Reliability Analysis of an Existing Ash Basin Embankment

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ABSTRACT

This paper presents a case study to demonstrate the use of reliability analysis to evaluate an existing large ash basin which is contained by a clay embankment. For this project, the purpose of the reliability analysis was to evaluate the probability of a deep-seated slope failure. The results of these analyses were used to assist the owner with decisions regarding the necessity to improve the existing embankment. The calculated reliability index (β) was compared to the available literature to categorize risk of failure. The analysis included the assessment of the variability of shear strength and phreatic surface levels. The variability in shear strength parameters was evaluated from consolidated-undrained triaxial compression test results on site soils. The variability in phreatic surface levels was evaluated considering both existing water levels from piezometers and long-term steady state conditions. The findings of this study were used to confidently conclude that no immediate improvement of the slopes was necessary, resulting in approximately $5 million in cost savings.

INTRODUCTION

This paper presents a case study to demonstrate the use of a reliability analysis at an existing ash basin site that is contained with a clay embankment. The embankment was primarily constructed with 2 Horizontal: 1 Vertical (2H:1V) side slopes that are up to 13.5-m (44-ft) high. The reliability analysis was used to evaluate the probability of a deep-seated slope failure along a critical cross section. The reliability index (β) was calculated and used to assess the risk of not providing slope improvements. The investigation to perform this evaluation included the assessment of shear strength variability within the clay embankment and phreatic surface levels. The reliability analysis was performed with a commercially available two-dimensional slope stability program.

In December 2014 the United States Environmental Protection Agency (USEPA) provided a prepublication of the Coal Combustion Residual Rules (CCR Rules) that address the integrity of CCR pond embankments. This paper provides an overview of how the CCR Rules are related to a reliability analysis approach.

BACKGROUND

The embankment that contains the ash basin was constructed more than forty years ago using on-site soils that were excavated within the footprint of the Ash Basin. The embankment material is characterized as stiff to very stiff, consisting of lean clay with some sand and trace gravel. Review of historical documents revealed that the embankment was constructed using standard engineering and construction methods including compacting each lift to a specified minimum compaction level based on maximum dry density and optimum moisture content of the soil material that was compacted. The embankment is considered to be fairly uniform based on the results of a number of field investigations and laboratory testing programs conducted since construction was completed.

The subsurface soil conditions at the site (below the clay embankment) consist of an approximately 9- to 15-m (30- to 50-ft) thick stiff to hard silty clay layer with trace to some sand and gravel that generally gets progressively stiffer with depth. The bedrock below this soil unit is characterized primarily as dolomite with occasional interbedded shale. There are two phreatic surfaces at the site below the embankment: the upper phreatic surface was observed at depths ranging from 3 to 12 m (10 to 40 ft) below natural ground, and the confined lower phreatic surface in bedrock.

Portions of the embankment had developed surficial sloughs during its operating life, mainly as a result of freeze and thaw cycles over the years, steepness (2H:1V) of the original slopes, and dense shrubs and woody vegetation keeping the sun from improving evaporation on the side slopes. The owner implemented various operations and construction activities over a period of five years to improve the condition of the entire embankment. In general, the mitigation program included reconfiguring 70 percent of the outer embankment slopes by removing the sloughed zones and reconstructing the grades to 2H:1V and 2.5H:1V depending on the severity of the sloughing. The remaining portions of the embankment were improved by clearing undesirable vegetation and restoring desirable vegetation where the embankment did not show evidence of slope distress.

In addition to the mitigation program, the owner took a proactive role and conducted a potential failure mode analysis (PFMA) to evaluate the anticipated long-term performance of the embankment. One of the concerns that was raised as part of the PFMA process was the potential failure of the embankment along a critical portion of the slope where only vegetation was cleared as part of the mitigation program (also corresponding to where the slopes remained at approximately 2H:1V). One method to evaluate this concern was to perform a reliability analysis to estimate the probability of failure. Herein, failure is assumed to correspond to a computed static slope stability factor of safety, FS, of 1.0 or lower.

One of the outcomes of the PFMA was to install slope inclinometers and piezometers. The data obtained from piezometers were used to estimate the existing phreatic surface. The data obtained from slope inclinometers indicates that no significant movement had been recorded to-date. These instruments are continuing to be used to provide an early warning system for the long-term performance of the embankment.

OVERVIEW OF RELIABILITY ANALYSIS

Reliability analysis provides a tool to assess the effect of uncertainties of important parameters in a slope stability analysis. A slope stability analysis alone does not account for the variability (or uncertainty) of parameters that affect the safety of the structure. For example, an embankment with a FS value of 1.5 may not necessarily mean it is "safer" than an embankment with a FS of 1.3. To appropriately account for this variability, slope stability analyses can be augmented with a reliability analysis. However, it is important to have a data set that is representative of the site and large enough to perform the analysis with confidence.

