Remedial Design of An Earth Dam – 20 Years later

Michael J. Mann, P.E., MASCE[1], Robert E. Snow, P.E., MASCE[2] and Andrew J. Klettke, EIT, AMASCE[3]

1. Principal, McMahon & Mann Consulting Engineers, P.C., 2495 Main St., Suite 432, Buffalo, NY 14214; mmann@mmce.net.
2. Principal, D'Appolonia, 275 Center Road, Monroeville, PA 15146, resnow@dappolonia.com.
3. Project Engineer, McMahon & Mann Consulting Engineers, P.C., 2495 Main St., Suite 432, Buffalo, NY 14214; aklettke@mmce.net

ABSTRACT: This paper is a sequel to an earlier paper presented by the authors at the 1992 "Stability and Performance of Slopes and Embankments-II" Conference. The original paper described the failure of the upstream slope of a 13.7 m (45 ft) high earth dam in eastern Ohio. The dam was constructed in 1983 and 1984 and the upstream slope failed upon completion of construction, prior to first filling. Subsurface explorations and monitoring identified the failure surface and revealed that the upstream slope continued to creep. A remedial design was developed that included an earthen berm to buttress the upstream slope. The analyses for the design of the stabilizing berm included laboratory creep tests to establish the appropriate level of stress to include for the portion of the failure surface that remained in place. A stress level was selected with the goal of little or no creep movement during operation.

The earthen berm was constructed in 1988 and the reservoir was filled. Performance monitoring includes inclinometer data that show that creep movements continued to occur at a decreasing rate over a 20-year period. This paper describes the post construction creep movements and relates the embankment performance to the testing and analyses completed for the remedial design. A finite element model of the post construction creep movement is developed, using the Plaxis 2D program, to relate the performance data to the design.

INTRODUCTION

The Scout Reservation Dam is located in Carroll County Ohio. It is approximately 13.7 m (45 ft) high and 244 m (800 ft) long, creating a 121,000 sq. meter (30-acre) recreational lake for the Buckeye Council of the Boy Scouts of America. The original dam design was for a homogeneous earth fill with a 3 horizontal to 1 vertical (3H:1V) downstream slope and a 2H:1V upstream slope. The dam was constructed of residual silty clay soil beginning in 1983 and was completed in the fall of 1984. The principal spillway consisted of a concrete intake tower and discharge pipe located near the west abutment. An emergency open channel overflow spillway was located on the east abutment.

A crack developed along the length of the crest immediately after construction and prior to first filling. The upstream slope moved downward and away from the rest of the dam resulting in a scarp that was 0.3 to 0.6 m (1 to 2 ft) high in January 1985, 0.9 to 1.2 m (3 to 4 ft) high in July 1985 and 1.5 to 1.8 m (5 to 6 ft) high by the summer of 1986. Shortly after the slope movements were noted, a soil and rock fill berm was placed upstream of the primary spillway tower and the crest was lowered in this area to limit damage to the structure. Figure 1 shows the configuration of the dam and instrumentation that was installed to study the slope failure.

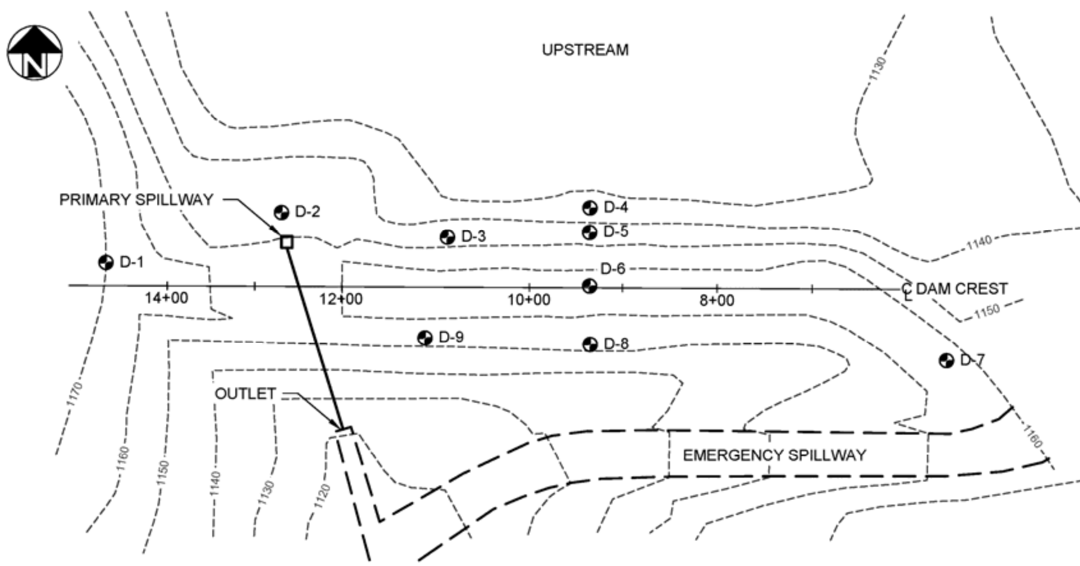
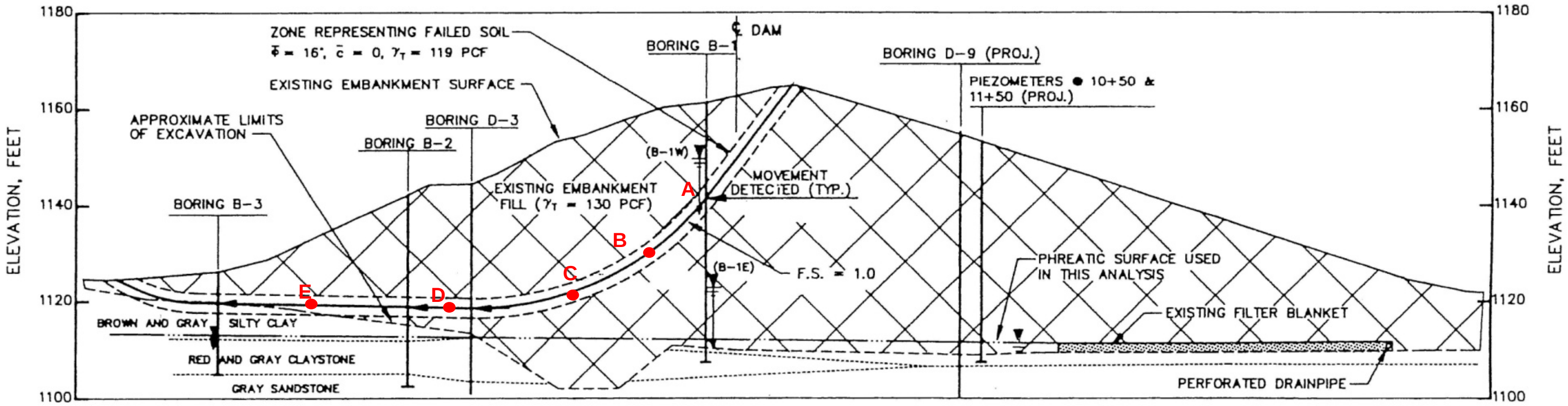


