Monitoring results from a full-scale field test of rockfill columns

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ABSTRACT
This paper will detail the performance monitoring of a rockfill column stabilized natural riverbank subjected to test loading. A 19m section of riverbank was stabilized with eleven (11) 2.1m diameter, compacted rockfill columns. The full-scale field test involved the staged placement of 1930 tonnes of material upslope of the columns causing lateral displacements in the columns and surrounding soil. The site was monitored extensively with slope inclinometers, in-place inclinometers, vibrating wire piezometers, and surveys. Monitoring results from a select number of the 17 total sub-surface installations are presented. Preliminary interpretation of the natural riverbank performance is presented.

RÉSUMÉ
Ce papier démontre le contrôle du rendement d'une rive naturelle stabilisée par une colonne d'enrochement, ceci est sujet à une charge d'essai. Une section de la rive, d'une taille de dix-neuf mètres, a été stabilisée avec onze (11) colonnes d'enrochement compressé, chacun 2,1 mètres en diamètre. L'essai en vraie grandeur comprenait le placement étagé de 1930 tonnes de matériels; la pente des colonnes suscite un déplacement latéral des colonnes et du sol autour. Le site a été instrumenté avec des inclinomètres de pente, des inclinomètres sur place, des piézomètres à cordes vibrantes et des levés d'arpentage. Certains résultats des dix-sept installations souterraines sont montrés. Une interprétation préliminaire du fonctionnement naturel de la rive est aussi présentée.

1 INTRODUCTION AND BACKGROUND

Rockfill columns have become a preferred form of riverbank stabilization in the Winnipeg area for practical, environmental and economic reasons. The common practice in Manitoba is to drill large diameter (1.5-3m) holes into till, along the mid to lower bank. The holes are backfilled from the surface with crushed limestone and may then be compacted with a vibratory lance. Although most applications of rockfill columns have been successful, there is a limited understanding as to how the columns behave when stressed. Some rockfill column installations have exhibited larger than expected deformations after their construction.

Large scale direct shear tests of rockfill material and composite rockfill/clay samples have provided valuable insight into how the columns behave when sheared along a plane (Abdul Razaq 2007), but observed field performance has suggested that serviceability, rather than peak strength may sometimes be the critical limitation (Yarechewski and Tallin, 2003). This research project was initiated by government and industry to gain a better understanding of the behaviour of the rockfill columns and reinforced slope during column installation and when placed under significant stress. The purpose of the field test was to determine the mobilized shear resistance of a stabilized riverbank. This is accomplished by measuring the displacements and calculating the in-situ stresses, resultant from loading since the mobilized shear resistance cannot be measured directly.

The experimental component of this project consisted of the installation, compaction, test loading and monitoring of full-scale, rockfill columns. The chosen test site was a 19m long section of natural clay riverbank on the Red River in Winnipeg, Manitoba. The two (2) rows of rockfill columns were installed with a triangular spacing pattern as commonly used in local practice. These columns were installed in November to December of 2007. The test loading took place between February 21 and February 29, 2008 (days 1-9) with unloading beginning on March 3, 2008 (day 12).

A further discussion of the site conditions and project design can be found in Thiessen et al. (2007). It should be noted that the final implementation of the field test varied from the published plans (Thiessen 2007) in a number of ways. The most significant variation was the installation of a row of weakening holes along each edge of the test section. Seventeen (17) 0.48m diameter holes were drilled into the clay to within 0.3m of the till contact elevation on each edge of the test section. These holes were left empty for the duration of the field test. This weakened vertical surface improves the plane-strain assumption used in much of the numerical modeling by reducing the "drag" from the outside edges of the mobilized soil mass. These weakening holes also provided a secondary means of containing displacements to the desired test area.

The scheduling of site works was controlled largely by outside factors. Works on the lower banks of the Red River are preferably done after the fall drawdown of the river, which begins in mid-October. The period between column installation and test loading was maximized to allow post construction slope movements to subside. As well, the slope loading and site restoration was to be completed prior to the spring freshet which usually occurs near the end of March.

