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EVALUATING STABILITY OF RIVERBANKS REINFORCED WITH ROCKFILL COLUMNS

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ABSTRACT: This paper presents the results of stability analysis of natural and stabilized riverbanks. Limit Equilibrium Method (LEM) and Finite Element Method (FEM) were used in the analysis. In the FEM, two approaches were employed: 1) combined LEM and FEM, and 2) FEM with the shear strength reduction (SSR) technique. The limit equilibrium method and finite element method of stability analysis produced similar calculated factor of safety values. This provides confidence that either method estimates similar factor of safety values for natural and stabilized riverbanks. Based on the FEM of analysis, it was shown that the centre-to-centre spacing between columns plays a major role in increasing the factor of safety. The calculated maximum shear strain in the stabilized riverbank is reduced with a closer spacing of rows. This is important since lesser slope displacements are required to mobilize the necessary shear resistance for closer rockfill column spacing.

Keywords: Stability analysis, rockfill columns, slope stabilization, finite element method, shear resistance

INTRODUCTION

The City of Winnipeg, Manitoba, Canada and surrounding area is flat, with the only major natural relief being the valleys of the Red and Assiniboine rivers and their tributaries. The City boasts over 240 km of natural riverbanks. The riverbanks provide value to both private owners and the public as recreational and parkland areas. The riverbanks are continuously undergoing change under natural erosion processes, resulting in riverbank failures that can have a dramatic impact on private dwellings and park areas (Fig. 1). As a result, there is a constant challenge of how to economically protect the riverbanks from landslides while maintaining the natural environment of the riverbank area. The City of Winnipeg is continuously examining methods to improve riverbank stability in areas that are prone to movement to ensure that the areas adjacent to the rivers will be preserved for future generations (Alfaro, et al. 2008).

The soil deposits in the City of Winnipeg consist predominantly of lacustrine clay rich soils and alluvial (river) deposits of layered clays, silts and sands. They were deposited by proglacial Lake Agassiz. As ice sheets melted northwards, runoff formed a large lake in what now covers large portions of Manitoba, northwestern Ontario and northeastern Saskatchewan in Canada; and eastern North Dakota, and western Minnesota, both of which are US states. At its largest extent, Lake Agassiz was larger than all of the current Great Lakes combined. The present Lake Winnipeg is a remnant of this ancient lake (Coduto 1999). The Lake Agassiz deposit can be considered 'lacustrine lowland' in the terms used by Miura et al. (1994). The lacustrine clay soils are relatively weak, resulting in instability along many stretches of the two primary rivers within Winnipeg. Field observations (Baracos and Graham 1981), (Peterson et al. 1960) on failed riverbanks have shown that potential slip surfaces passed through the weakest clay zone located at the clay-till interface.

Fig. 1 Instability along riverbanks

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Fig. 2 Drilling for installation of rockfill columns

Fig. 3 Placement of rockfill into the drilled shaft

Fig. 4 Compacting rockfill columns with vibrolance

A number of traditional stabilization alternatives use structural elements such as piles and walls to stabilize the riverbanks. These approaches are invasive to the environment and can be quite costly. A method that is gaining widespread attention is the use of rockfill columns to reinforce the weaker lacustrine clay soils (City of Winnipeg 2000), (Yarechewski and Tallin 2003). The rockfill columns (also known as stone columns) are large diameter (2m-3m) drilled shafts filled with coarse rockfill. The concept in principle is that weak clay is removed near the base of the riverbank and replaced with strong coarse rockfill. The common practice in Winnipeg is to drill the large diameter shafts (Fig. 2) with a piling rig into competent bearing strata below the lacustrine clay, along the lower riverbank region. The holes are backfilled from surface with crushed limestone (Fig. 3) and may then be compacted with a vibratory lance (Fig. 4). The coarse rock provides increased strength over the weaker clay that is removed and replaced. Installations of rockfill columns are usually done in winter primarily because winter construction in the soft soils at the lower bank of slope is easier. The lower water level during winter and ice cover over the river increases the work area and minimizes the potential environmental impacts of the work (Thiessen et al. 2010).

