

# BACK ANALYSES OF AN ANCIENT LANDSLIDE: A CASE STUDY IN ZAGROS



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## ABSTRACT

This paper investigates the behavior of an ancient landslide by the means of two dimensional slope stability analyses. Back analysis is conducted to derive the shear strength parameters involved in the translational slope. The result of the back analysis illustrates a large gap between the safety factors calculated from the optimized (noncircular) and the original method (circular). Usually the results of these two calculations are similar but in this case study the calculated factors of safety are far apart. This difference significantly influences the back calculated shear strength parameters. In the subsequent analyses it became clear that the higher shear strength parameters calculated from the optimized method (noncircular) which provided a lower factor of safety are not appropriate when the gap between the calculated factors of safety from the two methods (circular and noncircular) are large.

## Résumé

Cet article étudie le comportement d'un ancien glissement de terrain par le biais de deux dimensions, les analyses de stabilité. Retour analyse est réalisée pour calculer la résistance au cisaillement paramètres intervenant dans la pente de la traduction. Les résultats de l'analyse à illustrer un grand écart entre les facteurs de sécurité calculé à partir de l'optimisation (noncircular) et la méthode d'origine (circulaire). Habituellement, les résultats de ces deux calculs sont similaires mais dans cette étude de cas, les facteurs de sécurité calculées sont très éloignés. Cette différence influence de façon significative l'arrière calculé les paramètres de résistance au cisaillement. Dans les analyses subséquentes, il est devenu évident que la résistance au cisaillement des paramètres calculés à partir de la méthode optimisée (noncircular) qui a fourni un plus faible coefficient de sécurité ne sont pas appropriées lorsque l'écart entre les coefficients de sécurité calculé à partir de ces deux méthodes (circulaire et noncircular) sont grandes.

## 1 INTRODUCTION

It is conventional to adopt the lowest factor of safety in the design of slope instabilities. James Bay Dike case history discussed by Duncan et al. (2014) was initially analyzed by Christian et al. (1994); this case history is an example of comparing the results of a circular and noncircular method for obtaining the minimum factors of safety in a two dimensional slope stability analyses. The case history is discussed by Christian et al. (1994), they used a circular slip surface to analyze their case and came up with a minimum factor of safety of 1.45. The case is also described and reanalyzed by Duncan et al. (2003) and Duncan et al. (2014) which adopt a composite slip surface (noncircular) to illustrate that a lower factor of safety is obtained compared to a circular slip surface (they provide a value of 1.17). In this paper the behavior of an ancient landslide in Zagros (Iran) is investigated. The results indicate that the lower factors of safety obtained by a noncircular method may not be appropriate in all cases. Four phases are used to complete the back analyses of the ancient land slide; the results of the circular method (i.e. the original method) are compared with the results of the noncircular method (called the optimized method) for each phase to derive conclusions. As discussed by Duncan 1992 a two dimensional analysis for slope design is appropriate because it yields a conservative estimate for the factor of safety. There are exceptions for landslides with complicated topography and site conditions where a three dimensional analysis is required (Stark 2003).

All ancient landslides have a discrete shear zone which is generated due to the initial movement of the sliding mass. The results of an appropriate back analysis might be better than conducting in situ or lab tests on the slip plane or the discrete shear zone of an ancient landslide and that is because a back analysis provides an average value for the shear strength parameters, while conducting tests provides results only for the location where the test is conducted, Cornforth (2005). Ancient landslides are a jumbled mass of different soils from large boulders of rock to silt and clay material. Furthermore, their shear zone or slip plane is significantly weaker than the soil above and below them. Due to the very thin and remolded state of the discrete shear zone in an ancient landslide it is practically impossible to attain undisturbed samples for shear tests in the lab. As the shear zone of ancient landslides have undergone large displacements, a ring shear test or a shearbox test with multiple cycles is required to obtain the residual shear strength parameters for the discrete shear zone involved in an ancient landslide. Furthermore, these values might only represent the residual strength of the bored location in an ancient landslide. As discussed by Cornforth (2005), conducting SPT or CPT does not provide reasonable results for ancient landslides as the material in such slopes are significantly mixed and can also damage the instruments.

The concept of back analysis is to successfully capture the field mechanics (the rupture shape) through the modelling process with accepting a factor of safety equal to 1.0, which states that the mass is at the onset of failure. As a discrete shear zone exists in all ancient

landslides, it can assist the engineer in defining the shear strength parameters involved in the sliding mass. When there are uncertainties about particular aspects in a back analysis, it is recommended to conduct a parametric study (i.e. changing the values of one particular unknown while keeping the other values untouched) to define the effects and the importance of the particular unknown on the whole system, Cornforth (2005). The concept of a parametric study is adopted in other fields of Geotechnical Engineering, for instance Pedram (2015a), Pedram (2015b) and Pedram (2018) adopts the concept in the field of pile engineering to derive conclusions about the behaviour of deep foundations installed in sands.