1.1 Site Conditions and Stratigraphy

The research test site is located on an outside bend of the Red River. The subsurface conditions are typical of the Winnipeg area, with glaciolacustrine clay overlying silt

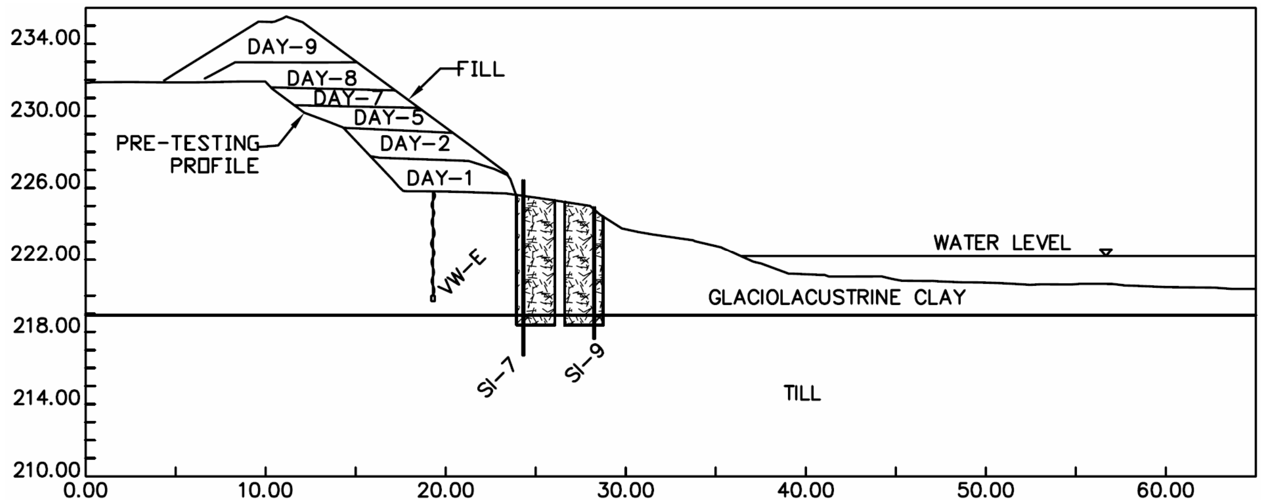

Figure 1. Cross section through centerline of rockfill column test site showing fill staging and select instrumentation

till. The elevation of the floodplain was approximately 232m, while the average till contact elevation was 219m. The upper till was considered loose, but competent enough to anchor the columns. The columns were installed from a working pad at an elevation of approximately 225m. A cross section along the centerline of the test site is shown in Figure 1. The thickness of the frost was measured at a number of places, and varied from 0.3 to 1m in thickness. To reduce the effect of the frost, the frozen soil was removed along the crest of the test site where it was found to be the most thick and critical to test performance.

2 INSTALLATION, PREPARATION AND TESTING

2.1 Column Installation

Column installation followed standard local practice. The columns are 2.13m in diameter and extend 1m (typical)

