensile steel wire mesh in combination with special system spike plates adapted to the high-performance steel wire mesh in its size and bending resistance based on numerous puncturing and bending tests. GEWI $d = 28$ mm steel bars with a length of 8 m and arranged in a pattern of 3.0 m x 3.0 m are used for the nailing of the slope whereas the mesh is connected to each nail head by the special spike plates. The forces are then transferred from the mesh over the plate into the anchors.

Rock and soil anchors offer the possibility to stabilise steep slopes comprising of soil or rock. When the slope inclination is restricted to 50 – 60 degrees for soil slopes and to 70 degrees for rock slopes, the anchoring can be combined with a complete slope protection system including
a tensed static system for surface stabilisation and a vegetation layer to prevent the slope surface from erosion caused by heavy rainfalls, snow, water outflows or even wind.

Figure 5. After successful installation of the TECCO® slope stabilisation SYSTEM³.

Figure 6. One year after the installation of the TECCO® slope stabilisation SYSTEM³.

The use of high-tensile steel wire meshes as a flexible surface stabilisation measure has proved its suitability in numerous cases and is often an alternative to massive concrete constructions (Kytzia et al., 2016 and Kühne et al., 2001). This success of this technology is decisively influenced by numerous laboratory and field tests as well as long-term experiences and practical applications worldwide (Rorem and Flum, 2003). The open structure of the meshes permits thereby to realize a full-surface vegetation face.
Furthermore, the open structure has the effect that no water pressure can be built up. Of course, to avoid any erosion problems, the surface needs to be revegetated and if there is noteworthy hillside water existing, corresponding drainage measures are required as well.

In standard layout, the high-tensile steel wire mesh TECCO® for surface stabilisation is made from a high-tensile steel wire of a tensile strength of the individual wire of at least 1’770 N/mm² of 3 mm diameter which has an aluminium-zinc coating (so-called GEOBRUGG SUPERCOATING®) for protection against corrosion. The diamond-shaped meshes measuring 83 mm · 143 mm are produced by single twisting. The TECCO® steel wire mesh provides a tensile strength of 150 kN/m. Thanks to its three-dimensional structure, the mesh clings to the soil in an ideal manner and, additionally, serves to optimally secure subsequent sprayed-on greening (Cala et al., 2012).

Special diamond-shaped system spike plates matching the TECCO® mesh serve to fix the mesh to soil or rock nails. By tightly pressing and if possible slightly impressing the spike plates in the ground to be stabilised, the mesh is tensioned in the best possible manner (Cala et al., 2012).

Figure 7. General scheme of flexible surface stabilisation system and its nail arrangement (left, Cala et al. 2012). Key feature is the diamond shaped high tensile steel wire mesh with an individual wire strength of at least 1’770 N/mm² (right).

The nailing was adapted to the static requirements based on the investigation of the overall stability considering sliding mechanisms with deeper-seated sliding surfaces. In addition to this, one needed to check if the flexible slope stabilisation system consisting of the mesh cover and corresponding system spike plates in combination with the existing nailing can withstand all stresses as a result of superficial instabilities. Based on the RUVOLUM® dimensioning concept world-wide published and accepted, local instabilities between the single nails as well as superficial slope-parallel instabilities as shown in figure 8 had to be investigated. Thereby, all proofs of bearing safety could be fulfilled (Flum et al., 2004 and Cala et al., 2012).
6 Revegetation

The application of a vegetation layer is limited by the soil or rock properties and is also dependent on a certain amount of water supplied from rainfalls and groundwater following the slope layers. Furthermore the consideration of the regional micro climate is a very important factor for the selection of the seed to be applied (Rüegger et al., 2001).

The steeper the slope cuttings are the harder it is to raise up a durable vegetation. The system has to be flexible to be able to slightly move under frost effects. This is not possible with a shotcrete cover. Additionally water exist at the surface of the slope should be taken over the full surface and be guided back to the natural circular cause (Rorem and Flum, 2003).

Based on the underground characteristic and the climate conditions, the organic mass has to be applied with high water restoring capacity. Because of the slope steepness, the erosion stability during strong rainfalls and frost is one of the most important points.

The seed mixture has to be specially adapted to the local conditions for getting a successful regreening. Aim of the first step is a fast surface covering greening, which is so planned that the used species can develop, during different steps of evolution, to a locally adapted dry biosphere.

Since this solution is based on the complete removal of the shotcrete coverage, a proper connection of the vegetation layer to the natural slope surface is guaranteed. This fact is very important for a continuous and sufficient water supply of the vegetation and also for the most reliable long term solution.

In steep slopes featuring fine-grained, non-cohesive loose rock or severely weathered rock there is a danger of erosion. Such fine material can be washed through the TECCO® mesh and flushed away underneath it. Hereby channels and hollows may be formed under the mesh.
The cause is emerging hillside, layer or fissure water, or in otherwise dry slopes also drain water from heavy rainfalls. Such water must generally be captured and drained. Permanent water outflows will always lead to problems and must be coped with before the slope stabilisation measure is started, since corrective action is hardly possible afterwards. Particular care must also be taken that no larger quantities of surface water from above flow over the slopes. If appropriate, drain channels must be provided above the edge of the slope so that the water is drained to the side in a controlled manner.

### 7 Conclusion

Two years from installation with dry and hot summers and cold, wet winter weather the slope is establishing a continuous vegetation cover itself, even so it is built into flysch mainly consisting of sand and siltstones. After an initial grass greening the local vegetation is slowly establishing back onto the stabilised cut slope with bushes and shrubs. No additional irrigation or revegetation has been carried out during this period. The visual impact of the slope as described is much more appealing compared to the big grey patches formed by the older shotcrete slopes.

Designed to be maintenance free, the slope will further grow into the surrounding landscape and contribute to a safe and economical operation of Egnatia Highway. Numerous applications have proven that the fully designable TECCO® slope stabilisation SYSTEM³ can ideally combine slope stabilisation with revegetation measures tailored to the actual climatic and environmental conditions.
References


Slope Stabilization and Erosion control of Gullies in Edo State, Nigeria

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Abstract

Numerous cases have been reported of destruction of properties and loss of lives due to formation of gullies in Nigeria. Subsequently, studies have been undertaken to understand the causes of these gullies as well as possible mitigation factors, however the government required cost effective solutions to stabilize existing gullies and prevent further land degradation of Nigeria soils. Aurecon was appointed by the World Bank for engineering, design and supervision services as part of the Nigeria Erosion and Watershed Management Project (NEWMAP) which aims to reinstate eroded lands and minimize long-term vulnerability.

Geotechnical studies undertaken at 6 different sites at Edo State confirmed that most gully sites are underlain by clayey sands of lateritic origin. To prevent further degradation of lateritic soils, various options for conveyance of flows were considered. It was recommended that the sides of the gully should be reshaped to a gradient that will achieve the required global Factor of Safety of 1.50. It was also recommended that extensive use of vegetation will improve the soil’s resilience to erosion.

Keywords: Slope-Stability, Nigeria, Gully, Erosion, Laterite.

1 Introduction

The formation of gullies due to excessive erosion in Nigeria has led to a number of deaths and destruction of property every year (Ibitoye & Adegbuyega, 2012). A number of studies have been conducted to understand the causes of gully formation as well as to investigate possible mitigation factors (Mangabara, 2012). Notwithstanding these studies the government required cost effective solutions to stabilize existing gullies and prevent further land degradation of Nigerian soils (Afegbua et al., 2015).

Following various assessments of this life-threatening challenge, Aurecon was appointed by World Bank and Edo State Government for engineering, design and supervision services as part of the Nigeria Erosion and Watershed Management Project (NEWMAP) which aims to reinstate eroded lands and minimize long-term vulnerability. Geotechnical engineering forms
a major part of this project with the objective to conduct slope stability analyses of reshaped gullies to design safe slopes (Aurecon, 2015).

