Use of Instrumented Test Fill to Assess Static Liquefaction of Impounded Fly Ash

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ABSTRACT

A 61-m (200-ft) high FGD waste landfill over an existing fly ash impoundment is currently under construction. A concern was whether typical waste filling rates would be sufficient for static liquefaction of the impounded fly ash to occur. In this paper, data from a large test fill is used to assess the potential for static liquefaction of the impounded fly ash. The test fill comprised a 9-m (30-ft) high preload fill which was constructed using much higher fill placement rates than for typical waste filling. Excess pore pressures in the impounded fly ash were measured. The time history of filling and measured excess pore pressures were used to back-calculate the average vertical coefficient of consolidation of the impounded fly ash, a parameter that is representative of the drainage characteristics of the fly ash. As measured excess pore pressures were primarily the result of volumetric loading, an analysis was conducted to estimate undrained shear-induced excess pore pressures for a condition with high potential for static liquefaction. Using these excess pore pressures and the interpreted rate of excess pore pressure dissipation, a global slope stability analysis was conducted. Analysis results and visual observations during test fill construction indicate that static liquefaction of the impounded fly ash is unlikely to occur for typical loading rates associated with waste filling.

INTRODUCTION

After the December 2008 slope failure at the Tennessee Valley Authority (TVA) Kingston Fossil Plant, the U.S. Environmental Protection Agency (EPA) and all electric utilities that used wet disposal techniques to manage coal combustion products (CCPs) raised concerns regarding geotechnical stability including the potential for static liquefaction of CCPs. Static liquefaction is a phenomenon in which the granular skeleton of a saturated, loose soil matrix collapses during an undrained shear loading resulting in a significant reduction in effective stress, resulting in the flow of the soil mass.

Using information from a subsurface investigation and laboratory testing program and information obtained from an instrumented test fill (hereafter referred to as a monitored preload fill (MPF)), the potential for static liquefaction of fly ash in an impoundment in Ohio (hereafter referred to as Fly Ash Reservoir 1 (FAR-1)) was evaluated. It is noted that these information sources were obtained prior to the Kingston incident and were obtained strictly as a means to assess the settlement potential of fly ash in an impoundment as part of the design of a FGD waste landfill expansion over FAR-1and were therefore not specifically focused on addressing the potential for static liquefaction. Herein, multiple lines of evidence are used to assess the potential for static liquefaction of the in-place fly ash in FAR-1, as described below.

- Cone penetration test (CPT) data was used to assess the “potential” for the in-place fly ash to undergo static liquefaction. This assessment does not predict whether the in-place ash has the “opportunity” to liquefy (i.e., is there a set of loading conditions which results in static shear stresses which exceed the undrained strength of the fly ash), it is merely used to identify whether the characteristics of the fly ash (e.g., density of the fly ash) are such that the material has the potential to liquefy.
- Using information from laboratory studies and the measured performance of the MPF, values of the coefficient of consolidation of the in-place fly ash were evaluated. While this value, by itself, is not an explicit indicator of static liquefaction, it can be used to assess whether field loading conditions are undrained, partially drained, or drained. That is, static liquefaction is usually associated with the development of relatively large excess pore pressures and no (or limited) drainage. Measured pore pressures from the MPF provide specific information on the magnitudes of pore pressures that would likely be developed in the in-place fly ash.
- Slope stability analyses were conducted using estimated excess pore pressures in the in-place fly ash which could lead to static liquefaction of the in-place fly ash. Computed factors of safety are shown to be relatively high, thus providing further confirmation of the limited potential for static liquefaction of the in-place fly ash.
- Finally, the fact that the 9-m (30-ft) high MPF was constructed in a relatively short period of time (relative to the amount of time required for typical waste placement) provides the most definitive evidence that the in-place fly ash at this impoundment is not susceptible to static liquefaction. Stated differently, if the in-place fly ash is susceptible to static liquefaction, high and sustained excess pore pressures would have developed and static liquefaction (involving a potential flow failure) likely would have occurred during construction of the MPF.

ASSESSMENT OF POTENTIAL FOR STATIC LIQUEFACTION USING CPT DATA

Due to the loose, saturated, and non-plastic character of the fly ash in FAR-1, this material was considered to be potentially liquefiable. It is noted that the maximum thickness of the fly ash in FAR-1 is 37 m (120 ft).