Results of stability analysis of typical natural and stabilized riverbanks in the City of Winnipeg are presented in this paper. Comparison of results using different methods of analysis is discussed. Two common methods of analysis are used to assess the stability of natural and stabilized riverbanks: Limit Equilibrium Method (LEM) and Finite Element Method (FEM). The LEM has been used extensively by geotechnical engineers mainly because of its simplicity. However, FEM has many advantages. One of the significant advantages of FEM is its ability to compute the stresses and strains in the riverbank. A slope stability analysis is available that combines LEM and FEM in which the FEM is used to determine the stresses more accurately in the soil mass and then use these stresses in the LEM to determine the factor of safety of the slope (Geo-Studio 2004). In this combined FEM and LEM approach, the numerically computed stresses from FEM are used to establish the resisting and driving shear forces along a potential slip surface and then use the summation of these forces to compute a safety factor. The pure FEM approach can be categorized into two general methods: shear strength reduction method (SSR) and gravity increase method (GIM).

A number of researchers (Shukla and Baker 2000), (Griffith and Lane 1999), (Dawson, et al. 1999), (Duncan 1996), (Matsui and San 1992) reported several advantages of the SSR technique including the ability to predict stresses and deformations of support elements, such as piles, anchors and geosynthetic reinforcements, at the verge of failure (Hammah, et al. 2005). However, Krahn (2006) cautioned the use of SSR because of its inherent limitations that affect its usefulness in practice. One limitation of the SSR technique is that the solution is based on non-convergence of the solution. He pointed out that this is very unconventional in engineering design analyses in that engineers usually rely on converged solutions rather than non-converged solutions. The second limitation, as Krahn pointed out, is that the strength of soil is reduced throughout the earth structure by an equal amount. This means that the local safety factor is taken to be constant along the entire slip surface which is not the case in reality. The local safety factor in fact can vary significantly along the potential failure surface. For the purpose of this study, the stability of the riverbank is evaluated using three approaches, namely: 1) LEM, 2) combined LEM and FEM, and 3) FEM-SSR. The results from these methods are compared.

LIMIT EQUILIBRIUM METHOD (LEM)

A factor of safety, *FS* is traditionally calculated using the limit equilibrium method. The Slope/W (GeoStudio 2004) and Slide 5 (Rocscience 2004), two commercial computer programs, which implement the limit equilibrium methods, were chosen. Two traditional limit equilibrium methods: Bishop, and Morgenstern and Price methods were used to analyze the slope stability of the riverbank being examined. These methods are arguably the most popular in the consulting industry. Typical riverbank geometry in the Winnipeg area is shown in Fig. 5. This geometry was used for all the analysis work done throughout this study. Typically the water table is about 2m below the ground surface (Tutkaluk 2000). In our analysis, however, this was positioned at the ground surface based on the observations by local contractors and engineers. They indicated that the water can be at the ground surface during spring flooding. Flooding can occur during spring season due to the combined snow melt and rainfall.

FINITE ELEMENT METHOD (FEM)

The FEM with shear strength reduction (SSR) approach was used in this study because it is more suited for analyzing the stability of natural slopes (Swan and Seo 1999). A combined FEM and LEM using Sigma/W and Slope/W (GeoStudio 2004) were also conducted for comparison of analysis results.

Phase 2.0 (Rocscience 2005), an SSR-based FEM computer program, was chosen to perform the numerical analysis. The model used 6-noded triangular elements, and was set to a maximum of 500 iterations and a tolerance value of 0.001. These settings are recommended values given by Rocscience.

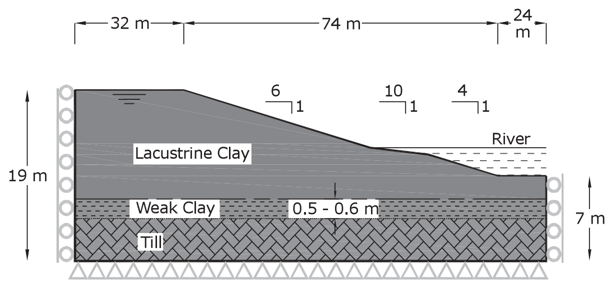

Fig. 5 Typical geometry of Red riverbank in Winnipeg (after Tutkaluk 2000)

The values of soil modulus for both high plastic clay and rockfill material are required for the soil constitutive model used to analyze the performance of the riverbank. Presently, the SSR method is limited to using elastic and elastic-perfectly plastic constitutive soil models. Values of equivalent Young's modulus, E and Poisson's ratio, ν, in combination with shear strength parameters are required to compute the stresses and deformations. A rigorous elasto-plastic model such as Modified Cam Clay model suitable for high plastic clays is yet to be incorporated into the computer program.