For an effective analysis (or long term analysis) it is common to consider the soil cohesion as zero (Wood 1990 and Budhu 2007). Software similar to Slope/W are not well suited for defining a cohesion equal to zero for conducting a long term analysis. If the cohesion is set to zero, the center of the circular slip surface may fall at the very edges of the defined grid which is used for drawing the center of the slip surface. This is not acceptable and consequently a small value for the cohesion is required in the analysis. This problem also appears for a short term analysis i.e. using constant undrained shear strength ( $S_u$ ) for the soil. In such cases a linear increase in the undrained shear strength is required, which is also a better representation of the field.

## 2 INFORMATION AND ANALYSIS OF THE ANCIENT LAND SLIDE

The land slide accrued in the mountainous regions of Zagros in western Iran close to Kermanshah during constructing the Right-Of-Way (R.O.W) for a gas pipeline route, Figure 1.



Figure 1. Location of the ancient landslide

The soil and rock sediments in this region of Iran are heavily crushed and anisotropic and this is due to the geology of Zagros. The ancient land slide was not initially detected in the site investigation process and the R.O.W passed through it.

Experience in this region of Iran has shown that slopes of 1:1 are acceptable and suite the requirements for constructing the R.O.W for gas pipe lines. Engineers and contractors usually remove the soil on top of the slopes with an angle of 45° by constructing steps. The steps have a width and height of 5 m as illustrated in Figure 2. The diameter of the pipe line for this route is 140

cm and the width of the R.O.W is around 28 m. After constructing the R.O.W, the pipe line must be buried under the ground at a depth of at least 1.5 m. The width of the R.O.W is large so that if there are any emergencies, machineries can easily get access to the required location.

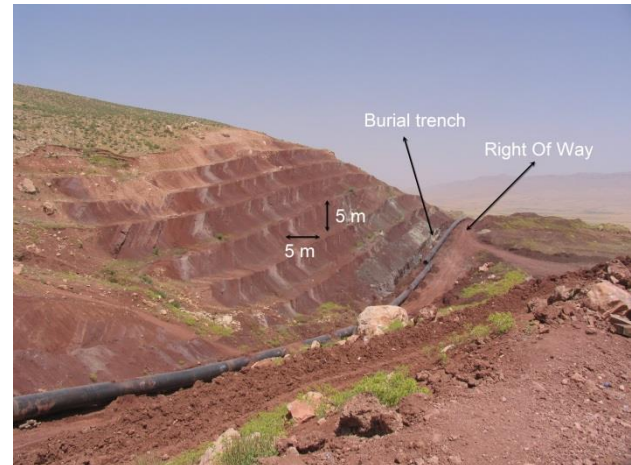


Figure 2. Steps used for stabilizing slopes in Zagros

As there are a few phases in the design of the ancient landslide, each phase is described in detail and subsequently the results of the modelling process are discussed in the same section.

### 2.1 Land Slide Geometry, Phases One and Two

As mentioned, during the initial design stage (site investigation), the ancient landslide was not detected and the R.O.W passed through the sliding mass. Stage one is considered as the initial shape of the land slide (i.e. before the R.O.W passing through it) and the second stage is when the R.O.W passed through the ancient landslide and the slope was modified for constructing the R.O.W. The ancient landslide did not cause any problems due to the removal of the soil for constructing the R.O.W, as no substantial movement or tension cracks were sighted. The engineers used the usual method for designing slopes in this region, which is constructing slopes of 1:1 with heights and widths of 5 m for stabilizing the cut slope. During May 2016 the region experienced a heavy rainfall for around a week. After the rainfall, the landslide was activated and tension cracks appeared at the top and edges of the sliding mass. Figure 3 illustrates the tension cracks generated at the top and around the sliding mass; the picture was taken on 16/May/2016 after the heavy rain fall.

Figure 4 illustrates the ancient landslide with its graben (the location were the initial soil mass was positioned) and the constructed steps.

Figure 5 illustrates the head scarp of the sliding mass after the heavy rainfall in the region. The depth of tension crack was 3 m. It must also be noted that no bulging was detected on the R.O.W during the inspection; this means that the slip surface is not exiting at the level of the R.O.W. After the rainfall, most constructed steps collapsed and cracks appeared at their base.

Figure 6 illustrates the R.O.W after the movement of the landslide, it is clear that the landslide contained all types of material from large boulders to fine grained soils (comparing Figures 6 and 8).

The width of the sliding mass at the level of the R.O.W is 350 m. The sliding mass did not contain a particular geometry along its length; this means that the slope changed at a point along the length of the sliding mass. From the upper step constructed by the contractors the slope moved with an angle of  $10^\circ$  for 105 m and then increased to  $20^\circ$  and continued for another 70 m up to the graben. The head scarp was generated after the rain fall at the intersection between the two described slopes (where the slope angle changed from  $10^\circ$  to  $20^\circ$ ). Furthermore, the geometry does not change along the width of the sliding mass, that means that a two dimensional analysis provides acceptable results. Figure 7 illustrates the landslide with the two slope geometries and the head scarp. The soil removed from constructing the steps was deposited along the sides of the R.O.W, as illustrated in Figure 7. Before depositing the removed soil, the slope was flattened so that a higher volume of material could be deposited along the sides of the R.O.W. It can also be noticed that the instability is a translational sliding mass, which the slip surface is shallow compared to its length (Figure 3).



