Deformation Performance of Waba Dam

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ABSTRACT
Waba dam is an earthfill dam founded on marine clay in Eastern Ontario. Deformation observations based on surveys and slope inclinometer readings are presented. Survey measurements have shown that the dam has settled over (5 ft) beneath its maximum section. The dam was built with a camber to accommodate the large predicted settlement without loss of required freeboard. Based on current settlements, future settlements are predicted based on Asaoka's method. Inclinometer measurements have indicated a foundation lateral spreading of 12 in. The trend of settlement versus lateral displacement at Waba Dam is comparable to well behaving embankments reported in literature.

RÉSUMÉ
Waba Dam est un barrage en remblai foundé sur les argiles marines de l'est de l'Ontario. Les déformations enregistrées par relevés topographiques et inclinomètres sont présentées. Les relevés topographiques montrent des tassements de l'ordre de 5 pieds sous la section la plus haute du remblai. Le barrage avait été construit avec une cambrure pour compenser les tassements anticipés, et maintenir le marnage. Sur la base des tassements observés, les tassements futurs sont prédits avec la méthode de Asaoka. Les inclinomètres ont indiqués une déformation latérale de l'ordre de 12 pouces. Les tendances tassement versus déformations latérales observées à Waba Dam se comparent favorablement aux remblais à bon comportement.

1 INTRODUCTION

Ontario Power Generation (OPG) owns and operates over 240 dams across Ontario. OPG has a comprehensive dam safety program encompassing dam engineering, design reviews, emergency preparedness and surveillance and monitoring. This paper describes the performance of the Waba Dam which is being monitored as part of OPG Dam Safety Program.

1.1 Description of Dam

Waba Dam is an earthfill dam about 3600 ft long, with a maximum height of about 60 ft above the foundation, designed and built by Acres Consulting Ltd. for Ontario Hydro and placed in service in 1976. The dam, consisting of compacted impervious clay fill with internal filter and drainage zones, was constructed on about 250 ft thick foundation consisting mainly of soft compressible and sensitive marine clays overlying bedrock (Figs. 1 and 2). Wide berms were provided upstream and downstream beyond the slopes of the main fill to ensure stability of the dyke on the soft clay foundation and the crest elevations were designed to allow for a large anticipated settlement in the foundation which would be overstressed by the dam loading.

An extensive instrumentation program, including foundation settlement gauges; surface monuments; slope inclinometers; load cells; and piezometers, has been in effect since the construction of the dam in 1975.

1.2 Soil Characteristics of Waba Dam and its Foundation

The foundation under Waba Dam is comprised of a weathered desiccated brown clay crust up to 5ft thick overlying an extensive deposit of sensitive grey marine clay up to 250ft thick. The grey marine clay has been found overlying a deposit of sand or sand and gravel up to 10ft thick, which directly overlies bedrock.

The weathered crust has been characterized as being strong, but fissured both in the horizontal and vertical directions. Its undrained strength is generally in excess of 192 kPa, although the macro strength, controlled by the fissures, etc., is considerably less. The direct tension test in the crust material gave tensile strength of about 29 kPa. The weathered clay was used as impervious fill material during the construction of the Waba Dam.

The underlying unweathered grey clay is a varved sensitive clay with occasional thin layers of silt or very thin layers of fine sand. Its natural water content, liquid limit, plastic limit, liquidity index and sensitivity values were found to be 40, 60, 30, 50, 20 to 25, 0.9 to 4.0 and 20 to 500 respectively. The unweathered clay has been previously estimated to have an average undrained strength of 36 kPa near the interface with the weathered clay zone and then increasing progressively with depth. The clay deposit is slightly overconsolidated with a steep virgin compression curve.

Waba Dam core material was obtained from the weathered clay crust excavated during the relocation of the adjacent CPR line, or borrowed from a local source.