One common laboratory strength test is the direct shear test as schematically illustrated in Fig. 6. In this figure, $\Delta\tau$ is the change in shear stress, Δx is the change in horizontal displacement, Δz is the change in vertical displacement, D is the sample diameter, and h is the sample height. Large direct shear testing was used in this study to accommodate the tests on rockfill materials. To define the appropriate Young's modulus, E, results from large-scale direct shear tests were used to estimate the shear modulus, G. Rough estimates for G can be obtained using Fig. 6 and the equation given by Davis and Selvadurai (1996)

$$G = \frac{\Delta\tau}{\Delta x} h \qquad (1)$$

The values of shear modulus were used to calculate the Young's modulus for both clay and rockfill soils using linear elastic assumptions

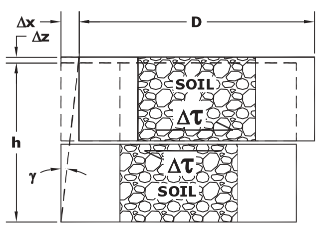

Fig. 6 Direct shear specimen after shearing (after Davis and Selvadurai 1996)

$$E = 2G(1+\nu) \tag{2}$$

The elastic parameters taken from direct shear tests must be used with caution given the fact that the tests create non-uniformity of both stress and strain fields in the soil sample (Davis and Selvadurai 1996). Hammah, et al. (2005) pointed out that Young's modulus had minimal impact on factor of safety results. However, this value will affect the calculations for instabilities. The Poisson's ratio, ν was calculated using the neutral earth pressure method (Tschebotarioff 1973)

$$K_o = \frac{\nu}{1-\nu} \tag{3}$$

The value of earth pressure, K_o was calculated using the Jaky (1948) equation

$$K_o = 1 - \sin\phi \tag{4}$$

ANALYSIS OF NATURAL AND STABILIZED RIVERBANKS

The soil properties used in the analysis are shown in Table 1. They have been determined from laboratory direct shear tests (both large-scale and conventional scale direct shear tests) conducted as part of this study. Details of the procedures and results of large-scale direct shear tests can be found in Alfaro, et al. (2009).

Natural Riverbanks

Figures 7 and 8 show the calculated factor of safety of a natural riverbank for both Slide 5 and Slope/W computer programs, respectively using Morgenstern and Price's method. As illustrated in these figures, the factors of safety were both close to unity. Although not shown, the factors of safety using Bishop's method were also close to unity. There was good agreement between the two software programs in terms of the estimated factors of safety. The results also showed that the potential slip surface passed through the weakest clay zone located at the clay-till interface, consistent with field observations on failed riverbanks (Baracos and Graham 1981), (Peterson, et al. 1960).

The calculated factor of safety from FEM using the shear strength reduction method was unity (Fig. 9). A combination of LEM and FEM using Geo-studio estimated the factor of safety was also about unity as shown in Fig. 10. These results were similar to the factor of safety estimated using the LEM.

Table 1 Soil properties for both lacustrine clay and the weak soil used in the analysis

Soil Type	E MPa	ν	γ kN/m^3	ϕ (°)	c kPa
Clay	5	0.4	17.0	15	4
Weak clay	3.5	0.4	15.7	12	3
Rock column	22	0.2	19.0	50	0
*Till Layer	170	0.2	22.0	50	0

Stabilized Riverbanks

The slip surface predicted by the numerical analyses

Fig. 7 Stability analysis for natural riverbank using LEM - Slide 5

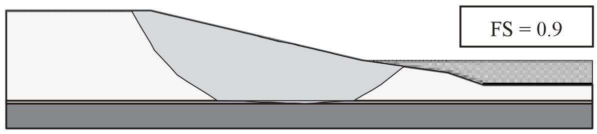

Fig. 8 Stability analysis for natural riverbank using LEM - Slope/W

Fig. 9 Stability analysis for natural riverbank using FEM - Phase 2.0

Fig. 10 Stability analysis for natural riverbank using FEM and LEM - Sigma/W

helped to determine the most efficient location to install the rockfill columns. Hassiotis, et al. (1997) recommended that reinforced concrete piles must be placed in the upper middle portion of the slope to provide the optimum improvement of factor of safety. Similarly, the FWHA (1983) recommended that stone columns should be installed in the middle of the predicted slope failure in engineered slopes. Consequently, the rockfill columns were placed at the mid-span of the potential failure surface. A triangular grid pattern of rockfill columns was assumed with a centre-to-centre spacing of columns equal to twice the column diameter. Since the analysis was two-dimensional, the rockfill columns had to be represented by an equivalent strip element as shown in Fig. 11. The conversion of rockfill columns to strip elements is

$$t = \frac{A}{s} \tag{5}$$

where t = the thickness of equivalent strip element, s = the spacing between columns, and A = the cross sectional area of the rockfill column.