Figure 3. The edges of the ancient landslide

At this point (after the heavy rainfall) it was clear that a discrete shear zone existed below the landslide and so two trenches were excavated to define the depth of the slip surface along the midsection of the slope. The first trench was excavated on the R.O.W and the second was excavated above the tension crack generated at the intersection of the two slopes of the landslide. This particular location was chosen to make sure that the discrete shear zone extended beyond the generated tension crack as well.

Figure 8 illustrates the excavated trench on the R.O.W and the discrete shear zone which contained fine grained material. The discrete shear zone was at a depth of around 3.0 m below the level of the R.O.W and at a depth of 10 m above the tension crack. This indicates that the discrete shear zone is almost parallel to the ground surface for both slope angles involved in the sliding mass.

During the excavations it became clear that the soil was a mixture of all types of materials, i.e. a mixture of fine grained soils up to large boulders. The average properties of the fine grained soil in the discrete shear zone are: LL 65, PL 34,  $w$  37% and PI 31.

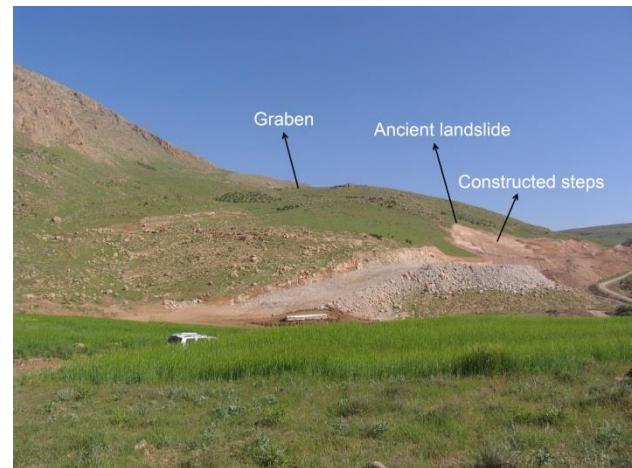


Figure 4. The ancient landslide with its graben



Figure 5. Tension crack generated at the head scarp

A simple and effective method for classifying silts and clays is discussed by Cornforth (2005); he compares the accuracy of his method with the Unified Soil Classification System (USCS) for landslides that he has investigated. He also discusses the problems associated with the USCS method for fine grained materials. Cornforth defines a parameter called Cohesive Index (CI) which is equal to the plasticity index divided by the plastic limit (PI/PL). The CI is between 0 to values above 1. The higher the value of CI means that the soil contains more clay particles. Fat clays have a  $CI \geq 1.0$ , cohesionless silts have a  $CI = 0$ , weakly cohesive silts have a  $CI = 0.2$  and  $CI = 0.8$  is close to being a clay. In the investigated case, the CI is equal to 0.91 which illustrates silty CLAY, an indication that clay particles are dominant. As discussed by Lupini et al. (1981), for soils with high clay fractions, above 50%, the difference between the peak and the residual strength is large. Tests were conducted by Stark and Hussain (2013) which also illustrate that soils with

clay fractions above 50% can provide small residual friction angles compared to clays which have lower clay fractions, the effects of the normal stress are also considered in their analysis.



Figure 6. Large boulders closing the R.O.W due to the soil movement

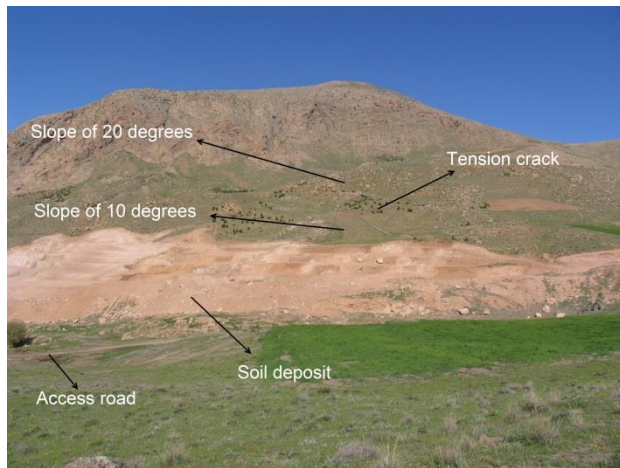


Figure 7. The two slopes involved in the ancient landslide



Figure 8. The discrete shear zone of the ancient land slide in the trench opened on the R.O.W

During the excavation of the trenches, the ground water was not encountered. As the engineers and contractors were obliged to complete the R.O.W for the installation of the gas pipe in a very short period of time, bore logging was not conducted to define the depth of the ground water level. Locals in the area with a good knowledge about the region were consulted, they pointed out that the ground water level in the region can be intersected at around 30 m below the level of R.O.W, this was sufficient to assert that suction played a major role in keeping the sliding mass in position before the heavy rain fall in May 2016. Not accounting for the effects of suction is conservative and it can be ignored in modelling (Duncan 2014).