Figure 1 shows the location of geotechnical borings and CPTs in FAR-1. The CPT and boring data from the downstream half of FAR-1 was used in making the assessments reported herein as the fly ash over this reach is thicker and generally less dense (as evidenced by low CPT tip resistance) than fly ash that is further upstream. Therefore, the results presented herein can be considered to represent potentially worst-case conditions at the site.

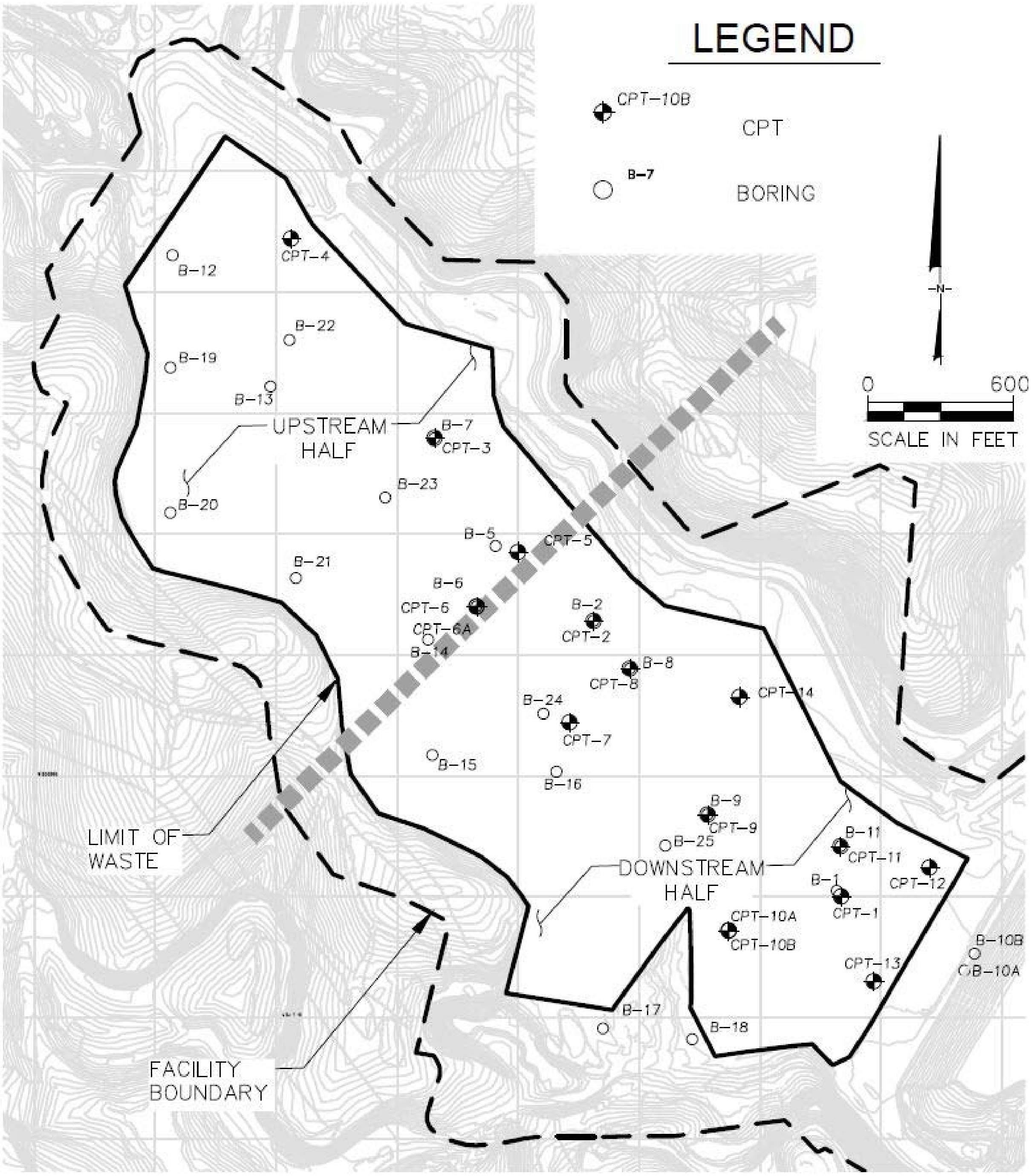


Figure 1. Boring and CPT Locations.