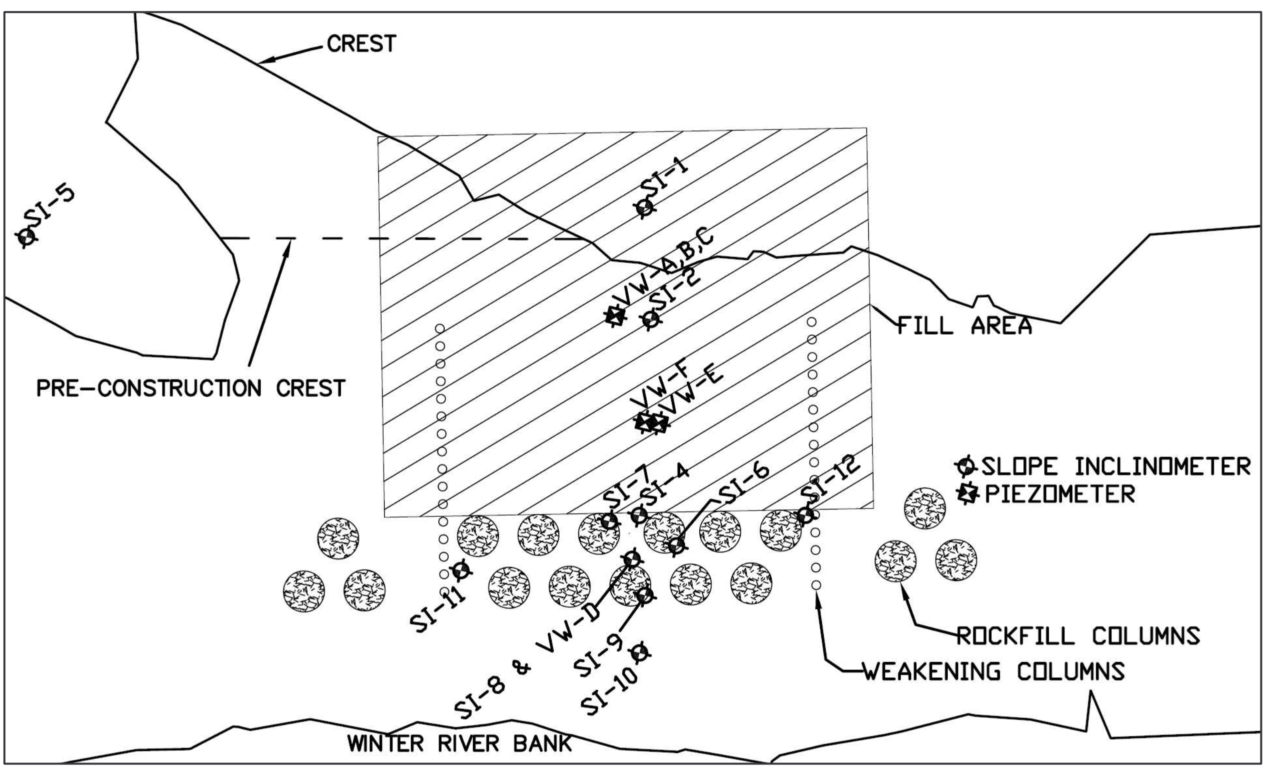

Figure 2. Layout of rockfill column test site

into the till. Figure 2 shows the general site layout and placement of the rockfill columns and weakening columns.

The columns were pre-bored using a pile drilling rig. The open shaft was immediately backfilled to the surface with 150mm down crushed limestone rockfill. The rockfill was compacted with the hydraulic vibrolance seen in Figure 3. The columns were charged with water to help the lance penetrate the rockfill, and also to increase the density achieved by the compaction effort. To advance the lance through the rockfill, air was jetted through the tip, while vibrating its way down.

Figure 3. Compacting columns with vibrolance

2.2 Field Test

The field testing of the columns was designed to happen over a relatively short time-span to induce excess pore water pressures, while still allowing time for the necessary monitoring. Having a break between consecutive lifts also allowed the observation of early time-dependant effects.

All time will be referenced to 08:00 February 21, 2008, unless otherwise noted, for the purpose of this discussion. Time 0d was the time that the first load of fill was delivered to the test site. The final load of fill was placed at time 8.2d and unloading began at 11d and finished on day 13. Re-grading work began at 18d and was completed on day 28.

The photos in Figures 4 and 5 were taken on day 7 and day 8 respectively

Figure 4. Aerial view of rockill column research site during test loading with photo taken from tethered aerostat

Figure 5. Rockfill Column research site during test loading

2.3 Loading

The field test consisted of loading the upper portion of the slope over a period of seven (7) working days. The fill material was 50mm clean limestone. The material was delivered by truck, and then placed with a tracked excavator. In total 1930 tonnes of fill material were placed. Figure 1 shows the centerline profile at the end of each day and Figures 7, 8 and 9 show the loading rate with respect to time. No additional load was placed on days 3, 4, and 6. In general, approximately one half of each day's load was placed in the morning after an initial round of inclinometer monitoring. The second round of fill was placed in the early afternoon, allowing for critical inclinometer installations to be surveyed in the intervening time.

3 INSTRUMENTATION

The purpose of the instrumentation program was to capture changes in ground conditions over an entire year preceding major site works and then during the installation of the columns, during the load test and after the completion of the major site works. Subsurface deformations and porewater pressures were of the greatest interest and to that end, slope inclinometers and piezometers were the main types of instrumentation. GPS-survey, video and both ground based and aerial photos were also used to track developments. Aerial photos were taken remotely from a tethered aerostat.

The locations and corresponding names of the subsurface installations are found in Figure 2.

3.1 Slope Inclinometers

A total of 11 slope inclinometer casings were installed, as is shown in Figure 2. The slope inclinometer (SI) layout generally follows the suggested plan of Turner and Schuster (1996) based on the expected area of movement.