## 2.2 Modelling Phases One and Two

The initial step is to work out the average shear strength parameters of the soil involved in the ancient landslide. Phase one is considering the landslide before it was cut through. The two dimensional analysis is conducted by Slope/W. All analyses are performed using the method of Spencer (1967) which calculates the interslice force inclination as an unknown in the equations of statics and considers all interslice forces to act with the same angle.

The initial soil mass before cutting the slope for the R.O.W had a slope of  $10^\circ$  for its initial section and it that was followed by a slope of  $20^\circ$  (similar to Figure 7). The discrete shear zone was also parallel to the ground level for both slopes involved in the landslide. The water depth (piezometric line) is at a depth of 30 m from the level of the R.O.W in the analysis. It is also assumed that at the time of failure no suction is present and the factor of safety is equal to 1. Furthermore, the slip surface in the analysis must pass through the discrete shear zone.

The results of this analysis are illustrated in Figures 9 and 10 for the circular and optimized methods respectively. Usually the calculated values for the factor of safety from the optimized method (noncircular) and the results of the ordinary method (circular) are close to each other. With the same shear strength values used to obtain a factor of safety of 1.0 for the ordinary method, a factor of safety of 0.529 is calculated for the optimized method (Figures 9 and 10).

The shear strength parameters obtained from the circular method for the soil on top and below of the discrete shear zone are as follows: unit weight  $20 \text{ kN/m}^3$ ,  $\phi' = 17^\circ$  and  $c = 4 \text{ kPa}$ . The shear strength parameters from the circular method for the discrete shear zone are also as follows: unit weight  $17 \text{ kN/m}^3$ ,  $c = 0 \text{ kPa}$  and  $\phi' = 6^\circ$ .

The soil shear strength parameters are changed for the optimized method to achieve a safety factor of 1.0 (as it provided a value of 0.529 through using the circular method). For the soil above and below the discrete shear zone the shear strength parameters are as follows: unit weight  $20 \text{ kN/m}^3$ ,  $c = 4 \text{ kPa}$  and  $\phi' = 34^\circ$ . For the discrete shear zone, the values are as follows: unit weight  $17 \text{ kN/m}^3$ ,  $c = 0 \text{ kPa}$  and  $\phi' = 11^\circ$ .

It is clear that there is a large gap between the friction angles calculated from the two methods for obtaining a factor of safety of one (the values of the ordinary method

are half the values of the optimized method). This can significantly influence the remedial process as it is not clear which value is appropriate for design purposes. As previously discussed, the case history of James Bay Dike analyzed by Christian et al. (1994) and reanalyzed by Duncan et al. (2003) and Duncan et al. (2014) provided a lower factor of safety when an optimized method was adopted (they were able to reduce the factor of safety from 1.45 to 1.17).

Note that Slope/W does not consider the effect of the water level in Figures 9 and 10 in its calculation as the slip surface is above the water level and the effect of suction must be introduced through the values of  $\phi_b$  provided by Fredlund (1987) or by providing a volumetric water content function, which were not introduced in this phase of the analysis. This meant that the soil mass adopted slopes of  $10^\circ$  and  $20^\circ$  after its initial movement and the effect of suction and the water level were not present (this is a conservative assumption).

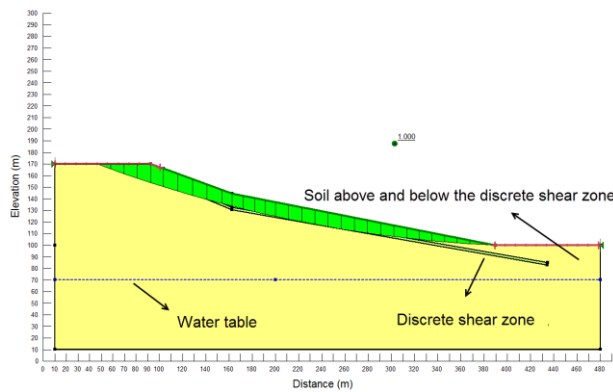


Figure 9. The safety factor calculated by the circular method (phase one)

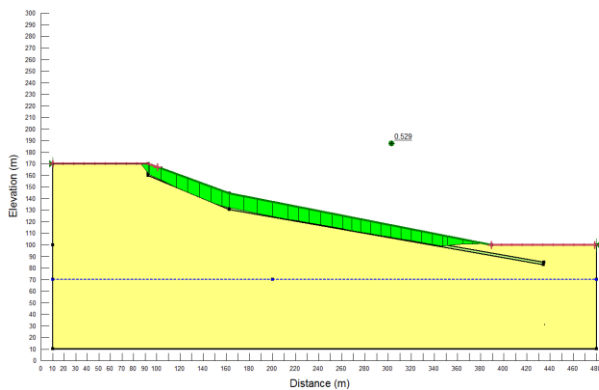


Figure 10. The safety factor calculated by the optimized method (phase one)