The procedure described in Been and Jefferies (1992)[1] to assess the so-called "state parameter" was used for the fly ash using CPT signature information. The computed state parameter, ψ, provides an indication as to whether the in-place fly ash would tend to contract or dilate during drained shearing. That is, if the computed state parameter is greater than zero, the material may generate positive excess pore pressures during undrained shear and, for the evaluations conducted herein, is considered to have the potential to undergo static liquefaction. If the computed state parameter is less than zero, the material may generate negative excess pore pressures during undrained shear and is considered to have limited potential for static liquefaction.

As proposed by Jeffries and Been (2006)[5], the potential for static liquefaction is considered negligible if ψ ≤ -0.05. Using the CPT results, "potentially liquefiable material" was therefore defined for depth increments over which the majority of the computed value for the state parameter exceeds -0.05. As an example, Figure 2 presents CPT tip resistance and the state parameter with depth for CPT-9. Figure 3 provides a summary of this assessment for the CPTs used and indicates that the thickness of potentially liquefiable fly ash ranges from 9 to 27 m (30 to 90 ft).

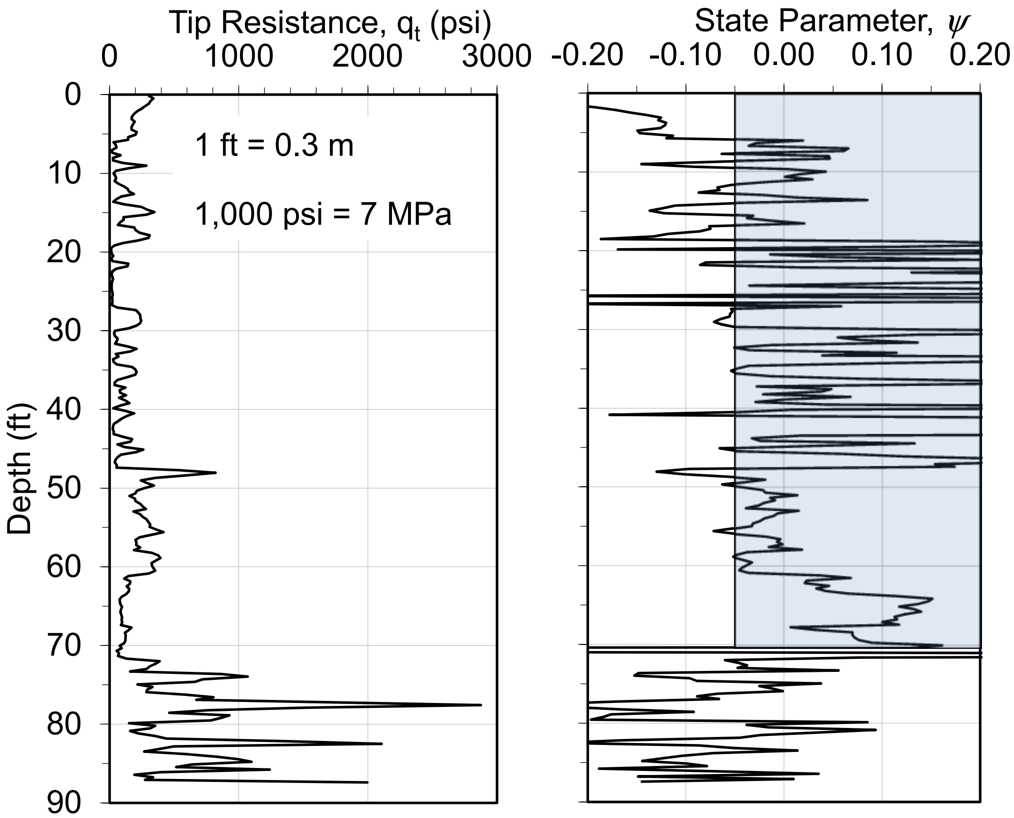


Figure 2. Tip Resistance for CPT-9 and State Parameter with depth (based on Been & Jefferies, 1992)[1]. Shaded area indicates potentially liquefiable fly ash.

