Slope Stability Assessment and Evaluation of Remedial Measures Using Limit Equilibrium and Finite Element Approaches

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ABSTRACT
Slope stability is a recurrent theme and is of major significance on large scale infrastructure projects such as highways, railways, dams or canals, where having more cost-effective designs becomes a crucial drive on any scheme.

This paper relates to the on-going study of the Covilhã's granitic residual soil, undertaken at UBI, which is commonly used as a construction material in the area, particularly in road schemes.

This paper is primarily focused on the slope stability assessment in granitic residual soils, using different calculation methods and for different soil parameters, geometries, applied loads and groundwater conditions. Additionally, a quantification of the merits of some of the most common remediation techniques are established, drawing a comparison between their effectiveness.

The parametric study makes use of one LE (SLOPE/W) and one FE (PLAXIS 2D) software and has revealed that changes in groundwater level are more detrimental to the stability than increases in the applied surcharge at the crest. Also, reductions in Factor of Safety using LE methods due to rises in groundwater levels appear to be inversely proportional to the cohesion of the intersected materials, i.e., the greater the c' of the soil the lesser are the consequences of groundwater rises.

The comparison between FE and LE methods reveals a significant divergence in the obtained FoS for purely granular materials, with all LE methods overestimating the safety of the slope up to circa 40%. Nevertheless, these differences may have little interest if there is a large uncertainty in the input parameters of the soils.

The remedial options discussed and analysed in this paper should be perceived as concept ideas as their gain in terms of FoS will likely vary from case to case. Furthermore, the benefits of combining the effects of more than one remedial option have been excluded from this study.

Keywords: Slope Stability, Limit Equilibrium, Finite Elements, Remedial Measures

1 INTRODUCTION

Slope stability plays a major role in civil engineering projects, particularly in transportation and infrastructure schemes, and its assessment requires not only an adequate understanding of its triggers but especially a solid understanding and critical analysis of the mechanics behind the software used to arrive at a given conclusion.
This work is part of the on-going study, currently being undertaken at UBI, of one of the region's most abundant resource, its granitic residual soil; which by being a very common construction material in the area, particularly in highway schemes, has raised an interest on its behaviour from a slope stability perspective.
This particular type of soil and its properties, although usually falling within a rather well defined range of values, have been the subject of significant analysis in order to

expedite the design and stability assessment of future and existing earthworks.
An experimental embankment was completed in November 2010 (Figure 1) using the granitic residual soil of the area to help with the study of its geotechnical properties. The embankment shoulders were constructed with varying gradients, from 45º to 80º to the horizontal, having remained stable since its construction. In addition, a surcharge of 10kPa was applied to the crest of the slope, near its steepest section, without any evidence of instability.

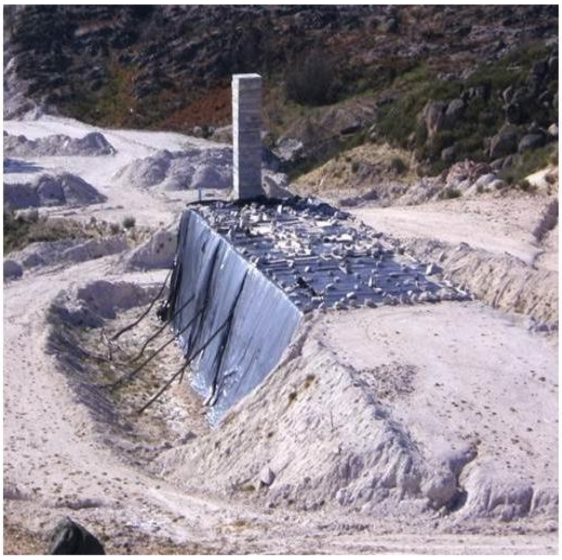

Figure 1 View of the experimental embankment.