To work out which factor of safety is acceptable for the ancient landslide in this case study, the calculated shear strength parameters from each method (phase one) is considered into phase two of the analysis were the steps on the sides of the slope are constructed for the R.O.W. This requires two different calculations with one geometry, in the first calculation the shear strength values from the ordinary method are adopted in to phase two and

subsequently only the result of the ordinary method is checked. For the second analysis, the shear strength values from the optimized method are adopted in phase two and only the factor of safety of the optimized method is checked. The correct factor of safety must be less than 1.0 because the soil mass moved and tension cracks appeared at the top and around the sliding mass, as the rain fall waived the effect of suction.

For phase two (when the soil mass was cut through for constructing the R.O.W) of the analysis an entry and exit point is considered for graphing the slip surface as the entrance point is known. The entrance is at the point where the ground slope changed and the exit point is picked sufficiently large for the analysis as it is not known where it might exit. It is also important to model the soil deposited along the R.O.W, the soil removed from the sliding mass was deposited alongside the R.O.W as the path was constructed.

For phase two of the analysis the factor of safety calculated by the optimized method and its corresponding shear strength parameters is equal to 1.388. For the ordinary method with its corresponding shear strength values, the factor of safety is equal to 0.973. Figures 11 and 12 illustrate the factors of safety calculated for the optimized and the circular method respectively. After the heavy rainfall the soil mass moved and tension cracks were generated on the top and its sides which indicates that the factor of safety dropped below one. This clearly illustrates that if the gap between the factors of safety calculated by the two methods (circular and noncircular) are large, the ordinary method must be used for back calculating the shear strength parameters involved in a translational sliding mass.

The angle that the head scarp of the ancient land slide made with the horizon was  $53^\circ$ , which states that the soil on the top and below the discrete shear zone must have a friction angle of around  $16^\circ$ . This is in line with the results obtained from the ordinary method (circular), which provides a friction angle half the value calculated through the optimized method (noncircular).

Through further investigating the numerical results of phase two it became clear that the circular method (ordinary) illustrates that the upper slices are in tension (i.e. tension cracks are anticipated). Through checking the slice information, the depth of the tension crack is about 3.5 m which is in a very close agreement with the depth of the tension crack generated in the field (Figure 5). The optimized method also predicts that the upper slices are in tension but it anticipated that the tension crack depth is over 20 m, which is much deeper than the depth sighted in the field.

One way to check the effects of suction for an ancient landslide is to use the values of  $\phi_b$  provided by Fredlund (1987) which is implemented in Slope/W. For ancient landslides it is appropriate to use this method compared to the volumetric water content function approach as ancient landslides are a jumbled mass of different soils. The range of  $\phi_b$  is between  $12^\circ$  to  $22^\circ$  and for a conservative analysis a value of  $12^\circ$  is adopted in the analysis.

For the original method (circular) with its back calculated shear strength parameters a value of  $\phi_b$  equal

to 12° is used for both soils involved in the ancient landslide and consequently a factor of safety of 2.461 is calculated. It is clear that this high factor of safety was the main reason for the cut slope not to move before the heavy rainfall (the rainfall waived the effect of suction which generated the tension cracks in the slope).

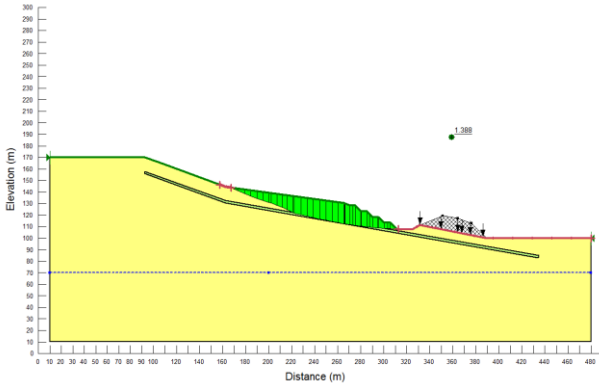


Figure 11. The results of the optimized method (phase two)

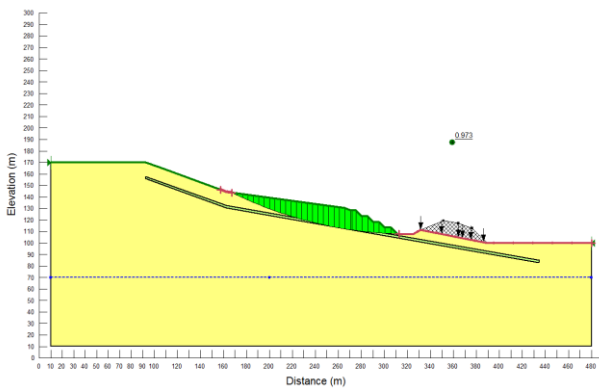


Figure 12. The results of the analysis for the circular method (phase two)