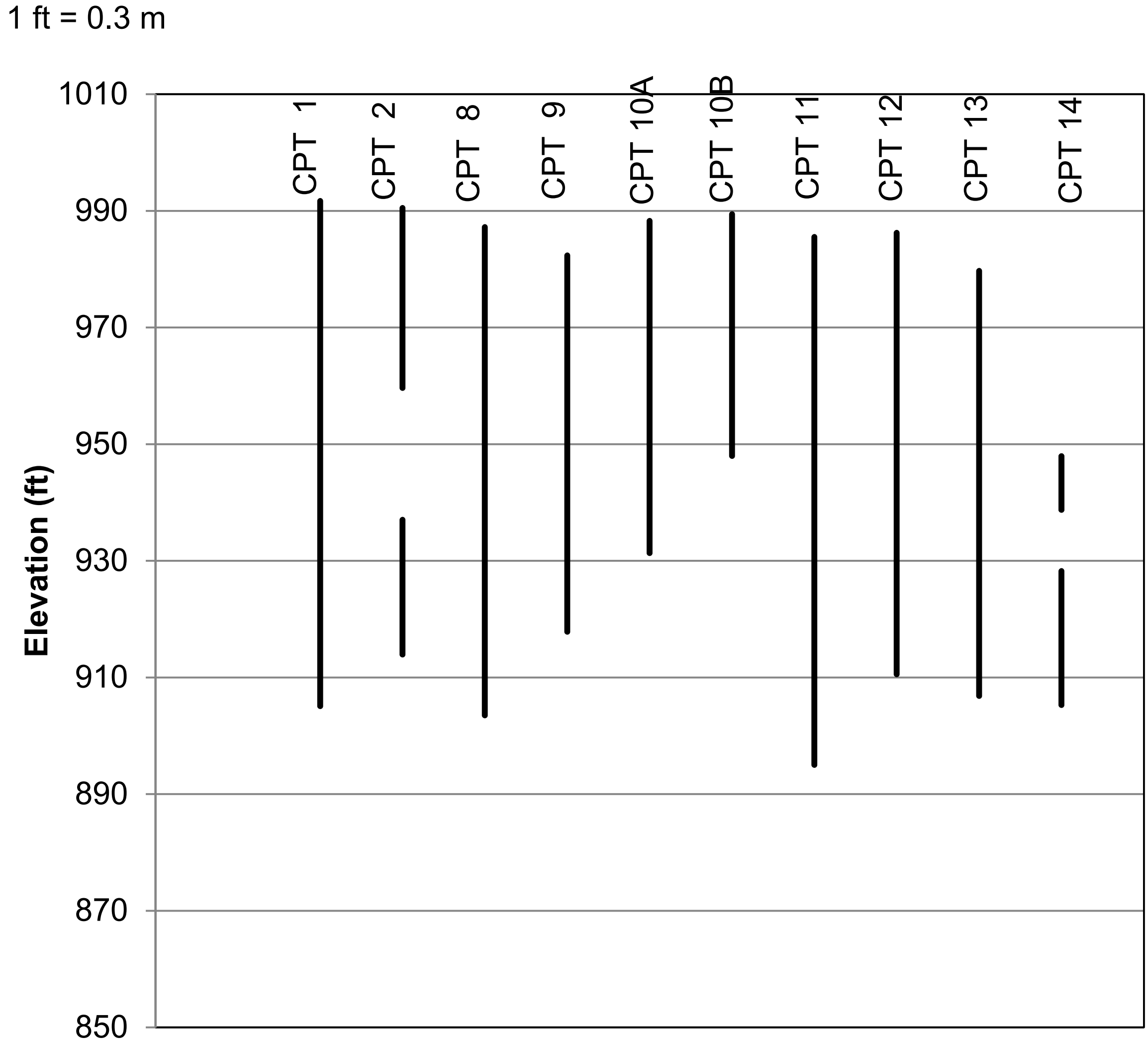


Figure 3. Assessment of Potentially Liquefiable Fly Ash Thickness (based on Been & Jefferies, 1992)[1].