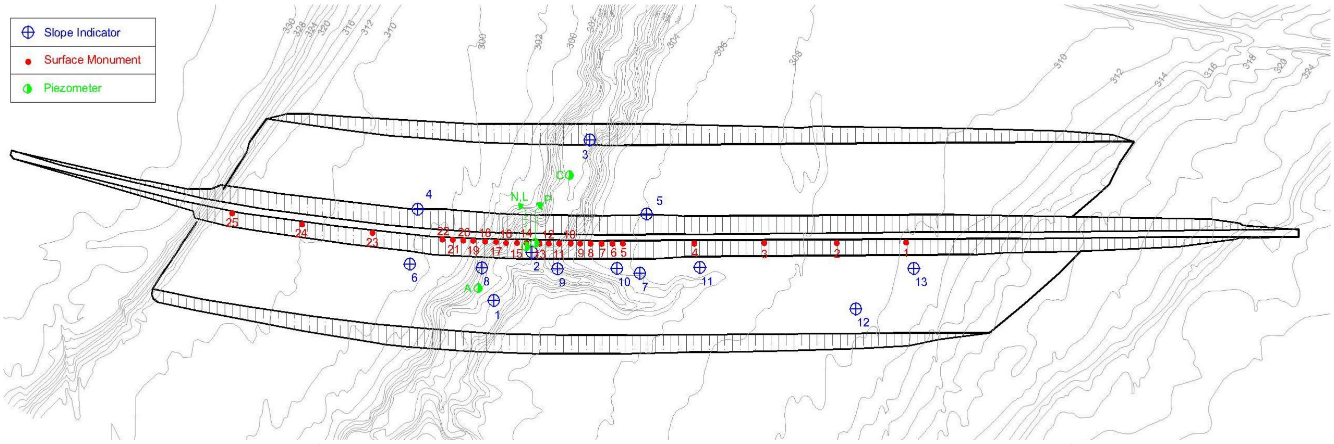

Figure 1. Plan view of Waba dam showing some piezometers; slope inclinometers; and surface monuments

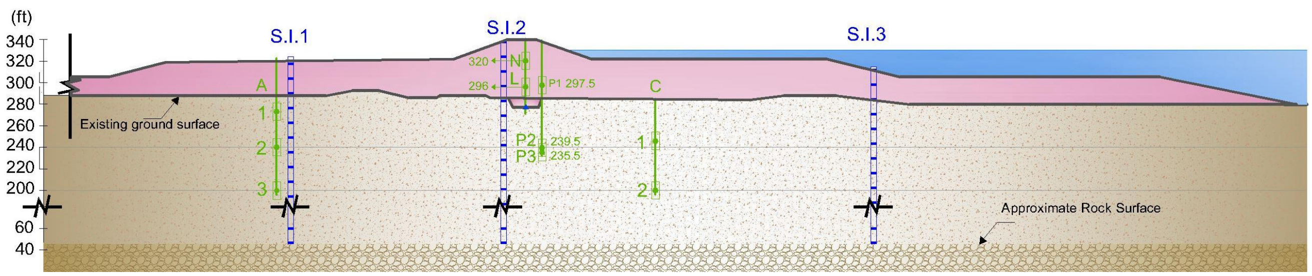

Figure 2. Typical cross section of Waba dam showing some piezometers and slope inclinometers

2 PERFORMANCE PREDICTION ISSUES

Significant settlement has occurred at Waba Dam since its construction in 1976. The current settlement observation indicates that the dam has settled about 10 percent of its original height. This is a significant settlement for earth dams, which usually settle about one percent of their heights (ASCE 2000). Although this type of settlement was anticipated and accounted for during design, it is still important, as part of dam performance monitoring, that vertical and horizontal deformations are monitored during the life span of the earth dam to continuously ensure its integrity. It is desirable that the future performance of this dam conform to predictions. Three potentially competing mechanisms make such prediction challenging: (a) weakening of the foundation clays due to loss of salinity, (b) weakening of the foundation soils by strain softening, and (c) strengthening of the foundation soils through consolidation.

The salinity of the pore water in marine clay has a major effect on its geotechnical behavior. This effect was investigated by many researchers particularly Torrance (1975), where he demonstrated that properties such as remolded shear strength, sensitivity, and liquid limit of Marine clay were significantly affected by changing the salinity of its pore water. A decrease in salinity results in an increase in sensitivity and a decrease in liquid limit and shear strength.

The dependency of marine clay behavior on the salinity of its pore water is of particular importance with regards to Waba Dam. As the fresh water from the reservoir seeps through the dam, leaching the salt from the marine clay will take place and may result in a reduction of salinity and hence an increase in sensitivity and a decrease in liquid limit and shear strength.

Two soil investigation programs, including in-situ and laboratory tests, were undertaken in 1972 and 1986. The undrained shear strength values measured in the 1986 laboratory tests appeared to be lower than those measured in 1972 in the Upper Clay layer but similar in the Lower Clay. However, these values were equal to or greater than those used in the original slope stability analyses (Cragg 1988). In 1984 another site investigation program was also undertaken to determine the dynamic characteristics of the fill and foundation materials. Based on the vane shear tests, there was no increase in the strength of the foundation soils which would normally be expected as a result of consolidation of deposits under the embankment loading after nine years. However, Law (1979) showed that vane shear tests may not reliably reflect strength gains under loading. This is due in part to the strong dependency of the apparent vane strength on the effective horizontal stress, which in turn influenced by a large number of factors related to soil compressibility and the boundary conditions.