2 FACTORS INFLUENCING SLOPE STABILITY

Slope movement often is a complex process which involves a continuous series of events from cause to effect, making it rather difficult to pinpoint a single trigger to the movement. It is largely determined by lithology and stratigraphy (influencing strength, deformability and permeability), as well as the hydro-geological conditions, the topography of the terrain and the weather conditions. A combination of these may trigger a failure event along one or more sliding surfaces, which induces the movement of the unstable mass.
Saturation, however, appears to be the primary cause of landslides, especially if resulting from rainfall. Its magnitude depends on both weather conditions (distribution and duration of precipitation and changes in temperatures) and topography. Additionally, human activities can also play a crucial role in slope stability. Disturbing or changing drainage patterns, destabilising slopes and removing vegetation are common human-induced factors that may trigger instability. Other examples include steepening of slopes by undercutting its toe, placing loads on its crest or even the presence of leaking pipes.
At the same time, it is common to witness the development of cracks on the crest of slopes, some of which have been monitored for dozens of years without any noticeable unstable behaviour. According to some researchers (Guidicini and Nieble, 1984) these are often a result of minor shear movements within the slope; which, although individually small, when accumulated may lead to significant slope movement, sufficient to cause a vertical separation on the materials at the top of the slope, thus forming tension cracks.
The fact that these structures are a result of shear movements is extremely relevant, as when a tension crack becomes visible it can be an indicator that a slip surface may have already been formed within the slope and that a shear failure process is underway. Neverthless, it is nearly impossible to quantify just how hazardous this phenomenon is, since it represents the beginning of a complex and progressive failure mechanism. Moreover, there is also the possibility that, in some cases, the appearance of tension cracks is purely associated with a relief in porewater pressure.

3 RESIDUAL SOILS

A residual soil has, by definition, been formed in situ by the decomposition of a parent rock and has not been transported to any significant distance. For this to occur, the decomposition processes tend to typically be quicker than the erosion and transport of the resulting soil grains. However, these soils can also result from erosive processes, as long as they are not transported afterwards, which is often in the genesis of the Portuguese granitic residual soils (Figure 2).

Figure 2 Example of granitic residual soil slopes in the Covilhã region with the parent rock at the bottom.

In fact, and although residual soils cover a very significant extent of the Earth's surface, Soil Mechanics has paid much less attention to them then to sedimentary soils, as its principles were mainly studied and defined in countries where the latter were the most common and problematic. Nevertheless, residual soils can present some particularly complex features, exhibiting a significantly different behaviour from sedimentary soils with similar grading, void ratio and moisture content.
This is thought to be the result of interparticle connections, inherited from the original parent rock, or due to the chemical reactions that occur through the weathering process (Fernandes, 2011). As such, it is questionable just how representative grading curves are for these soils and whether their geotechnical parameters should be extrapolated from them, given that sieving necessarily affects and/or breaks this bond.

3.1 Granitic residual soils of the Covilhã region

Granite rock masses are predominant in the northern part of Portugal and in the surroundings of the Serra da Estrela mountain complex. Here, granitic residual soils are abundant, covering more than 50% of the surface, and can extend to maximum depths of over 18.0m (Cavaleiro, 2001). The typical composition of a granitic residual soil from this region contains kaolinite as the most common clay mineral, which is associated with soils with good engineering properties.
The fine fraction in these soils varies, although predominantly low, and with sand typically being the predominant fraction. Figure 3 below illustrates the grading envelope obtained from circa 15 samples of the soil in analysis, plotted against the historical results obtained from circa 80 different samples of granitic residual soils of the Covilhã region (Cavaleiro, 2001).

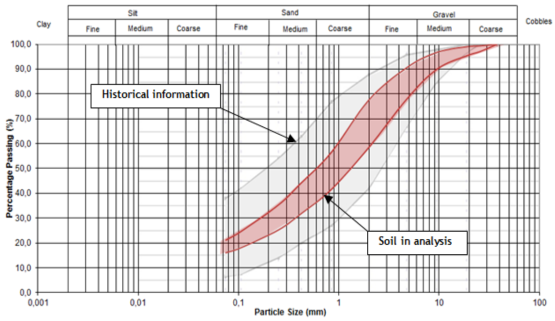

Figure 3 Grading envelopes of historical data and the residual soil in analysis (Neves, 2015).