ASSESSMENT OF TIME RATE OF CONSOLIDATION OF FLY ASH FROM LABORATORY AND FIELD TESTING RESULTS

The potential for static liquefaction of the in-place fly ash in FAR-1 should consider the drainage characteristics of the fly ash as compared to typical loading rates. Even if the low-density fly ash is relatively permeable, positive excess pore pressures may develop as a result of construction loads. However, if the rate of loading is relatively slow, the magnitude of such excess pore pressures may be sufficiently small to preclude static liquefaction. Herein, several data sources are reviewed to assess the coefficient of vertical consolidation, c_v, and coefficient of horizontal consolidation, c_h, of the in-place fly ash at FAR-1, as these parameters will be used to assess the relative permeability and excess pore pressures of the in-place fly ash.

The following sources of information were reviewed to assess c_v and c_h of fly ash:

- interpreted c_h values from piezocone dissipation tests in fly ash at FAR-1;
- interpreted c_h values from piezocone dissipation tests in TVA Kingston fly ash; and
- measured c_v values from an Ohio State University laboratory testing program including constant rate of strain consolidation testing on fly ash from FAR-1 (see 2006 OSU Report)[2].

Values for c_h and c_v from the data sources noted above are shown on Figure 4. These results indicate that vertical or horizontal consolidation coefficients for FAR-1 fly ash (based on either CPT or OSU laboratory data) range from 0.1 to more than 10 cm^2/sec (9 to more than 940 ft^2/day). While not specifically evaluated for this paper, we believe that the interpreted vertical or horizontal consolidation coefficients are the result of layering in FAR-1 (e.g., possible zones of higher and lower permeability fly ash).