The reduction in shear strength confirmed by the 1972 and 1986 tests maybe attributed to different phenomena. First, is the leaching of the marine clay due to reservoir water seeping through the dam and washing out the salt from the clay. Second, strain softening of the clay due to shearing failure taking place in the foundation. Third, a combination of both phenomena is also possible.

A comparison of soil chemistry tests in 1972 and 1986 could not confirm the leaching phenomenon. This is likely due to the fact that the underlying methods used to estimate the pore water salinity were different. Torrance (1974) showed that the early method of determining salinity, which the 1972 test was based on, resulted in overestimation of the pore water salinity. However, what the 1986 test did show was that the salinity is low in the clay fill and high in the native clay with values decreasing towards the underlying sand layer. A third soils investigation may be needed to confirm the leaching phenomenon.

This paper focuses on the observation and analysis of measured performance parameters at the dam such as, settlement, lateral spreading, and pore water pressure, to assess the deformation performance of Waba Dam and make predictions for future settlements. The analysis provides some indications of the relative contributions of the competing mechanisms. It is acknowledged that an extensive site investigation program is needed in conjunction with this performance analysis to fully understand the underlying behavior of the dam

3 SETTLEMENT

Settlement performance of Waba Dam has been monitored via the annual surveying of surface monuments and foundation settlement gauges installed at the dam. The initial settlement prediction during the original design process suggested that final settlement would range (based on the estimation of the pre-consolidation pressure) between a maximum of 8ft to 13 ft and a minimum of 4ft to 6 ft at the highest dam section. The time for completion of the primary consolidation was estimated to be in excess of 100 years based on a low value of coefficient of consolidation.

Figure 3 shows the settlement curves of the foundation. These curves somewhat resemble the consolidation settlement curves observed in oedometer tests. The shape of the settlement curves may be an indication that the primary consolidation stage is still underway. Figure 4 shows the piezometric elevations in the dam fill (Piezometers N, L, and P1) and foundation (Piezometers P2 and P3). These piezometers are located in close proximity to the highest section of the dam. It can be seen from the piezometric elevations that the dissipation of excess pore water pressure is still underway which, is also an indication of the primary consolidation stage. Other piezometers installed at different locations of the dam suggest the same general trends observed by these piezometers. In addition to indicating that consolidation is still underway, the piezometric level above headpond suggests that fresh water steady state seepage flows and the anticipated associated leaching has not yet fully developed. The settlement profiles of the dam crest are shown in Figure 8. It can be seen that some minor differential settlements are also apparent in the profiles.

Based on these settlement observations it is possible to predict future settlements and compare them to those initially predicted during the design phase of the dam.