The grading results are in agreement with the historical information and classify the soil as a well-graded silty very gravelly fine to coarse sand, with a low fines content (less than 20%). Given the low percentage of fines and the fact that its constituent minerals exhibit low plasticity, this residual soil is generally classified as non-plastic. Although the representativity of the grading curves of these materials may be questionable, it is clear that the predominant fraction is typically sand, which is originated from the quartz of the parent rock. Additionally, the clay fraction is often reduced and its minerals present low activity, which explains the non-plasticity of these soils. As a result, their behaviour can be better approximated to a granular than to a cohesive soil.
Most of the geotechnical characterisation of the soil has been undertaken through laboratory testing, in particular drained and undrained consolidated triaxial tests on remoulded samples. These have been used to assess their drained and undrained tangent and secant deformability moduli, as well as

the peak and residual angles of shearing resistance. The results are summarised in Table 1 below, along with the correspondent values for both dry and saturated unit weights. Note the values in brackets correspond to the average results of approximately 15 samples.

The values reported exceed the ones obtained through a wider triaxial test campaign conducted on granitic soils of the region (Cavaleiro, 2001), which reported angles of shearing resistance between 26.9º and 37.7º. However, these were established considering effective cohesion values of up to circa 20kPa, while the values in Table 1 have been derived for null cohesion. When applying the same principle to the historical results, angles of shearing resistances between 31.3º and 42.5º are obtained (Neves, 2015), which in turn agree well with the results of this study.

Table 1 Geotechnical parameters assessed from the triaxial test campaign.

Parameter	Range of values	Average
E_{us} [MPa]	16.6 – 23.3	18.5
E_{ut} [MPa]	25.2 – 37.0	29.4
E_s' [MPa]	12.0 – 34.5	25.8
E_t' [MPa]	21.8 – 46.0	34.3
γ_d [kN/m³]	19.3 – 19.8	19.5
γ_{sat} [kN/m³]	21.0 – 21.9	21.5
φ'_{peak} [°]	37.5 – 43.6	40.6
$\varphi'_{residual}$ [°]	35.3 – 36.9	36.1

Direct shear tests conducted on undisturbed samples of the same materials in Cavaleiro (2001) report effective cohesion values from 4kPa to 42kPa and angles of shearing resistance between 35º and 45º (Figure 4).

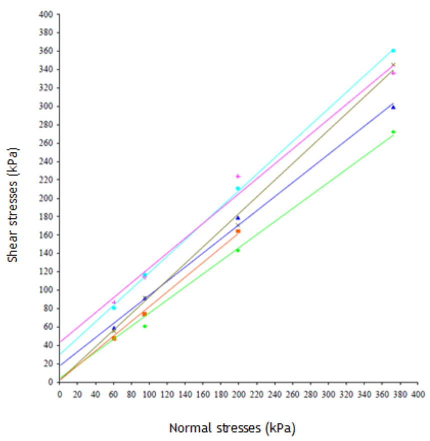

Figure 4 Direct shear test results in samples of Covilhã's granitic residual soils (Neves, 2015).

3.2 *Experimental embankment*

As mentioned earlier, an experimental embankment was constructed using granitic residual soils from the Covilhã area. The controlled embankment was built with a 16.0m footprint and a 20.0m development (Figure 5) and completed in November 2010.

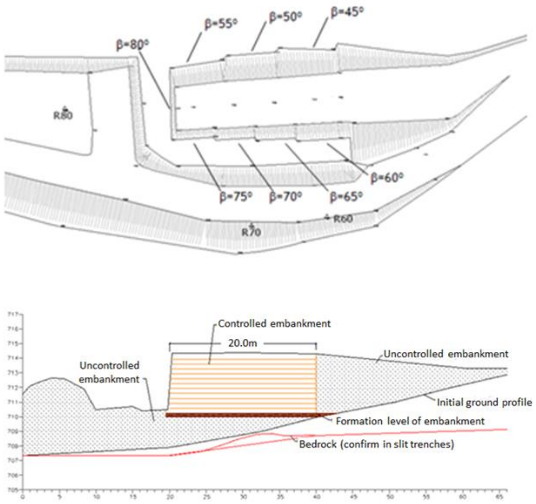

Figure 5 Plan and elevation of the controlled embankment (Neves, 2015).