### 2.3 Phase Three

In this phase a consultant (not the author) provided advice for stabilizing the moving mass. It was advised that from the constructed steps, 30 m of soil must be removed into the landslide and steps must be constructed with a slope of 1:1 (this is essentially removing the slope's toe and increasing the width of the R.O.W). The constructed steps for stabilizing the slope must also have lengths and breadths of 5 m. It was also mentioned that the R.O.W's level must be decreased by 2.5 m. In a case history described by Cornforth (2005) (case history number 6, Pelton Upper Slide) it is mentioned that a consultant made such an advice in the US in 1990 which generated movements in a sliding mass. The reason that the consultant made such a decision for the gas pipe line route was to open enough space for materials that might fall down from the slide. He was also looking forward to bury the gas pipe line beneath the discrete shear zone of the ancient landslide. By using such a technique, he could

save the pipe line from rupture if there were any large activities in the sliding mass and in the case of materials falling down from the slope there was enough space so that the R.O.W would not be seriously damaged. In such projects, the main goal is to save the pipe line from rupture and as the R.O.W for these structures are not made for human transport, any minor damages to it is insignificant.

As advised, the contractors removed 30 m of soil from the location of the destructed steps into the slope and constructed the required steps but they were not able to decrease the level of the R.O.W as the landslide experienced large displacements. Figure 13 illustrates the tension crack on top of the landslide after removing the toe of the landslide (compare Figure 5 and Figure 13).

It was also advised that the soil removed from the slope must be deposited on the sides of the R.O.W so it can act as a semi buttress for the landslide.

As the soil is removed (30 m into the sliding mass) it is clear that the lower part of the slope (that is where the steps are located in phase two) will intersect the discrete shear zone of the ancient landslide and it will cause a rupture in the shear zone (that is because a huge amount of soil is displaced towards the shear zone of the ancient landslide). Essentially one segment of the discrete shear zone sits along the original slope and the other starts beneath the R.O.W as illustrated in Figure 14.



Figure 13. The head scarp after removing the landslide's toe

Figure 15 illustrates the intersection between the discrete shear zone of the ancient landslide with the initial step above the R.O.W after removing the landslide's toe. This also confirms that the discrete shear zone moved parallel to the ground level. It must be pointed out that the discrete shear zone was shaved during the process of generating the initial step and only a small segment of it was present when the author investigating the site after the landslide's toe was removed (Figure 15).

It is evident that if suction was not present, the soil mass would have toppled down during removal of the slope's toe and it would heap up on the R.O.W.

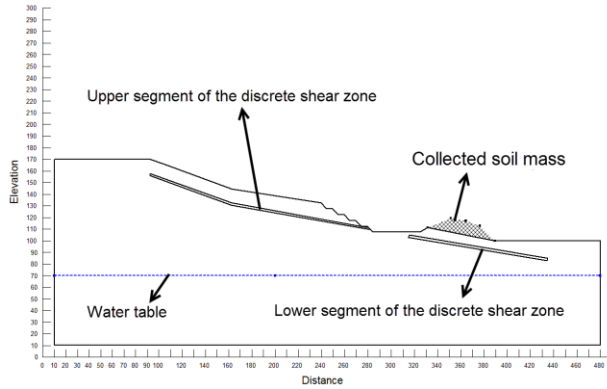


Figure 14. The discrete shear zone segmented due to soil removal (phase three)



Figure 15. The intersection of the discrete shear zone with the initial step above the R.O.W

## 2.4 Modelling Phase Three

As discussed, for modelling phase three, the R.O.W had to be expanded by 30 m with slopes of 1:1 with steps which have lengths and breadths of 5 m while there is no decrease in the level of the R.O.W. Furthermore as a tension crack existed from phase two, this component is also included into the model (the depth of the tension crack is 3 m for this phase). As in the previous phase, the entrance location of the slip surface is at the point where the geometry of the landslide changed and so consequently the entry and exit option is picked for generating the slip surfaces in Solpe/W.

It must be pointed out that the factor safety must drop in this phase compared to phase two of the analysis as the slope experienced larger displacements. The analysis is conducted using the shear strength parameters obtained from both the noncircular (optimized) and the circular (original) method and their results are compared in this section.

As mentioned, the original method of Spencer (circular) provided shear strength parameters for the soil on top and below the discrete shear zone, they are as follows as: unit weight  $20 \text{ kN/m}^3$ ,  $\phi' = 17^\circ$  and  $c = 4 \text{ kPa}$ , while the shear strength parameters for the discrete shear

zone are as follows: unit weight  $17 \text{ kN/m}^3$ ,  $c = 0 \text{ kPa}$  and  $\phi' = 6^\circ$ . The values are adopted in the third constructed phase and a factor of safety equal to 0.884 which is less than the second phase (0.973) is calculated. The result of this analysis is illustrated in Figure 16. It is interesting that as the discrete shear zone is segmented during the construction process, the minimum calculated factor of safety is only related to the upper part of the ancient land slide and consequently the existence of the semi buttress constructed by heaping up the removed soil does not influence the minimum factor of safety. Figure 17 illustrates the same analysis without the existence of the heaped soil along the R.O.W. As anticipated, changes made to the geometry significantly influenced the behavior of the ancient landslide.