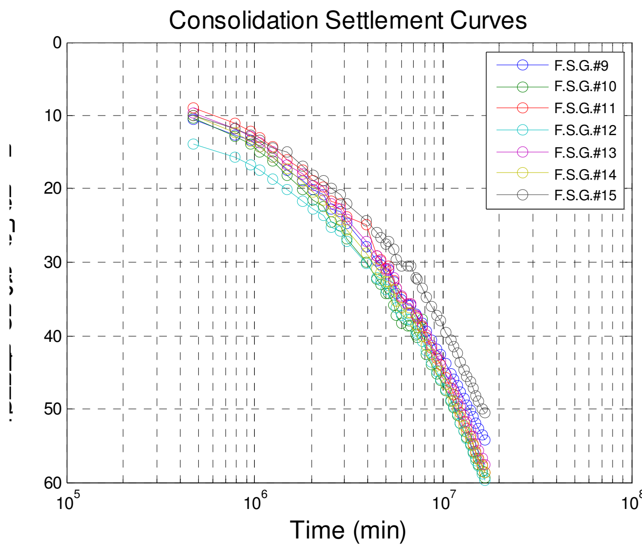

Figure 3. Foundation consolidation settlement curves

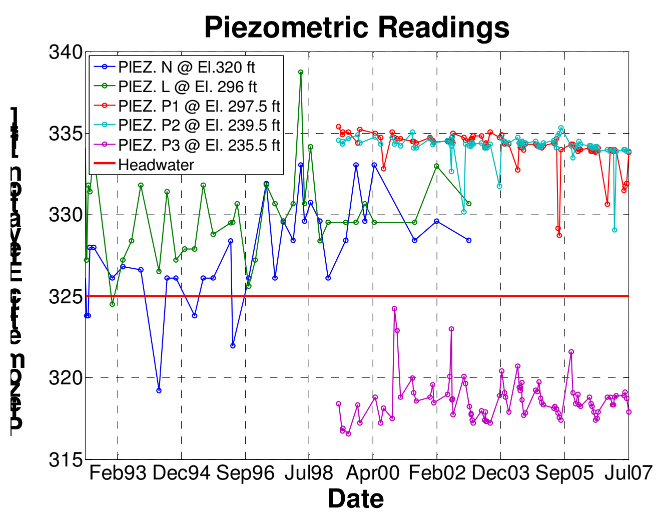

Figure 4. Piezometric elevations in dam

3.1 Settlement Prediction: Asaoka Graphical Method

Asaoka (1978) developed a method for settlement prediction based on observations. In this method, the one-dimensional consolidation equation is first used to arrive at a time-settlement relationship. Then, based on a difference form (an autoregressive equation) of the time-settlement relationship an observational procedure was developed where future settlement is predicted using past observations. For a simple graphical prediction of settlement, a first order approximation of the difference equation was used. This equation takes the following form:

$$\rho_j = \beta_0 + \beta_1 \rho_{j-1} \qquad [1]$$

where
ρ_j denotes $\rho(t_j)$, the settlement at the time $t=t_j$
β_o and β_1 are unknown parameters
The final settlement, ρ_f, is reached when $\rho_j=\rho_{j-1}$, thus:

$$\rho_f = \frac{\beta_0}{1-\beta_1} \qquad [2]$$

So, for $n+1$ settlement observations, $(\rho_0, \rho_1,\ldots,\rho_n)$ generated by a constant external load, we can plot n points, (ρ_k, ρ_{k-1}) for $k=1, 2, \ldots, n$, on the (ρ_j, ρ_{j-1}) co-ordinate. If all the plotted points lie on a straight line, then the first order approximation, Eq.1, gives a good fitting to the settlement observations. In that case, β_0 and β_1 are the intercept and the slope of that straight line. The above method is illustrated in Figs. 5 and 6 below.

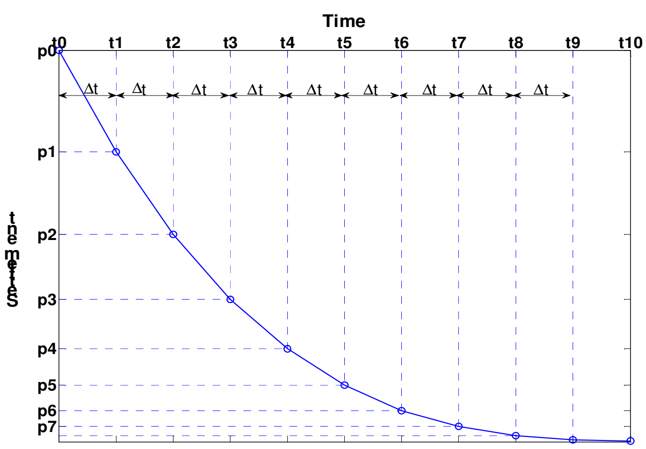

Figure 5. Illustration of Asaoka method: establishing settlement observations into fixed time intervals

By a recursive operation to the first order approximation equation, it is also possible to predict the settlement at any future time as follows:

$$\rho_j = \frac{\beta_0}{1-\beta_1} - \left[\frac{\beta_0}{1-\beta_1} - \rho_0\right]\cdot(\beta_1)^j \qquad [3]$$

Using the settlement observations along with Asaoka method, future settlement performance of Waba Dam, with reasonable accuracy, could be achieved. Figure 7 shows that the predicted final settlement of Waba Dam at the Surface Monument 14 (S.M.14) is about 6.13 ft.