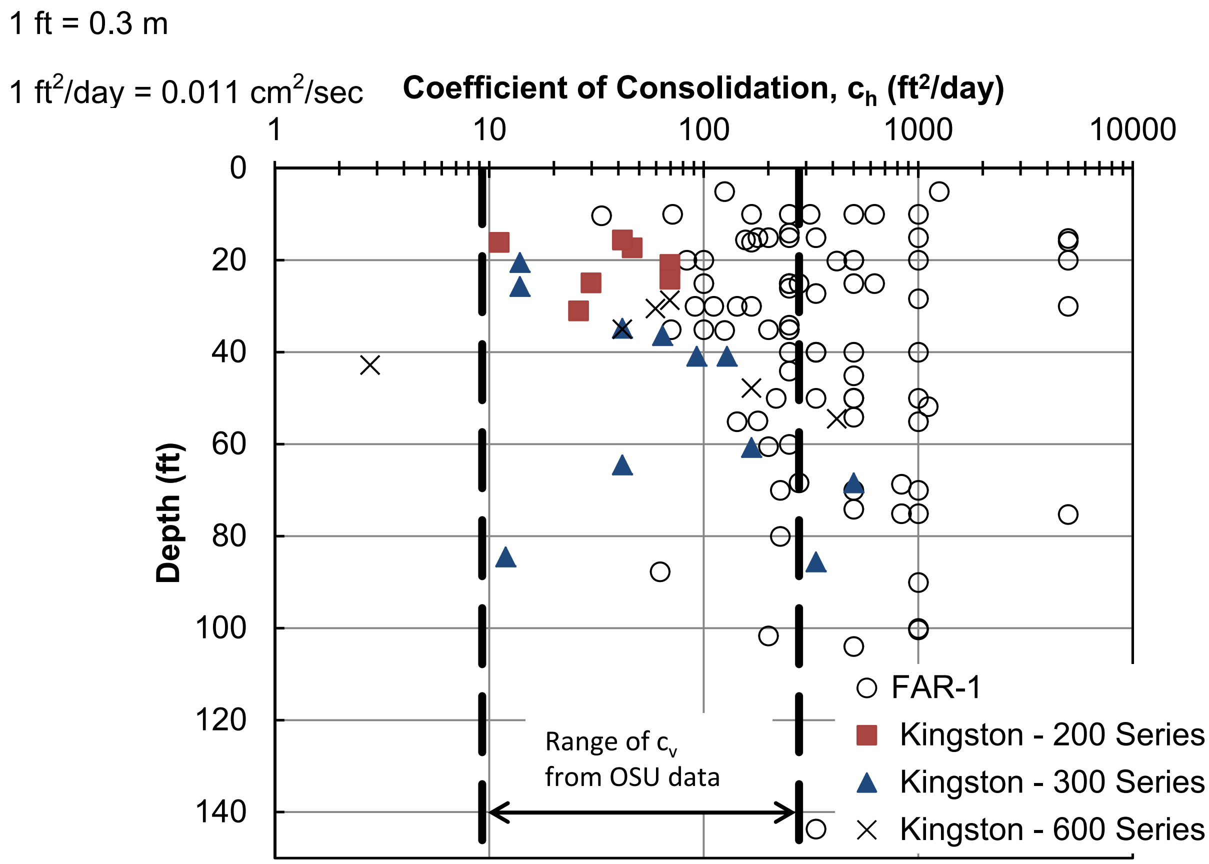


Figure 4. Coefficient of Consolidation Values from Field and Laboratory Testing Program.

ASSESSMENT OF TIME RATE OF CONSOLIDATION OF FLY ASH BASED ON RESULTS OF MONITORED PRELOAD FILL

Between March and August 2007, a 9-m (30-ft) high MPF was constructed and monitored. The MPF and the underlying fly ash were instrumented using settlement measuring devices and piezometers. The location of the MPF is shown on Figure 5. A detailed description of the MPF is provided in Haydar et al. (2008)[3].

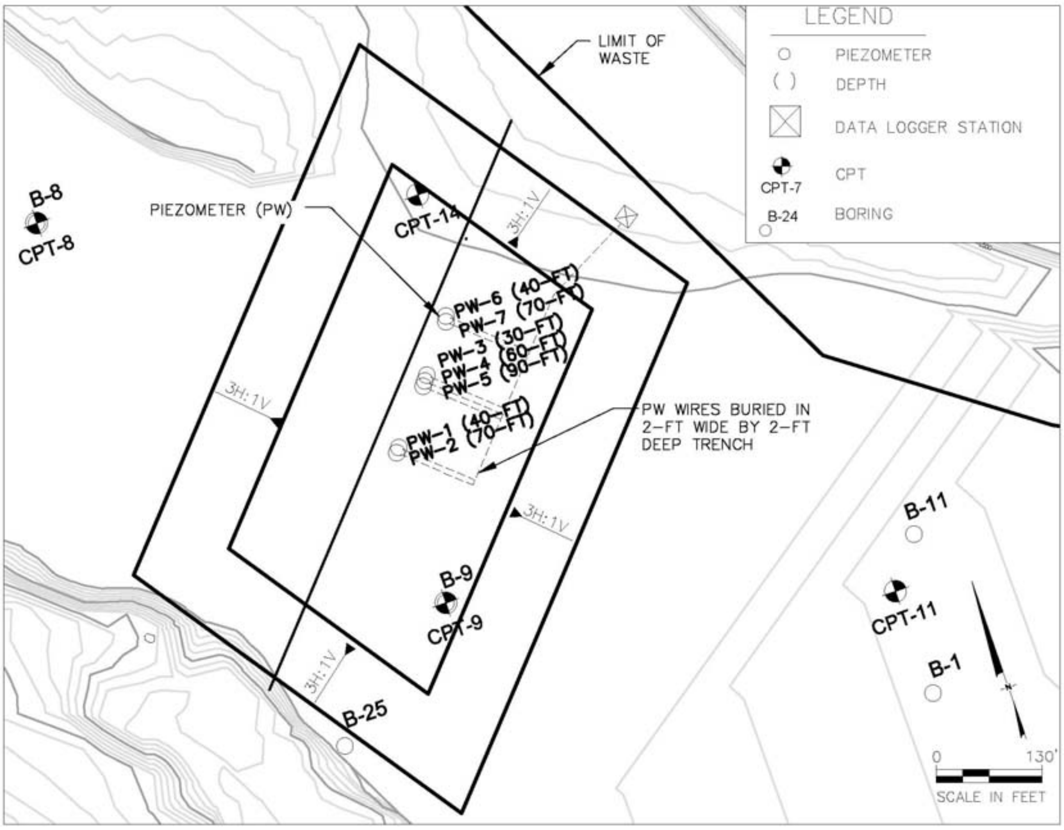


Figure 5. Location of MPF in the Downstream Half of FAR-1.