Nigeria experiences a tropical climate defined by very high temperatures and heavy rainfalls. In most tropical areas laterite soils are formed from the weathering of parent igneous rocks, metamorphic rocks and sedimentary rocks, producing rusty red to reddish brown soils which are rich in iron and aluminium. Geotechnical studies done at 6 different sites at Edo State confirmed that most gully sites are underlain by clayey sands of lateritic origin (as seen in Figure 1 below). Gullies in Edo State developed as a result of poor road drainage which collected stormwater from the catchment area and concentrated them to the lowest point.

![Gully at Ewu site](image)

Figure 1. Gully at Ewu site

To prevent further degradation of lateritic soils in Edo State, various options for conveyance of flows were considered. It was recommended that diversion channels and discharge chutes should be constructed at the gulley head and the sides of the gully should be reshaped to a slope that will achieve the required global Factor of Safety of 1.50. It was also recommended that extensive use of vegetation will improve the soil’s resilience to erosion.

The geotechnical design which comprised slope stability analysis was performed to assess the safety of the eroded gulley and terraced backfill whereby it was concluded that the recommended geometries of the slopes are sufficient to achieve the required Factors of Safety and to maintain the gully slopes stability.

The main objective of this paper is to present findings from the geotechnical investigations conducted at various gullies in Edo State and also provide geotechnical design comprising of slope stability analyses.

2. Site location and geology

2.1 Site location
Edo State is located in the southern part of Nigeria coordinated at 6°30’N 6°00’E. It is positioned at the south west of Abuja and west of Niger River. More than 110 sites with cases
of eroded gullies across Edo State have been reported (Aliu & Egbejule, 2012) however, this paper focuses on six (No.6) sites in Edo State which were investigated namely:

The Ibore site is located in the Esan Central District, approximately 6 km to the east of Ekpoma-Auchi Road. A large erosion gully has grown to a depth of approximately 20 m and has caused the collapse of a main road and several buildings in the residential area of Ibore.

Emu site is located in Emu town, approximately 95 km north east of Benin City and 2 km east of Agbor road. The problem at Emu relates to the formation and growth of a deep erosion gully that developed alongside a road that was under construction prior to formation of the gully.

Ewu is located about halfway between Benin City and Auchi, just north of the Benin City – Auchi road. A massive gully has developed near the lower part of Ewu town where the main road crosses the drainage line.

The Edo College site is located in the Ikpoba-Okha districts to the south-east of the center of Benin City. The problem at this site relates to the formation and growth of a deep erosion gully that is undermining the foundations of buildings at the college that resulted from uncontrolled runoff from a relatively small catchment area.

The Ambrose Alli University (AAU) site is located in the Esan West district, approximately 5 km to the west of Ekpoma on the Benin-Auchi Road. The problem relates to the development of a large erosion gully that has grown to a depth of approximately 10 m and threatens to undermine a road and other infrastructure within the university estate.

The Fugar-Agenebode Road site is located approximately 2 km to the east of the village of Fugar, on the road to the town of Agenebode in the Estako Central Local Government Area of Edo State. The problem relates to the formation of erosion gullies alongside the Fugar-Agenebode Road.

2.2 Climate
Edo State experiences a tropical climate, defined by very high temperatures and heavy rainfalls. Edo State normally receives about 2025 mm of rain per year, with most rainfall occurring during summer. It receives the lowest rainfall (9 mm) in January and the highest (338 mm) in September. Edo state is warmest in April with an average temperature of 27.5 °C and coldest in July with an average temperature of 24.5 °C. The average annual temperature is 26.1 °C.

This kind of climate influences the formation of laterite soils in tropical areas. Of utmost significance, tropical rainfall has been the major cause of gully formations at Edo State and other surrounding areas.

2.3 Geology
According to the published 1:2 000 000 Geological Map of Nigeria (NGSA, 1994) shown below in Figure 2, Ibore and Ewu sites are underlain by Imo Clay-Shale Group lithologies consisting of clay and shales with limestone intercalations of the Ewekoro Formation. The presence of clayey sands was confirmed by the geotechnical investigation and laboratory test results.

Emu and AAU site is underlain by Bende Ameki Group lithologies consisting of clay, clayey sands and shale of the Ilaro Formation. The presence of clayey sands was confirmed by the geotechnical investigation and laboratory test results. Fugar site is underlain by Asata Nkporo Shale Group lithologies consisting of shale and mudstones of the Nkporo Formation. Edo College Gully site is underlain by Coastal Plains Sands consisting of sands and clays of the Benin Formation. The presence of clays and sands was confirmed by the laboratory test results.
Clayey sands found at Edo state are of lateritic origin, which means they are formed by the chemical weathering of parent rock under hot and moist conditions. Under very wet conditions, calcite and silicates are washed out leaving iron oxide which gives the soils a distinctive red colour (Lyon Associates Inc, 1971).

Figure 2. The general geology map of Nigeria (NGSA, 1994)

3 Site Investigation

Geotechnical investigations were conducted between March 2016 and August 2016 at the six gully Sites in Edo State, Nigeria where field assessments and soil sampling was undertaken. The predominant material encountered across the sites is described as moist, reddish brown, loose to medium dense, slightly clayey sand of a lateritic origin. At Emu gully tail, a moist, creamy to greyish brown, firm sandy clay was encountered. No water was encountered in most of the sites.

Most sections of the gullies were not accessible due to vegetation, steepness and instability of the side slopes as seen in Figure 1 above. Soil samples were taken at various positions along the gullies and tested in the laboratory. The following tests were conducted; Sieve Analysis Test, hydrometer test, Atterberg limit test, specific gravity test, compaction test, California bearing ratio and undrained triaxial test. The soil samples taken were very similar in their grading and Atterberg limits and the details of three sites are given in the table below.
Table 1. Summary of laboratory results

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Sample No.</th>
<th>Depth (m)</th>
<th>Soil composition (%)</th>
<th>Atterberg limits</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>GM</td>
<td>LL</td>
<td>WPI</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gravel</td>
<td>Sand</td>
<td>Silt &amp; Clay</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0</td>
<td>68</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>33</td>
<td>12</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.89</td>
<td>36</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.91</td>
<td>35</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.92</td>
<td>34</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.88</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.89</td>
<td>47</td>
<td>27</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.08</td>
<td>36</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.08</td>
<td>37</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.97</td>
<td>26</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.95</td>
<td>27</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.96</td>
<td>26</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.95</td>
<td>26</td>
<td>10</td>
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<tr>
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<td></td>
<td></td>
<td>1.96</td>
<td>26</td>
<td>9</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td>1.97</td>
<td>26</td>
<td>10</td>
</tr>
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<td></td>
<td></td>
<td>1.95</td>
<td>27</td>
<td>10</td>
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<td></td>
<td></td>
<td></td>
<td>1.96</td>
<td>26</td>
<td>9</td>
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<td>1.97</td>
<td>26</td>
<td>10</td>
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<td></td>
<td>1.95</td>
<td>27</td>
<td>10</td>
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<td></td>
<td></td>
<td>1.96</td>
<td>26</td>
<td>9</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>1.97</td>
<td>26</td>
<td>10</td>
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<td></td>
<td>1.95</td>
<td>27</td>
<td>10</td>
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<td></td>
<td></td>
<td>1.96</td>
<td>26</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.97</td>
<td>26</td>
<td>10</td>
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<td></td>
<td></td>
<td>1.95</td>
<td>27</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.96</td>
<td>26</td>
<td>9</td>
</tr>
</tbody>
</table>

The laboratory results indicated that the site is underlain by clayey sands and are classified as ‘SC’ by the unified soil classification system (USCS). Typical strength parameters were derived from the available data and used to complete a preliminary slope stability analysis.