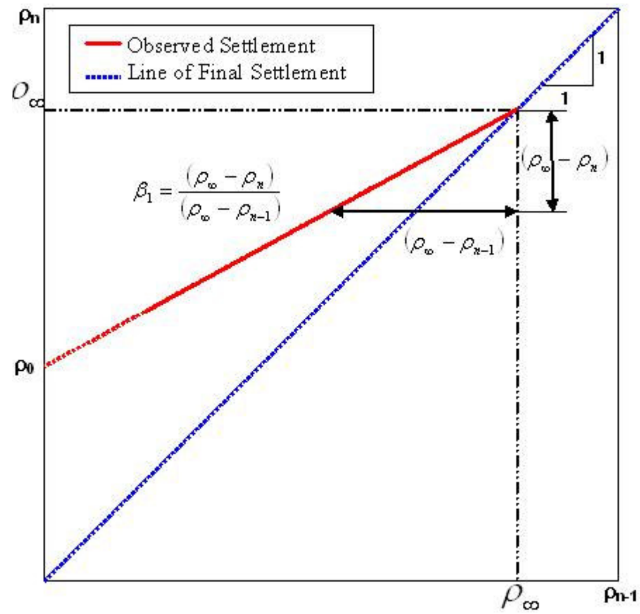

Figure 6. Illustration of Asaoka method: projection of observed settlement line to final settlement line

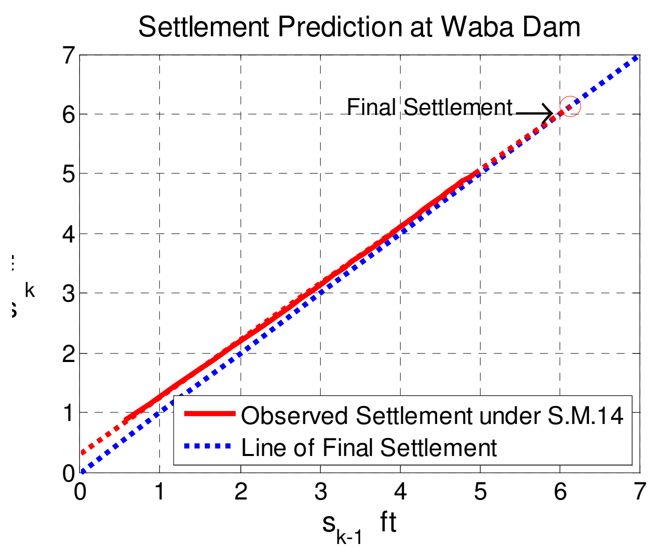

Figure 7. Prediction of final settlement of Waba dam at surface monument # 14

The predicted settlement for the year of 2009 is about 5.08 ft. As a validation step, the Asaoka method was applied to the data up to 2007 (inclusive) and a settlement prediction for the year of 2008 was made and compared with the observed settlement of that year. The observed versus predicted settlements for 2008 were virtually identical as can be seen form Table 1 below. The time needed to reach the final settlement is projected to be in the order of 70 years from now under S.M. No. 14. Since 33 years have already passed, the total consolidation time compares well to original design calculations. However, the rate of consolidation will significantly decrease after the year 2030. The final settlement profile for the entire dam is shown in Figure 8.

Table 1: Observed and predicted settlements at Waba dam for selected surface monuments

Surface Monument	Observed Settlement, (ft)	Predicted Settlement, (ft)		
	2008	2008	2009	**Final**
S.M. 1	4.45	4.45	4.50	**5.26**
S.M. 3	5.06	5.07	5.12	**5.95**
S.M. 5	5.21	5.22	5.27	**6.11**
S.M. 7	5.23	5.21	5.27	**6.14**
S.M. 9	4.97	4.98	5.03	**5.97**
S.M. 11	4.81	4.81	4.88	**6.00**
S.M. 14	5.03	5.02	5.08	**6.13**
S.M. 17	4.71	4.71	4.77	**5.80**
S.M. 19	5.11	5.12	5.17	**5.95**
S.M. 21	5.00	5.00	5.05	**5.91**
S.M. 23	4.34	4.34	4.38	**5.23**
S.M. 25	2.30	2.31	2.32	**2.70**

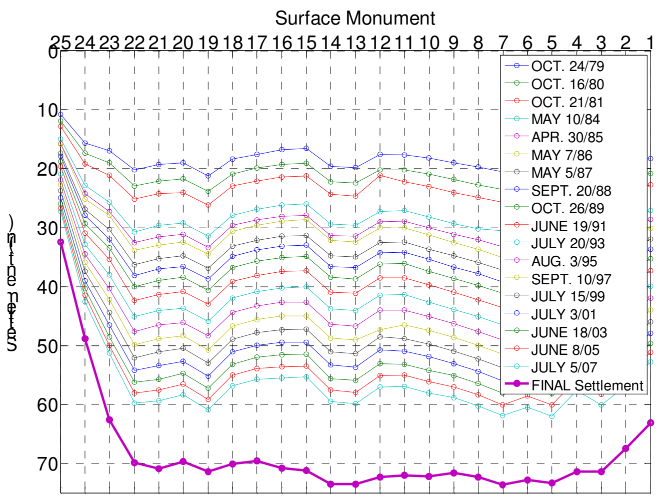

Figure 8. Settlement profiles at Waba dam – predicted final settlement profile also shown

