HISTORY OF A LANDSLIDE IN HEAVILY OVERCONSOLIDATED CLAY OVER AN ARTESIAN AQUIFER

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ABSTRACT

A cantilever wall located along the tailrace channel of the Hodenpyl Hydroelectric Dam was undergoing significant lateral displacement. A geotechnical investigation revealed that the soils at the site consisted of high-plasticity, highly overconsolidated clay with OCR values greater than 10. The clay was underlain by an artesian aquifer, which created an upward seepage within the clay. Design of the remediation of the wall required a significant analysis effort to understand the mechanism of movement of the wall and its evolution since construction of the wall.

This paper describes the geotechnical conditions at the site. It also describes some of the stability analyses performed, which included traditional slope stability analyses as well as two- and three-dimensional finite element analyses. It also describes the selected remediation alternative, which consisted of a combination of unloading above the wall and installation of tiebacks through the existing wall. One of the most significant findings is the evolution of the stability conditions of the wall since its construction. The variation of factor of safety of the cut over time is intrinsically related to the permeability of the clay and to the magnitude of displacements along the failure surfaces.

INTRODUCTION

The Hodenpyl Hydroelectric Dam is located on the Manistee River in Michigan. The facility, constructed between 1923 and 1925, consists of an

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earthen embankment running roughly in the north-south direction. The powerhouse is located on the southern end of the embankment and discharges the water to a tailrace channel that was excavated in natural soils. Unlike most dams, the tailrace channel is nearly parallel to the core of the dam. Cantilever retaining walls of varying height were originally constructed on the east and west sides of the tailrace channel. The east wall is approximately 250 feet long and 30 to 50 feet high (see Figure 1).

FIG. 1. View of the tailrace channel and east retaining wall in 2000-2001. Note the existing tiebacks installed in 1996. The sheetpiles were also added in 1996 as a facing and do not penetrate below the footing.

Tiebacks were installed in 1996 as depicted in Figure 1. Subsequent monitoring of the wall revealed continuous lateral displacements and the need for additional stabilization measures.

Investigation of the causes of the displacement and the design of the additional stabilization was performed in stages. Initially, a geotechnical investigation was performed to assess the potential causes for movement while collection of movement data continued. Inclinometers and piezometers were installed to determine the geometry of the sliding mass, and variation of the groundwater table behind the wall. Several potential mechanisms that could cause the observed movements were identified. It was concluded that a global slide encompassing the existing wall and 1996 tiebacks was the cause for the movements.

Based on the results of these analyses, a preliminary remediation was implemented consisting of tiebacks installed through the existing wall. This preliminary remediation plan was predicated on the observation approach, and included additional monitoring measures to enhance the existing monitoring program, to verify the preliminary design assumptions, and to gain additional information for development of a final remediation scheme if needed.

The preliminary stabilization significantly reduced the rate of movement of the wall. However, subsequent monitoring of the wall movement and tieback load revealed that additional remediation was required to prevent eventual failure of the tiebacks under continued wall movement.

Additional geotechnical investigation, laboratory testing, and instrumentation data revealed important aspects of the mechanics and evolution of the instability of the slope. In addition, consolidation tests of undisturbed clay specimens showed that the preconsolidation pressure of the clay was larger than previously thought.

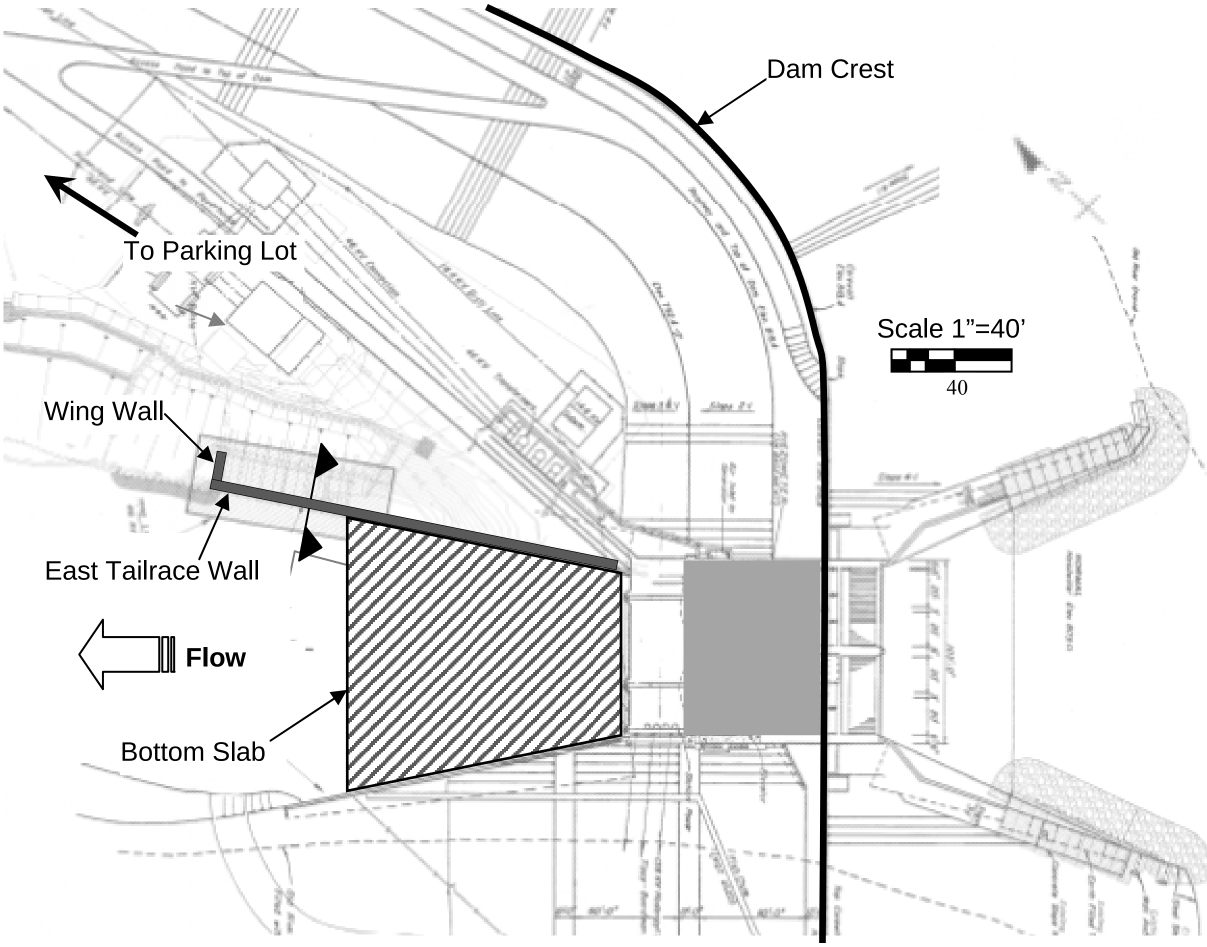
CHARACTERISTICS OF THE SITE

Figure 2 shows a general plan view of the site. The tailrace channel shown in Figures 1 and 2 was carved into natural soils during construction of the dam. The retaining walls on both sides of the channel were originally constructed as reinforced concrete cantilever walls. A bottom slab extends from the powerhouse to approximately the midpoint of the east wall. Natural clays are exposed in the bottom of the channel downstream of this slab.

During the 1996 remediation attempt, tiebacks were installed throughout the length of both walls as seen in Figure 1. The face of the walls were protected with sheetpiles bearing directly on the concrete footing of the wall and supported laterally by the tiebacks.