Based on the USCS classification, effective strength parameters were selected from the Swiss Standard (1999). The parameters for the fill material were conservatively selected also using the Swiss Standard. Varying effective friction angle and the effective cohesion values were used for sensitivity analyses. The details are given in the table below:

Table 2. Soil strength parameters used for modelling

<table>
<thead>
<tr>
<th>Soil</th>
<th>Effective Angle (Degrees)</th>
<th>Friction</th>
<th>Effective Cohesion (kPa)</th>
<th>Unit Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SC</td>
<td>32 (±4)</td>
<td>1 (+4)</td>
<td>19</td>
<td></td>
</tr>
<tr>
<td>Fill 90% Comp</td>
<td>32</td>
<td>1</td>
<td>17.5</td>
<td></td>
</tr>
<tr>
<td>Fill 93% Comp</td>
<td>32</td>
<td>1</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>Rip Rap</td>
<td>38</td>
<td>0.5</td>
<td>22</td>
<td></td>
</tr>
</tbody>
</table>

4 Slope stability analyses

Slope stability analysis is performed to assess the safety of man-made or natural slopes and their equilibrium conditions. Slope stability is the resistance of an inclined surface to failure by sliding or collapsing. The main objectives of slope stability analysis include: finding endangered areas, investigation of potential failure mechanisms, determination of the slope
sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics and designing possible remedial measures. Stability analyses can take many forms and may include finite element modelling, the use of empirical circular failure graphs and hand held calculation checks taking into account the soil cohesion, friction angle and ground flow conditions together with any imposed loads. Slope stability analyses were conducted to determine if the proposed slope gradients of the gullies meet the required acceptable Factor of Safety (FOS). The USBR Design Standard (2014) states that a long term steady state seepage condition requires FOS ≥ 1.5. It also states that Rapid drawdown conditions from the normal water surface to an inactive water surface require FOS ≥ 1.3.

Due to the nature of the gullies and catchment areas, it was recommended that gullies at AAU, Fugar and Edo College should be backfilled to the top. Therefore the following subsections present slope stability analyses at Ibore, Emu and Ewu sites.

Slope stability analysis was performed using GeoStudio Slope/W (Morgenstern Price method) for the Ibore, Emu and Ewu gully sites to determine whether the required Factor of Safety (FOS) is met for each of the cross sections. The cross sections are typically sloped at 1V:2H and will contain flowing water when the gully is operational. The presence of water has been accounted for in the analysis with the design level (at 1.85 m depth) taken as the 1:50 year flood event. No appreciable seepage flow is expected to occur within the in-situ soils beneath the slopes and therefore seepage flow has not been accounted for. Sensitivity analyses were conducted by varying the effective friction angle and the effective cohesion values.

4.1 Ibore

The results for Cross sections are presented in the table below. The current geometry of the gully sloped at 1V:2H is sufficient to achieve the required FOS values given the soil parameters derived from the Geotechnical Investigation Report.

<table>
<thead>
<tr>
<th>Location</th>
<th>Left Slope</th>
<th>Right Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without water</td>
<td>With water</td>
</tr>
<tr>
<td>CH 120</td>
<td>1.553&gt; 1.50</td>
<td>1.590&gt; 1.50</td>
</tr>
<tr>
<td>CH 340</td>
<td>1.649&gt; 1.50</td>
<td>1.619&gt; 1.50</td>
</tr>
<tr>
<td>CH 860</td>
<td>1.800&gt; 1.50</td>
<td>1.784&gt; 1.50</td>
</tr>
<tr>
<td>CH 1360:</td>
<td>1.641&gt; 1.50</td>
<td>1.603&gt; 1.50</td>
</tr>
<tr>
<td>CH 1820</td>
<td>1.892&gt; 1.50</td>
<td>1.905&gt; 1.50</td>
</tr>
</tbody>
</table>

Note: Left and right are defined looking down stream

4.2 Emu

The results for the analysis of typical cross sections is presented in Table 3 below. Analysis of cross sections yielded FOS values greater than 1.50 for dry slopes and for the slopes with water. The current geometry of the slopes is sufficient to achieve the required FOS values.
Table 4. Summary of FOS values for Typical Cross Sections at Emu site

<table>
<thead>
<tr>
<th>Location</th>
<th>Left Slope</th>
<th>Right Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without</td>
<td>With</td>
</tr>
<tr>
<td></td>
<td>water</td>
<td>water</td>
</tr>
<tr>
<td>Cross Section at CH 800</td>
<td>1.5= 1.5</td>
<td>1.5= 1.5</td>
</tr>
<tr>
<td>Cross Section at CH 860</td>
<td>1.6&gt; 1.5</td>
<td>1.6&gt; 1.5</td>
</tr>
<tr>
<td>Cross Section at CH 940</td>
<td>1.6&gt; 1.5</td>
<td>1.5= 1.5</td>
</tr>
<tr>
<td>Cross Section at CH 1040</td>
<td>2.0&gt; 1.5</td>
<td>1.7&gt; 1.5</td>
</tr>
</tbody>
</table>

Note: Left and right are defined looking down stream

4.3 Ewu
The results for the analysis of typical cross sections is presented in the table below. The current geometry of the slope is sufficient to achieve the required FOS values given the soil parameters derived from the Ewu Gully Investigation Report.

Table 5. Summary of FOS values for Typical Cross Sections

<table>
<thead>
<tr>
<th>Position</th>
<th>Left Slope</th>
<th>Right Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without</td>
<td>With</td>
</tr>
<tr>
<td></td>
<td>water</td>
<td>water</td>
</tr>
<tr>
<td>CH 140</td>
<td>2.778 &gt; 1.50</td>
<td>2.750 &gt; 1.50</td>
</tr>
<tr>
<td>CH 180</td>
<td>2.517&gt; 1.50</td>
<td>2.563&gt; 1.50</td>
</tr>
<tr>
<td>CH 180</td>
<td>1.569&gt; 1.50</td>
<td>1.548&gt; 1.50</td>
</tr>
<tr>
<td>CH 260</td>
<td>1.905&gt; 1.50</td>
<td>1.966&gt; 1.50</td>
</tr>
<tr>
<td>CH 520</td>
<td>1.800&gt; 1.50</td>
<td>1.767&gt; 1.50</td>
</tr>
<tr>
<td>CH 740</td>
<td>1.641&gt; 1.50</td>
<td>1.603&gt; 1.50</td>
</tr>
</tbody>
</table>

Note: Left and right are defined looking down stream
The typical models and slope stability results at different gully sites are presented below:

**Profile and Water Levels with Critical Failure Surface**

Ibore CH 660, Right slope

![Ibore CH 660, Right slope](image)

Emu CH 940 Left slope with water

![Emu CH 940 Left slope with water](image)

Ewu CH 260 Left slope without water

![Ewu CH 260 Left slope without water](image)

**Sensitivity**

![Sensitivity Data](image)

Figure 3. Typical models and slope stability results at different gully sites
5 Recommendations and conclusions

Analysis of gullies cross sections yielded FOS values greater than 1.50 for dry slopes and for the slopes with water. The proposed geometries of the slopes are sufficient to achieve the required FOS values. It is recommended that backfill material will be stabilized by shaping the earthworks for improved drainage resistance gullies. The earthworks will be compacted to a minimum mod AASHTO density of 93%, and where necessary, the surface will be vegetated. The flow will be managed as sheet flow to reduce erosive streamflow.

It is also recommended the extensive use of vegetation to improve the soil’s resilience to erosion. The grass species to be planted are *Vetiveria zizanioides* and *Pueraria sp* and the tree species is *Acacia sp*. This species, which has a wide distribution, can be a tree or a shrub. It is also used as a pioneer species in land rehabilitation, as it is very resilient and able to tolerate extreme temperatures and rainfall. The figure below presents typical cross sections of the gully with recommended vegetation.

![Figure 4. Typical cross section of the gully with vegetation (Design report, 2016)](image)

References


Use of Lime Piles as an Alternative Method for Stabilisation of Road Embankments

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Abstract

The durability and performance of an embankment depends on its stability. Several options are available for controlling stability and settlement problems associated with embankment slopes. One of them involves using stabilising agents which are suitable for the existing embankment. This research focused on improving the engineering properties of clay soil in situ by using lime pile technique. The clay soil was obtained from a failed embankment along Kamwenge – Fort portal road, chainage 18 + 900. Preliminary tests were carried out to determine if the soil required stabilisation. It had a high liquid limit of 58.6%, plastic limit 26.5% and plasticity index of 32.1. It was classified as CH using the Unified Soil Classification System. Various tests were carried out, for curing days of 14, 21 and 28, to investigate the effect on the engineering properties of the soil. Results showed an increase in Maximum Dry Density, shear strength and a decrease in Optimum Moisture Content and plasticity index hence improved soil properties for embankment slope stability.