Three (3) of the slope inclinometer casings, SI-6, SI-7 and SI-9 were installed directly into rockfill columns. The SI-6 casing was placed into the column boring before backfilling. The casing was held in place while the boring was backfilled from the surface. The backfilling and subsequent compacting of the rockfill deformed the casing somewhat, but it remained readable. To install SI-7 and SI-9, a steel casing was placed in the open column boring before backfilling. After the rockfill was compacted, the ABS casing was placed inside the larger diameter steel casing. As the outer steel casing was removed the annulus was backfilled with silica sand. Both of these methods worked sufficiently well, and have provided valuable deformation data. All other casings were backfilled with a cement-bentonite grout.

3.1.1 In-Place Inclinometers

Six (6) in-place inclinometers (IPI) were used to provide a continuous record of subsurface deformations. Three (3) bi-axial sensors were installed in SI-8 and three (3) uni-axial sensors were installed in SI-9. The bi-axial sensors were chosen for SI-8 in order to pick up any potential lateral displacement of the clay around the columns. The sensors in each hole were placed in a continuous string, each with a 1m span for a total of 3m of coverage in each casing. The sensors were installed so that they bracketed the weak soil zone immediately above the clay-till interface where the greatest deformations were anticipated.

The in-place inclinometers were temporarily removed, and the full length of the casing surveyed, after the completion of the loading and at the end of the unloading stages to provide a full depth deflection profiles.

3.2 Piezometers

Six (6) vibrating wire (VW) piezometers were installed as the primary means of measuring pore water pressures. VW piezometers were chosen for their ability to respond to rapid changes in porewater pressure in soils with low hydraulic conductivities. The vibrating wire piezometers were backfilled with a cement-bentonite grout in accordance to the methods proposed by Mikkelson and Green (2003). A string of three (3) vibrating wire piezometers (VW-A, B and C) were installed in a single borehole during the initial site investigation in August 2006. An additional two (2) push-in vibrating wire piezometers, VW-E and F were installed near the center of the loading pad. VW-D was installed in the same borehole as SI-8 to measure potential porewater pressure changes in the *in-situ* clay amongst the columns.

To measure till porewater pressures, the casing for SI-4 was slotted, wrapped in filter cloth and backfilled with sand to 1.5m up from the bottom. The rest of the annular volume was backfilled with cement-bentonite grout. SI-7 was used as a standpipe piezometer in the same way to measure the water levels in the rockfill column.

3.3 Survey

The site was surveyed daily to record the fill profile and volume as well as to pick up any surface displacements relative to the base receiver. The tops of the inclinometer casings were also surveyed regularly throughout the load test but it was found that the displacements were small enough that the survey equipment did not record any meaningful trends.

3.4 Data Collection

The vibrating wire piezometers and in-place inclinometers were monitored continuously with a Campbell Scientific CR-23x data-logger and appropriate peripherals. The data could be downloaded at any time to a spreadsheet for quick updates on any changes to the subsurface conditions. A full round of data was collected at ten-minute intervals, which provide sufficient resolution.

An electrical short in the wiring of a relay board caused the data-collector to fail six (6) hours after the last load of fill was placed on day 8 of the field test. The problem was corrected on day 10, but unfortunately, all intervening observations were missed.

4 MONITORING DATA

Only selected data will be included in this discussion and a complete presentation and analysis of all the data is forthcoming as the research progresses. Data from SI-7 and SI-9 as well as pore water pressures from VW-E will be presented and discussed.

4.1 Deformation Monitoring

Regular monitoring of SI's 1,2,4 and 5 as well as VW's A, B and C began in September 2006 and carried on through to the present, (where instrumentation has not been damaged). Reading intervals were approximately four (4) to six (6) weeks during the summer and winter months, with increased monitoring in spring and fall when river levels and groundwater conditions combine to theoretically reduce the factor of safety. These four (4) inclinometers were monitored closely during and immediately following the installation of the columns in November and December 2007.

A monitoring schedule was set up for the duration of the loading and unloading so that all inclinometers would be read at least twice per day. Priority installations, including SI's 4,6,7,and 10 were read at least three (3) times per day when loading was taking place, and 1-2 time per day on days when no additional load was placed.

All inclinometer data has been corrected for orientation, with the A+ direction being parallel to the dip direction of the natural slope, and thus perpendicular to the rows of rockfill columns. Other minor corrections have been applied to the raw inclinometer data as judged to be necessary.