The final settlement predicted based on Asaoka method is comparable to the minimum upper bound of settlement of 5.90 ft predicted during the original design phase Based on this prediction about 80-85% of consolidation settlement has occurred. As porewater pressures have not yet dissipated and consolidation is only partially complete, only nominal improvement in foundation strength and performance can be expected at this time.

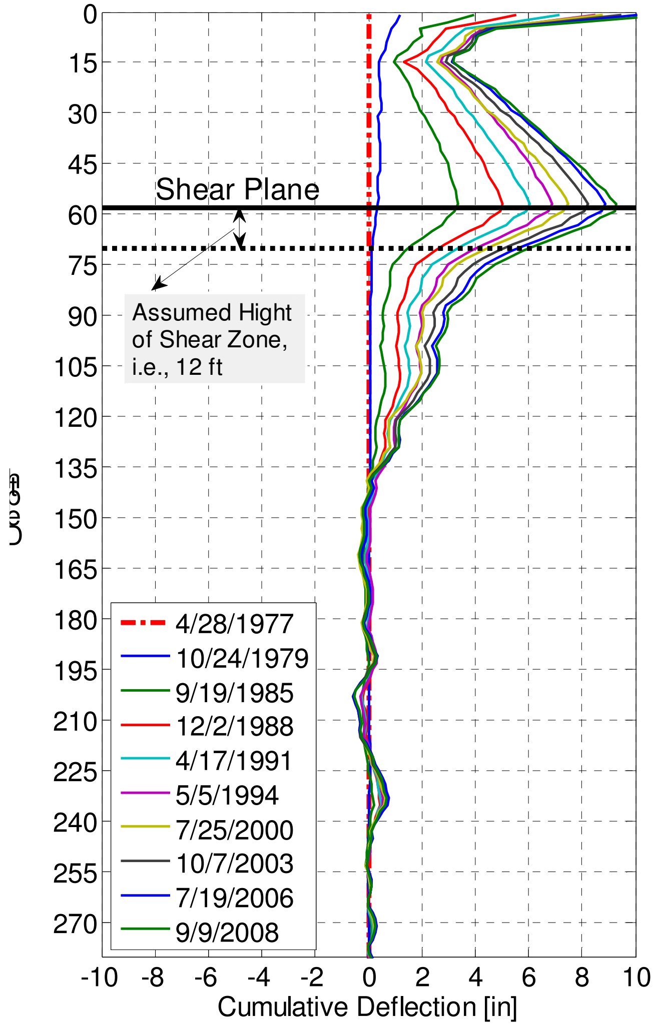

Figure 9. Slope inclinometer SI-2 in D/S edge of crest

4 LATERAL DEFORMATION

There are 13 slope inclinometers installed at Waba Dam in the downstream and upstream sides to measure the lateral deformation. Figure 9 shows a cumulative horizontal deformation measured by Slope Inclinometer No. 2 installed near the edge of dam crest. It is clear from the figure that there is a progressive downstream movement (bulging) at around 60 ft below crest of dam. The location of this "shear zone" corresponds to the relatively thin and weak normally consolidated layer (5 ft thick) underlying the topmost crust in the foundation soil. The same shear zone is also evident in slope inclinometer 1 (not shown here).

This lateral deformation is inevitable with the large settlement observed at the dam. Quantifying this lateral deformation in terms of shear and axial strain with time may provide a better understanding of the underlying soil behavior at the identified "shear zone".