Figure 1. Site plan prior to remediation

REMEDIAL DESIGN

The following are some of the highlights of the remedial design development. The previous paper, Mann and Snow (1992), provides additional details regarding the subsurface explorations, laboratory testing and the basis for of the remedial design.

A subsurface exploration program was completed in 1987 and 1988 to study the failure mechanism and develop a remedial plan for the dam. The subsurface explorations included soil sampling, rock coring and inclinometer and piezometer installation. Figure 2 is a cross section from the original paper, showing the instrument locations and typical subsurface conditions.



From Mann and Snow (1992)

Figure 2. Cross section prior to remediation

The dam was constructed with low to medium plasticity silty clay with typically 70 to 90 percent by weight finer than the 200 sieve. The soil's liquid limit ranges from 30 to 42, the plasticity index varies from 8 to 21 and water contents vary from 16 to 20 percent.

As shown on Figure 2, inclinometer measurements indicated that movement was continuing to occur along a surface that extended from near the base of the dam up through the embankment to the crest. The lower, nearly level portion of the failure surface, (approx. El 1120) corresponded approximately to the elevation of fill placement during the shutdown for the 1983-1984 winter.

Slope stability analyses were completed for the existing conditions using the computer program STABL2. The results indicated that a factor of safety of unity corresponds to a friction angle of 16 degrees along the failure surface. This friction angle is less than the residual friction angles measured in direct shear tests on soil samples from in or near the slide zone (17 to 30 degrees). It was hypothesized that the soils along the failure surface were deforming at stress levels below the peak levels measured in the direct shear tests. The concept of deformation occurring at stress levels below the peak values measured in short duration tests has been discussed by many researchers including Mitchell (1976) and Tavenas and Leroueil (1981).

Constant stress direct shear tests were completed to characterize the creep behavior of the soils along the failure surface. The purpose of the tests was to explain the ongoing creep movements and develop an appropriate friction angle for the portion of the failure surface that would remain in place in the remedial design. The test procedures and the results are described in the previous paper. The results for all of the samples showed a significant increase in deformation rate when the shear stress exceeded a friction angle of about 16 degrees. Figure 3 is a typical plot of the results.