Figure 6 provides the time history of preload filling for the MPF. Using this information, time rate of consolidation analyses were conducted using the computer program “SAF-TR” (SAF-TR, 1993)[6] to back-calculate a “best-fit” c_v value for the entire thickness of fly ash. For these analyses, it was assumed that the bottom of the valley is “impervious” and that one-way drainage occurs vertically at the top of the profile. Information from borings and CPTs indicates that the valley floor soils (below the fly ash) are clayey and would therefore be significantly less permeable than the in-place fly ash. Figure 7 shows the results of the analysis for the assumption of c_v = 3.2 cm^2/sec (300 ft^2/day) for filling up to approximately 6 m (21 ft), corresponding to the first four load steps for the MPF.

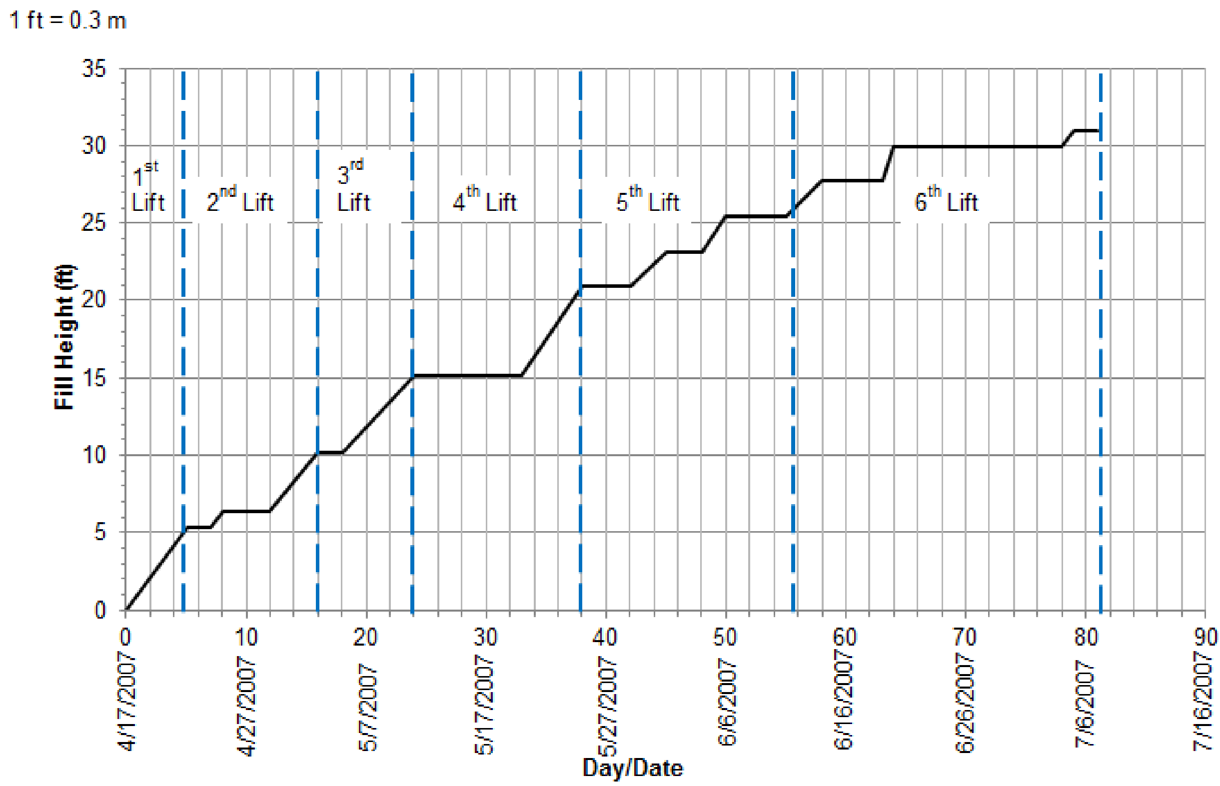


Figure 6. Filling Time History of MPF.

Information in Figure 7 indicates that computed excess pore pressures are greater than measured pore pressures at depths of 18 m (60 ft) and 27 m (90 ft) whereas computed and measured excess pore pressures at a depth of 9 m (30 ft) are reasonably similar. This comparison is discussed further below.