The shear strain and consequently the axial strain can be calculated as per Gould (1960), shown in Figure 10:

$$\gamma = \frac{H}{T \times \cos^2 \theta} \qquad [4]$$

where:

γ: shear strain in radians
H: observed horizontal movement
T: vertical dimension of shear zone
θ: angle of shear zone with horizontal

The angle with the horizontal is taken to be zero in our case. The axial strain can also be approximated as:

$$\varepsilon_a = \frac{2}{3} \times \frac{\gamma_f}{\cos \phi} \qquad [5]$$

where:

ε_a: axial strain
γ_f: shear strain on failure plane
ϕ: angle of shearing resistance

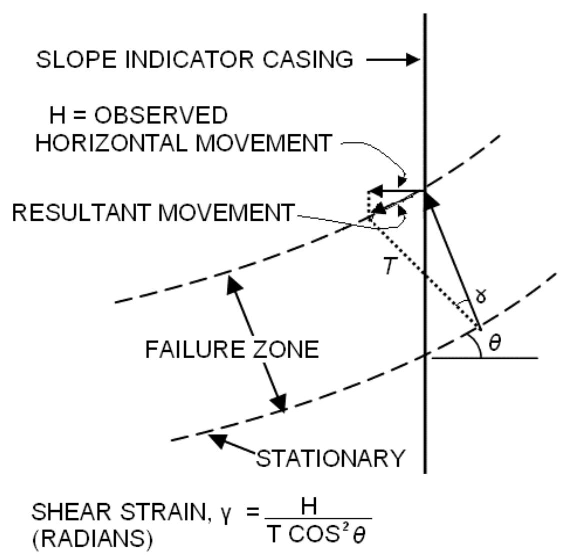

Figure 10. Shear strain calculation from slope inclinometer

The angle of shearing resistance is taken to be 23° based on the 1986 site investigation program. Figure 11 shows the axial strain with time approximated from SI.2 at the yield zone. Depending on the thickness of the shear zone considered in the approximation, the maximum axial strain ranges between 1.4-1.75%. Figure 12 shows the strain rate versus time at the "yield zone" indicated by Slope Inclinometer No. 2. The ongoing decrease of strain rate with time along with decreasing levels of horizontal and vertical deformation suggest that these strain softening and long term creep may not be long term concerns. However, the decrease in strain rate with time resembles literature reports of soil creep behavior, at a constant stress level, where creep rate decreases to a minimum value before it accelerates to failure. Future laboratory tests to examine creep behavior and ongoing monitoring of this site are planned to resolve this issue.

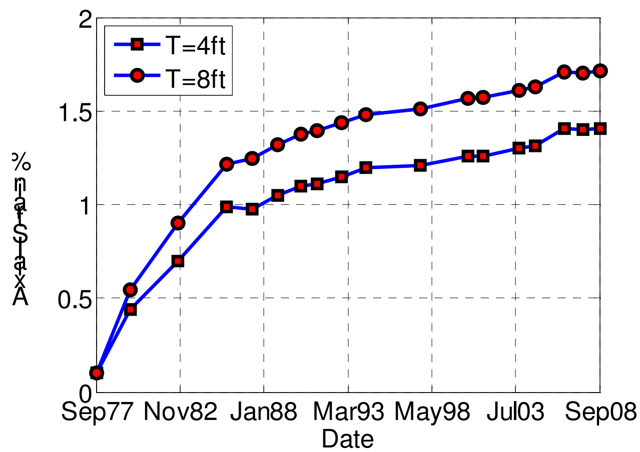

Figure 11. Axial strain vs. date at various depths depicted by SI-2

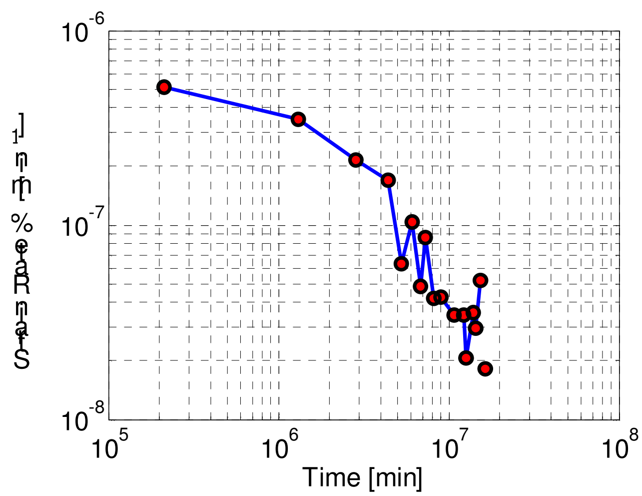

Figure 12. Strain rate vs. time at "Yield Zone" depicted by SI-2. In this case H=6 ft

Tavenas et al. (1979) investigated the variation of the maximum lateral displacement with the settlement under several embankments of different geometries and stability conditions (Figure 13). The period for these observations did not exceed five years and thus corresponds to the initial stages of consolidation. According to Tavenas et al. (1979), the sample case histories considered in the analysis was not sufficiently large to establish a clear definition of parameters affecting the $\Delta y_{max}/\Delta s$ ratios, such as the factor of safety.