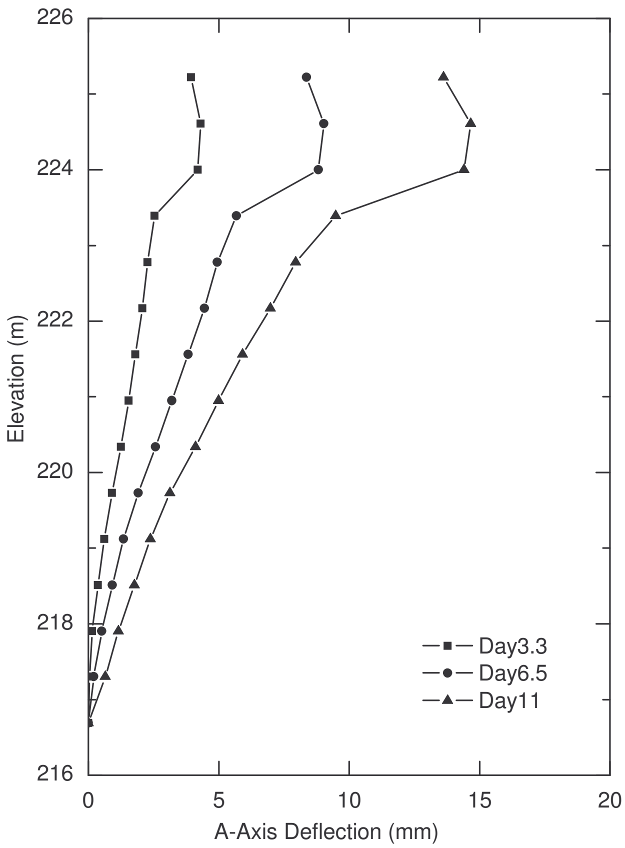

Figure 6. Deflections vs. depth as measured with a slope inclinometer. Baseline prior to test loading

Figure 6 shows the displacement vs. depth for SI-7 at three different times referenced to a baseline at the start of the test loading. One can see that from the base of the inclinometer casing to the surface, it is "leaning" in a nearly linear fashion. The one zone of increased displacement is between 223.5m and 224m where more significant shear deformations developed. A preliminary hypothesis is that increased deformations occur above 223.5m because practical limitations prevent the upper portion of the columns from being compacted to the same density as the main part of the column. The fact that the entire column is displacing indicates that the entire column is being stressed, and that the column has not reached a state of yielding at depth. Future work and publications will aim to verify column performance with numerical modeling results. Figure 7 plots total deflection vs. time for three (3) elevations. The displacement profiles of the clay both up-slope and down-slope of the columns are similar to the displacement profiles of SI's-6, 7 and 9 in the columns.

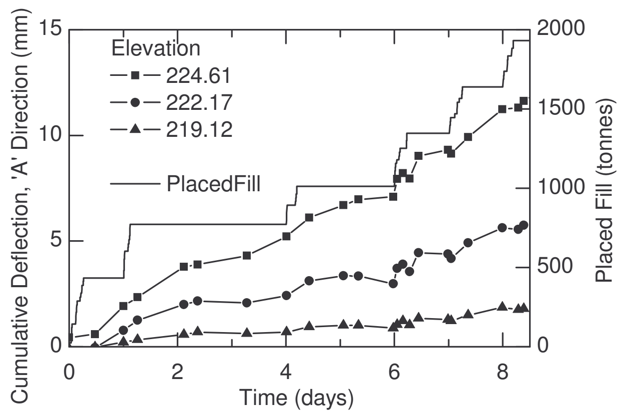

Figure 7. SI-7 displacements at three (3) depths and placed loading fill vs. time

Figure 8 shows the deviations measured by an in-place inclinometer situated over the span between 219.0 and 220.0m in SI-9. These observations demonstrate the rapid initial response to each additional lift of fill, followed by a decreasing rate of displacement with time, prior to the placement of the next lift of fill. Despite some technical difficulties with the in-place inclinometers, they did provide a very valuable set of data, especially when complimented by the regular full surveys of neighbouring standard inclinometer casings. The final observed displacement was confirmed at the end of the test with a standard inclinometer survey of the full length of the casing. The displacement response with respect to time of this in-place inclinometer corresponds fairly well with that shown in Figure 7.