The embankment height (4.0m) was kept constant across its full extent and its shoulders were built with varying gradients, so as to try to experimentally establish what would be the steepest stable configuration. The side slopes gradients varied between 45º and 80º with the horizontal, in 5º increments (refer to Figure 5 for the location of each of these faces in plan).

Additionally, a surcharge of 10kPa was applied at the top of the embankment, in August 2011, and near its steepest face. Circa 4 years after its application there are still no signs of instability in the slopes. The embankment was initially protected from weathering effects by means of a plastic film, which has deteriorated with time, leaving the embankment unprotected against the erosive effects of rain and wind.

4 METHODOLOGY

There is probably no analysis conducted by geotechnical engineers which has received more programming attention than the limit equilibrium methods of slices. This is due to the fact these methods tend to require iterative procedures, which are usually numerically undertaken. The available commercial software which assesses slope stability, with both circular and non-circular slip surfaces, has the advantage of enabling a large number of calculations to be carried out in a very short period of time. Additionally, they have the advantage of providing a graphical output, which shows the geometry of all of the analysed slip surfaces and their correspondent MoS.

However, LEM methods consider forces acting on one or several discrete points along the slip plane, whilst assuming that failure occurs instantaneously and that the available shear strength is mobilised along the whole slip plane at the same time. As an alternative, stress-strain methods can be used to overcome these limitations. By considering the stress-strain relationship of the materials during deformation and failure, software can output the type and magnitude of the displacements in the slope which are consistent with its state of equilibrium. They can also provide the slope's MoS, which may be different from the one obtained with limit equilibrium analyses, as no specific failure surface is defined (Matthews et al., 2014).

For the purpose of this work, both LEM and FEM numerical analyses have been undertaken. The commercial software used was SLOPE/W and PLAXIS 2D, respectively for LEM and FEM approaches. Within SLOPE/W the Fellenius', Bishop's simplified and Janbu's simplified methods were used.

For the stability analysis undertaken in this work, the envelope of ground conditions and soil parameters, the latter based on the soil's characterisation and testing, presented in Table 2 was considered.

Table 2 Envelope of parameters and ground conditions.

Parameter	Range of values
Slope angle	1(V):2(H), 1(V):1.5(H), 1(V):1(H) &1(V):2(H)
Slope height	2.0m – 8.0m, with 1.0m intervals
φ'_k	35º, 36º, 37º & 38º
c'_k	0kPa, 5kPa & 10kPa
Groundwater	Dry, Low & High groundwater level
Surcharge	0kPa, 5kPa & 10kPa

Please note that all analysis have been undertaken in compliance with current standards (EC7) and exclusively to Design Approach 1 Combination 2, as this is typically the most critical combination for slope stability. As such, the concept of Margins of Safety (MoS), as opposed to the use of a lump Factor (FoS), has been used.

5 STABILITY ANALYSIS RESULTS AND DISCUSSION

The stability assessments undertaken in this study have selected a few key parameters, internal and external, which have been deemed to most influence the stability of slopes in the granitic residual soil in analysis. With regard to groundwater, and as briefly described in Table 2, three different scenarios have been analysed. Firstly, the different models have been run without the presence of a water table, which then served as baselines to analyse the influence of increasingly higher groundwater levels on the stability of the slope.

A second set of runs has considered the water table to coincide with the toe of the slope, and raising moderately behind the slope's face. A final set of calculations has been considered with the water table passing again through the toe of the slope but rising sharply behind its face, reaching only one metre below ground level at the crest. It should be noted the highest groundwater table is deemed as very conservative and highly unlikely to occur. Therefore, this is considered as an upper limit to the rises in groundwater levels.

As for the surcharges applied on the top of the slope, they have also been factored according to the EC7. As such, surcharges of 0kPa, 6.5kPa and 13kPa have been applied in the models. Also, all analysis have considered a minimum depth of 0.50m for the slip surfaces, as failures shallower than that are not thought of endangering the overall stability of the slope. No consideration has been given to the formation of tension cracks.