The effects of suction at this stage can also be checked by the means of  $\phi_b$ . If a  $\phi_b$  of  $12^\circ$  is considered for this analysis a safety factor of 2.103 is calculated, which is the main reason for the soil mass not to heap up on the R.O.W after removing the slope's toe.

The same analysis is conducted with the shear strength parameters calculated by the optimized method without considering the effect of suction. In this phase the minimum factor of safety is equal to 0.972 for the noncircular method (in phase two it was 1.388).

As anticipated, the slip surface with the minimum factor of safety passes through the defect and will come out at the intersection of the discrete shear zone and the constructed step above the R.O.W, which is also illustrated in Figure 15.

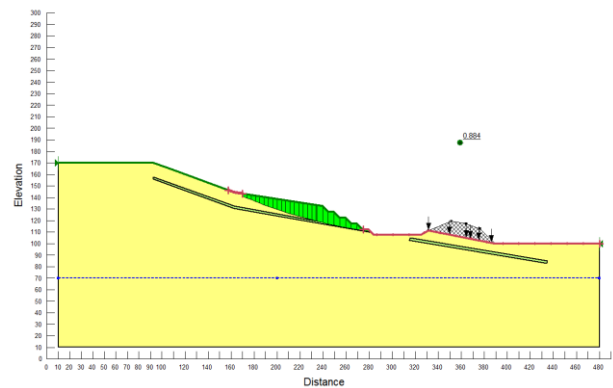


Figure 16. Results of the circular method for phase three

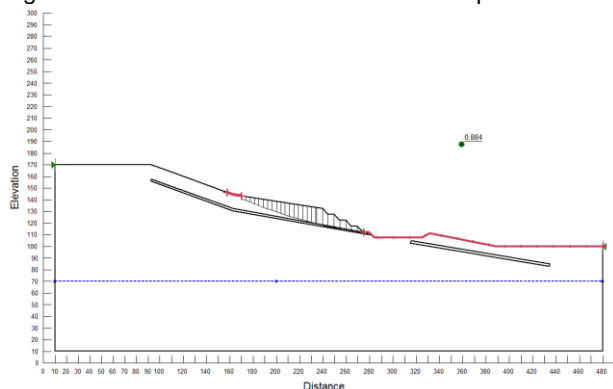


Figure 17. Results of the circular analysis for phase three, without considering the semi buttress

## 2.5 Phase Four

In this phase it was recommended that the soil must be removed from the head scarp downward (i.e. towards the R.O.W) with an angle of 45° and then to bury the gas pipe line 3.5 m below the ground level (at this stage the discrete shear zone is broken up into two segments). This is removing the soil from the upper segment of the slope which has an angle of 10° (Figure 7) up to the R.O.W; this requires removing a large amount of soil and further enlarging the R.O.W. As the geometry of the landslide changes and the defect is present in both sections of the slopes above the R.O.W (i.e. along the slopes of 10° and 20°), this recommendation did not sound acceptable as the minimum slip surface passes through the defects. This recommendation is considered as phase four and it is modelled in this section. Figure 18 illustrates the results of this analysis with the circular method with the back calculated shear strength parameters derived from phase one. It is clear that by removing the soil, the upper segment of the ancient landslide is activated and it can cause failure (the calculated factor of safety is 0.478). To increase the factor of safety in this case, soil must also be removed from the segment of the slope which has an angle of 20° (Figure 7) in the ancient landslide.

As in other phases, the effect of suction is investigated through adopting a value of  $\phi_b$  equal to 12. In this analysis the minimum factor of safety is equal to 1.543. This essentially means that this phase can also be completed but a heavy rainfall can waive the effect of suction and trigger a land slide.

It is also interesting to note that if the route is changed to the base of the landslide (i.e. behind the deposited soil mass, Figure 7) the heaped soil will cause instability for the R.O.W. Furthermore, as this site is not for human use, removing the soil from the section of the ancient landslide which moves up with an angle of 20° is costly and time consuming. Furthermore, constructing a buttress with a shear key that passes through the slip surface is also an expensive option for a R.O.W which is not used by humans.