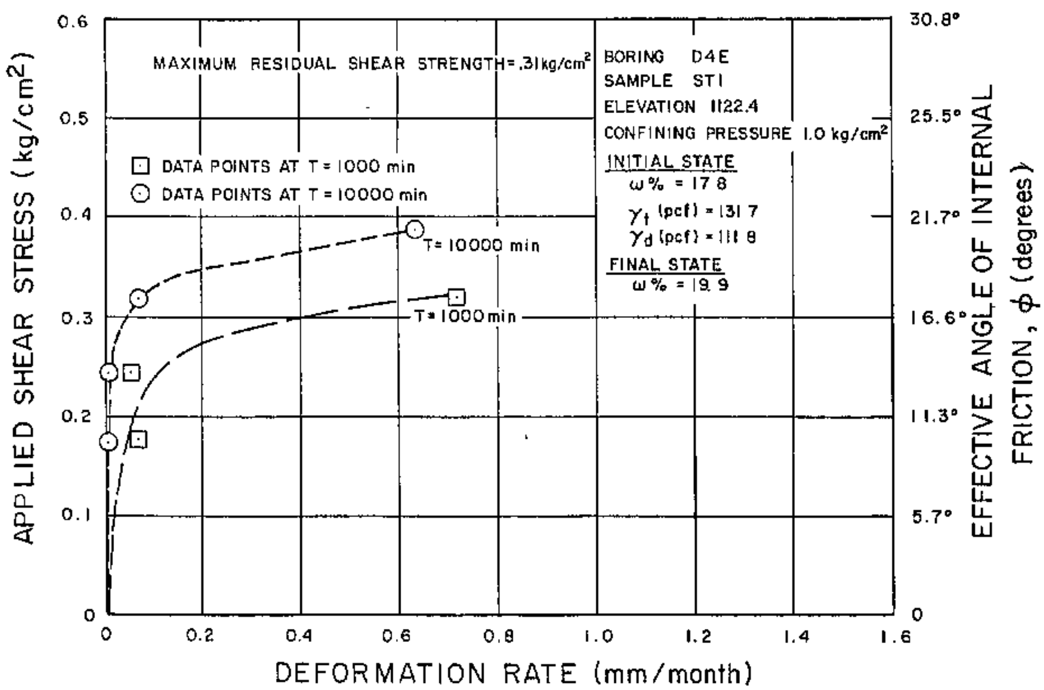


Figure 3. Typical plot of stress controlled test results

The close agreement between the back-calculated friction angle and the stress level at which creep movement initiated in the constant stress tests, supported the hypothesis that the soils along the failure surface were deforming at a stress level below the residual shear strength.

The remedial design included construction of a buttress against the upstream slope to stabilize the embankment. Other features of the remedial design included; removal and replacement of the upper 3 to 4.5 m (10 to 15 ft) of the embankment crest, removal of the fill around the spillway tower and repair of the tower and lake drain pipe and additional seepage control measures. Figure 4 is a plan of the remedial design.

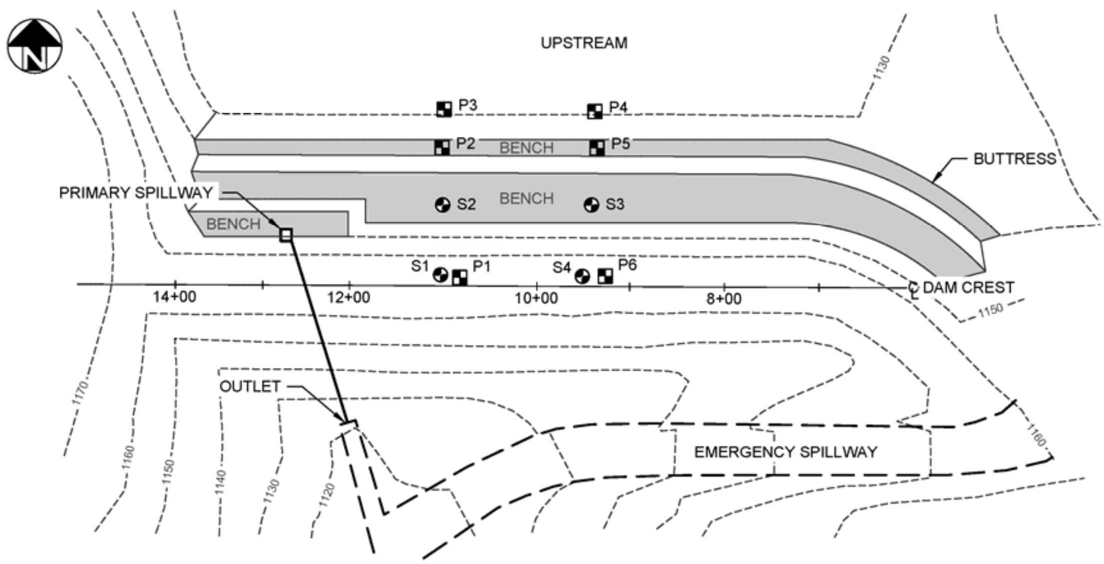


Figure 4. Site plan after remediation

The buttress configuration was based on slope stability analyses. Figure 5 is a cross section near the middle of the dam showing the buttress configuration. In order to restrict creep movement after remedial construction, the design analyses used a friction angle of 14 degrees to represent the portion of the failure surface that was left in place. As shown on Figure 3, little or no creep movement was observed in the constant stress tests at this level of shear stress. The previous paper describes the other material properties used in the analyses.

The analyses included a factor of safety of at least 1.5, therefore it was expected that the shear stress on the failure surface would be less than that represented by a friction angle of 14 degrees. At this low stress level, no significant additional movement was expected after the remedial design was constructed.