To help interpret the results from the numerical trials these have been split to individually cover the effects of an increase in surcharge on the crest, the consequences of variations in groundwater levels, a direct comparison between the outcomes of all three LEM methods for all sets of conditions and a comparison between the LEM and FEM approaches on all the common sets of conditions.

5.1 Interpretation of the LEM results

Through the application of the three LEM methods the following conclusions have been drawn. It should be noted at this point that although the analyses have been undertaken numerically for the Janbu's method, the correction factor applied to the attained MoS has been assessed manually for each individual case.

5.1.1 Effects of surcharge increases

It has been observed that, in non-cohesive soils, increases in surcharge to 5kPa and 10kPa typically result in maximum reductions in the MoS of circa 5% and 10%, respectively. Also, these changes are quite often negligible. Furthermore, these results are fairly consistent throughout all slope gradients and independent of groundwater conditions. It appears that the critical slip surfaces tend to mostly develop at a shallow depth along the slope's face and therefore are not considerably affected by surcharge variations at the crest.

When an effective cohesion of 5kPa is considered, the repercussions of a surcharge increase are more critical. If groundwater is absent, the reduction in MoS is up to 15%, for a surcharge of 5kPa, and 20% (Bishop) to 25% (Fellenius and Janbu) for a surcharge of 10kPa. However, these extreme values only occur in slopes with heights of 2 to 3m. When ignoring the latter the differences in MoS are reduced to a maximum of circa 5% and 10% for 5kPa and 10kPa surcharges. The conclusions are similar for the remaining groundwater conditions. As opposed to the previous set of conditions (non-cohesive soil) the differences in MoS are noticeable over the full range of heights, whereas before from a height of 5m to 6m the differences were negligible.

If an effective cohesion of 10kPa is considered, and ignoring groundwater, the reduction in MoS is up to 15%, for a surcharge of 5kPa, and 25% for a surcharge of 10kPa. Again, these extreme values occur for small heights (2m to 3m high). When ignoring the latter, the differences in MoS are confined to maximums of circa 10% and 15%, respectively for 5kPa and 10kPa surcharges. The conclusions are similar for the remaining groundwater conditions.

For all LEM methods used in this thesis, increases in surcharge at the crest have been ascertained as more detrimental to the overall stability within cohesive materials, than in purely granular soils. In the latter case, for slopes above 3.0m high the differences are quite often negligible. Lastly, and even though the conclusions are similar regardless of the position of the water table, i.e. reductions in MoS of the same magnitude, having a higher water table leads to a slightly more penalising effect from surcharge increases.

5.1.2 Effect of groundwater level rises

When solely varying groundwater levels in the models, the differences in MoS are far more expressive. For purely granular soils, the consideration of a low groundwater table does not seem to affect the MoS obtained when groundwater is absent. However, rising these levels to what has been defined has a high groundwater table, leads to reductions in the MoS of over 50%.

Nevertheless, and with both high and low water levels, the reductions in MoS appear to be independent from the surcharge applied at the crest, although strongly linked to the slope angle.

For the Bishop's and Janbu's methods, the slopes modelled with the two slacker gradients, 1(V):2(H) and 1(V):1.5(H), have a similar response to groundwater level changes. A greater effect is reported in steeper slopes, particular in a 2(V):1(H) gradient. Sharp drops in the MoS are obtained for slopes over 3m high, when using the Bishop's method, or over 2m to 3m high for the Janbu's method.

A different trend emerges when analysing the results of the Fellenius' method. Here, the reductions in MoS seem to be more expressive for slacker slopes, diminishing as the slope angle is increased. The maximum reductions are reported under 35%.

Similar conclusions can be drawn from the SLOPE/W runs on slopes in soils with a 5kPa and 10kPa effective cohesion. The only exception to this is that in these cases and when considering slacker slopes, 1(V):2(H) and 1(V):1.5(H), differences of up to 15% are reported between models where groundwater is absent and having a low water table. This is more noticeable as height increases. This might be explained by the fact that as these materials have a certain cohesion value, they drive the critical slip surfaces deeper. As a result, in slacker slopes they will more easily intersect what has been defined as a low groundwater table.