A reliability analysis uses probability-based methods to deal with reliability in a more comprehensive way than do stability methods (USACE, 2006)[1]. According to Christian (1996)[2]:

> *"The reliability index, β provided a better indication of how close the slope is to failure than does the factor of safety alone. This is because it incorporates more information on the uncertainty in the values of the factor of safety. Slopes with large values of β are farther from failure than slopes with small values of β regardless of the value of the best estimate of the factor of safety".*

The reliability index, β, is calculated as:

$$\beta = \frac{FS_{mean}\text{-}1.0}{\sigma_{fs}} \qquad \text{Equation 1}$$

where FS_{mean} is the average FS of all the analyses results and σ_{fs} is the standard deviation in the computed FS. For example, even if both slope stability evaluations resulted in the same FS_{mean} of 1.5, one result may have a σ_{fs} = 0.2 resulting in β = 2.5, and the other result of σ_{fs} = 0.1, which would result in and β = 5.0. This analysis would indicate that the slope with β = 5.0 would be safer than the slope with β = 2.5.

Based on USACE (2006)[1], values of β greater than 3 represent a "stable" slope condition whereas values of β less than 2 represent slopes expected to be poor performing.

FIELD INVESTIGATION AND DATA INTERPRETATION

Several field investigations and laboratory testing programs were conducted during the design phase of the mitigation program. Samples were obtained from test pits and borings advanced through the embankment and subgrade soils at various locations around the perimeter of the Ash Basin.

The index properties of the embankment and subgrade soils are provided in Figures 1 and 2. Subgrade and embankment soils are grouped in these figures as the embankment was constructed from the on-site soils within the footprint of the structure. Both soils are classified as low plasticity clay.

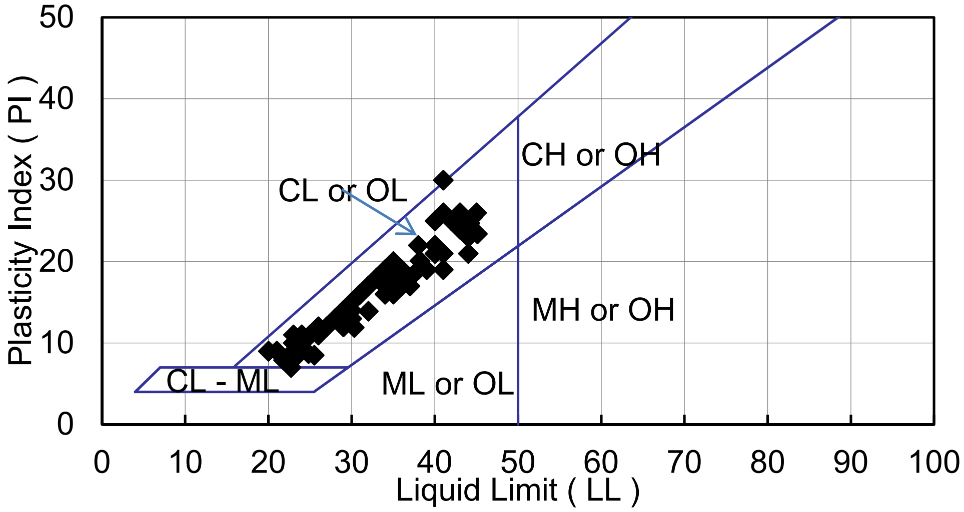

Figure 1. Atterberg limit results from embankment and subgrade soils.