For a comparison purpose, the observations at Waba Dam covering a period of 30 years (1977-2007) are superimposed on the same figure. A linear trend can also be established for Waba Dam such that $\Delta y_{max} \approx 0.22\Delta s$. Although there could be other case histories analyzing the $\Delta y_{max}/\Delta s$ ratios and correlating them with several embankment parameters, at the time of writing this paper, the cases presented by Tavenas et al. (1979) were the only cases utilized by the authors.

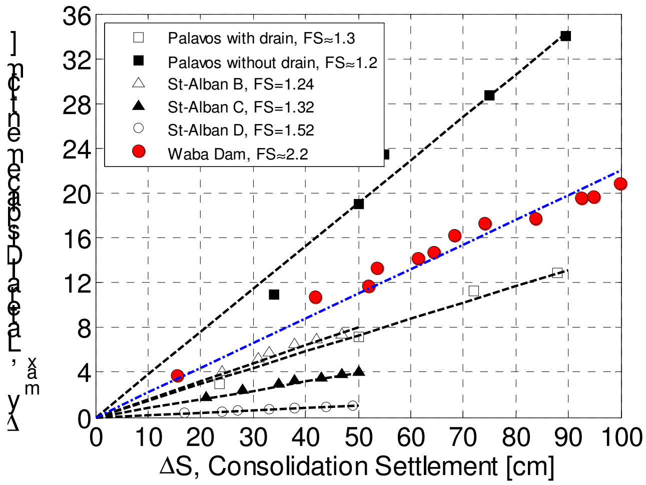

Figure 13. Relationships between maximum lateral displacement and settlement during consolidation, for different embankment geometries and stability conditions (after Tavenas et al. 1979)

It can be seen from the figure that the factor of safety decreases with increasing with $\Delta y_{max}/\Delta s$ ratios. However, the observation at Waba Dam does not follow this trend, which has a ratio of $\Delta y_{max}/\Delta s$ of 0.22 and a factor of safety of 2.2. The reasons for this could be attributed to several factors, first the geometry of Waba Dam is significantly different to those presented by Tavenas et al. 1979, second, the period of observation at Waba Dam is significantly longer than those presented by Tavenas, third, the marine clay present at Waba dam.

For a future performance evaluation, a linear trend between lateral displacement and settlement is established in Figure 14.

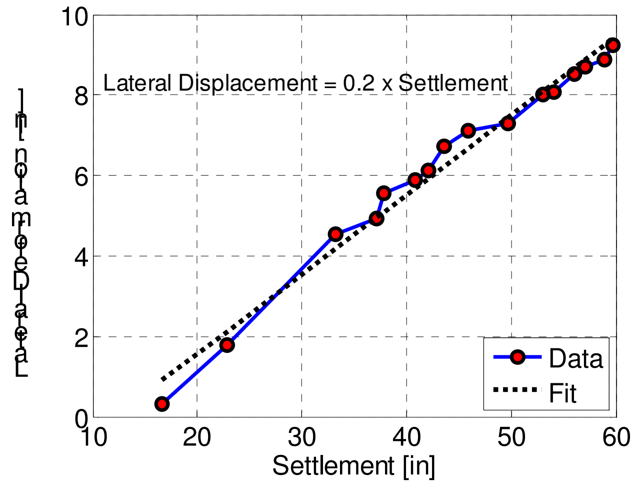

Figure 14. Lateral displacement vs. settlement at Waba dam

5 CONCLUSION

The case study of Waba Dam over 30 plus years shows settlements at the low end of original predictions based on laboratory tests. Predictions of long term deformation by the Asaoka's graphical method match well with observations. The time to final consolidation also matches well to the original design of 100 plus years. Changes in soil properties due to long term leaching of saline porewater in the marine clay have not yet been observed. However, any leaching may be delayed by high pore pressures due to ongoing consolidation. Likewise, performance improvements expected from consolidation are also delayed. Lateral strain rates are decreasing with time, consistent with decreasing deformations and advancing consolidation, indicating no current adverse trend in strain softening. The lateral versus vertical deformations are comparable to well behaving embankments reported in the literature. Taken together, these analyses indicate that Waba Dam is performing well and should continue to perform well into the future. OPG will continue to monitor the Waba Dam to ensure these trends in performance are within expected limits.

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