Keywords: Embankment, Slope stability, Lime piles, Soil improvement

1 Introduction

Failure of embankments is mainly due to clay soil associated with high organic matter, low bearing capacity and high compressibility that exists in unconsolidated condition leading to excessive settlement. Various options are proposed for embankment slope stability including; re-gradation of slope material to a more suitable angle, removing all slumped and unstable materials, benching slopes, constructing berms of soil/gravel at the toe of the cut, provision of appropriate vegetation on the slope (including fast growing shrubs), drainage measures to intercept surface and seepage water and lead it away from the slope, repairs to the damaged road section and also reconstruction of the concrete lined side drain. However, all these don’t change the soil properties for a long lasting solution to the problem. Stabilisation techniques, on the other side, increase the shear strength of the soil. It involves the use of various stabilising agents. This paper investigated the use of lime piles to stabilize an embankment slope.
According to Kennedy et al. (1987), the geotechnical behavior of lime treated soils depend on their physical and chemical properties which is related to soils formation conditions and the mineralogical compositions of the matrix.

2 Lime Stabilisation

2.1 Definition
Lime stabilisation is one of the commonest and most economical techniques of soil stabilisation. This technique is unique in that, lime reacts with the soil forming a two material system. This technique may be applied for heavy wet clays during construction of road sub bases by providing a stable working platform. Many significant engineering properties are improved with lime treatment and they include; increase in soil bearing capacity, reduction in shrinkage properties of the soil, reduction in plastic index, reduction in soil compressibility and immobilization of heavy metals. There are three main common types of lime which include quick lime, hydrated lime and slurry lime. Lime has been used as deep stabilizer in several countries for example Sweden, Japan, USA among others and the stabilisation techniques include:

- Lime slurry pressure injections
- Lime columns
- Lime piles

Lime stabilisation is suitable for soils having plasticity index above 10 (Christopher et al., 2006)

2.2 Lime Piles
These are basically holes augured in the ground filled with lime. These use a mechanism of cation exchange in their work. Ingles and Metcalf (1972) showed one method of lime pile construction, in which a hollow tube is pushed into the soil to the required depth of pile and quicklime is forced into the tube under pressure as it is withdrawn. The other method is auguring holes in the ground and filling them with lime and compacting in layers for stabilization.

2.3 Mechanism of Lime Pile Stabilisation
Mechanism of stabilisation proposed by several authors include pile expansion and clay hydration.

Lime Pile Expansion: Quicklime in the piles reacts with the water in the in-situ soils, drawing excess water from neighborhood of the piles. This leads the piles to expand due to reaction, causing lateral consolidation of the nearby clay. According to authors that proposed the mechanism, using lime piles improves the bearing capacity and settlement characteristics of soft ground. $\text{CaO} + \text{H}_2\text{O} \rightarrow \text{Ca(OH)}_2$

Clay Hydration: The migration of calcium ions from the pile into the surrounding clay is aided by soil moisture. For the clay-lime reaction to take place, clay must be in a highly alkaline condition at a minimum pH of 12.4. Clay particles have negatively charged surface absorb positively charged calcium ions forming a slaked lime.

3 Material and Methods

3.1 Clay Soil
Clay soil was sampled along Kamwenge - Fort portal road at km 189 + 900. This was carried out using axel, spade and sacks at a depth of 1 m from the ground within the road shoulders. Several tests were undertaken out on the neat sample and lime pile treated samples to evaluate the effect of lime piles on the engineering properties. Table 1 shows the chemical composition
of the obtained soil sample. The main component was Silica (SiO$_2$) at 55.65%. Table 2 shows the index test values of the clay soil.

Table 1. Chemical composition of the obtained clay soil

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Composition (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alumina Al$_2$O$_3$</td>
<td>20.16</td>
</tr>
<tr>
<td>Calcium Oxide CaO</td>
<td>0.59</td>
</tr>
<tr>
<td>Iron Oxide Fe$_2$O$_3$</td>
<td>7.47</td>
</tr>
<tr>
<td>Magnesium Oxide MgO</td>
<td>0.73</td>
</tr>
<tr>
<td>Manganese Oxide MnO$_3$</td>
<td>0.97</td>
</tr>
<tr>
<td>Silica SiO$_2$</td>
<td>55.65</td>
</tr>
</tbody>
</table>

Table 2. Soil classification properties.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit (%)</td>
<td>58.6</td>
</tr>
<tr>
<td>Plastic Limit (%)</td>
<td>26.5</td>
</tr>
<tr>
<td>Plastic Index (%)</td>
<td>32.1</td>
</tr>
<tr>
<td>Percentage Retained (%)</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>17.4</td>
</tr>
<tr>
<td>40</td>
<td>80.7</td>
</tr>
<tr>
<td>10</td>
<td>1.9</td>
</tr>
</tbody>
</table>

The soil was classified using the Unified Soil Classification System. From the sieve analysis test, 17.4% of the soil sample was retained on sieve No. 200 (75µm) which was less than 30% but greater than 15% as sand dominated more than gravel. Thus the soil was classified as CH, which is inorganic fat clay with sand.

3.2 Lime

Hydrated lime, Figure 1, was obtained by slaking quick lime with water. Hydrated lime was used as a chemical stabilizer to modify the properties of the soil and was bought from a hydrated lime dealer.

3.3 Sample Preparation

Using an evaporating dish, stove and digital weighing balance, the sample from 1 meter deep was measured and evaporated until no moisture was indicated on glass plate cover. The evaporated soil sample was left to cool and weighed again. The difference in weight was noted as weight of in-situ moisture content.

The extracted soil was air dried and then mechanically ground into a pulverized form. It was then mixed with water equal to the amount of the in-situ moisture. The prepared wet soil was placed in an airtight polythene bag for 24 hour to attain uniform moisture distribution and prevent moisture loss.

The wet soil was then compacted with the modified rammer in four equal layers in the test tank (1000mm*1000mm) to a predetermined in situ bulk density using core cutter and water content values to simulate the natural field conditions of the sample.
3.4 Lime Pile Installation

This was done in the laboratory, Figure 2, involving the installation of nine piles in the compacted soil block. Each of these piles were 100mm in diameter and 350mm height. It was done using a hollow polyvinyl chloride (PVC) pipe that had openings at both ends. The PVC pipe had internal diameter of 100mm so as to aid the penetration into the soil block and create the hole without interfering with the properties of the compacted soil block. Each drilled hole was filled with hydrated lime of uniform mass applied in three equal layers and lightly compacted to form the lime piles. The piles were spaced at 180mm center to center.

The setup was then covered with synthetic fabric and then after lake sand on top to facilitate the physicochemical reactions between lime and the clayey material. This also helped to minimize sudden lateral expansion of the lime piles. The lime and soil block absorbed water and calcium and magnesium ions were diffused in each pile. The soil block was left to cure for a period of 14, 21 and 28 days.
3.5 Sample Extraction
The sample was extracted between the lime piles using extrusion devices at different curing periods. During the sample extraction, the soil specimens were handled carefully and extracted at slow rate to prevent disturbance, openings or voids, loss of water content and cracking in the samples. The samples were immediately kept in air tight polythene bags after extrusion before conducting different tests.

3.6 Laboratory
Different laboratory tests were carried out on the lime treated samples including classification tests, compaction tests and strength tests. The tests were carried out in reference with BS 1377.