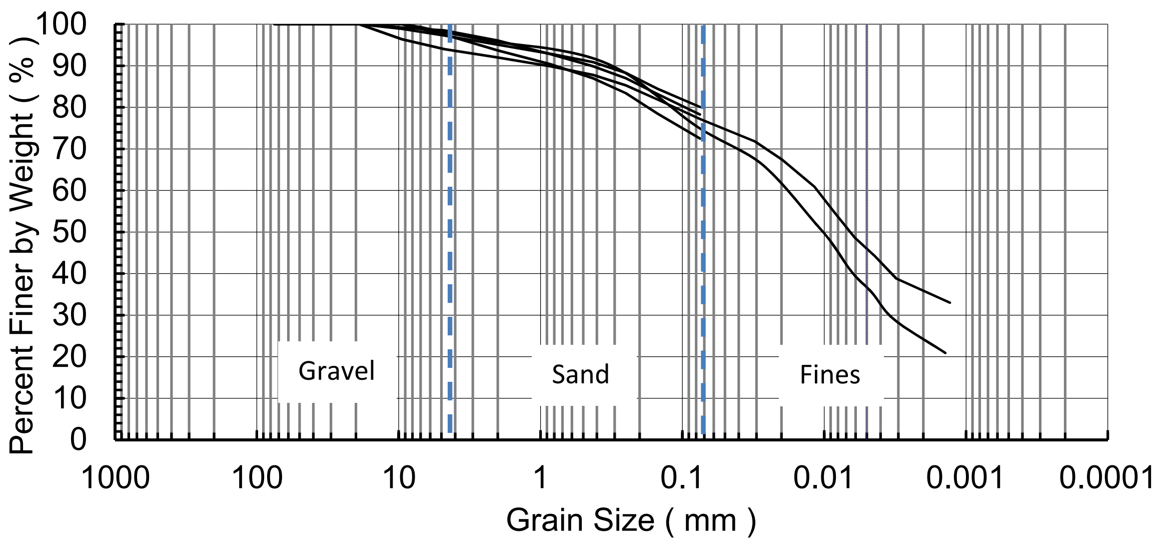

Figure 2. Typical grain size distributions of the embankment and subgrade soils.

A total of 31 Consolidated – Undrained (CU) triaxial compression tests were conducted; 23 on embankment soil samples and 8 on subgrade soil samples. Tests were conducted at stresses that would be expected in the field with consolidation stresses ranging from 17.5 to 175 kPa (2.5 to 25 psi). The data obtained were used to define the mean and standard deviation values for drained shear strength parameters.

The variability in phreatic surface levels was evaluated considering both existing and interpreted long-term conditions. The phreatic surface for the existing condition was derived from data obtained from piezometers installed in the embankment in the critical cross section location. The phreatic surface for the long-term condition was estimated from the water levels within the ash basin and at the toe of the embankment using a finite element seepage model.

INPUT FOR THE RELIABILITY ANALYSES

The reliability analyses were performed by selecting a critical cross section where the embankment was the highest and steepest within the area of concern. This implies that the reliability index for other portions of the embankment where slopes are shallower or the slope height is lower would have a higher reliability index (assuming other parameters were the same).

Analyses were performed for a given range of input values that influence the calculated FS, including cohesion intercept, c', and friction angle, ϕ'. The phreatic surface within the embankment and subgrade were input as constants to estimate the probability of failure for the “existing” condition and long-term conditions separately. The locations of the piezometers, and the interpreted existing and long-term phreatic surfaces are depicted in Figure 3. The unit weights of the soils were input as average values and were kept constant for the analyses because the effect of variability in unit weight was judged to have only a minor effect on the reliability index.

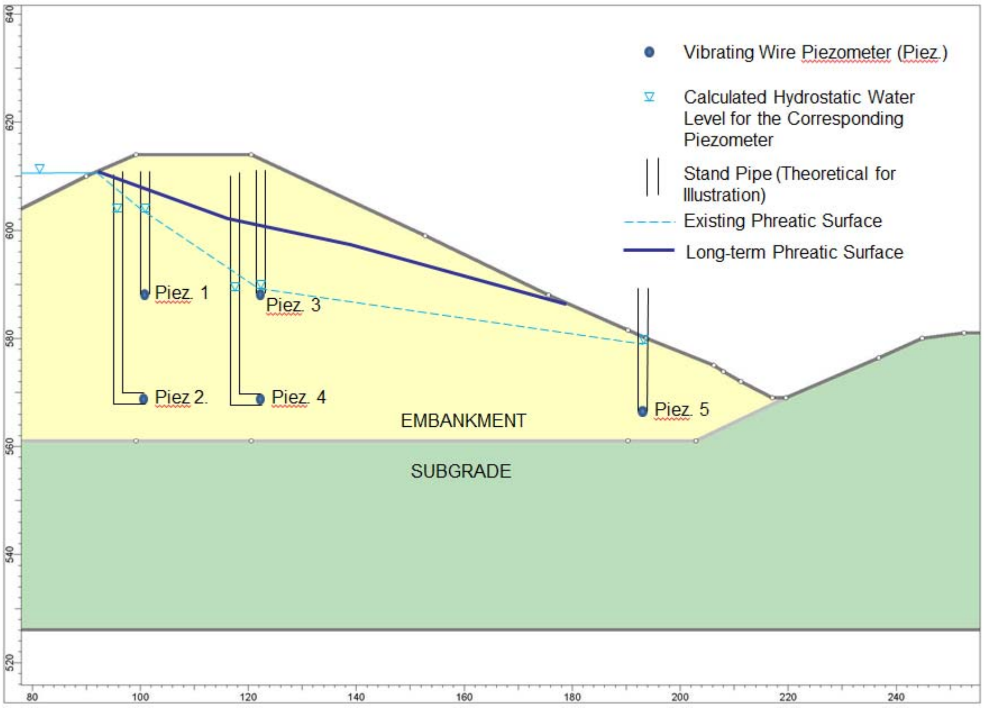

Figure 3. Existing and long-term phreatic surfaces used for the analyses.