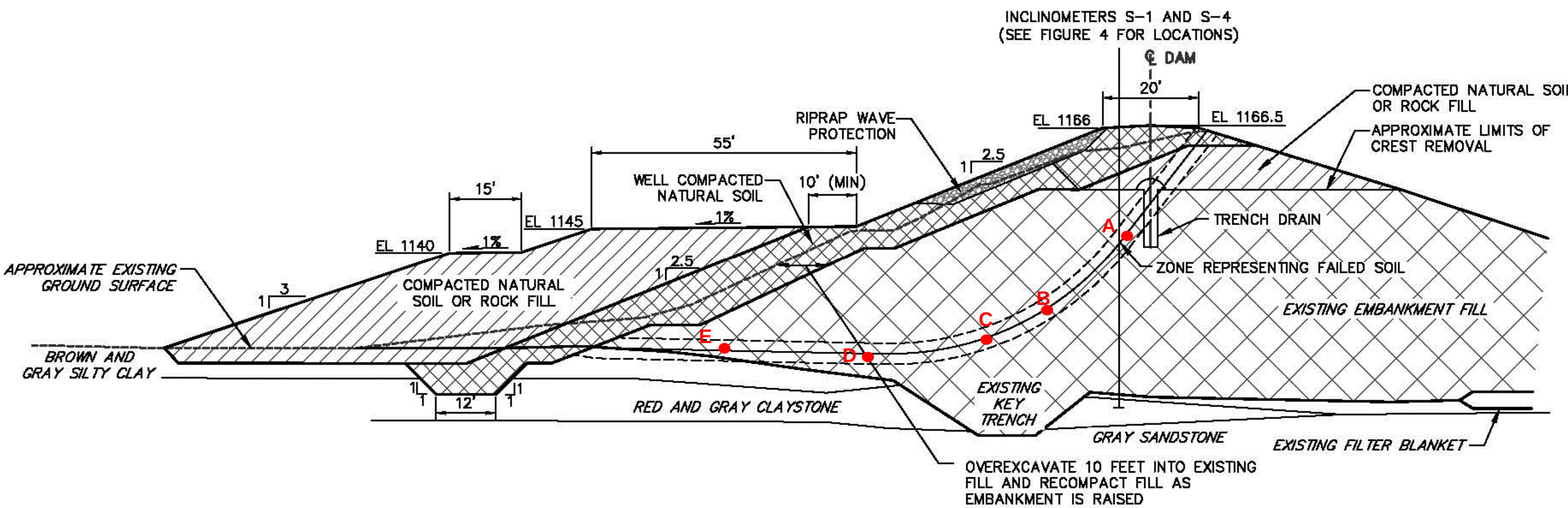


Figure 5. Cross section of remedial design

The buttress was constructed in the summer of 1988. Figure 6 is a picture of the upstream slope shortly after remedial construction was complete.

Figure 6. Upstream slope after completion of remedial construction

PERFORMANCE MONITORING

The upstream slope of the Boy Scout Dam has been monitored since 1986 after the failure was observed, through remedial design and construction and for the 20 plus years since remedial construction. Figure 7 is a plot of the inclinometer data through remedial design and construction, reflecting movement at a depth corresponding to the maximum deformation and at the approximate shear plane. The locations of the inclinometers that were monitored during the remedial design phase and for the initial portion of construction are shown on Figure 1. Inclinometers D-4, D-5 and D-6 are located near the center of the dam and show that during the remedial design the upstream slope was creeping at a rate of about 2.5 mm (0.1 in) horizontally per month. It was also observed that minor regrading of the slope and other factors such as heavy rainfall increased the rate of movement.

During remedial construction, the upper 3 to 4.5 m (10 to 15 ft) of the crest was removed, the buttress was constructed and then the crest was replaced. Monitoring of the inclinometers was continued through removal of the crest until reconstruction work required their removal. As shown on Figure 7, the creep movements stopped after the crest was removed.

Slope stability analyses completed for the case with the crest removed, but without the new buttress in place, showed that the limit equilibrium stress level on the failure surface was less than the value that caused creep movements in the laboratory tests but was slightly more than with the crest replaced and the new buttress in place.