Figure 3 is a cross section of the east tailrace wall, which depicts the soil strata identified from the site explorations. The soils at the site consist of a shallow layer of sand extending to a depth of 5 ft. The sand is underlain by an interval of overconsolidated clay. The clay typically contained a fraction of fine to coarse sand and traces of gravel-size particles. Sporadic lenses and/or seams of sand were also detected at some locations within the clay mass. The results of the laboratory testing revealed that the plasticity of the clay materials increased with depth. An approximate and somewhat idealized boundary between low and high plasticity clay is represented in Figure 3.

The clay stratum is underlain by an artesian aquifer consisting of fine to coarse sands. The hydrostatic head of the aquifer was not known accurately. However, based on readings from nearby piezometers, an elevation of 790 ft (237 m) was conservatively estimated for the piezometric surface.



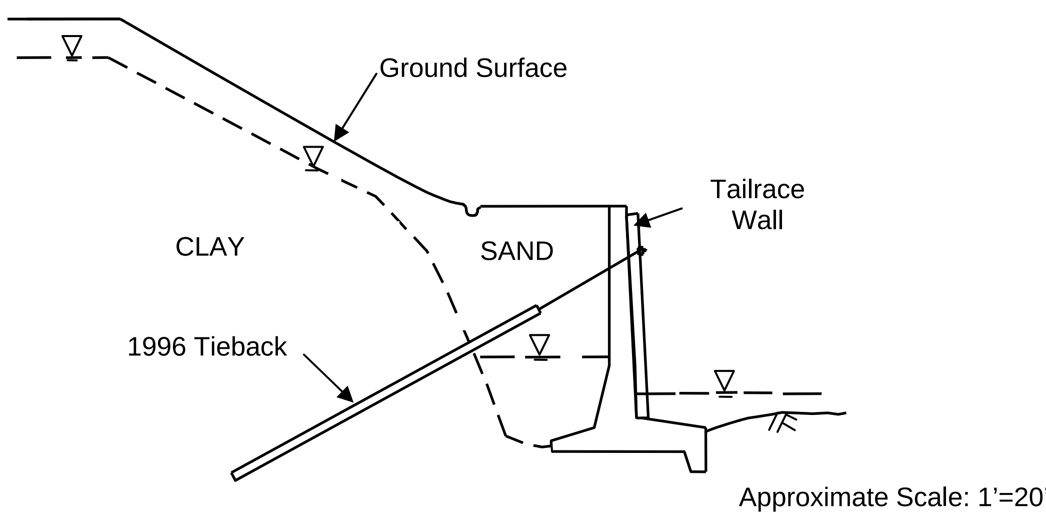


FIG. 2. General Site Plan and Section after 1996 remediation looking upstream.

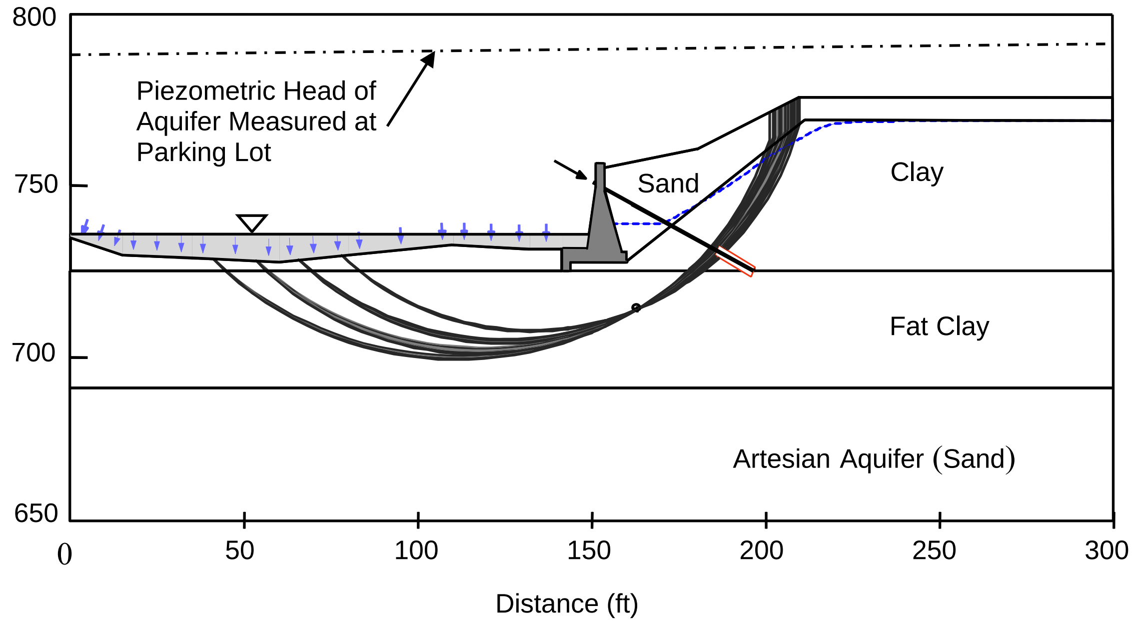


FIG. 3. Typical Section through East Tailrace Wall looking Downstream.

MASS MOVEMENT

Figure 4 is a plan view showing the measured movement of the East Tailrace Wall. The magnitude of movement increases almost linearly in the downstream direction of the wall. Thus, the downstream end of the east wall rotates around a vertical axis located approximately at the termination of the bottom tailrace channel slab. It appears that the slab restrained lateral movement of the wall. In addition, it was found that the wall did not move downstream as observed in inclinometers.

Figure 5 shows the measurements from various inclinometers installed behind the wall. The inclinometer data revealed the existence of a predominant slip surface located from EL 713 to 717. The inclinometer data also showed a significant downstream component of the movement.

Visual observation of the slope above the retaining wall and data from monitoring of surface survey points on the slope revealed the existence of a semi-circular failure scarp that extended to the crest of the slope, next to the electrical substation above. There was no discernible movement throughout the substation or further away from the slope.

These data suggested the existence of a spoon-shaped unstable mass of soil oriented at an angle with the wall. The instability did not affect the toe of the dam itself, which was a significant distance behind the wall.

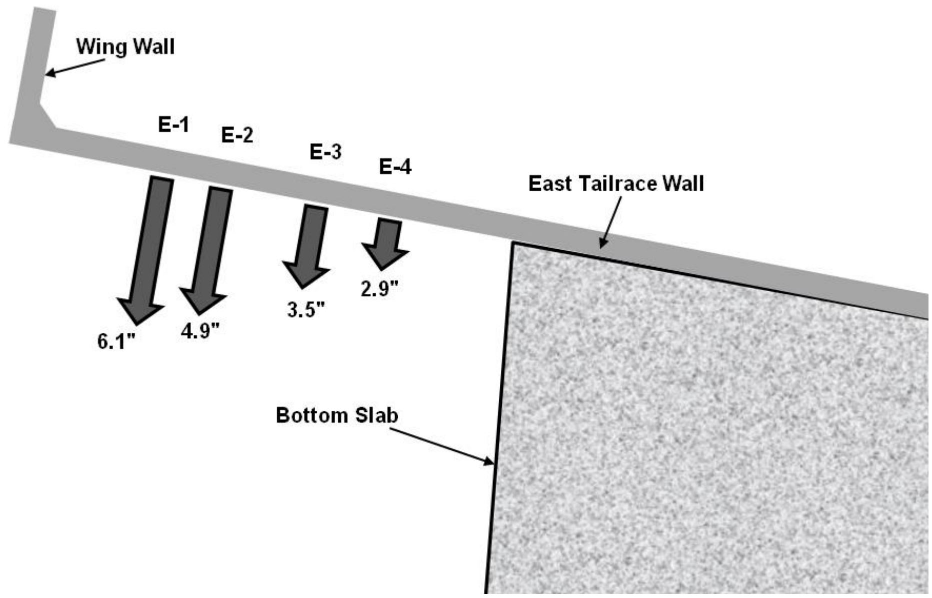


FIG. 4. Magnitude of movement of the downstream end of the East Tailrace Wall.