The shear strength values used in the reliability analyses were selected based on their respective standard deviations (σ) and the relationship developed between the cohesion intercept, c', and the friction angle, ϕ', both of which are explained in detail below.

Standard deviations for c' and ϕ' were estimated based on a reasonable lower bound values and the "six-sigma rule". The six-sigma rule indicates that the average (or mean) value of a normally distributed variable is 3σ away from the "highest possible value" and "lowest possible value" (USACE, 2006)[1]. The following procedure was implemented to estimate σ values for c' and ϕ':

1) Calculated the average c' and ϕ' values with a "best-fit" line.
2) Establish the "reasonable" lower bound shear strength values.
3) Calculate the difference between the average and lower bound c' and ϕ' values and divide it by three to estimate the standard deviation (σ) values for c' and ϕ'.

Figures 4 and 5 provide shear strength envelope boundaries and standard deviations for strength parameters for embankment and subgrade soils, respectively.

The correlation factor between c' - ϕ' was established based on shear strength values that were interpreted from individual set of tests that constitutes the entire data set of 31 CU tests (see Figure 6). In general, for soils, as the friction angle increases the cohesion intercept decreases; similarly as the friction angle decreases the cohesion intercept increases. This relationship would correspond to a correlation factor. For the data set used herein, the correlation factor between c' and ϕ' is dependent on the data circled in Figure 6. If this particular data point is excluded, the c' - ϕ' correlation factor would be -0.44. Therefore, the reliability analyses were performed using c' - ϕ' correlation factors of -0.44 and -0.86.

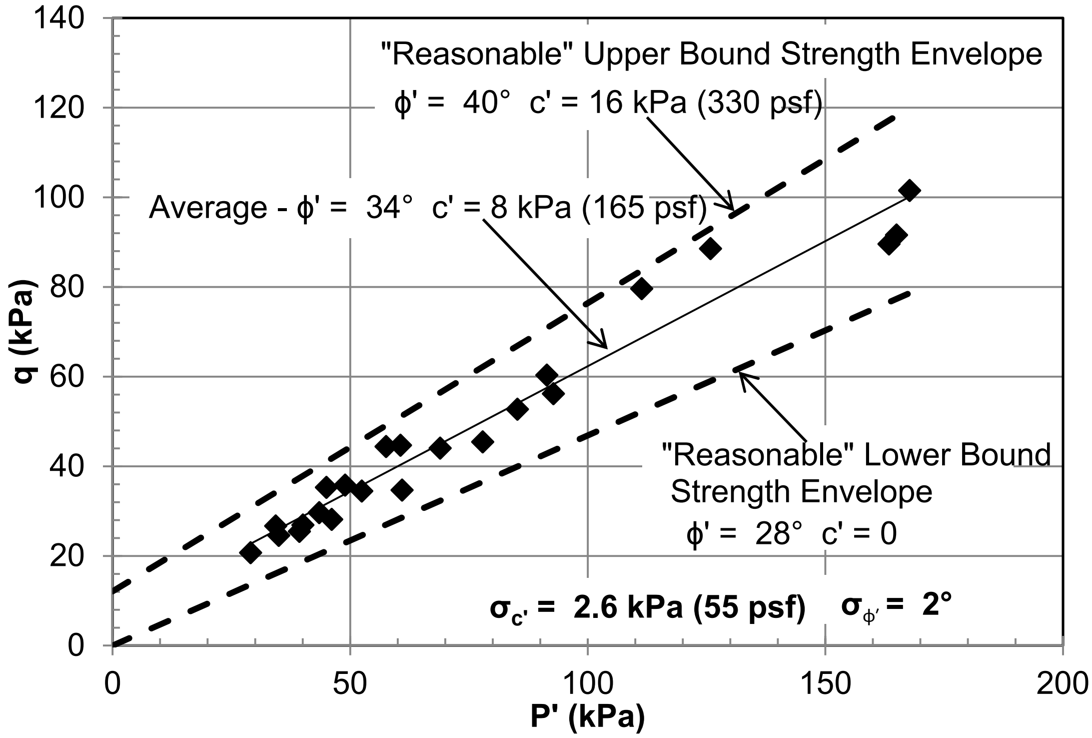

Figure 4. Shear strength envelopes used for embankment for reliability analysis.