4 Results and Analysis
4.1 Effect of Lime Piles on Atterberg Limits
Figure 3 shows the variation of the liquid and plastic limits of the soil sample due to the lime piles for the different curing days that is 0, 14, 21 and 28 days. The neat soil sample was found to have a liquid limit of 58.6% which consequently reduced by 5.1%, 4.9%, and 2.7%. The plastic limit of the neat sample was found to be 26.5% which decreased with increase in curing days by 10.2%, 7.1% and 3.6%. These variations were because of reaction formed calcium silicate hydrates of high surface area and crystallized calcium aluminate hydrate which are larger than the initial particle size of soil sample. The increase of particle sizes continuously in the soil block consequently reduced the fines thus reducing liquid and plastic limits.
Figure 3. A graph of liquid limit and plastic limit against curing days

4.2 Effect of Lime Piles on Compaction Parameters
Figure 4 shows the variation of maximum dry density (MDD) and optimum moisture content (OMC) with curing days. The maximum dry density of neat soil sample was 1.801 kg/m$^3$. It increased by 22.2%, 27.3% and 7.8% for 14, 21 and 28 days of curing. The maximum dry density increased with increase in curing days due to the increase in bond strength in the flocculation and agglomeration in soil block resulting from the physicochemical reaction hence causing mineralogy changes. The reactions formed calcium silicate hydrates of high surface area and crystallized calcium aluminate hydrate of high strength and larger than initial particle size of soil sample. This reduced the amount of fines in the soil block which consequently increased the maximum dry density as the curing days increased.

There was a decrease in the optimum moisture content in the soil block as the curing days increased. This was caused by flocculation and increased surface area of soil particles which increased the volume of voids in the soil while reducing amount of fines that absorb moisture. And also the increase in hydroxyl ions in the surrounding soil mass which decreased the affinity of surface of clay particles to water consequently decreasing water demand thus reducing optimum moisture content from 13.4% for the neat sample by 9.7%, 3.3% and 10.2% within the 14, 21 and 28 days.

4.3 Effect of Lime Piles on the Shear Strength Parameters
The neat sample gave initial shear strength parameters of cohesion of 19 kPa and angle of internal friction of 13°. The migration and diffusion of calcium ions from the lime piles caused pozzolanic reaction, altering the mineralogy and physicochemical properties of the soil. With the use of lime piles which dehydrated surrounding soil mass under evaporation caused by increased temperature and also reduced percentage of fines by agglomeration forming high strength crystallized with increased soil particle sizes. This significantly increased the soil cohesion by 20.8%, 4.0% and 7.4% within 14, 21 and 28 days. The angle of internal friction increased by 7.7%, 12.5% and finally 11.1% with in 14, 21 and 28 days of curing. Figure 5 shows the variation of shear strength parameters with curing days.
5 Conclusion

There was signified reduction of water permeability in the soil which also increased the stability of the soil to be used for road embankment. The water required to achieve maximum compaction was reduced however water content for ion migration was increased which required attention. There was increase in cohesive strength between the stabilised soil particles which also contributed to the stability of the embankment. The internal angle of friction of soil particles increased which reduced the ability to slide off on the road section.

References

Reliability Assessment of Limit State Designed Slopes Using First Order Reliability Methods Coupled with Response Surface Methodology

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Abstract

Slope stability analysis is traditionally undertaken using limit equilibrium methods (LEM) and a lumped factor of safety (FOS) approach, which has been found to be a poor indicator of stability. Reliability methods effectively assess the performance of a design and its probability of failure but have not been widely used in South Africa due to the complexity of transformed space and the statistical knowledge required for such assessments. The use of a response surface method (RSM) is confirmed to be a suitable and relatively simple bridge between LEM slope stability assessment methods and the first order reliability method (FORM) of reliability analysis. The application of FORM is illustrated in an a priori silty slope which has been designed according to EC7 principles. Although a deterministic minimum FOS of 1.43 is calculated for this slope, lower than typically required, the probability of failure is acceptable according to South African standards.

Keywords: FORM, response surface, slope stability

1 Introduction

1.1 Background

Slope stability is traditionally assessed using a lumped factor of safety (FOS). This index has been shown to be a poor measure of safety with slopes of similar FOS values having widely differing margins of safety (Whitman, 1984).

Limit state design using partial factors presents a superior approach in which uncertainties are accounted for in a more robust way. Many countries have followed the example of the European Union who, after realizing the need for a geotechnical code compatible with structural engineering, introduced EN 1997-1. This geotechnical code is based on the limit state design frame-work laid out in EN 1990. Limit states design is increasingly being implemented in the geotechnical engineering sector in South Africa after provision was made for geotechnical design in Part 5 of SANS 1060 in 2010.
As the scope of SANS 10160 covers buildings and industrial structures, no provision is made for slope stability analysis in SANS 10160-5 and designers of slopes are referred to EN 1997-1 for guidance (Day, 2013). The geotechnical division of SAICE is currently looking to increase the scope of the current geotechnical design code to include a wider range of geotechnical structures (Day and Retief, 2009). In order to develop design codes, a reliability assessment of the limit state design approach is required to ensure the probability of failure of each design is maintained below a certain threshold (EN 1997-1).

The simplest form of reliability assessment is to statistically assess the capacity and demand of a system as separate random variables. In order to meet reliability requirements, the resistance of a system is to exceed the effect of the actions acting on it with a certain degree of confidence, an entity which has a quantitative value when variables are described statistically.

In the case of many geotechnical problems, including slope stability assessment, the capacity and demand are not independent, as both are affected by the self-weight of the ground. In these circumstances, the introduction of a limit state is required to assess reliability through the use of approximate reliability assessment methods.

The first order reliability method is useful due to its invariance to the formulation of the limit state. Reliability can be calculated as the minimum distance from the origin to the limit state function in a standardized cartesian space (Hasofer and Lindt, 1974) This is highlighted in Figure 1 showing a dispersion ellipse (or equivalent form for $n$ parameters $>2$) of one standard deviation ($1\sigma$) in dimension, in the original space of 2 random variables (which in this study comprise $c'$, $\tan\phi$, $\gamma$ and $Q$) with a center corresponding to the means of each variable. The ellipse will have each of its two ($n$) axis at an angle to the corresponding co-ordinate axis determined by the correlation of the variables (Low, 2007). The point of tangency to the ellipse and the $1\sigma$ dispersion ellipse are measured from the mean; the quotient of these values being equal to the $\beta$ value, providing a visual representation of the reliability (Low and Tang, 2007).

![Figure 1. Reliability index determined in the original space of random variables X1 and X2.](image)

The application of limit state design of a slope compatible with EN 1997 and SANS 10160 principles was illustrated in this study using an a priori slope, first presented by Länsivaara and Poutanen (2013). The slope was designed using procedures laid out in EN (1997-1), applying the partial factors stipulated in SANS 10160-5 for the ‘GEO’ limit state, together with ancillary works for guidance (e.g. Frank et al., 2004; Schuppener, 2007). This was followed by a reliability assessment of the design through FORM coupled with a response surface methodology.
1.2 EN 1997-1 design

To take uncertainty into account, the design of geotechnical structures is undertaken through careful description of mean values, the selection of characteristic values and the application of partial factors to attain design parameters which impart a suitably high degree of safety to the design (ENV, 1997-1, ISO 2394:2015). The verification requirement for the ultimate limit state according to EN1990 and SANS 10160 is given by Equation 1:

\[ E_d > R_d \]  \hspace{1cm} (1)

Where \( R_d \) is the design action and \( E_d \) is the design resistance. \( R_d \) and \( E_d \) are determined using the procedures given in EN 1997-1.

The mean value is the first moment or central tendency representing the average value of the relevant parameter. If sufficient testing is undertaken within a soil horizon for instance, this value would equal the 'true mean' of the soil that the model is representing (Schneider, 1999). This is typically not the case as testing is generally limited by cost and time restraints, creating a degree of uncertainty when assigning this value.

EN 1997-1 defined the characteristic value of a variable as “a cautious estimate of the value affecting the occurrence of the limit state”. If statistical methods are used, the characteristic value should be chosen such that the likelihood of a worse value controlling the occurrence of the limit state is less than 5%. This selection must take account of the variability of the parameter, the volume of ground affected and the number of test results. The characteristic value may conveniently be expressed as shown in Equation 2:

\[ X_k = X_m[1-nV_x] = X_m - n\sigma \]  \hspace{1cm} (2)

Where \( X_k \) is the characteristic value, \( X_m \) is the sample mean and \( V_x \) is the coefficient of variation. Schneider (1999) proposed that where the value of the parameter is averaged over a large volume of ground, the parameter varies in a homogeneous random manner and at least ten test results are available, \( n \) may be taken as 0.5, i.e. the characteristic value may be taken as 0.5 standard deviations below the mean value.