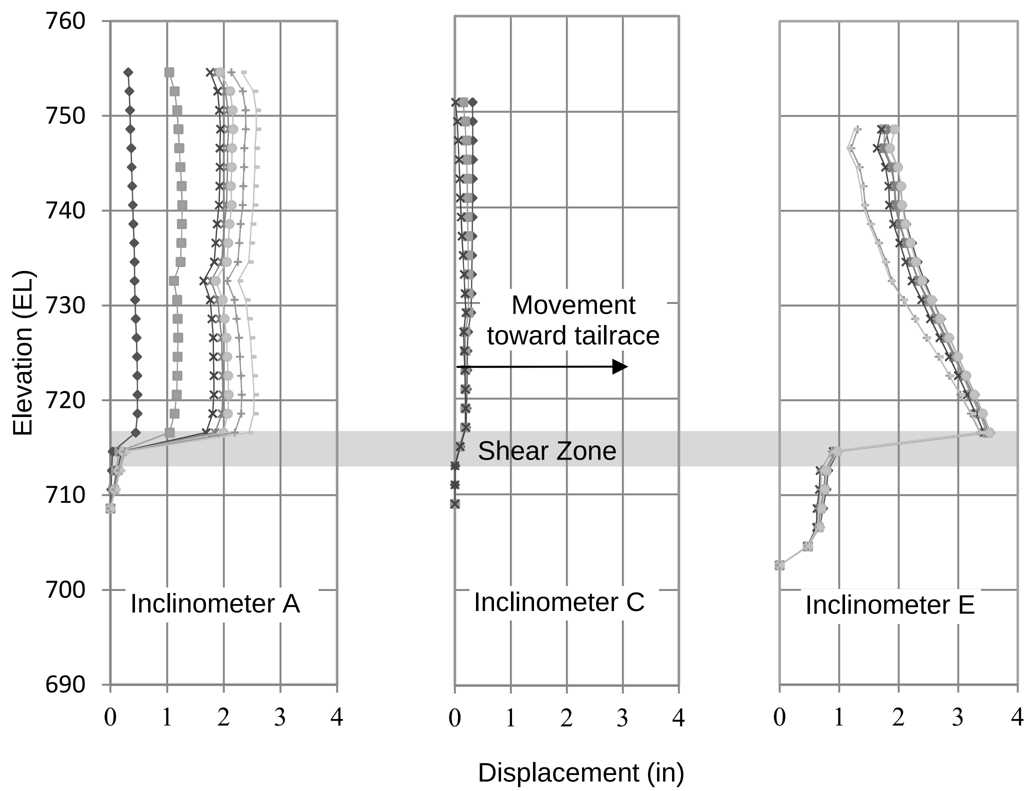


FIG. 5. Selected Inclinometers through the slip surface. Baseline data and last data reading for Inclinometer A (4/2001-1/2009), Inclinometer C (8/2009-7/2010), and Inclinometer E (6/2005-11/2010).

ARTESIAN AQUIFER AND ITS EFFECT

Figure 6 shows the piezometric head data inferred from pore pressure measurements from vibrating wire piezometers embedded at various elevations within the inclinometer grout. This data shows that there is a significant gradient from the artesian aquifer into the clay. It is possible that there are narrow seams of relatively permeable material within the clay that are connected to the artesian aquifer and where the piezometric head is even larger than that shown in the figure.

Figure 7 is a seepage analysis performed using SEEP/W, assuming a steady state flow condition across the clay layer. This analysis considers that there is a horizontal gradient across the aquifer. The reason for this assumption is that the piezometric head measured within the aquifer below the parking lot area could not be sustained in the tailrace channel area because the overlying, thinner clay stratum would heave under the sub-pressure. Also, it is likely that the piezometric head within the aquifer increased during construction and filling of the dam, which would create flow within the aquifer away from the dam embankment.

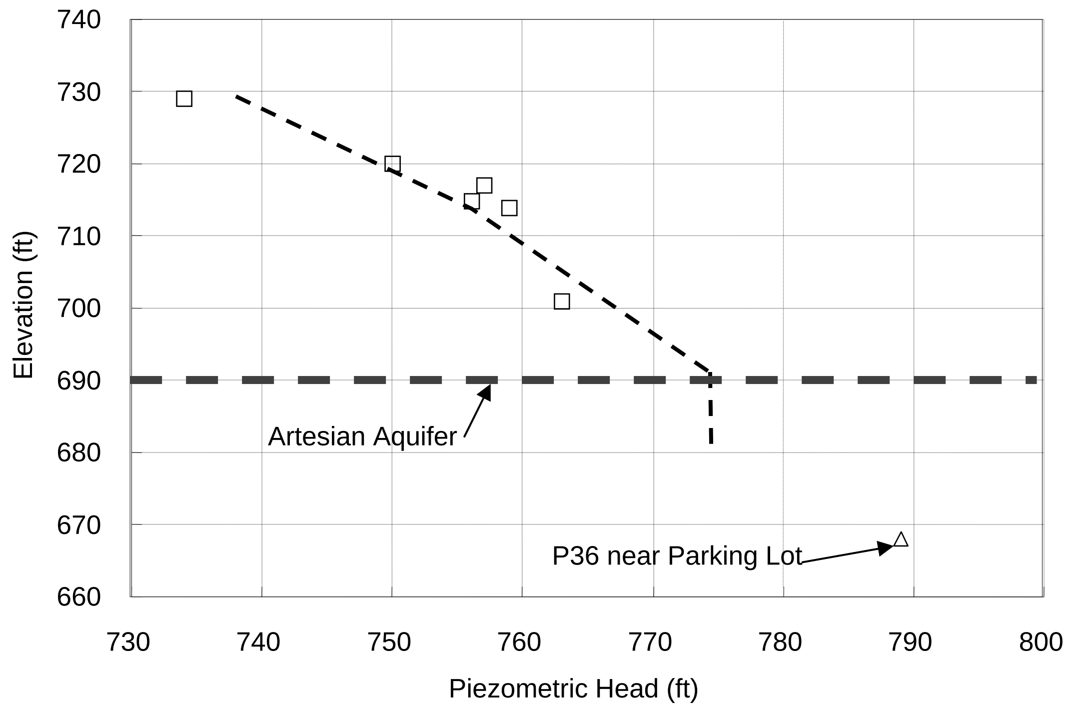


FIG. 6. Piezometric head data with depth showing effect of artesian aquifer below.

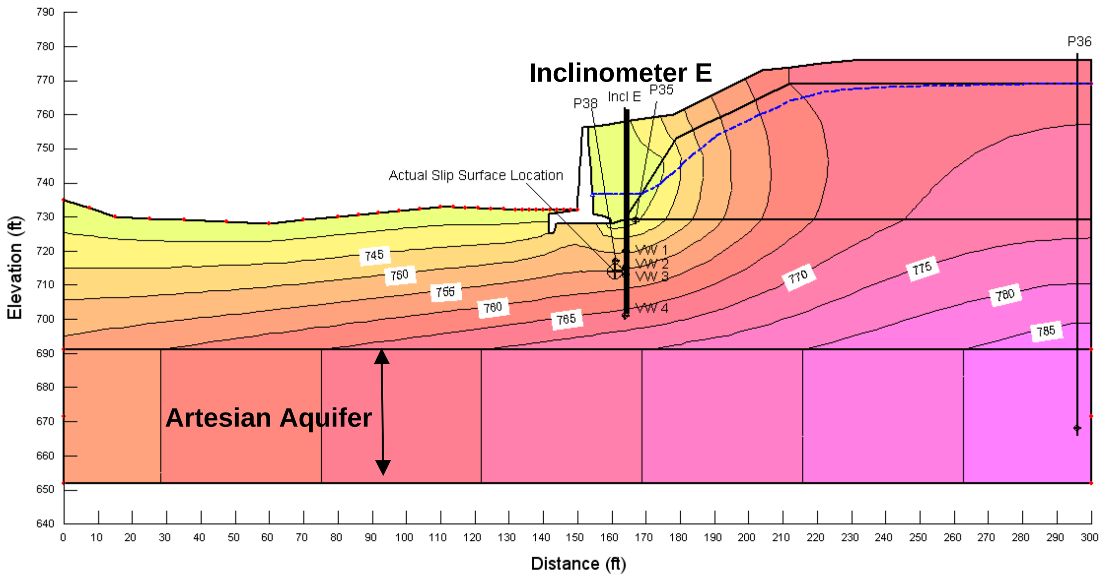


FIG. 7. Steady state seepage analysis output from SEEP/W.