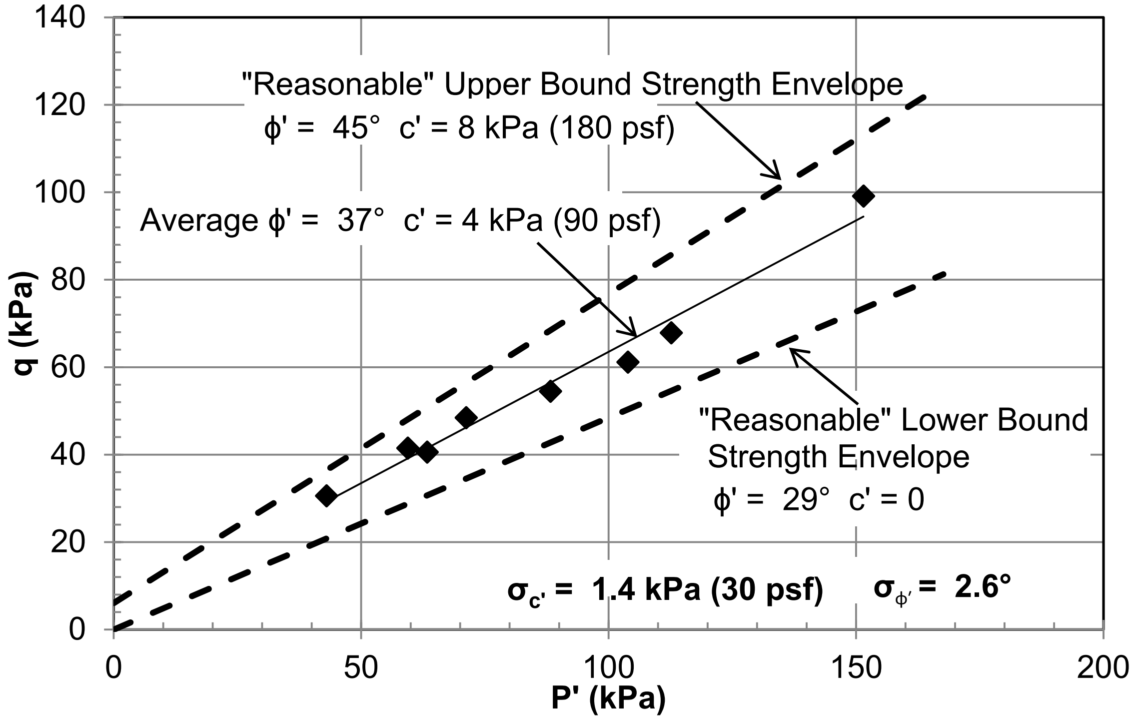

Figure 5. Shear strength envelopes used for subgrade for reliability analysis.

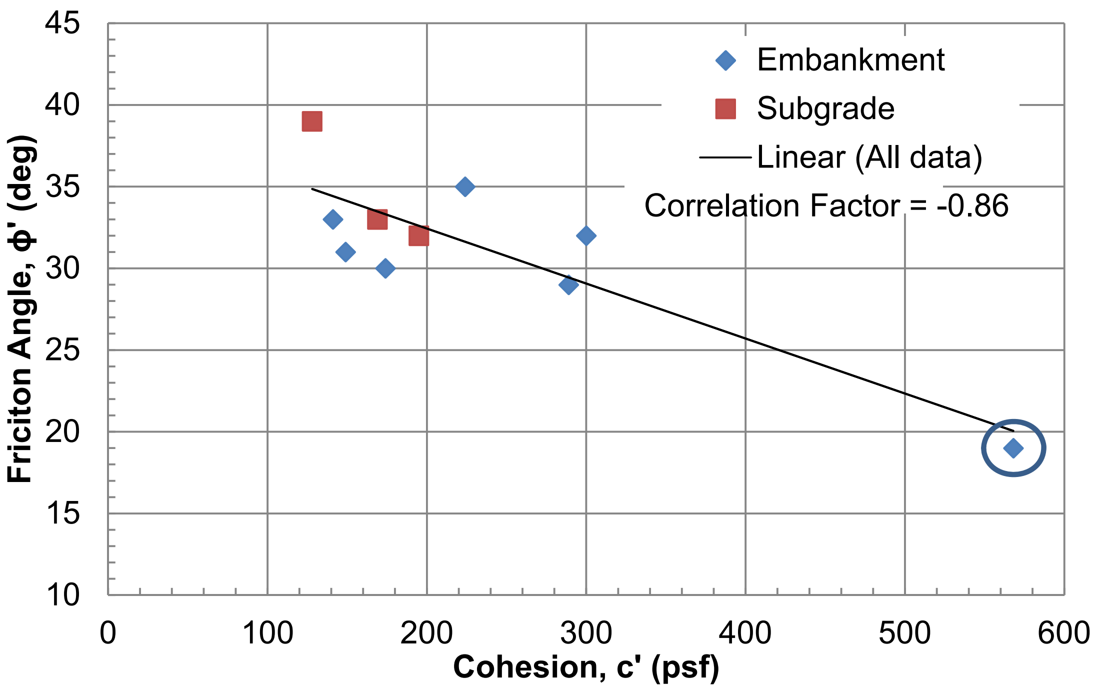

Figure 6. c' - ϕ' correlation.