Limit states design requires the application of a partial factor on the characteristic values which takes account of uncertainties influencing each parameter. These uncertainties include measurement error, statistical uncertainty, transformation or model uncertainty as well as the natural variations occurring within the subsurface which are likely to affect the characteristic parameter value (Länsivaara and Poutanen, 2013). This process involves reducing parameters such as the shear strength and/or increasing loads using partial factors according to the principles given in EN (1997-1) or SANS 10160-5.

EN1997-1 allows for three different design approaches, each with its own set of partial factors. South Africa, like the United Kingdom, uses Design Approach 1, Combination 2 for the GEO limit state. In the case of slope stability, this approach is identical to Design Approach 3 which is used by the majority of EU countries for slope design. All EU countries have agreed on the use of a partial factor of 1.0 on weight density as prescribed in Table A.4 of EN 1997-1 (Schuppener, 2007). This is because of difficulty in correctly factoring the influence of the weight of soil (Länsivaara and Poutanen, 2013). SANS 10160 uses a partial factor of 1.25 on effective strength parameters (\( c' \) and \( \phi' \)) in the GEO limit state and partial factor of 1.0 and 1.3 on permanent and variable actions respectively.
Reliability analyses such as FORM can be used to assess whether the partial factors have suitably increased the safety of the design to adequately take uncertainties into account.

The target reliability value of $\beta=3$ has been adopted in South Africa (Retief & Dunaiski, 2009). This is lower than the $\beta$ target values of 3,8 in EN 1990-1, prescribed for class R2 structures such as residences and offices (EN, 1990-1). However, the financial implications in the context of South Africa means that the target $\beta$ value of 3 and the associated failure probability of 0.1% is unlikely to change (Dithinde and Retief, 2016).

2 Methodology

2.1 Case study

The case study involved a slope stability problem adapted from an assessment by Länsivaara and Poutanen (2013), comprising an approximately 10m high slope consisting of a single layered homogenous, silty soil with mean effective strength parameters of $\phi=26^\circ$, $c'=8kPa$ and a unit weight of 18 kN/m$^3$. A 10m wide surcharge is located 3m from the crest of the slope with a characteristic value of 50 kPa. No water table is present.

A deterministic assessment of the slope using LEM software is undertaken to determine the critical slope dimensions that satisfy the requirements of limit state design. The mean, characteristic value and design values based on EN 1997 / SANS 10160-5 are given in Table 1.

![Table 1. Summary of limit state design values](image)

Figure 2. Limit state deterministic design of the problem slope

To determine the most critical design, the slope is increased in height and gradient, using the design soil parameters and design actions, until the over-design factor ($R_d/E_d$) is approximately equal to unity, resulting in a maximum height of 10.92m as per Figure 2.
2.2 Response surface methodology
To assess the reliability of the design using FORM, an explicit performance function separating
the safe and failure limit states was approximated. This can be effectively achieved using
response surface methodology to solve a polynomial such as that in Equation 3, by least squares
regression, using results from a number of LEM assessments as inputs.

\[
g = g(x) = a + \sum_{i=1}^{n} b_i x_i + \sum_{i=1}^{n} c_i x_i^2
\]

where \( g \) is an approximate response surface function, \( n \) the number of basic variables,
and \( x \) vector of input variables (\( x_1, \ldots, x_n \)). The coefficients \( a, b \) and \( c \) were determined using
a least squares method as described by Myers et al. (2009).

For the coefficients to be determined, a minimum of \( 2n+1 \) results were required. Statistical
consideration is given to determine the sampling points, resulting in sampling undertaken at \( x_i \)
(mean) points ± \( f \sigma \) points where \( f \) is a sampling factor (Bucher and Bourgond, 1989). A
sampling factor of between 1 and 2 was used in this study.

The \( y \) response values were determined for each point in the experimental design using the
Spencer’s limit equilibrium method in SLOPEW (GEO-SLOPE International Ltd., 2016).
Spencer’s method is one of many prescribed by EN 1997-1, however, other rigid LSM or FEM
methods advocated by EN 1997-1 could be employed.

The polynomial used to approximate the response can be of first or second order, with the
matrix formulation adjusted accordingly. A quadratic second order polynomial without cross
terms is typically used to approximate the performance function in similar cases with only the
terms that influence performance included in the regression (e.g. Xu and Low, 2006).

2.3 FORM methodology
The FORM method was efficiently implemented using the Microsoft Excel add-in, ‘solver’, to
perform a constrained optimization process, determining the reliability index using Equation 4
(Low and Tang, 2007):

\[
\beta = \min_{x \in \mathbb{R}} \left[ \frac{x_i}{\sigma_i} - \frac{\mu_i}{\sigma_i} \right]^T R^{-1} \left[ \frac{x_i}{\sigma_i} - \frac{\mu_i}{\sigma_i} \right]
\]

Where \( \beta \) represented the Hasofer Lindt (1974) reliability index, \( R \) represented the correlation
matrix (\( cm \)), \( x_i \) the critical point on the limit state surface, \( \sigma_i \) and \( \mu_i \) the standard deviations and
means of the input parameters respectively, as illustrated in Figure 3. The limit state \( g(x) \) in this
investigation represented the combination of variables at which the slope attained equilibrium,
determined using the Spencer LEM.

Table 2. Statistical parameters used in this study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>COV ( \phi )</th>
<th>COV ( c' )</th>
<th>COV ( \gamma )</th>
<th>COV Q</th>
<th>( \rho_{\phi c'} )</th>
<th>( \rho_{\gamma c'} )</th>
<th>( \rho_{\phi \gamma} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>0.1</td>
<td>0.4</td>
<td>0.05</td>
<td>0.4</td>
<td>-0.49</td>
<td>0.14</td>
<td>-0.63</td>
</tr>
<tr>
<td>Source</td>
<td>a</td>
<td>a</td>
<td>a</td>
<td>b</td>
<td>c</td>
<td>c</td>
<td>c</td>
</tr>
</tbody>
</table>

The standard deviation values were derived from the relevant coefficient of variation (COV) values indicated in Table 2. The high COV value prescribed to the surcharge load $Q$ is due to the uncertainty arising from loading associated with wind and snow (Länsivaara and Poutanen, 2013). The correlation coefficients were sourced from laboratory tests on impervious borrow material undertaken by Holtz and Krizek (1971).

The statistical distributions of $\phi$ and $c'$ were taken as log-normal to ensure that unacceptable negative values were avoided Uzielli et al., (2007). The probability density functions (pdf) of $\gamma$ and load were represented by normal distributions (Lacasse and Nadim, 1996; Poutanen, 2011).

Once the statistical parameters were finalized, the solver function was invoked to change cells containing $x_i$ values with the constraint that $g(x)$ equals 0. This constrains possible $x_i$ combinations to those lying on the limit state defining the boundary between stable states and failure states. The limit state is explicitly represented by a performance function which for this study takes the form of a second order polynomial. The third condition is to minimize $\beta$, which corresponds to the largest probability on the cumulative pdf and as such, the most probable failure point (MPP) on the limit state (Low and Tang, 2007).

Figure 3 provides the spreadsheet formulation used in the final iteration of the FORM analysis. The $\mu$ and $\sigma$ values represent the equivalent normal mean and standard deviations of their respective lognormal counterparts and are calculated using a VBA code of a typical Rackwitz Fiessler transformation. The calculation for the $vn$ term is described by Equation 5.

$$vn = (x_i - \mu_i^N) / \sigma_i^N$$

(5)

Where $vn$ is the distance in normalized standard deviation units between the $x^*$ and the respective normalized mean.