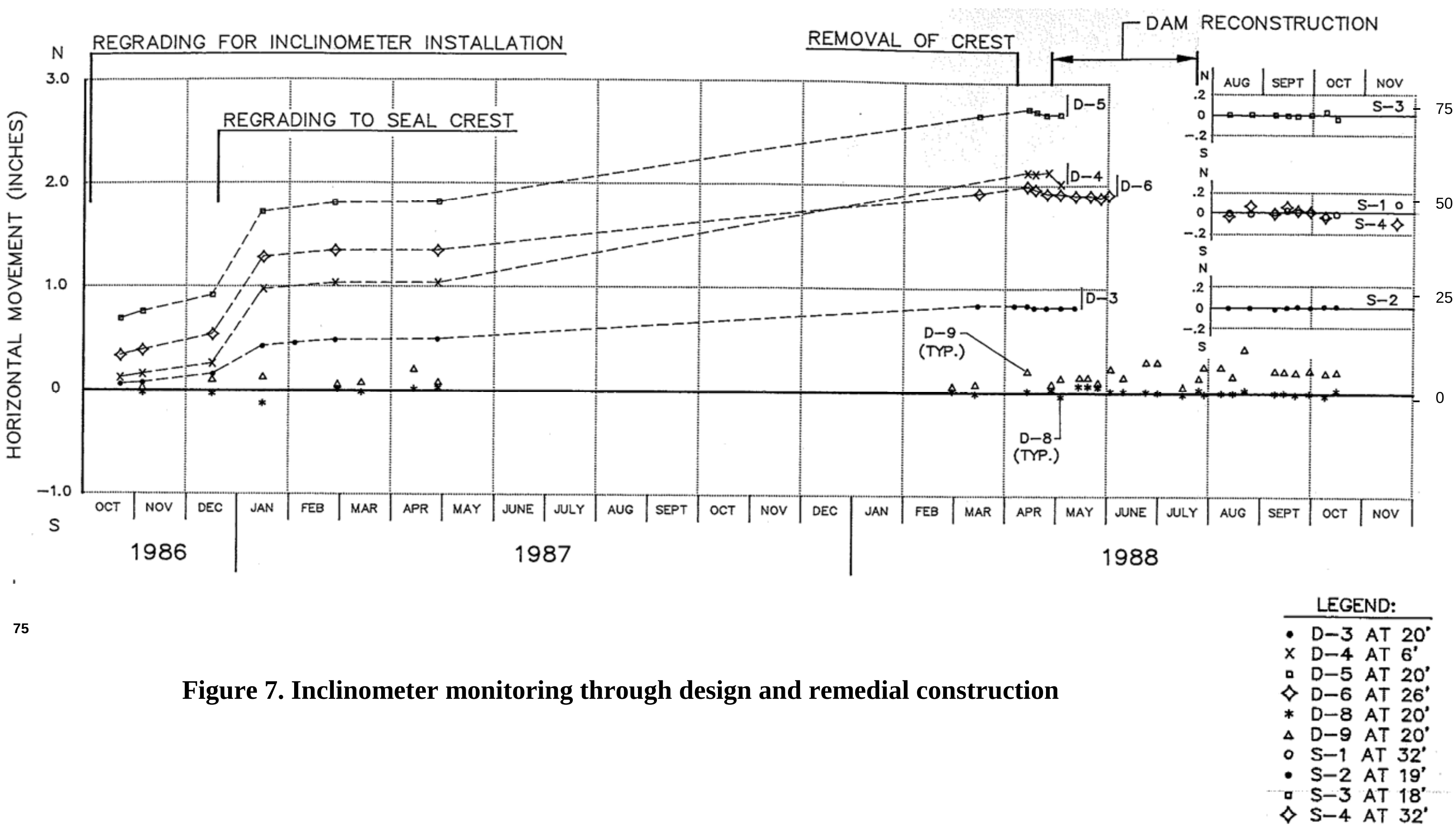


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Figure 7. Inclinometer monitoring through design and remedial construction

This observation provided a level of confidence that movement would not occur after remedial construction was complete.

After construction, a new set of inclinometers were installed at the locations shown on Figure 4. Inclinometers S-1 and S-4 are located on the dam crest and S-2 and S-3 are located on a bench approximately at the mid-height of the upstream slope. As shown on Figure 7, for the first several months after remedial construction, the new inclinometers showed no movement of the upstream slope as expected. However, as the reservoir was filled and the embankment began to become saturated, movement of the upstream slope began to occur. As the reservoir filled, inclinometers S-2 and S-3 were no longer accessible leaving S-1 and S-4 available for measurement.

Figure 8 includes two graphs, one showing the pool level and the piezometric levels at pneumatic piezometers P-4 and P-5 (see Figures 4 and 5 for location) and the other showing horizontal movement at inclinometers S-1 and S-4, at a depth corresponding to the position of maximum displacement and the approximate original shear plane.

The final normal pool is El. 1160 however the reservoir was initially raised to El. 1150 and remained there for about 6 months. As the reservoir filled, the piezometric levels in P-4 and P-5 increased and movement was observed in both S-1 and S-4.

Figure 9 is a plot of the rate of movement with time for S-1 and S-4 and shows that during initial filling, the horizontal movement was at a rate of about 6.3 mm (0.25 in) per year. Once the reservoir level reached El. 1150, movement in both inclinometers appeared to diminish. After 6 months, the reservoir was raised to its final elevation at El. 1160. As the reservoir level increased, horizontal movements were again detected, at post-remedial construction rates, in both inclinometers. The movement in S-4 subsided a few months after the reservoir reached its maximum level and very little movement has been observed in this inclinometer since the reservoir reached its maximum level.

Horizontal movement continued at inclinometer S-1 for about 20 years after the reservoir reached it maximum pool level. As shown on Figure 9, the rate of movement in S-1 gradually decreased from about 6.3 mm (0.25 in) per year in 1991 to less than 1.5 mm (.05 in) per year in 2010. This extended period of movement was not anticipated given the remedial design stress levels estimated based on limit equilibrium analyses, and resulted in questions about the design assumptions and the basis for establishing the expected performance of the dam.