In addition, the reductions in MoS by rising groundwater seem to be inversely proportional to the cohesion of the intersected materials, i.e. greater reductions are reported for lower or null effective cohesion values.

5.1.3 Comparison between LEM methods

The comparison between methods has been undertaken considering the Bishop's method as a reference. As such, all the conclusions below refer to comparisons between the latter and the remaining two methods.

In non-cohesive materials the MoS seems to converge for slacker slopes and for greater heights. Also, and in line with the results of the sensitivity analysis to groundwater changes, results show that for steeper slopes, 1(V):1(H) and 2(V):1(H), the Fellenius' method gives MoS significantly higher than the Bishop's method (up to circa 70%) for high groundwater levels. This is in agreement with the findings of Whitman and Bailey (1967), which reported the Fellenius method to result in errors as much as 60%.

This contrasts with the findings of the Janbu's methods which report a significant reduction in the MoS for the same models by up to 25% when compared to the Bishop's. Again this may be the result of the lesser aptitude/inadequacy of this method to circular slips in non-cohesive soils.

Overall, for slacker slopes, 1(V):2(H) and 1(V):1.5(H), a good correlation is achieved between all methods, with maximum differences of 10% and frequently under 5%.

Different conclusions are drawn for slopes in cohesive materials, with the exception of the Fellenius' method. This still reports significantly higher MoS than the Bishop's method for high groundwater tables within the steepest slopes. On all remaining models the Fellenius's method reveals lesser MoS than the Bishops' but only up to circa 10%.

When comparing the outcome of the Janbu's method, it is clear that overall these results are closer to the Bishop's than those obtained with the Fellenius's method. However, greater divergences are noted for high groundwater levels and in steeper slopes, 1(V):1(H) and 2(V):1(H).

Nevertheless, when groundwater is absent or at a low level, this method generally reports greater MoS than the Bishop's method, albeit marginally.

5.2 Interpretation of the FEM results

By using the HSM available within the PLAXIS 2D software it has been possible to arrive at the following conclusions.

5.2.1 Effects of surcharge increases on the slope's crest

When considering a purely granular soil, significant reductions in the MoS have been obtained by increasing the surcharge to 5kPa and 10kPa, respectively up to circa 20% and 30%. Greater reductions are reported for the 5.0m models than for the 8.0m ones and surcharge increases appear to be less critical for higher water levels.

When an effective cohesion is considered, the repercussions of a surcharge increase are overall less critical (as opposed to the LEM analyses), with reductions generally under 5% and up to a maximum of 6.3%. The effect of the surcharge increase appears to be similar when groundwater is absent and in slopes with a low water table. A lesser effect is noted when groundwater rises to high levels.

These changes report lesser reductions to the MoS as the slope gradient increases, although the pool of results is not sufficiently large to arrive at a definite conclusion.

5.2.2 Effect of groundwater level rises

As opposed to the above, changes in groundwater levels appear to be more critical for cohesive than non-cohesive materials. However, it should be noted that only two non-cohesive models were used and that on these it has not been numerically possible to assess their stability for a high water table.
For cohesive materials no significant differences are reported for effective cohesions of 5kPa or 10kPa. Also, negligible reductions occur for the steepest configuration 1(V):1(H). 1(V):2(H) and 1(V):1.5(H) models show reductions in the MoS of circa 30% and 35% for a high groundwater table, respectively for 5.0m and 8.0m heights.
When considering a low groundwater table reductions are up to 10% and 15%, respectively for 5.0m and 8.0m heights in 1(V):2(H) slope. For a slightly steeper slope, 1(V):1.5(H), these values are reduced to up 5% and 10%, respectively for 5.0m and 8.0m heights.
There is no significant variation with the increase in surcharge, which leads to the conclusion that variations in groundwater levels are substantially more critical than increases in surcharge.