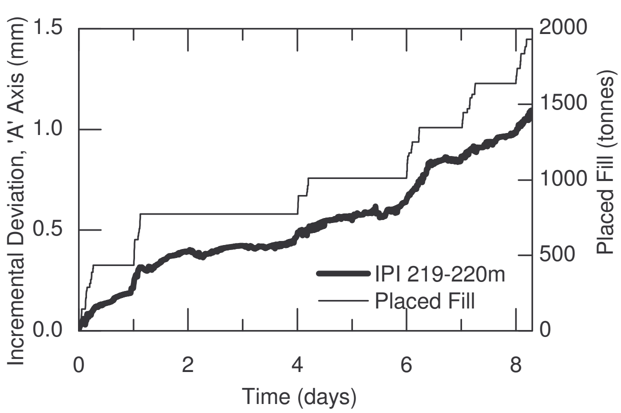

Figure 8. SI-8 IPI deviations and placed fill load vs. time

4.2 Porewater Pressure Monitoring

Figure 9 shows the pore pressure response as reported by VW-E, which is a push in piezometer installed beneath the initial loading footprint. Note that the response is the greatest over the first two (2) days of loading, even though incremental increase in total load is similar to the loading of following days. After the second day of loading the additional fill was no longer being placed directly over VW-E, and therefore the porewater pressure response is decreased with additional loads. Piezometers VW-A, B and C, which are not presented in detail here, respond more dramatically as fill began to be placed immediately above their locations.

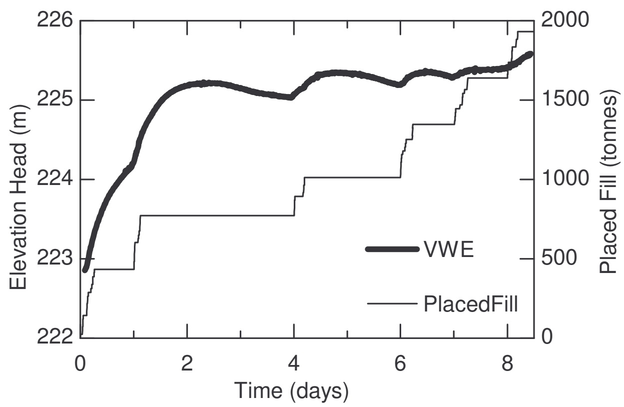

Figure 9. VW-E porewater pressure response and placed fill load vs. time

Piezometer VW-E shows a slightly delayed response, with pressures only reaching equilibrium approximately 1.5 days after the placement of the load. This may be due to air entrapped in the tip from the push-in installation process.

Very little change was seen in the till or rockfill porewater pressures measured in inclinometer casings SI-7 and SI-2 because of the relatively high hydraulic conductivity of the rockfill and till materials. The interconnectivity between the columns and the till was reflected in the observed hydraulic heads.

5 FUTURE WORK AND CONCLUSIONS

The full-scale field testing of the rockfill column has been a success. The test has shown that compacted rockfill has a friction angle significantly higher than has previously been assumed in designs. The test has also shown how proper compaction can affect the stiffness of the columns, and thus the deformations of the slope. The instrumentation worked well, providing all the necessary observations to validate numerical models, despite some technical difficulties.

The intent of the test was not to “fail” the columns, although the possibility of this happening had been accounted for. The columns were loaded to well beyond what current design practice would recommend while observing how the columns behaved under the increased stress. In the end, the columns held up very well, showing deformations as they developed their resistance but without large plastic strains. Large scale direct shear testing of the rockfill material had produced a Coulomb-Mohr failure envelope, but laboratory procedures were not able to achieve the same densities achieved in the field when the columns were compacted with the vibrolance. A calculated final density of 22.5kN/m^3 was measured for the compacted columns compared to a density of 19.1kN/m^3 which was achieved in the laboratory for full scale direct shear testing. The higher density of the field compacted rockfill had the dual effect of increasing the peak strength, and increasing the stiffness of the rockfill material. This in turn resulted in lower displacements than were predicted by preliminary numerical modeling which used material parameters derived from the lab testing program.

There would have been some benefit in allowing the site to sit under the final load for extended monitoring of creep and consolidation effects, but seasonal, economic and personnel limitations made this impractical. With decreasing porewater pressures, the slope was returning to a more stable condition with time. The consolidation process of the loaded bank would have reflected more on the performance of the clay, rather than that of the rockfill columns.

Further analysis and numerical modeling work is ongoing. Preliminary numerical models were able to successfully predict the strain characteristics of the slope, but there is additional work to be done to more accurately calibrate the model against the observed performance. The numerical model will be used to determine a mobilized shear resistance as well as improved modulus values. It is anticipated that the current research will allow engineers to design rockfill column stabilization works with more confidence in strength and serviceability characteristics of the technology. These contributions will likely lead to significant cost savings for clients, and hopefully a more widespread acceptance of the technology.

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