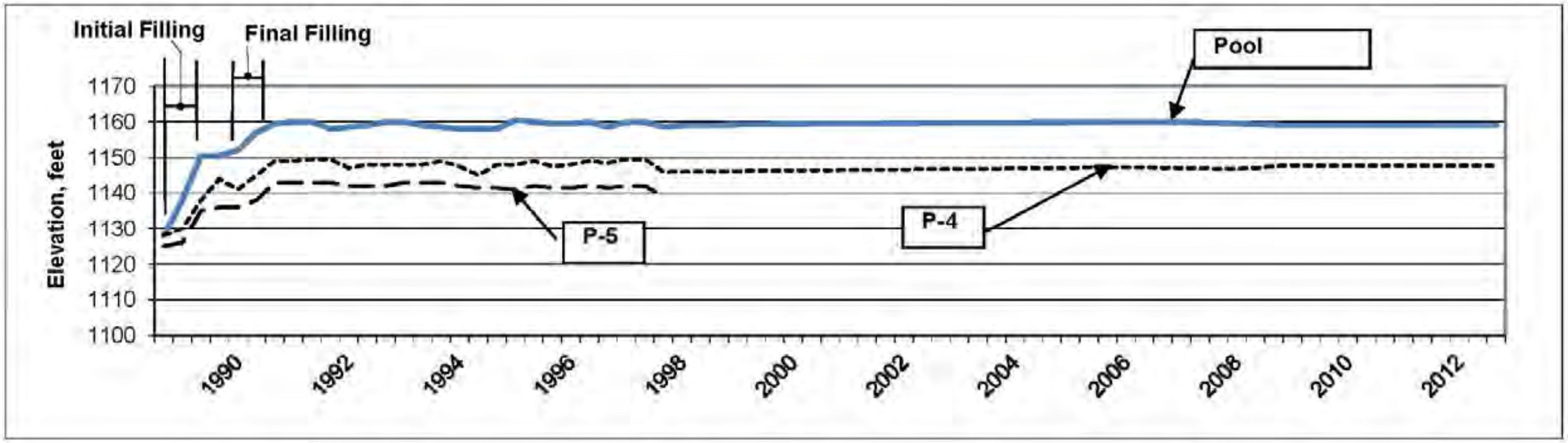

Figure 8a. Reservoir pool level and piezometer measurements

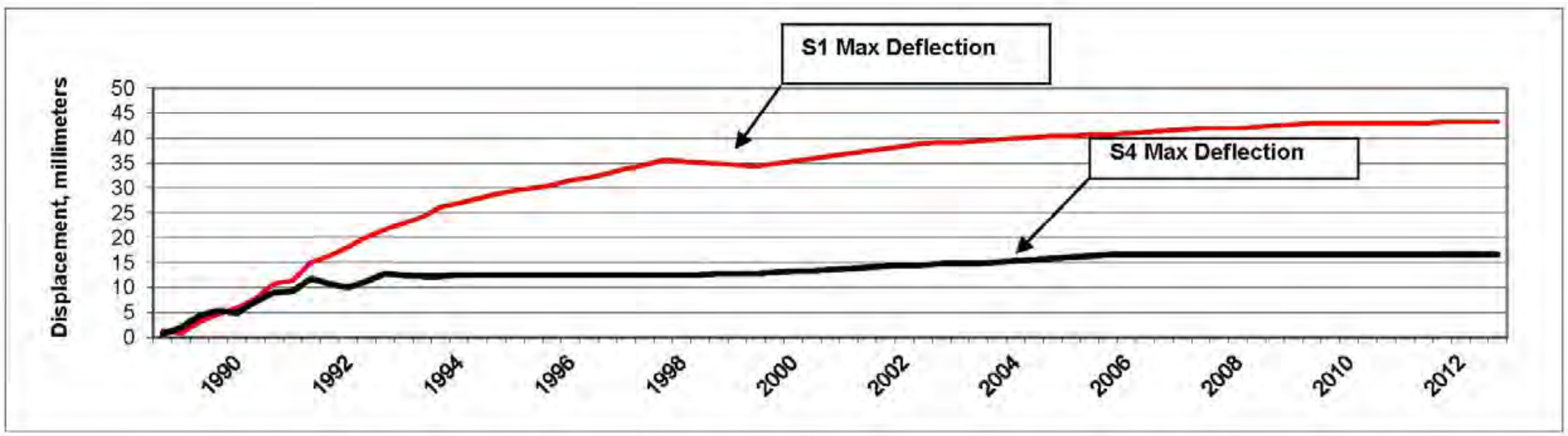

Figure 8b. Inclinometer measurements after remedial construction

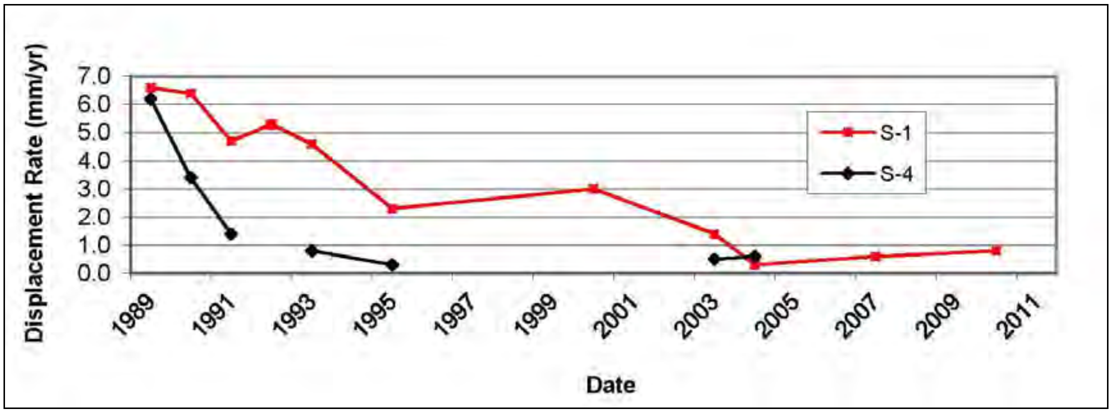

Figure 9. Rate of post remedial construction inclinometer movement

STRESS ANALYSIS

To help explain the observed long term movement at the Boy Scout Dam, the Plaxis (2011) two dimensional finite element computer program was used to estimate the stress conditions at selected points on the failure surface. The cross-sections shown in Figure 2 and Figure 5 were used to develop a plane strain finite element model. The Soft Soil Creep soil model along with an interface element was used to model the existing embankment soil and the known failure surface through the embankment fill. The Mohr-Coulomb soil model was used for the remaining soil layers. All layers were modeled as drained behavior types. Parameters selected for the soil layers, including the embankment fill, were based on the subsurface investigations completed during the remedial design of the dam. Parameters for the underlying rock layers were based on typical values for similar material. Figure 10 shows the finite element grid and the input parameters for the model.

Four conditions, representing significant changes to the dam, were considered in the analyses. The conditions include:

1. The existing conditions after failure of the upstream slope. Creep movement was occurring under these conditions.
2. After the buttress was constructed and the top of the dam was replaced but prior to filling the reservoir. No movement was detected during this period under these conditions.
3. After remedial construction was complete and the reservoir was filled. Creep movement was observed again under these conditions, and,
4. After the reservoir was filled and sufficient time elapsed for equilibrium conditions to be established.