5.2.3 Comparison between LEM and FEM approaches

The comparison between FEM and LEM methods has revealed there is a significant divergence in the obtained MoS for purely granular materials. All LEM methods appear to overestimate the safety of the slope by up to circa 40%. These differences are more expressive as the height and the applied surcharge at the crest increase.
In soils with apparent cohesion the results of the Bishop's simplified method are between circa 5% and 10% higher than the FEM outcomes. The differences appear to increase with the gradient of the slope, with rising groundwater level and the slope's height. No noticeable differences are reported for surcharge increase.
The Fellenius' method presents MoS values both over and under the FEM ones, with differences generally under circa 5% but with values between 5% and 10%. No tendency is apparent for changes in groundwater levels or applied surcharges.
The majority of the MoS values obtained with the Janbu's simplified method present the best correlation with the FEM approach (for soils with apparent cohesion), with differences reported predominantly under 5%. The best correlation occurs for greater applied surcharges and slacker slope gradients.

6 REMEDIAL MEASURES

Remedial measures, as their name suggest, are those carried out after a slope failure event or when excessive deformation is reported which may trigger instability. As such, their design requires an estimate of the relevant soil parameters, a prediction of the geometry of the failure surface and especially an assessment of the factors causing the instability. In general terms, they result from either one or a combination of the following options:

- Modifying slope geometry;
- Installing/improving drainage;
- Installing resisting elements on the slope;
- Setting up retaining walls/elements at the toe.

When a slope becomes unstable, it is very useful to perform a back analysis, to allow an estimation of the "real" geotechnical parameters found on site.
In order to help quantify the benefits of each type of measure, six distinct ground models have been selected (Table 3) to provide a better insight on the most appropriate remedial measure for each set of conditions.

Table 3 Sets of conditions to analyse the benefits of the different remedial measures.

Ground model	**1**	**2**	**3**
Parameter	**Range of values**		
Slope angle	1:1	1:1.5	1:1.5
Slope height	8.0m	5.0m	5.0m
φ'_k	35°		
c'_k	10kPa	5kPa	10kPa
Groundwater	Low	Low	High
Surcharge	0kPa, 5kPa and 10kPa		
Ground model	**4**	**5**	**6**
Parameter	**Range of values**		
Slope angle	1:2	1:2	1:2

Slope height	5.0m	8.0m	8.0m
φ'$_k$	35°		
c'$_k$	0kPa	0kPa	10kPa
Groundwater	Dry	Dry	High
Surcharge	0kPa, 5kPa and 10kPa		

For each of the above models the installation of the following structural elements has been analysed on both LEM and FEM approaches: sheet piles, soil nails and the use of gabion wall (Figures 6 to 8).

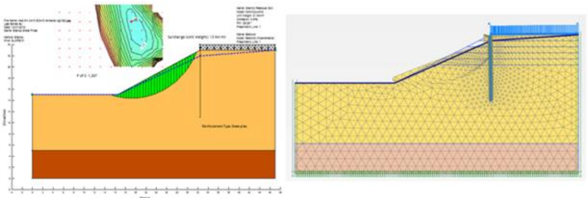

Figure 6 LEM & FEM sheet piles reinforcement and associated failure mechanisms.

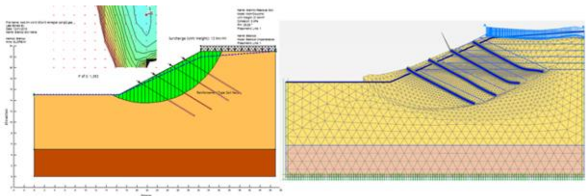

Figure 7 LEM & FEM soil nail reinforcement and associated failure mechanisms.

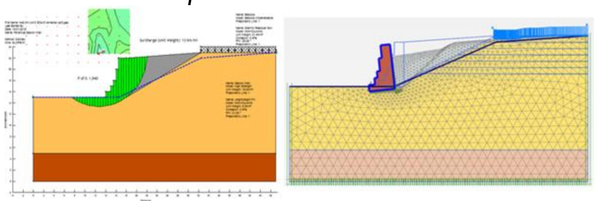

Figure 8 LEM & FEM gabion wall reinforcement and associated failure mechanisms.

6.1 Comparative analysis of remedial solutions

To help in the assessment of the most suited remedial option for each of the scenarios listed above, the gains in MoS for both LEM and FEM approaches are summarised in Table 4 below.

Table 4 Gain in MoS (percentage) of the different remedial measures.