PROPERTIES OF THE CLAY SOILS

Figure 8 is a summary of Atterberg Limit tests on specimens of the overconsolidated clay stratum. The values of liquid limit and plastic index measured in the Lean Clay specimens differ significantly from those of Fat Clay specimens. Similarly, the measured values of liquid limit tend to increase with depth within the Fat Clay interval. The figure shows an idealized boundary differentiating low plasticity from high plasticity clay. The figure also shows the natural water content measured in the clay specimens. The measured water contents generally range between 20 and 32 percent and tend to increase with depth. It is interesting to note that the water content shows a larger scatter near the location of the slip surface.

Grain-size analyses with hydrometer testing performed on Fat Clay specimens collected from different elevations showed that the sand and gravel content was generally less than 2 percent. The hydrometer tests showed that the clay fraction ranged from 84 to 91 percent and the remaining fraction was silt.

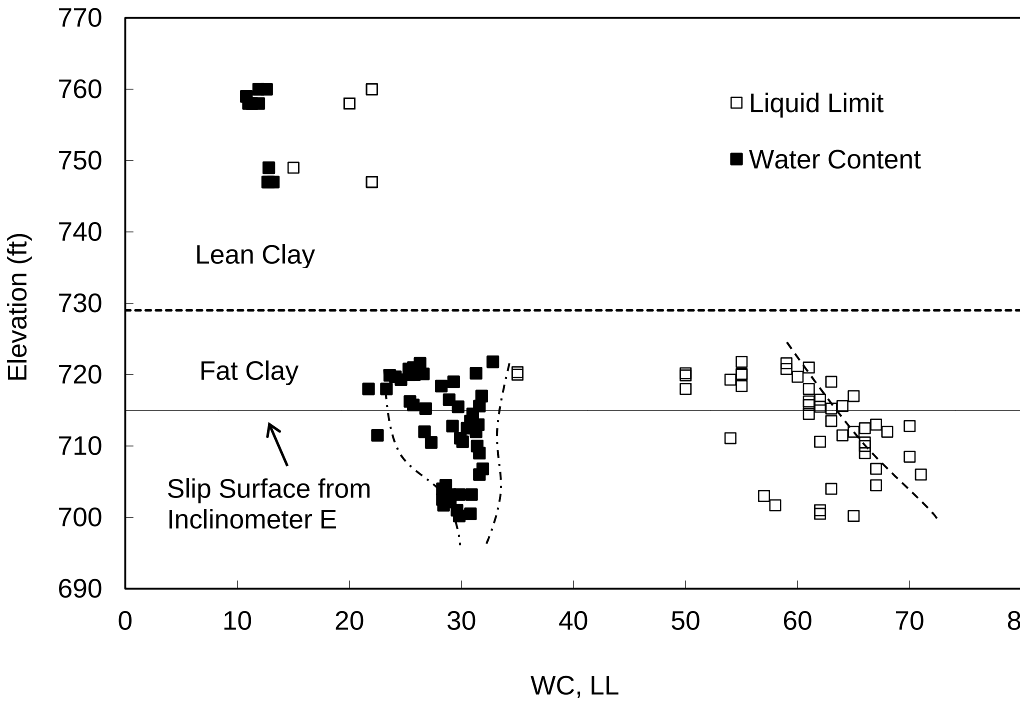


FIG. 8. Atterberg limits and natural water content across clay stratum.

A number of strength tests were performed on undisturbed specimens of the clay, including Unconfined Compression (UC) tests, Unconsolidated Undrained (UU) triaxial tests, Consolidated Undrained ($\overline{\mathrm{CU}}$) triaxial tests with pore pressure measurement, and ring shear tests on remoulded specimens. The UU triaxial tests were performed under a total confining stress similar to the estimated in situ confining stress of each specimen. Figure 9 is a summary of the undrained strength data across the clay stratum. The measured undrained shear strength values ranged between 2 to 5 ksf (143 to 240 kPa).

The results of $\overline{\mathrm{CU}}$ triaxial tests indicate peak effective stress friction angle values ranging between 26.4 and 28 degrees, with an apparent cohesion intercept ranging from zero to 0.4 ksf. Ring shear tests on remolded specimens yielded a value of residual friction angle ranging between 33 and 34 degrees. It is possible that these large friction angle values are associated to an unusually large sand fraction in the specimens tested and thus were not considered representative.

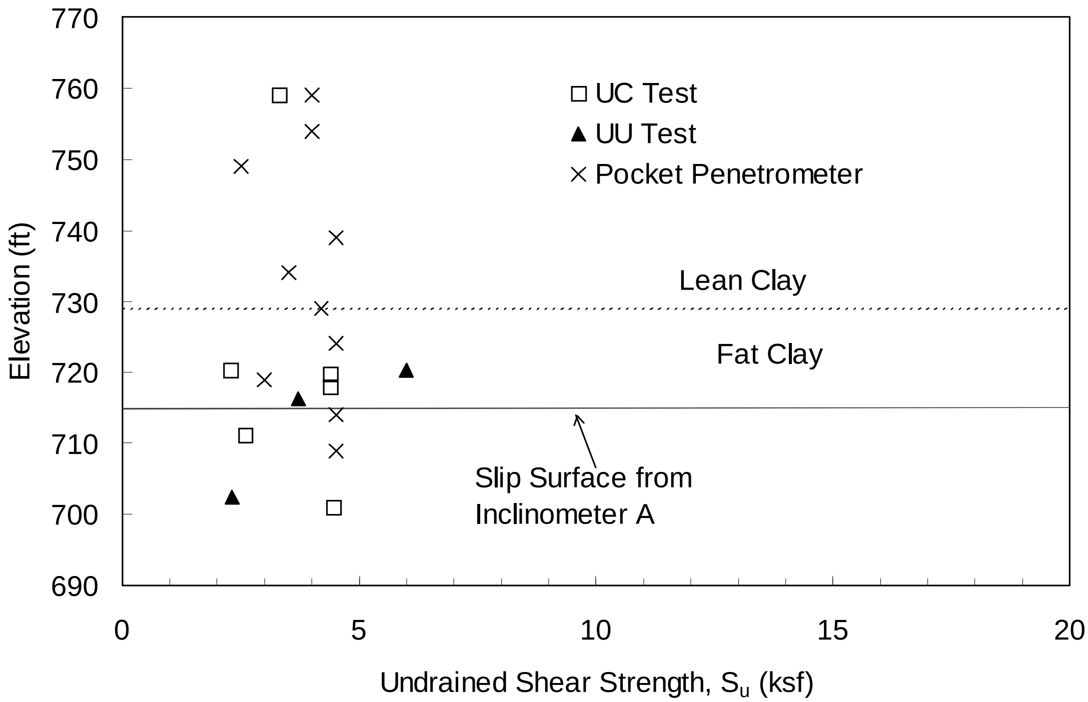


FIG. 9. Undrained strength across the clay interval interpreted from various laboratory tests.