RELIABILITY ANALYSIS RESULTS

Results of the reliability analysis based on the existing phreatic surface condition are provided in Figures 7 and 8. The probability of failure is calculated to be less than 0.0001% and the FS value is calculated to be 1.62, which is obtained based on the average shear strength parameters. The mean FS is estimated to be 1.63, which is the average of FS calculated based on 10,000 different c' - ϕ' combinations. The reliability index, β is estimated as 11.97 for c' - ϕ' correlation factor of -0.86, and 6.08 for c' - ϕ' correlation factor of -0.44. These are high reliability indices.

The results of the reliability analysis based on the estimated long-term phreatic surface are provided in Figures 9 and 10. The probability of failure is calculated to be less than 0.0001% and the FS is computed as 1.39. The mean FS is estimated to be 1.39, which is the average of FS calculated based on 10,000 different c' - ϕ' combinations. The reliability indexes were determined as 7.59 for c' - ϕ' correlation factor of -0.86, and 4.01 for c' - ϕ' correlation factor of -0.44.

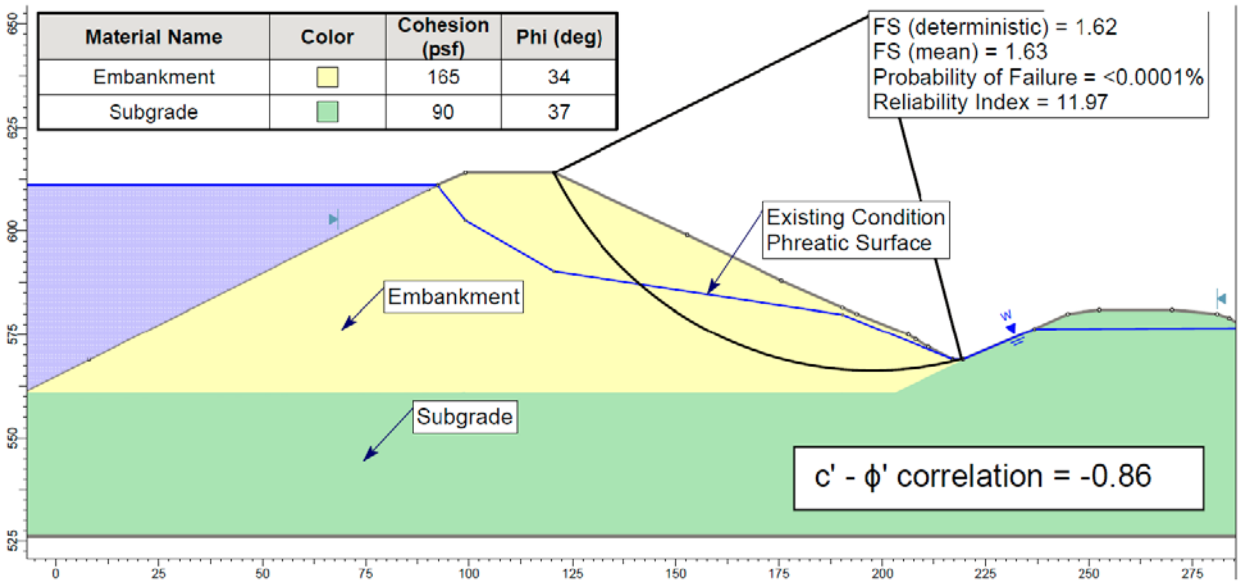

Figure 7. Reliability analysis result for existing condition using c' - ϕ' correlation factor of -0.86.