<table>
<thead>
<tr>
<th>$\tan\phi$</th>
<th>$x^*$</th>
<th>$\mu$</th>
<th>$\sigma$</th>
<th>$\mu^N$</th>
<th>$\sigma^N$</th>
<th>$vn$</th>
<th>correlation matrix $[cm]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>LN$^1$</td>
<td>0.38</td>
<td>0.49</td>
<td>0.05</td>
<td>0.47</td>
<td>0.04</td>
<td>-2.35</td>
<td>1.00 -0.49 -0.63 0.00</td>
</tr>
<tr>
<td>LN$^1$</td>
<td>4.29</td>
<td>8.00</td>
<td>3.2</td>
<td>6.64</td>
<td>1.65</td>
<td>-1.43</td>
<td>-0.49 1.00 0.14 0.00</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>N$^2$</td>
<td>19.9</td>
<td>18.0</td>
<td>0.9</td>
<td>18.0</td>
<td>0.9</td>
<td>2.16</td>
</tr>
<tr>
<td>$Q$</td>
<td>N$^2$</td>
<td>71.5</td>
<td>50.0</td>
<td>20.0</td>
<td>50.0</td>
<td>20.0</td>
<td>1.08</td>
</tr>
</tbody>
</table>

$[cm]^{-1}$  

$[vn]^T$  

$[cm]^{-1}[vn]$  

$[vn][cm]^{-1}[vn]$  

$\beta$ 3.92  

1Lognormal distribution  

2Normal distribution

Figure 3. Spreadsheet formulation of FORM method

The MPP on the limit state and the RSM performance functions themselves are both approximations. An iterative technique was required to ensure convergence to the ‘real’ surface and improve accuracy. To check whether the obtained function correctly represented the performance of the corresponding slope, the $x^*$ point was used as an input into SLOPE/W (2106) to check the validity the FOS equaled unity and that $g(x)$ equaled 0.
Where this was not satisfied, the RSM analysis was undertaken again using another experimental set. The second experimental design used $x_n^*$ values as the central points to generate the subsequent response surface data. The resulting response was used in another FORM iteration until the MPP criterion was achieved. The starting $x_n^*$ value for each iteration of FORM in EXCEL should be the resulting $x_n^*$ value from the previous iteration. (The fist $x_n$ values should be set to their respective mean values).

After results generated from an iteration indicated that the $x^*$ point suitably represented the MPP and that the $x_n^*$ values were located on the limit state function, the $g(x^*)$ function was accepted as a good approximation (Low and Tang, 2007). The $P_f$ of the problem slope was subsequently calculated in Excel using the “NORMDIST” function.

### 3 Results

#### 3.1 Deterministic factor of safety
The factor of safety for the most critical slip surface associated with mean values of all parameters was calculated as 1.43 when using the deterministic Spencer’s method to assess the limit state designed slope.

#### 3.2 Response surface methodology
The RSM analysis generated the terms for the performance functions of the limit state. These were used in the FORM analysis in the form of a second order polynomial as indicated by Equation 6.

$$
\bar{g}(x) = (1, \tan \phi, c', \gamma, Q, \tan \phi^2, c'^2, \gamma^2, Q^2)^* - 1
$$

#### 3.3 First order reliability method analysis
The results of the FORM analysis determined after 2 iterations are provided in Table 3. The final parameters related to the MPP were $x_\phi^*=28.54^0$, $x_c^*=0.00kPa$, $x_\gamma^*=17.86kN/m^3$ and $x_Q^*=50kPa$. The factor of safety checks for the $x^*$ values generated by the second iteration produced a FOS value of unity to a precision of 1 one thousandths of a unit. The $\beta$ value of 3.93 corresponds to a probability of failure of 0.004%.

<table>
<thead>
<tr>
<th>Iteration #</th>
<th>Correlation</th>
<th>$x_{\tan\phi}$</th>
<th>$x_{c'}$</th>
<th>$x_\gamma$</th>
<th>$x_Q$</th>
<th>FOS</th>
<th>$\beta$</th>
<th>$P_f$(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>see matrix#</td>
<td>0.40</td>
<td>4.54</td>
<td>19.68</td>
<td>79.38</td>
<td>1.04</td>
<td>3.50</td>
<td>0.02</td>
</tr>
<tr>
<td>2</td>
<td>see matrix#</td>
<td>0.38</td>
<td>4.29</td>
<td>19.94</td>
<td>71.53</td>
<td>1.001</td>
<td>3.93</td>
<td>0.004</td>
</tr>
</tbody>
</table>
3.4 MCS analysis

Monte Carlo Simulation (MCS) was undertaken in SLOPE/W (2016) using the same mean values, standard deviations, distributions and performance function as used in FORM. An assessment of all MCS simulations ranging from 1000 to 80 000 indicates a probability of failure of less than 0.1% with an associated degree of confidence of 0.88.

4 Discussion

The deterministic minimum factor of safety of 1.43 for the mean parameters and LS designed slope geometry was lower than the FOS value of 1.5, generally stipulated for long term stability.

Table 4. Summary of reliability assessments

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Case</th>
<th>Reliability index $\beta$</th>
<th>$P_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FORM</td>
<td>1- Lognormal</td>
<td>$\beta=3.93$</td>
<td>0.00004</td>
</tr>
<tr>
<td>MCS</td>
<td>2- Lognormal</td>
<td>-</td>
<td>&lt;0.001</td>
</tr>
</tbody>
</table>

The results of the FORM method indicated reliability values of 3.93 where the shear strength parameters were lognormally distributed. The probability of failure corresponding to this level of reliability was 0.004% whilst the MCS results indicated a $P_f$ of less than 0.1% for this problem. This results from the limitation of the computational precision of the MCS version used, not unexpected as MCS’s typically encounter inaccuracies at low probabilities of failure.

As such, the probability of failure determined by both FORM and MCS achieved the requirements laid out in SANS 10160 which stipulates a target minimum reliability index of 3, corresponding to a $P_f$ of around 0.001 or 0.1%. FORM takes considerably less computing time than the MCS method used (+2 hours vs 100’s -1000’s of hours) and is more precise. This has clear practical advantages.

The use of RSM to generate the performance function also provided insight into the effects that soil parameters and loading conditions had on the factor of safety and reliability of the design. The results of the RSM analysis indicated that both $c'$ and $\phi'$ had significant bearing on the performance and reliability whilst unit weight ($\gamma$) and load ($Q$) had significantly less impact. The structure of RSM allows for the input of any values (LEM or FEM), provided the experimental design and least squares matrix formulation are adhered to.

5 Conclusion

The use of reliability methods to assess design performance is superior to the traditional factor of safety approach because data-specific parametric effects on design can be assessed and a probability of failure can be calculated for the design. Where a central FOS value of 1.43 indicated an unsuitable slope design for long term stability, the FORM method indicated a probability of failure of 0.004% which was sufficiently low and indicated a safe design. The partial factors laid out in SANS 10160 were adequate to ensure safe slope design for this case study and the statistical distributions describing the materials involved.

Although the COV values in this study were based on published data, there lies a fair degree of inaccuracy resulting from the use of published values and care should be taken to accrue site specific data rather than published values.
Due to the differences expected in COV values from site to site, it is suggested that the use of partial factors to take uncertainty into account is not likely to ensure sufficient reliability across all slope stability problems. It is suggested that rather, reliability based design using FORM and principles laid out in this paper or statistical first principles, are employed for design. This is recommended until more statistical data describing soil parameters is made available in South Africa.

The FORM method was suitably employed to determine the reliability of a design. This was confirmed through comparison with MCS results. The FORM method is also more precise and accurate than MCS components of a commercially available probabilistic slope stability software.

The methodology is clear and can easily be employed by practicing engineers using readily available slope stability analysis programs together with Microsoft Excel, after invoking the solver add-in. The methodology used in this study allows the user to transparently assess the data at all stages. This includes manipulation of the data statistically through adapting the correlation, standard deviation, parametric means and statistical distributions according to site specific information, as well as changes to the structure or formulation of the performance function.

The structure of RSM allows for the input of various values and parameters to create a performance function for FORM, provided the experimental design and matrix formulation used in this assessment is adhered to. This study illustrated how response surface methodology can be suitably used as a bridge between limit equilibrium methods and FORM in a format that lends itself equally well to input from other techniques, such as finite element methods as well as geotechnical problems requiring different performance functions.