IMPORTANT FINDINGS REGARDING STRESS HISTORY OF THE CLAY

Several consolidation tests (ASTM D2435-04) were initially conducted on intact specimens of the Fat Clay. The results of these tests suggested an Over Consolidation Ratio (OCR) ranging from 1.2 to 2.5. The range of preconsolidation pressures determined from these initial tests was 4 to 8 ksf. The tests also suggested that the Compression Index (Cc) values ranged from 0.12 to 0.24, and that the Coefficient of Consolidation ranged from 0.05 to 0.13 ft^2/day in virgin compression.

These initial consolidation test results were inconsistent with typical values encountered in fat clay materials, and with the results of the strength tests performed. In particular, the interpreted preconsolidation pressures suggested Su/p' values that would reach or exceed unity. It must be noted that the range of consolidation pressures selected for these tests by the local geotechnical consultants was based on the widespread notion that these were post-glaciation deposits.

These results prompted the designers of the remediation to request additional consolidation tests under a larger range of consolidation pressures. A typical result is presented in Figure 10. The results of these new tests showed that preconsolidation pressures exceeded 30 ksf in most specimens, which yielded OCR values greater than 10 in most cases. It is important to note that the maximum consolidation pressure was 64 ksf which was limited by the load capacity of the consolidation equipment.

This region underwent repeated transgression and regression sequences of glaciation. Glacier thickness has been estimated between one and two miles. It appears that, contrary to previous belief, the clay deposits at the site were not scoured during the most recent or potentially previous sequences of glaciation and are thus highly overconsolidated.

The large OCR values interpreted from the second set of consolidation tests are important in understanding the behavior of the clay. It would be expected that the clay stratum would be heavily fissured, and that it would have been subject to significant in situ horizontal stresses before excavation of the tailrace channel. It could be expected that stress relief within the clay resulted in expansion and in relatively significant displacement along the fissures. Some of these fissures could have become potential sliding planes where the shear strength was relatively low even before the excavation for the tailrace channel commenced.

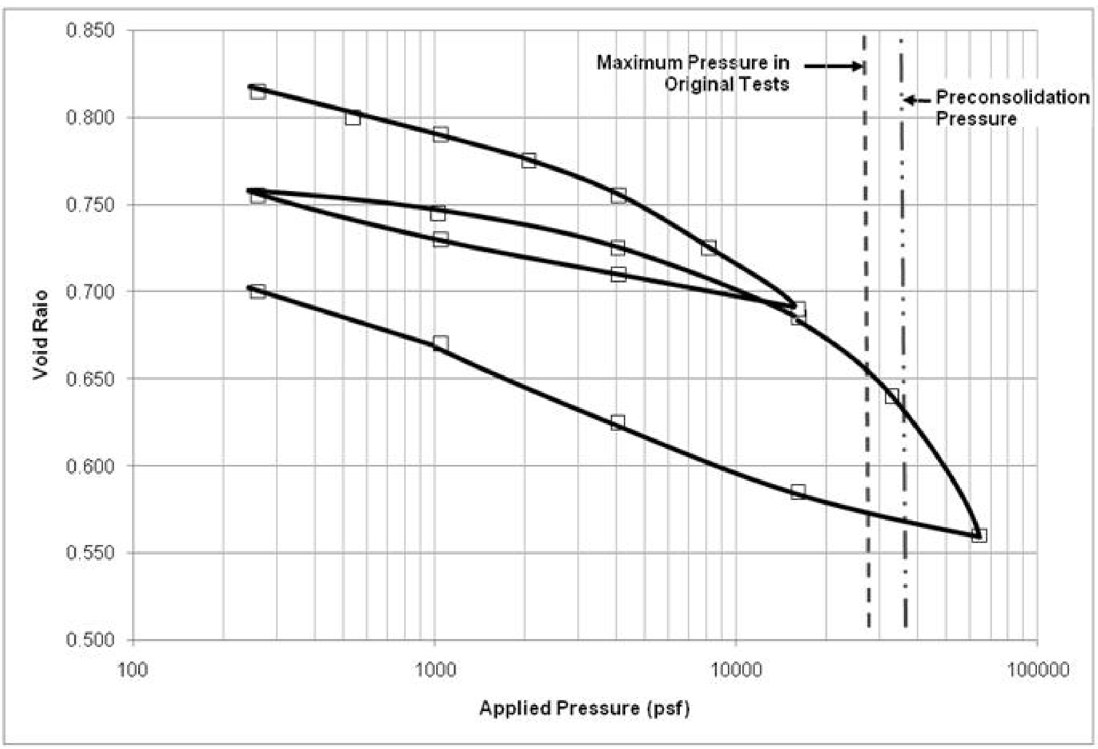


FIG. 10. Consolidation test of fat clay sample.

STRENGTH PARAMETER DETERMINATION THROUGH BACK ANALYSIS

Limit equilibrium analyses were performed using the computer program SLOPE/W to back-calculate the strength properties of the soils. The initial analyses modeled a section near the downstream end of the East Wall. The initial strength parameter values used for the upper sand interval and for the aquifer were estimated based on typical correlations with SPT blowcount and were used, unchanged, for all subsequent analyses. The strength properties of the clay interval were established based on the results of the strength tests performed and on correlations with index property values, and are shown in Table 1.

The initial slope stability analyses showed that the factor of safety against global instability obtained using the residual friction angle of the clay was low, and ranged between 0.65 and 0.8 depending on the method used to compute the factor of safety. Considering that the rate of the movement of the slope was very small, about 3/4 in/year, it was expected that the factor of safety would be closer to 1.0. It was therefore reasoned that the low values of factor of safety in the analyses could be due either to the use of strength property values that were too conservative, or to modeling errors.

Because the total cumulative movement of the wall was estimated at about 6 inches or more, it is likely that mobilization of the residual strength of the clay has occurred along the slip surface. Consequently, the authors believe that the low factor of safety obtained in the initial analyses was primarily due to the inaccuracy of the model rather than to incorrect values of shear strength parameters.

The portion of the East Tailrace Wall subject to movement is approximately 65 ft long. This yields a length-to-height ratio approximately equal to three. The two-dimensional slope stability analyses performed assume a plane strain condition, which does not likely develop in retaining walls with these proportions. Instead, it is likely that the slip surface is spoon-shaped, as suggested by the shape of the scarp observed on the slope above the wall. Based on the monitoring data collected since 1997, it appears that the upstream section of the wall and its retained soils are not moving significantly. Therefore, shear stresses at this upstream end may be significant and beneficial to the stability of the downstream portion. These end effects likely take place at the downstream end of the wall as well.

Because the two-dimensional slope stability analyses performed do not consider explicitly three-dimensional effects, end effects were included by adding an apparent cohesion intercept to the strength parameters of the clay. Therefore, the back analyses consisted of determining a value of apparent cohesion of the clay that would yield a factor of safety of 1.0 determined using the Morgenstern-Price Method, Half-sine function along a slip surface established based on the inclinometer data and field observations.

Two mechanisms of sliding were analyzed. A circular slip surface using Morgenstern-Price Method was first considered as illustrated in Figure 3. Composite or block slide mechanisms were also analyzed using the modified

Janbu's method, where the extent of the block behind the wall was limited to match field observations. The composite slide mechanism is not represented in the figure. Further details on stability analysis techniques can be found in Duncan (1996). Although the existing tiebacks were included in the analyses, their contribution to stability was negligible as the sliding mass typically encompassed them entirely.

The value of cohesion determined from these back analyses was 370 psf as shown in Table 1. This value is consistent with apparent cohesion estimates based on redistributing the estimated shear resistance at the ends of the sliding mass on the slip surface area.

TABLE 1. Soil Properties estimated from Correlations and back analysis.