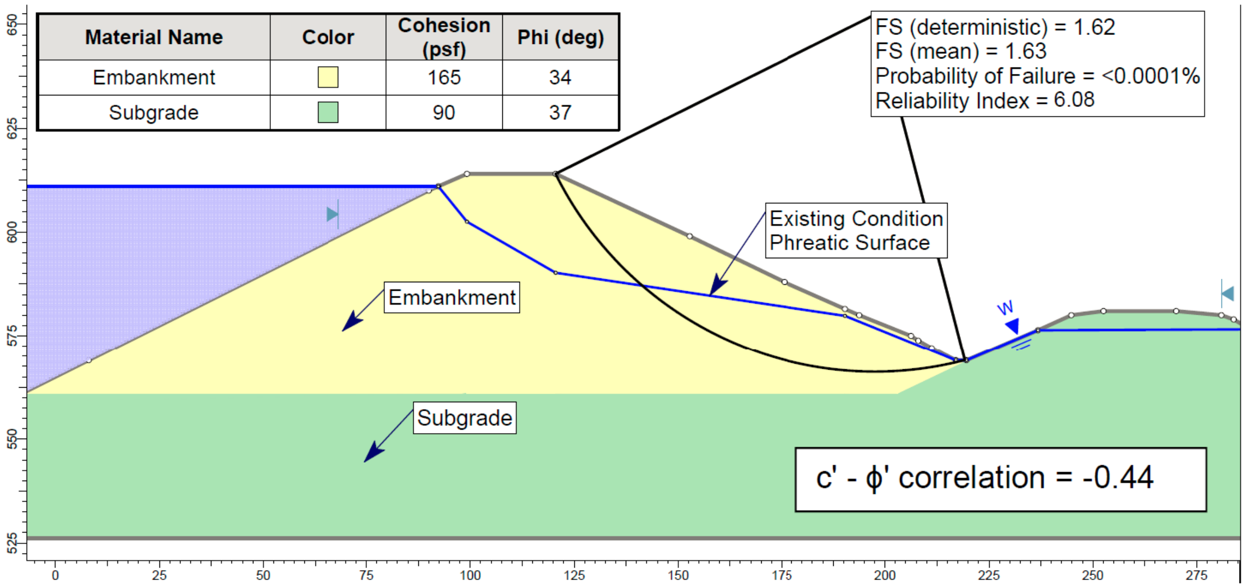

Figure 8. Reliability analysis result for existing condition using c' - ϕ' correlation factor of -0.44.

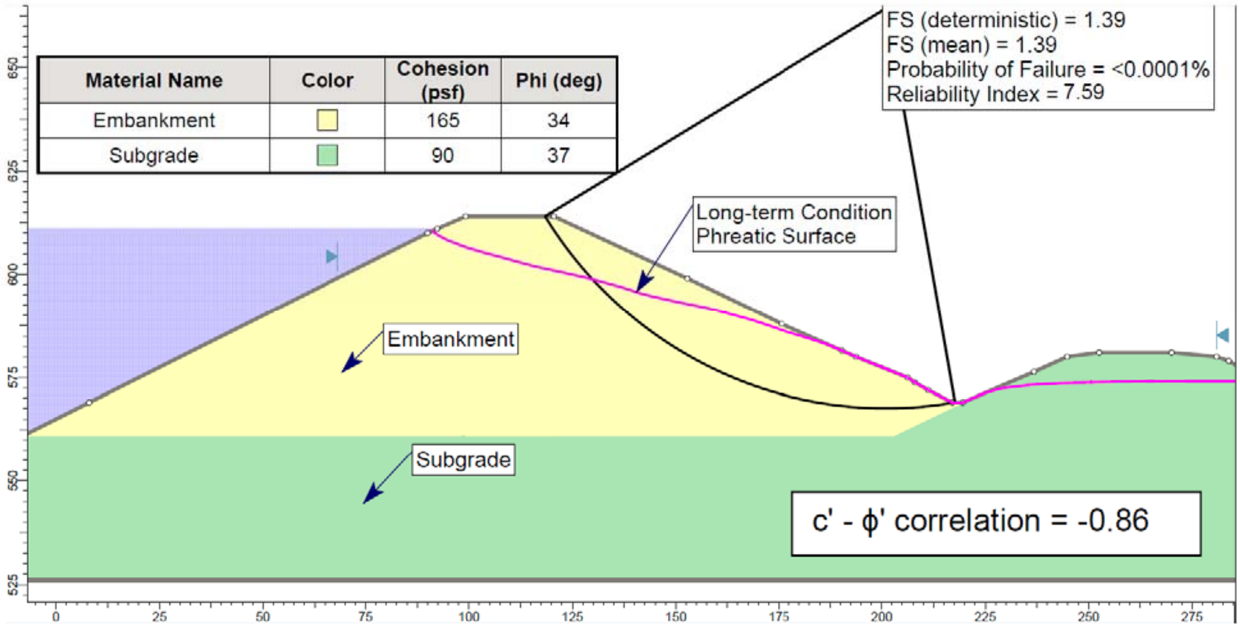

Figure 9. Reliability analysis result for long-term condition using c' - ϕ' correlation factor of -0.86.