References

SANS 10160-5:2011. Basis for structural design and actions for buildings and industrial structures Part 5: Basis for geotechnical design and actions. SABS Standards Division, Pretoria.
The Back Calculation of Shear Strength Parameters for Soils on Steep Slopes

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Abstract

Engineers often rely on databases, empirical relations and conservative design to compensate for difficulties associated with limited geotechnical information. A method is presented where back calculation is used as a tool to provide some justification for the choice of geotechnical design parameters. The method includes observation of the local topography, the use of databases, slope stability calculation, and the effects of vegetation.

Keywords: Residual Soils, Slope Stability, Back calculation

1 Introduction

Limited geotechnical information is a familiar challenge faced by Geotechnical engineers. The immense variability in the properties of natural materials such as soil, coupled with the challenges of limited budgets and timeframes make soil parameters difficult to estimate. The problems stem from poor drilling and logging practices, scarcity of good geotechnical laboratories, difficulty in bringing samples back to South Africa or elsewhere for testing, and limited budgets. Frequently the best information that is available is in the form of basic soils tests such as foundation indicators and test pit or borehole logs. The engineer is then faced with the challenge of determining adequate soil strength parameters using potentially unsuitable empirical correlations that may not be applicable and/or using databases that may not have been developed for the particular soils or applications in question. The method shown in this paper, although by no means definitive, offers a means of vetting the chosen design parameters, by observations of the topography and back analysis.

2 Site Information and Design Requirements

The site is characterised by deeply weathered residual granite soils which classify as a silty and clayey sands, SC-CL and SC-CH according to the United Soil Classification System (USCS). The site extends for a length of approximately 400m and is built along the hillside. The natural
hillside slopes are between 1V:1H and 1V:1.5H. It is important to note that the slope had previously been used for forestry and was well vegetated with grass at the time of site investigation.

The site investigation included test pits, sampling and soil testing. Due to project constraints only foundation indicators, compaction and CBR (California Bearing Ratio) testing was completed. Unfortunately the test pits only extended to depths of about 2m and the depth to bedrock was not established for this area of the project. However boreholes from an adjacent area confirmed that the depths are potentially variable.

Based on the USCS classification effective shear strength parameters were selected from the Swiss Standard: Characteristic Coefficients for Soils. The material is favourably comparable with a SC-CL material. According to the Swiss standard an SC-CL material would typically have an effective friction angle of 28° (±4°) and effective cohesion of 5 kPa (±5 kPa). Although useful, data bases such as the Swiss Standard, are not a definitive tool for design and their indiscriminate use may result in designs that are either overly conservative or even insufficient. Other methods exist whereby the engineer may refine his estimate of the shear strength of the soils in question.

3 Method by Wesley (2001)

In a paper describing methods for conducting coulomb wedge analyses in very steep slopes Wesley (2001) suggests a relation for back calculating the effective friction angle, φ', and the effective cohesion intercept, c', by using a simple mathematical relation. It is assumed that the slope is at limiting equilibrium with both the seepage surface and the slip surface parallel to the surface of the slope. This implies that we are assuming a translational and not a circular failure shape. The relation given by Wesley is as follows:

\[ SF = \frac{c'}{\gamma H \cos \beta \sin \beta} + \left[1 - \frac{\gamma_w}{\gamma} \left(1 - \frac{H_w}{H}\right)\right] \frac{\tan \phi'}{\tan \beta} \]  (1)

Where:

- SF = Safety Factor
- c' = Effective cohesion intercept
- φ' = Effective friction angle
- γ = Soil bulk density
- γ_w = Density of water
- H = Depth to failure surface
- H_w = Depth to phreatic surface
- b = Slope angle
The relation will not give definitive values for $c'$ and $\phi'$ but rather by assuming reasonable values for $H$ and $H_w$ one may generate a set of plausible shear strength values. Implicit in the relation is that the slope failure is translational and that the failure plane is parallel to the surface of the slope.

Assuming a slope angle of 45°, bulk density of 18 kN/m$^3$, depth to failure plane of 3m, and a dry or drained slope, three possible combinations of shear strength parameters were calculated (see Table 1). The slope was assumed to be at quasi-equilibrium with SF = 1.0. Considering that there was no evidence of slope instability on site this is deemed a conservative assumption.

Table 1. Possible combinations of shear strength parameters for a SF = 1.0 using the relation by Wesley (2001).

<table>
<thead>
<tr>
<th>Combination</th>
<th>Friction Angle, $\phi'$ (deg)</th>
<th>Cohesion, $c'$, (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28</td>
<td>13</td>
</tr>
<tr>
<td>2</td>
<td>30</td>
<td>11</td>
</tr>
<tr>
<td>3</td>
<td>32</td>
<td>10</td>
</tr>
</tbody>
</table>

We now have both, a general range of $c'$ and $\phi'$ values, from the Swiss standard, and three plausible combinations of values from the method by Wesley. It is noted that the method by Wesley (2001) requires several critical assumptions which, may not be an accurate representation of conditions on site.
4 Slope Stability Calculation

Although useful, especially for first pass calculations, the method proposed by Wesley (2001) does not take into account any site specific geometry and assumes a predefined failure shape. Modern slope stability software offers the facility to easily bring these factors into consideration. For the purpose of this project, Slope/W was used to analyse the site specific geometry. Further than taking a possible circular slip shape into account it is able to check for non-circular slips finding the lowest Safety Factor that the geometry, shear strength parameters and phreatic surface could generate.

Stability analysis was completed for the slope with the following inputs:
- Friction angles of 28, 30 and 32°
- Cohesion values of 10, 11 and 13 kPa
- Slope angle of 45°
- Slope height of approximately 50m
- A drawn down phreatic surface (considered appropriate considering the sandy nature of the soil and the well vegetated slope)

The Morgernstern-Price method was used with Mohr-coulomb strength parameters and the analysis considered non-circular failure planes.

The results of the analyses showed that the case analysed with \( \phi' = 32^\circ \) and \( c' = 10 \) kPa yielded a SF closest to 1.0. The results are shown in the figure below:

![Figure 2. Result of the slope stability analysis conducted for \( \phi' = 32^\circ \) and \( c' = 10 \) kPa](image)

From this analysis it is proposed that the appropriate shear strength parameters are \( \phi' = 32^\circ \) and \( c' = 10 \) kPa. The cohesion value of 10 kPa may be considered high for a clayey/silty sand, however there are aspects that have not been taken into account. The effects of the vegetation, the level of the bedrock and possible matrix suction in the unsaturated zone. Should the bedrock be shallower than assumed in the stability analysis this would explain the ‘increased’ cohesion as much of the profile would be rock. Considering that we are not able to confirm the depth to bedrock perhaps the increased cohesion could be explained by the effects of vegetation.
5 Effects of vegetation

It is widely known that vegetation, such as grasses, bushes and trees, can bolster the stability of the slopes. Vegetation improves the stability of the slope by drawing down the phreatic surface by evapotranspiration and yields mechanical stabilisation by virtue of the penetration of the roots systems of the plants.

Coppin and Richards (1990) have quantified the effects of vegetation on slopes. They provide a rigorous treatment of the subject, considering all the impacts that vegetation may have on the stability of the slope, including the mechanical strength of the roots, the effects of evapotranspiration, dead loading from the weight of the vegetation, wind loading and surface erosion. Of interest in this study is that they quantify the added strength to the soil as an increase in soil cohesion, $c_n$, which may be in the order of 1 to 10 kPa.

A cohesion value of 10 kPa is considered high for a silty or clayey sand but considering the effects of vegetation on the soil may explain why this slope proved to be stable, $SF = 1.0$, at slope angles of $45^\circ$. Considering that the site was previously used for forestry it is reasonable to assume that there would be many large tree roots present in the soils on the slope. This may serve to strengthen the proposed values of shear strength for the soil.

Conclusion

It is not the purpose of this paper to provide a definitive method for back calculations or verification of design parameters but rather to encourage the design engineer to look further than a test results, parameter databases, taking note of the topography and using more of the tools available to support him/her in the design decisions that have to be made. These methods have been used to help build a defendable choice of shear strength parameters.

References

Swiss Standard SN 670 010b. Characteristic coefficients of soils. Association of Swiss Road and Traffic Engineers.