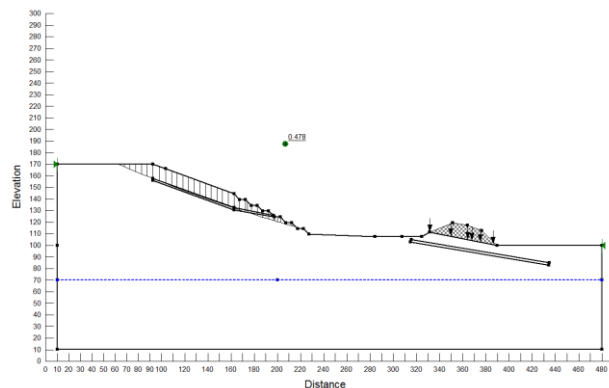


Figure 18. The minimum factor safety calculated with the circular method for phase four

It was recommended that for the width of the ancient landslide the route must be changed to the right of the access road (Figure 7) as it does not contain any steep

slopes and it is far from the soil deposited generated by removing the toe of the ancient landslide.

It is clear that if the ancient landslide was detected at earlier stages, all problems could have been avoided.

## 3 CONCLUSION

From back calculating the shear strength parameter of a translational ancient landslide with the aid of a two dimensional analysis the following conclusions can be derived:

When the difference between the calculated safety factors of the circular and noncircular methods are large, the results of the circular or the original method must be adopted. Usually the noncircular or the optimized method provides a lower factor of safety compared to the circular method. If the difference in the safety factors calculated between the two methods are large, the results can significantly influence the back calculated shear strength parameters. The case analyzed in this paper is a translational sliding mass and due to the large discrepancy between the results of the two methods, the back calculated shear strength parameters for the discrete shear zone and the soils above and below it are large as well. The back calculated shear strength parameters by the noncircular method are almost twice the values calculated by the circular method for the discrete shear zone and the soil above and below it in the investigated landslide.

It became clear that not only the shear strength parameters calculated from the circular method were appropriate; the depth of the tension crack anticipated by the circular method agreed with the tension depth generated in the field. In contrast, the noncircular method did not correctly estimate the depth of the tension crack at the head scarp for this case study.

For investigating the effects of suction in ancient landslides it is recommended that the values of  $\phi_b$  published by Fredlund (1987) to be used. This method is adopted due to the very mixed nature of soils in ancient landslides. It is evident that suction played a major role in the behavior of the analyzed slope for each investigated phase.

Changes in the geometry of a landslide can significantly influence the remedial process as any changes in the slope affects the minimum factors of safety and the location of the critical slip surface. It was clear that as the ancient landslide moved towards its graben with two different slope angles, the remedial process was significantly influenced.

Removing the toe of an ancient landslide will significantly influence the remedial process. This is due to the fact that the residual shear strength of the discrete shear zone is very small and consequently extensive measures must be taken to increase the slope's factor of safety after the toe is removed.

## 4 REFERENCES

Budhu, M. 2007. Soil Mechanics and Foundations. John Wiley and Sons. p. 578-588.



- Christian, J.T., Ladd, C.C. and Baecher, G.B. 1994. Reliability applied to slope stability analysis, *Journal of Geotechnical Engineering (ASCE)*, 120(12), 2180-2207.
- Cornforth, H.D. 2005. *Landslides in practice: investigation, analysis and remedial / preventative*.
- Duncan, J.M., Wright, S.G. and Brandon, T.L. 2014. *Soil Strength and Slope Stability, Second Edition*. John Wiley and Sons.
- Duncan, J.M. 1992. "State of the art: Static stability and deformation analysis." *Proceedings of stability and performance of slopes and embankments-II, ASCE*, 1, 222-266.
- Duncan, M.J., Navin, M. and Wolf, T.F. 2003. Discussion of Probabilistic slope stability for practice, by H. El-Ramly, N.R. Morgenstern and D.M., Cruden, *Canadian Geotechnical Journal*, 39, 665-683.
- Fredlund, D.G. 1987. Slope stability analysis incorporating the effects of soil suction. *Slope stability: Geotechnical Engineering and Geomorphology*, John Wiley and Sons, 113-144.
- Lupini, J.F., Skinner, A.E. and Vaughan, P.R. 1981. The drained residual strength of cohesionless soils. *Geotechnique* 31 (2) 181-213.
- Pedram, B. 2015a. Effects of pile shape in improving the performance of monopiles embedded in onshore clays. *Canadian Geotechnical Journal*, 52, 1-15.
- Pedram, B. 2015b. A numerical study on offshore wind turbines embedded in sand. *Journal of Geotechnical Research*, 2, 49-65.
- Pedram, B. 2018. Behaviour of hybrid piled footing structures in sands. *Journal of Geotechnical and Geological Engineering*. DOI 10.1007/s10706-018-0461-7.
- Spencer, E. 1967. A method of analysis of the stability of embankments assuming parallel inter-slice forces, *Geotechnique*, 17 (1), 11-26.
- Stark, T.D. 2003. Three dimensional slope stability method in geotechnical practice. 51<sup>st</sup> Annual Geotechnical Engineering conference.
- Stark, T.D. and Hussain, M. 2013. Empirical correlations: Drained strength for slope stability analyses, *Journal of Geotechnical and Geoenvironmental Engineering*, 139 (6), 853-862.
- Wood, D.M. 1990. *Soil Behaviour and Critical State Soil Mechanics*. Cambridge University Press. p. 219.