Interval	γ (pcf)	Fully Softened Strength Parameters	Residual Strength Parameters
Sand	135	φ' = 30deg c' = 0	φ' = 30deg c' = 0
Lean Clay	138	φ' =32 deg c'=(370)*	φ' = 30 deg c'=(370)*
Fat Clay	126	φ' = 30 deg c'=(370)*	φ' = 15 deg c'=(370)*
Artesian Aquifer (Sand)	135	φ'=38 deg c' = 0	φ'=38 deg c' = 0

* Apparent cohesion to account for three-dimensional effects.

REMEDIATION EFFORTS

Continuous movement of the wall prompted the 2002 remediation effort, which was conceived following the observation approach. The 2002 remediation consisted of tiebacks installed through the retaining wall and bonded beyond the critical slip surface determined from the inclinometer data as shown in Figure 11. One of the difficulties encountered during this design was the limited volume of soil available for tieback placement. The existing 1996 tiebacks did not allow the installation of the new tiebacks within the upper half of the wall. The artesian aquifer limited the inclination and length of the new tiebacks, which were not to be drilled into the aquifer. The tiebacks were then installed from the water line with relatively shallow inclinations. Details of the design and installation of these tiebacks are presented Bruce et al. (2007), Bruce et al. (2004), and Gomez et al. (2004).

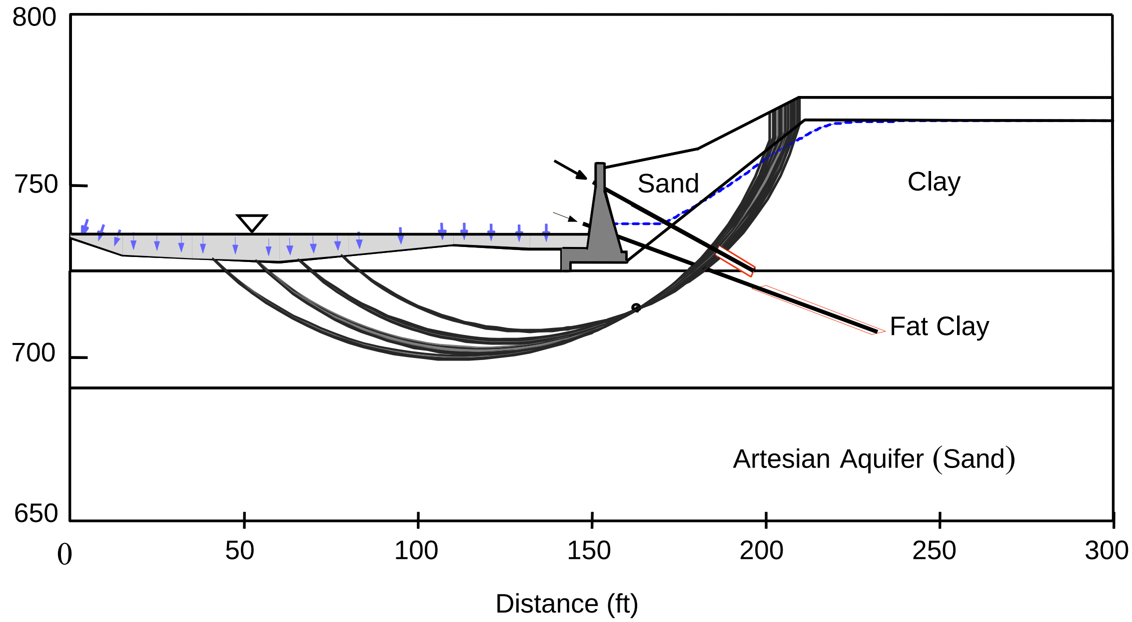


FIG. 11. Typical Section through East Tailrace Wall with installed 2002 tiebacks looking Downstream.

The remediation design included additional instrumentation and monitoring of the wall, accompanied by a complete action plan in case certain thresholds of movement were exceeded. The 2002 remediation was successful in that it reduced the rate of movement of the wall and gained additional time for a more comprehensive study of the soils and development of final remediation alternatives. The monitoring and action plan included in the remediation design was invaluable for the collection of additional data on the wall movement.

The last remediation effort was completed in two phases and is depicted in Figure 12. The first phase of the remediation was to further stabilize the known failure mass. The second phase of the stabilization was to increase the safety margin against a future deep failure mass. As part of the Phase I remediation in 2009, a significant volume of soil was excavated from above and behind the wall. This was accompanied by the construction of a soil nail wall adjacent to the existing switch yard, and the installation of 125 ft long soil nails at the base of the soil nail wall. The purpose of the soil nail wall was to provide stability to the substation and included drainage elements to reduce pore pressures in the clay behind. The length of the bottom row of soil nails was set at 125 feet to increase the factor of safety against potential deep-seated failure.

In 2010, the second phase was completed, which consisted of tiebacks installed through the existing retaining wall and through the downstream sheetpile wall to further increase the factor of safety against slope instability. The monitoring data collected shows that the movement of the wall has stopped, or reversed in some instances.

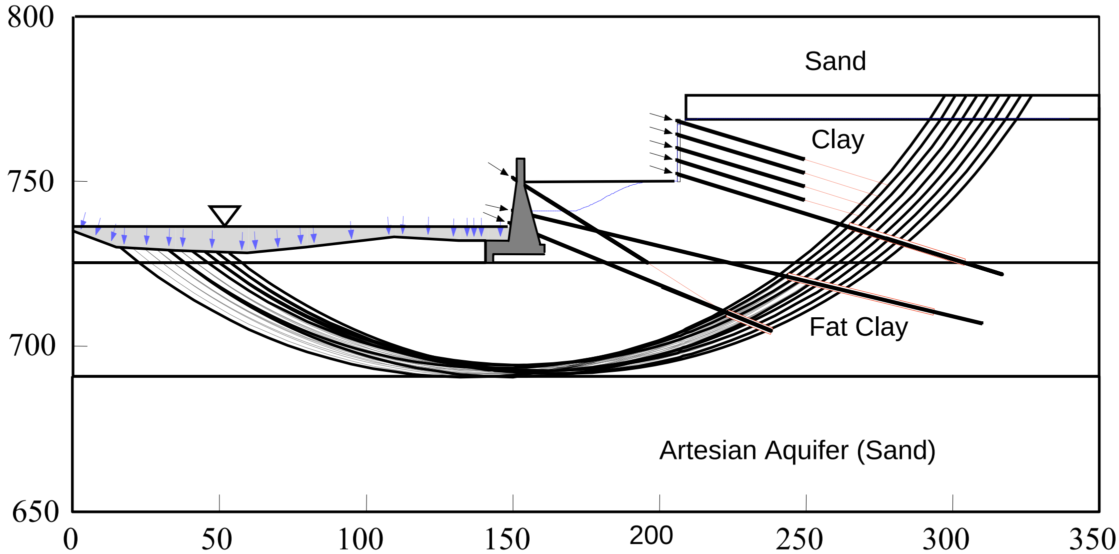


FIG. 12. Typical Section through East Tailrace Wall with 2009 Phase I and Phase II soil nails and tiebacks looking downstream.

Figure 12 also depicts potential deep slip surfaces where the factor of safety was increased by the remediation. The minimum factor of safety was 1.54, which met the minimum factor of safety of 1.5 specified by the Federal Energy Regulatory Commission (FERC). The tieback lengths were primarily driven by the effect of the artesian aquifer-induced pore pressures within the clay and the need to extend the bond zone beyond potential deep seated slip planes. Because the ratio between effective stress and depth is much lower than in a hydrostatic environment, there is a relatively small increase in drained shear strength with depth.

EVOLUTION OF FACTOR OF SAFETY

Figure 13 depicts the variation of the factor of safety over time since construction of the East Tailrace Wall. All the values in the plot are based on finite element analyses except the initial factor of safety from 1924. The initial factor of safety was estimated based on the undrained shear strength of the clay determined from the test results shown in Figure 9. This estimated factor of safety is not likely accurate because it is based on recent shear strength tests, and because it does not consider the presence of fissures in the clay, among other factors. However, it does serve to illustrate that the wall was stable immediately after construction.