Rockfill columns also act as a drain to relieve the pore pressure at the interface layer between the weak soil and the till. The hydraulic conductivity of rockfill columns is relatively high compared to the surrounding clay. This was assumed equal to 0.02 meter/sec for FEM analysis.

The average shear strength along the potential slip surface will increase significantly as a result of rockfill column installation, serving to stabilize and reinforce the riverbank. The rockfill columns should extend deep into the stiff till to act as a key (pin) into the stiff layer in order to force the failure surface through the rockfill columns instead of beneath them at the rockfill-till interface.

Figure 12 shows the layout of five rows of rockfill columns with diameter of 2.3 m. The factor of safety using the limit equilibrium method (Slide 5) was calculated to be about 1.6 using the Morgenstern and Price method. This shows that that replacing weak soil in the discrete shear zone of a failing riverbank with well compacted rockfill columns improved the stability of the riverbank. The results from FEM using the SSR method indicated a factor of safety equal to 1.4 as shown in Fig. 13. This means that SSR provides a more conservative solution compared to LEM. The results of the comparison of factors of safety between LEM and FEM-SSR are consistent with those observed by Hammah et al. (2005) for slopes stabilized with piles wherein the factors of safety calculated using FEM-SSR are generally less than those calculated from LEM.

The maximum shear strain contour shown in Fig. 13 is a good indication of the potential slip plane. It was observed that the potential slip plane moved from the interface between the till layer and the weak clay upward and through the rockfill columns. The mobilized shear resistance of rockfill columns is highly influenced by the effective normal stress applied at the sheared plane. The increase in depth does not increase the shear resistance in the clay as much as it increases in the rockfill columns. This phenomenon forces the shear failure plane to move upward where the applied normal stress is less.

The stability results obtained using Slide 5 and Phase 2 software programs have been compared with the results from Slope/W and Sigma/W in order to perform combined LEM and FEM analysis. Results obtained showed strong agreement between the software programs. Figure 14 shows the factor of safety from combined LEM and FEM analysis is equal to 1.4. Moreover, the modes of shear failure were similar. Good agreement was found between these analyses and the field observations in terms of the size and depth of the shear failure plane in the natural riverbank in Winnipeg

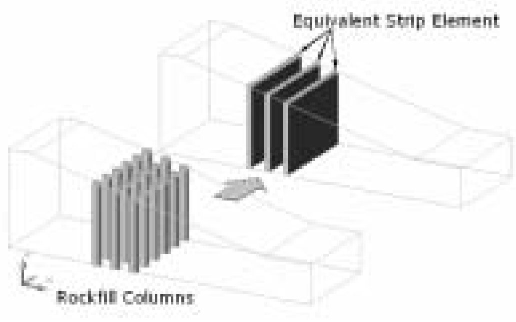

Fig. 11 Equivalent strip elements of rockfill columns

Fig. 12 Stability analysis for stabilized riverbank using LEM - Slide 5

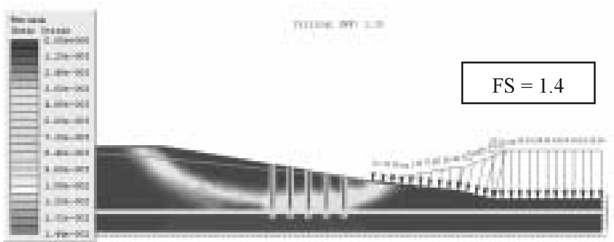

Fig. 13 Stability analysis for stabilized riverbank using FEM – Phase 2.0

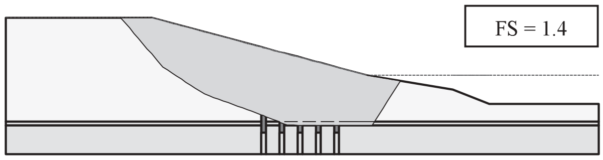

Fig. 14 Stability analysis for stabilized riverbank using FEM and LEM– Sigma/W and Slope/W

(see Baracos and Graham 1981, Peterson, et al. 1960).

To illustrate the importance of properly selecting the locations of rockfill columns along the riverbank, analysis was carried out with columns located further up and down in the slope. In addition, analysis was carried out to examine the design geometry such as the spacing and configuration of columns that provide optimum performance of rockfill column installation. The assessment of the stability has been conducted using limit equilibrium method (LEM) to calculate the factor of safety.