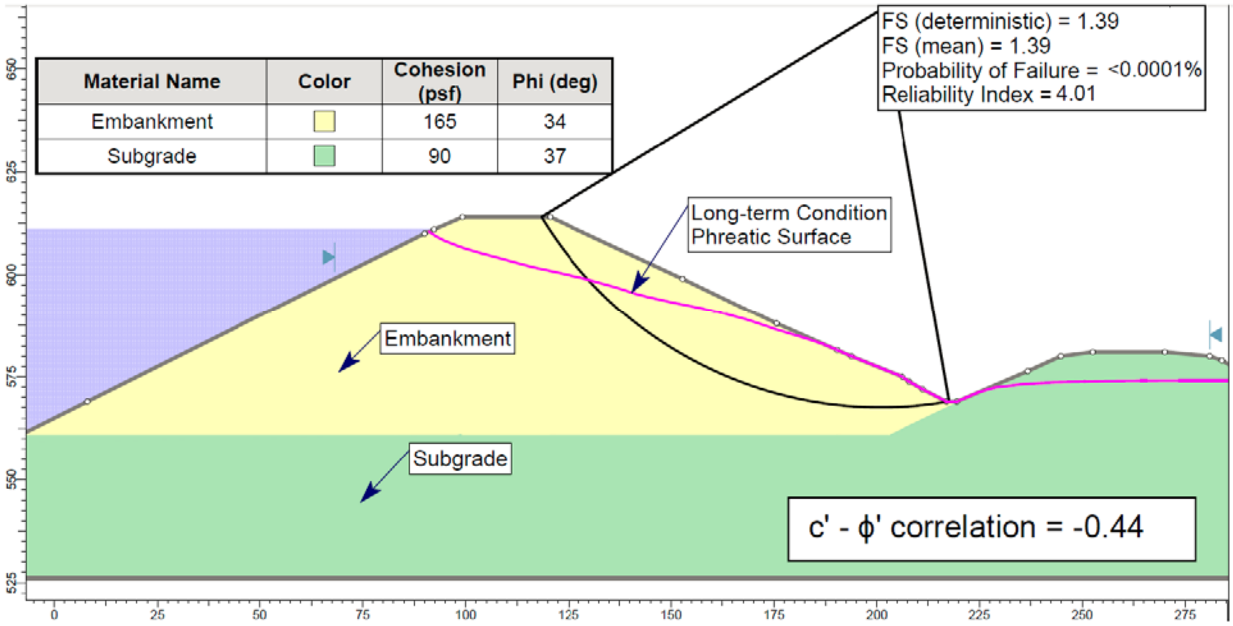

Figure 10. Reliability analysis result for long-term condition using c' - ϕ' correlation factor of -0.44.

CONCLUSIONS AND RECOMMENDATIONS

The reliability index values computed in this study are greater than 4. These reliability index values are greater than 3, which represents a stable slope condition based on USACE (2006)[1]. Based on these results, it was concluded that no immediate mitigation measures were necessary for the section of the embankment that was analyzed. The owner decided to continue with the implementation of the existing inspection, monitoring, and maintenance programs. It is recommended that if, in the future any changes were noted to the geometry of the embankment or there are changes to operational procedures, the stability of the embankment should be further assessed.

The findings of this study provided an approximately $5 million in cost savings as the results showed no immediate mitigation was necessary.

THE NEW CCR RULES AND RELIABILITY ANALYSIS

On 19 December 2014, USEPA provided a pre-publication of the coal combustion residuals (CCR) rules (Rules) providing criteria on design, operations, monitoring procedures and how to demonstrate structural integrity of CCR units. Under the Rules, a reliability analysis is not considered as a method of demonstrating structural integrity of CCR units. Instead, the Rules prescribe a Safety Factor (a.k.a. “Factor of Safety”, FS) values that should be attained for different loading conditions ranging from 1.00 for seismic condition to 1.50 for the long-term static condition for existing and new structures.

The authors’ disagree with the prescription of a FS of 1.50 as the only measure for evaluating existing slopes as it is contrary to the well-established engineering slope stability assessment practice as documented by the USACE (2003)[3]. The USACE engineering manual (2003) was used in the USEPA's development of the FS criteria but the USEPA did not follow the USACE approach for existing slopes. The excerpt below is based on USACE (2003)[3], which is referenced in the Rules as one of the main documents used for determining static factor of safety criteria:

> "*What is considered an acceptable factor of safety should reflect the differences between new slopes, where stability must be forecast, and existing slopes, where information regarding past slope performance is available. A history free of signs of slope movements provides firm evidence that a slope has been stable under the conditions it has experienced. Conversely, signs of significant movement indicate marginally stable or unstable conditions. In either case, the degree of uncertainty regarding shear strength and piezometric levels can be reduced through back analysis. Therefore, values of factors of safety those are lower than those required for new slopes can often be justified for existing slopes.*
>
> *Historically, geotechnical engineers have relied upon judgment, precedent, experience, and regulations to select suitable factor of safety for slopes. Reliability analyses can provide important insight into the effects of uncertainties on the results of stability analyses and appropriate factors of safety."*

In the authors' opinion, the reliability analysis process provides a tool that should be used as a means of demonstrating structural integrity for embankments exhibiting FS < 1.50. A FS of < 1.50 may be considered as "acceptable" given an acceptable reliability index and provided the evaluated slope has an inspection, monitoring, and maintenance program. Under the proposed rule, using the average shear strength parameters as appropriate for a stability analysis, the analysis result provided in Figures 9 and 10 for the long-term water pressure condition with FS value of 1.39 would, by itself, not be a sufficient demonstration of stability. However, the coupling of this FS value with a well-developed reliability analysis should be considered a sound approach to evaluate existing embankments.

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