The factor of safety decreases continuously over time until 2002, when the second remediation effort was carried out. The introduction of tiebacks through the wall increased the factor of safety to a value of 1.13. This factor of safety was calculated using residual strength parameters for the clay and introducing the apparent cohesion to account for three dimensional effects as described previously. The factor of safety value is low compared to the factor of safety of 1.34 considered during the design of the 2002 tiebacks. It is noted that the line depicting the decrease of factor of safety after construction of the wall is only indicative of the average rate of decrease of the factor of safety.

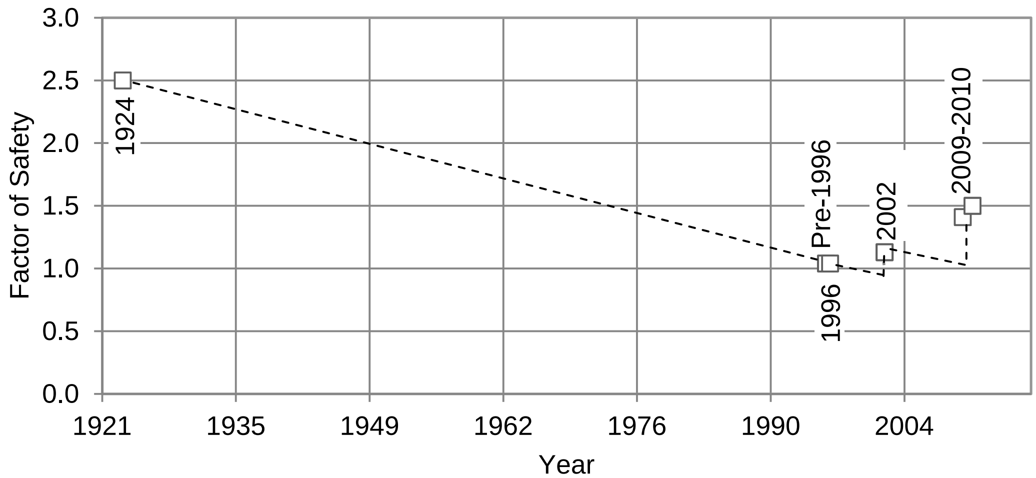


FIG. 13. Variation of factor of safety with time

Figure 14 shows the wall displacement over time. The following events are depicted in the figure:

(1) Tie-back construction period and dewatering was from about 7/18/02 to 9/9/02
(2) Start of Phase I of the 2009-2010 remediation
(3) Start of Phase II remediation
(4) Completion of Phase II remediation

The data in the figure shows that movement of the wall stopped for approximately six months to one year after installation of the 2002 tiebacks. The movement then resumed although at a significantly slower rate. The 2002 remediation monitoring/action plan required, among other things, periodic monitoring of the tension load of the 2002 tiebacks. The tieback load was measured through lift-off testing as described by Bruce et al. (2007), Bruce et al. (2004), and Gomez et al. (2004).

Figure 15 shows the results of the lift-off test measurements. It is seen that the loads of the 2002 tiebacks increased progressively since their installation as the wall moved outward. Careful analysis of these data showed that the tiebacks were bonded outside the sliding mass, and that they were not yielding geotechnically.

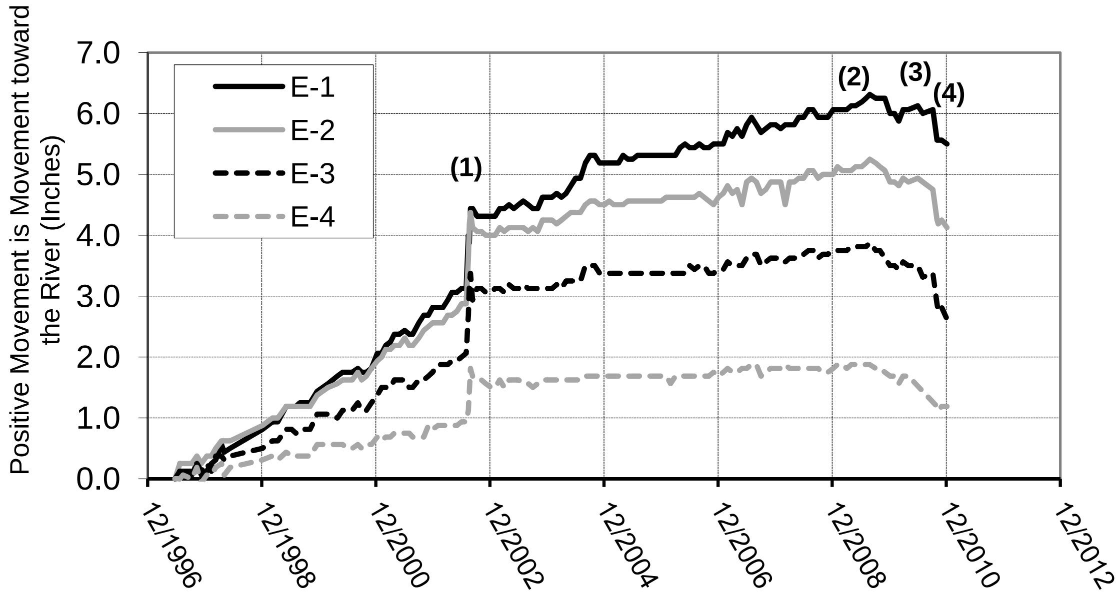


FIG. 14. Movement of the east downstream retaining wall cap, since 6/23/97. (See Figure 4 for location of monitoring points).

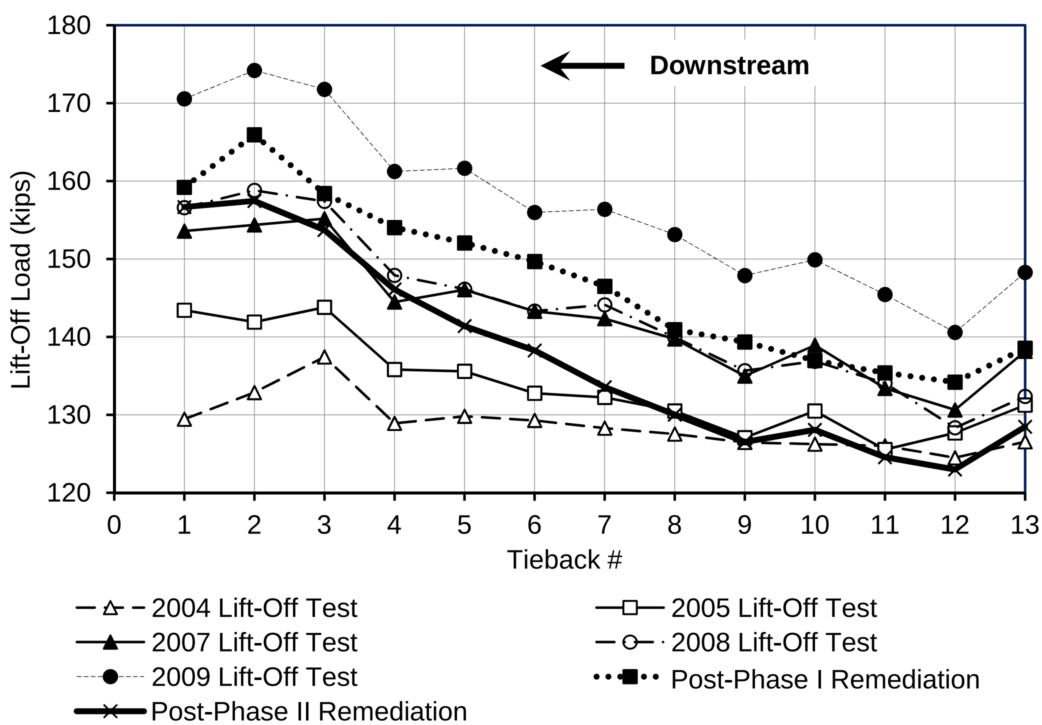


FIG. 15. Results of Lift-off Testing of SBMA Anchors

The wall movement continued at a steady rate after the installation of the 2002 tiebacks, but it is not entirely clear by what mechanism. The fact that the wall movement stopped temporarily after installation of the 2002 tiebacks, and that it then resumed and continued at a steady rate under increasing tieback loads suggests that the forces on the wall were steadily increasing. The authors theorize that this increase in force may have been due to a combination of progressive reduction in the shear strength of the clay and to its continued expansion or rebound.