Figure 15 illustrates, that the vicinity of the centre of the potential shear failure is the best position for rock columns to maximize the factor of safety. These results confirm the conclusions by Hassiotis, et al. (1997) and Ito, et al. (1979) on pile-stabilized slopes. They reported that the optimum location of reinforcement is at mid-span of the potential shear failure surface. However, they are in contrast with the results by others (Ausilio, et al. 2001), (Lee, et al. 1995) where they indicated that piles appear to be very effective when they are installed in the region from the middle to the toe of the slope. The reason for the conflicting analysis results may be due to deep-seated failure surface for the case of our studies as opposed to shallow sliding surfaces for the case studied by Ausilio, et al.

The effect of the ratio of the clear distance between columns, S_2 to the centre to centre distance, S_1 was investigated. The FEM with SSR technique was used in the investigation. When the spacing between rows of rockfill columns is altered, the location of the potential slips failure changes (compare Figs. 16 and 17). The lower value of S_2/S_1 the columns work as a group, while at the higher ratio the columns behave individually. This is demonstrated by the higher concentration of shear strain in the vicinity of the columns for S_2/S_1 = 0.40 compared to that for S_2/S_1 = 0.6. Another finding observed from FEM analysis is that the calculated maximum shear strain is reduced with the closer spacing of rows. This is important as lesser slope displacements are required to mobilize the necessary shear resistance for stabilization for closer rockfill columns.

Some cases (Yarechewski and Tallin 2003), (Tweedie, et al. 2004) have shown that movements can occur following installation of rockfill columns. Uncertainty regarding the magnitude of these movements required to mobilize shear resistance in the rockfill columns has resulted in situations where the stability of riverbanks following remediation has been questioned. The results of the analysis shown above provided an improved understanding about how the rockfill columns will be best laid out.

SUMMARY AND CONCLUSIONS

Limit equilibrium method and finite element method of stability analysis produced similar calculated factor of safety values. This provides confidence to use either method to estimate the factor of safety of natural and stabilized riverbanks.

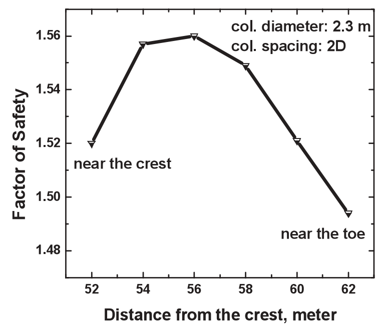

Fig. 15 Effect of column locations on the estimated factor of safety

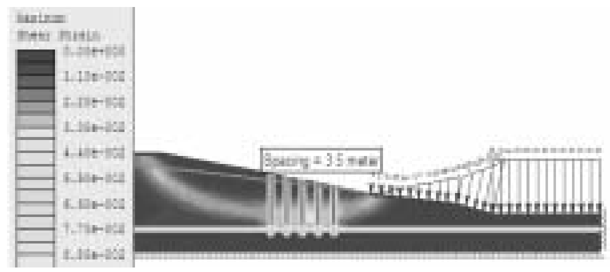

Fig. 16 Maximum shear strains for spacing ratio S_2/S_1 = 0.40

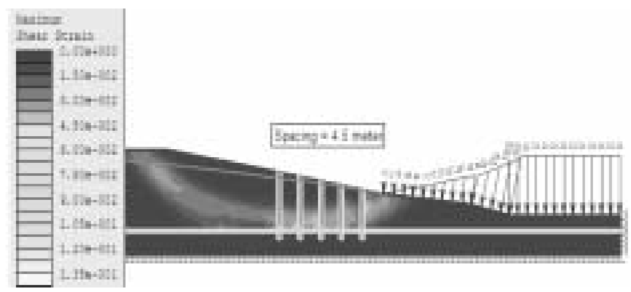

Fig. 17 Maximum shear strains for spacing ratio S_2/S_1 = 0.60

Installations of rockfill columns increase the factor of safety significantly, and the potential shear failure will move from the zone of weak clay at the clay-till interface upward into the clay zone. It was shown that the optimum location for a group of columns is in the vicinity of the centre of the potential shear failure surface.

The centre-to-centre spacing between columns plays a major role in increasing the factor of safety. It was found that the calculated maximum shear strain in the stabilized riverbank is reduced with the closer spacing of rows. This is important as reduced slope displacements are required to mobilize the necessary shear resistance for closer rockfill columns.

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