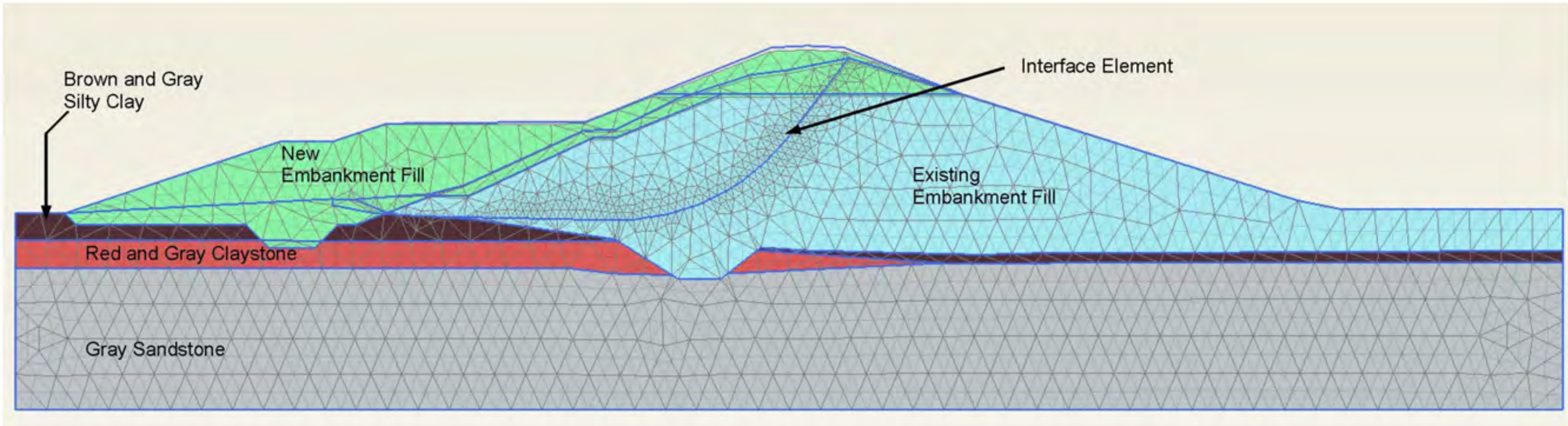

Figure 10a. Finite element mesh

Soil Layer	Material Model	Behavior Type	Saturated Unit Weight (lb/ft^3)	Unsaturated Unit Weight (lb/ft^3)	Horizontal Permeability (ft/day)	Vertical Permeability (ft/day)	Young's Modulus (lb/ft^2)	Poisson's Ratio	Compression Index	Swelling Index	Secondary Compression	Initial Void Ratio	Cohesion (lb/ft^2)	Friction Angle (°)	Dilatancy Angle (°)	Overconsolidation Ratio	Interface Strength
Symbol			γ_{sat}	γ_{unsat}	k_x	k_y	E	ν	C_c	C_s	c_α	e_o	c'	ϕ	ψ	OCR	R_{inter}
Existing Embankment Fill	Soft Soil Creep	Drained	130	112	0.00283	0.00283	-	-	0.20	0.015	0.010	0.52	0	24	0	1.5	1.0
Interface Element	Soft Soil Creep	Drained	130	112	0.00283	0.00283	-	-	0.20	0.015	0.010	0.52	0	24	0	1.5	0.5
New Embankment Fill	Mohr-Coulomb	Drained	130	112	0.00283	0.00283	400,000	0.3	-	-	-	-	0	28	0	-	1.0
Brown and Gray Silty Clay	Mohr-Coulomb	Drained	130	112	0.0283	0.0283	400,000	0.3	-	-	-	-	0	28	0	-	1.0
Red and Gray Claystone	Mohr-Coulomb	Drained	140	130	2.83	0.283	3,000,000	0.3	-	-	-	-	0	38	0	-	1.0
Gray Sandstone	Mohr-Coulomb	Drained	140	130	2.83	0.283	3,000,000	0.3	-	-	-	-	0	38	0	-	1.0

Figure 10b. Model parameters

Using the program, we estimated the stress conditions for each case for points A through E (see Figures 2 and 5 for locations). The normal stress and shear stress at which creep movement was observed in the constant stress creep test (Figure 3), were used to draw the Mohr's circle and estimate p′ and q for this state of stress. The straight line shown on Figure 11, labeled (K- creep line), corresponds to the values of p′ and q from the constant stress tests where creep movements were observed. Plastic deformation is expected at points above this line and movement is not expected for points that plot below this line.