Remedial option	c'=0kPa	c'=5kPa	c'=10kPa
Sheet piles	Minor	5-10% (LEM) <5% (FEM)	10-20% (LEM) 5-10% (FEM)
Soil nails	5-20% (LEM)	20-30% (LEM) 25% (FEM)	5-30% (LEM) 40-45% (FEM)
Gabion wall	30->50% (LEM) 15-35% (FEM)	30% (LEM) 20% (FEM)	25->40% (LEM) 25-40% (FEM)
GWL lowering*	Not assessed	Not assessed	20->30% (LEM)
Slacking slope	Not assessed	15% (LEM)	15-25% (LEM)

* Only valid when reducing ground water level from high to low. Previous analyses have shown differences in MoS between a low water table and a fully dry slope are insignificant.

By analysing the outcome of the different analyses the following points have been inferred.

Driving sheet piles from the crest of the slope does not appear to result in a significant improvement in the MoS. However, having these elements in place forces the critical slip plane to be shallower and purely along the slope's face. As such, and although the overall safety is not increased substantially, if the priority was to just ensure the safety of the crest of the slope, and accepting that a failure may occur below it, there would be great benefit in this option. FEM modelling also reveals that the overall maximum displacements along the face of the slope are not greatly reduced by adopting this remedial option.

The use of soil nails along the slope appears to be a more effective mean of enhancing its stability, particularly in cohesive soils, than the use of a sheet pile wall. However, the effectiveness of this remedial option is strongly dependent on their length and spacing (both vertical and horizontal).

In terms of displacements this option has proven to reduce the anticipated maximum total displacements along the slope's face between circa 30% and 50%. The FEM approach in these cases has provided a noticeable higher MoS than the LEM approach. This is likely explained by the abrupt change in the failure mechanism, with the resultant critical slip surface being significantly deeper in the PLAXIS 2D models than within the SLOPE/W runs.

The option to construct a retaining wall, for the purpose of this study only a gabion wall has been considered, has proven to have a

similar effect across all options and to result in the greatest increases in terms of MoS. In fact, the failure mechanism after the wall is in place appears to be very much dependent on the properties of the backfill rather than the existing slope's materials.
Further analyses have confirmed that, apart from when lightweight aggregate is used as a backfill to the new wall, the merits of this option are left in between the use of sheet piles and soil nails. However, it should be noted that the above has not taken into account the bearing capacity assessment of the gabion wall foundation.
The benefits of changes in groundwater levels and slope angles have only been assessed through the use of LEM analyses on the six ground models presented above. Changes in the water table are only particular relevant for high water levels and can equate to gains of 20% to 30% in the MoS. As for the slope angle, slacking the slope can result in increases of 15% to 25% in the overall stability of the slope.

7 CONCLUSION AND CRITICS

The majority of slope stability analyses performed in practice still use traditional limit equilibrium (LEM) approaches involving methods of slices that have remained essentially unchanged for decades. Nevertheless, the user-friendliness, simplicity and proven good record of LEM methods are enough to still make them a valuable tool against the use of formulations based on finite element (FEM) principles. The latter however, can help predict stress concentration problems and forecast deformations/displacements within the slope, which have been experienced problematic in LEM analysis and are often crucial in evaluating the performance and acceptability of some slopes which are sensitive to movement.
Also, LEM methods are especially useful when assessing the stability of a slope with a MoS below unity. On such cases, when remedial/strengthening measures are to be installed, it is then possible to quantify their merits in the overall stability of the slope using this approach. FEM methods, on the other hand, can rarely be used to compute a stability problem with a MoS value significantly below unity.
Consequently, it is quite common to undertake LEM back analyses when prescribing remedial measures whilst assessing the factors leading to the instability. These analyses usually try to establish the 'real' strength parameters of the soil (φ' and c') when little information is available and usually by considering the soil as a homogenous material for simplicity.
All stability calculations have been based on the principles of saturated soils. However, for situations where a failure occurs above the groundwater table, thus within the partially saturated zone, slope stability would have been better evaluated using an assumption of an unsaturated soil, which could have been more cost effective, although requiring an advanced understanding of matric suction contribution to slope stability.

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