It is possible that progressive failure of the clay was partially responsible for the increasing forces acting on the wall and that portions of the slip surface already at residual strength may have experienced a further reduction in shear strength caused by an increase in water content. It is noted that development of steady-state seepage through the low permeability clay after excavation of the tailrace channel and filling of the reservoir would take significant time. A finite element analysis was performed that showed that a steady-state flow through the clay would develop only after 70 years or more following a change in the artesian aquifer piezometric head. It would not be surprising that steady-state seepage has not been attained to date.

On the other hand, the large OCR values of the clay suggest that large horizontal stresses existed before excavation of the tailrace channel, and that the clay mass is likely heavily fissured. The subsequent lateral and vertical expansion of the clay after excavation would likely be a slow process due to its low permeability. Restraining such movement would require the application of significant lateral pressure on the clay mass, which would be much larger than the pressure generated by the 2002 tiebacks. It is thus possible that this expansion process continued after 2002.

Consequently, the wall movements after installation of the 2002 tiebacks may be due to a combination of the remaining portion of the slip surface progressing to residual strength as well as rebound of the clay during relief of large locked-in lateral stresses. It should be noted that a potential deeper slip plane was observed in Inclinometer A some time after the 2002 tiebacks were installed. This further supports both mechanisms for increased load on the 2002 tiebacks.

Figure 13 shows the increase in factor of safety during the 2009-2010 remediation. The largest increase in factor of safety was due to removal of soils during Phase I. The significant increase in the estimated factor of safety during the 2009-2010 remediation event is supported by the data presented in Figures 14 and 15. The wall experienced reversal of the direction of movement, and the load of the existing 2002 tiebacks decreased significantly. This reversal of movement and decrease in tieback loads is still ongoing.

CONSTRUCTION ASPECTS OF THE 2009/2010 REMEDIATION

The Phase I excavation extended five feet below the top of the East Wall and about 48 feet behind the East Wall. A soil nail wall was installed to retain the face of the excavation next to the existing substation. The second phase of the slope stabilization included installation of shallow tiebacks through the retaining wall and anchored behind a potential deep seated failure plane. A photograph of the final condition is shown in Figure 16.

FIG. 16. View of the tailrace channel and east retaining wall. Note the tiebacks from top to bottom, installed in 1996, 2010, and 2002.

The soil nail wall facing varies in height up to 26 ft. The upper rows of the soil nails ranged in length from 20 to 50 ft and were double corrosion protected. The bottom row of soil nails were 125 ft long to provide additional resistance to deep seated failures. The design considered a factor of safety of 2.0 for bond with confirmation through verification and proof load testing. The soil nail wall was designed assuming that full water pressure develops behind the wall, in addition to potential ice lensing in the lean clay adjacent to the soil nail wall face. Drainage panels were placed behind the shotcrete, and perforated PVC pipe drains were installed in predrilled holes through the excavation face. Figure 17 shows a photograph of the soil nail wall during construction.

Phase II included additional tiebacks through the existing East Wall. The tiebacks had a design load of 200 kips and were installed with a six foot lateral spacing. They extended 165 ft behind the wall. The tiebacks are composed of six 7-wire strand Class 1 corrosion protected Single Bore Multiple Anchors (SBMAs). A photograph of the performance and proof load test setup is shown in Figure 18.

Coring through the 5 to 7 ft thick concrete wall was performed prior to duplex drilling. Rotary drilling of the concrete wall was not performed due to limitations on the hole trajectory tolerances. The core hole provided a guide for installation of the long tiebacks. A photograph of the coring and tieback installation within the cofferdam is shown in Figure 19.

FIG. 17. Soil nail wall during construction with temporary shotcrete face applied.

FIG. 18. Photograph of the performance and proof load test setup for the SBMA anchors.

FIG. 19. Photograph of the coring and tieback installation through the tailrace wall.

CONCLUSIONS

One important finding during this project was that the clay deposits in the area are heavily overconsolidated with OCR values in excess of 10. For future projects in that involve excavations, it will be important to consider the effect of the locked-in lateral stresses and the presence of clay fissures. Residual strength values should be considered for design of such excavations.

The presence of the artesian aquifer, especially in the proximity of the dam, created significant complications. The increase with depth of the effective stress, and thus of the drained strength is less than that typically found in soils of this nature. Therefore, a significant increase of factor of safety against global instability requires the stabilization of large volumes of soil through the installation of very long tiebacks.

The authors theorize that steady-state flow across the clay had not been attained by 2002, after over 75 years from construction of the wall. This was one of the causes for continued decrease in the shear strength of the clay. Expansion or rebound of the clay was also taking place which resulted in further reduction of the shear strength.

Excavation of the soils behind and above the wall had a greater beneficial effect on stability than installation of additional tiebacks through the wall. It also caused heave of the soils above the wall and a clear reversal in the direction of wall movement.

The 2009-2010 remediation appears to have been successful. Data from subsequent monitoring of the wall and the instrumentation will be fundamental to confirm the continued stability of the wall over time.

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REFERENCES

Barley, A.D. (2000). “*Trial soil nails for tunnel face support in London Clay and the detected influence of tendon stiffness and bond length on load transfer*.” Proceedings of the Third International Geosystems, London, UK, June.

Barley, A.D., Windsor, C.R. (2000). “*Recent advances in ground anchor and ground reinforcement technology with reference to the development of the art*,” GeoEng 2000, International Conference on Geotechnical and Geological Engineering, Melbourne, November 12-19, pp, 1084-1094.

Bruce, M.E., Gómez, J.E., and Traylor, R., (2007). "*Repeated Lift-off Testing of Single Bore Multiple Anchors for Dam Retaining Wall over a 5-Year Period.*" International Conference on Ground Anchorages and Anchored Structures in Service 2007, London, England, November 2007.

Bruce, M.E., Traylor, R.P., Barley, A.D., Bruce, D.A., Gómez, J.E. (2004). "*Use of post grouted Single Bore Multiple Anchors at Hodenpyl Dam, Michigan*," Geosupport 2004, Geotechnical Special Publication 124, Orlando, FL, January 2004.

Duncan, J.M. (1996). "*Soil slope stability analysis*," Transportation Research Board, Special Report 247, National Academy Press, Washington, D.C.

Gómez, J.E., Bruce, M.E., Bruce, D.A., Basinger, D., and Cadden, A.W. (2004). "*Stabilization of Retaining Wall along Tailrace Channel of Hodenpyl Dam*," Geosupport 2004, Geotechnical Special Publication 124, Orlando, FL, January 2004.

Littlejohn, G.S. and Bruce, D.A. (1977). "*Rock Anchors - State of the Art.*" Foundation Publications, Essex, England, 50 pp. (Previously published in Ground Engineering in 5 parts, 1975-1976.)

Mc Gregor, J.A., and Duncan, J.M. (1998). "*Performance and use of the Standard Penetration Test in Geotechnical Engineering practice*," Center for Geotechnical Practice and Research, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg.

Post-Tensioning Institute (2004). "*Recommendations for prestressed rock and soil anchors*," Fourth Edition, June, 2004.