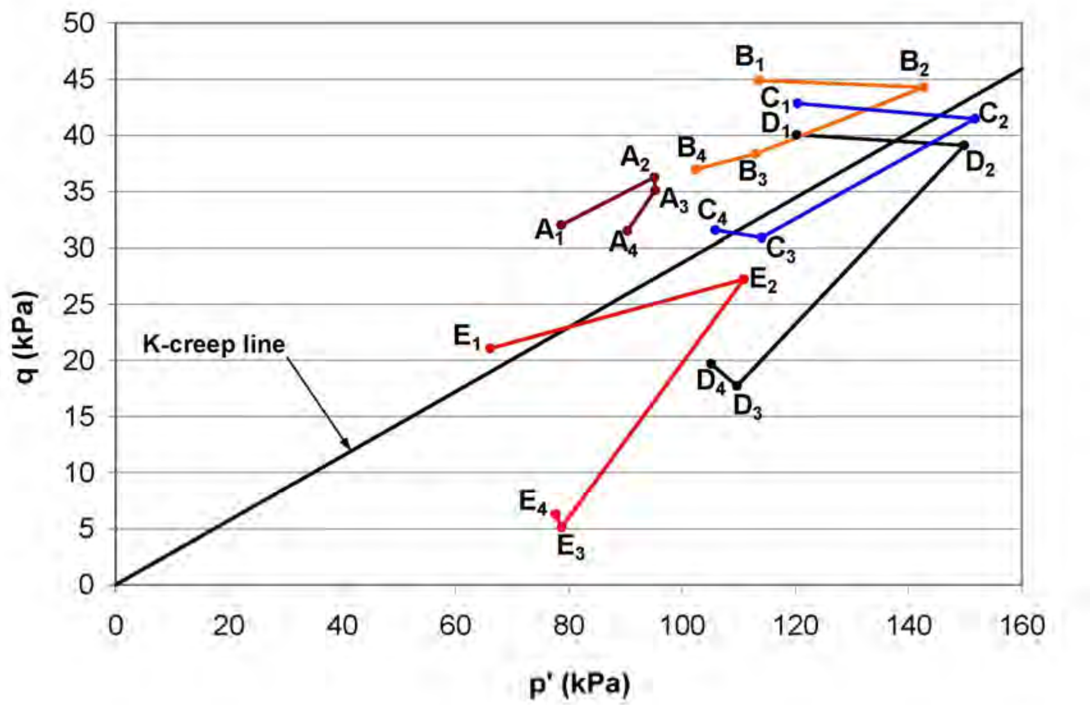

Figure 11. Stress paths for points on failure surface

The stress paths for conditions 1 through 4 at points A through E are also plotted on Figure 11. The stresses at all points plot above the K_{creep} line for the existing conditions (Case 1), suggesting that movement would be occurring at this state of stress. This is consistent with the observations of creep movements for the existing conditions.

After the buttress is constructed and the top is replaced (Case 2) the stress paths shift to the right. The stress conditions at point A remain above the K_{creep} line while the stresses at B point move closer to the K_{creep} line and the stress conditions at points C, D and E move below the K_{creep} line. The stresses suggest that creep movements would be occurring on the upper portion of the failure surface but not along the lower portion of the failure surface. The inclinometer measurements after construction and prior to filling did not show horizontal movement (Figure 7). Either creep movements were not occurring at this point or they were too small to be detected during the short time between construction and filling the lake.

As the reservoir is filled and the upstream slope is saturated (Case 3) p′ and q both decrease and the stress path shifts down and to the left. The reduction in q is consistent with the observations of Bishop and Bjerrum (1960) that there is a reduction in shear stress upon filling due to the stabilizing effect of the water pressure. This is accompanied by a reduction in effective stress, p′, (and shear resistance) due to saturation of the upstream slope. The net effect is to either improve or reduce the stability depending on the point in question and the geometry of the slope. The stress conditions at points on the upper portion of the failure surface become less favorable upon saturation while those on the lower portion become more favorable. Point A, on the upper portion of the failure surface, stays above the K_{creep} line and Point B, also on the upper failure surface, moves above the K_{creep} line as the upstream slope becomes saturated. Point C stays below the K_{creep} line but moves closer to it. Points on the lower portion of the failure surface (D and E) appear to become more stable (i.e., move away from the K_{creep} line). The movement of the stress paths to less favorable positions for points on the upper portion of the failure surface could explain the continued deformation detected in the inclinometers in the upper portion of the failure surface upon saturation of the upstream slope.

A time passes (Case 3 to 4), there is a stress transfer from the soil in the upper portion of the failure surface to the soil in the lower portion of the failure surface. The shear stress at points A and B is reduced while at points C, D and E it increases. This results in a reduction in the creep rate of the upper soils and coincides with the condition that the dam is approaching some 20 years after remedial construction is complete.

Another way to look at the results of the stress analysis is to consider the mobilized friction angle at different points on the failure surface for the different conditions. For Case 1 (existing conditions) all of the mobilized friction angles are above the level where creep movements were observed in the laboratory test (Figure 3). After the buttress is constructed (Case 2), the mobilized friction angles along the lower portion of the failure surface drop below the creep level however, the points on the upper portion stay above the friction angle where creep was observed. When the reservoir is filled and the upstream slope is saturated (Case 3), the mobilized friction angle along the lower portion of the failure surface is reduced but on the upper portion it stays the same or increases slightly. Finally, as time passes (Case 4), stress is transferred to the lower portion of the failure surface, resulting in an increase in the mobilized friction angle there. Because the mobilized friction angle at the upper portion of the failure surface is greater than the threshold established by the constant stress direct shear tests for ongoing creep movements, small continued creep could be expected and has been observed at the inclinometers located on the crest of the dam.

Condition	Mobilized Friction Angle, ϕ				
	Point A	Point B	Point C	Point D	Point E
1	24.0	23.3	20.8	19.5	18.6
2	22.4	18.1	15.9	15.1	14.2
3	21.6	19.8	15.7	9.3	3.7
4	20.4	21.1	17.3	10.8	4.6

Table 1. Mobilized friction angles

CONCLUSIONS

The results point out the weakness of limit equilibrium analyses as a predictor of dam performance. The limit equilibrium analyses provide an accounting of the available strength relative to the destabilizing forces along a failure surface and provides an indication of the "safety" of the dam relative to failure. The analysis considers that stresses are distributed along the failure surface in proportion to the available strength. This characteristic of limit analyses can lead to unrealistic expectations regarding dam performance, especially when the analysis involves a creeping failed slope. While the potential resistance associated with remedial construction such as a buttress is available, it is not mobilized until there is a redistribution of stress. This may take years to occur for a creeping slope and may never occur to the extent suggested by a limit analysis.

To supplement limit equilibrium analyses and gain insight into dam performance, an analysis of the stresses at critical points on the failure surface is recommended. Comparison of the stresses at key points with the results of strength tests (including the results of creep tests if appropriate) provides a much better indication of dam performance, particularly expectations with respect to residual movements, and additional basis to establish a surveillance and monitoring plan.

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