Figures 8 through 10 compares excess pore pressures at various time intervals at depths of 9 m (30 ft), 18 m (60 ft), and 27 m (90 ft) as measured in PW-3, PW-4, and PW-5, respectively, with the computed excess pore pressures at these depths. These piezometers were located along the approximate centerline of the MPF (see Figure 5) and at this location the response is approximately one-dimensional. For all depths, computed excess pore pressures are greater than the corresponding values from the pore pressure measurements for the MPF, with the difference between measured and computed increasing with depth. Reasons for this may include: (1) actual drainage boundary conditions may be different (i.e., more complex) than the assumed one-way vertical drainage; and (2) actual c_v values may vary with depth such that the use of a single value of c_v for the entire fly ash profile may not be sufficiently accurate. It is noted, however, that for purposes of predictions of excess pore pressures resulting from one-dimensional loading, the use of c_v = 3.2 cm^2/sec (300 ft^2/day) is reasonable and conservative since this value results in computed excess pore pressures greater than actual pore pressures in the fly ash.

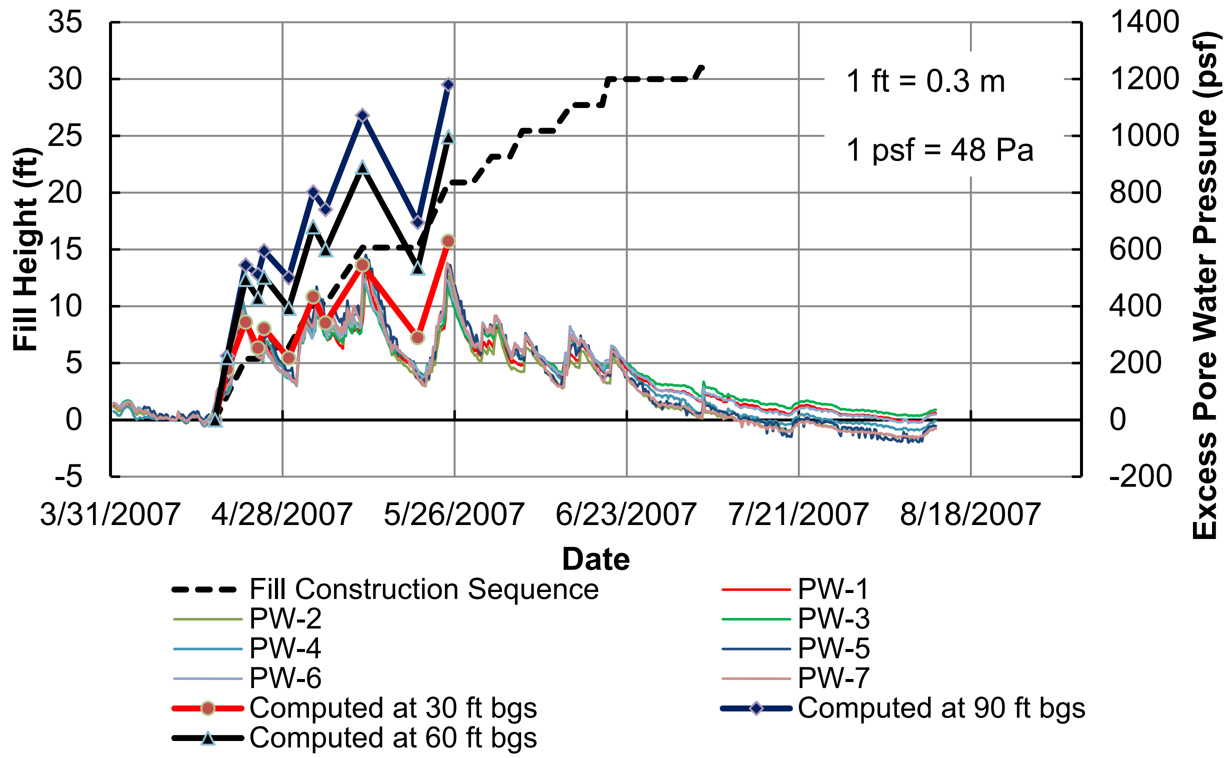
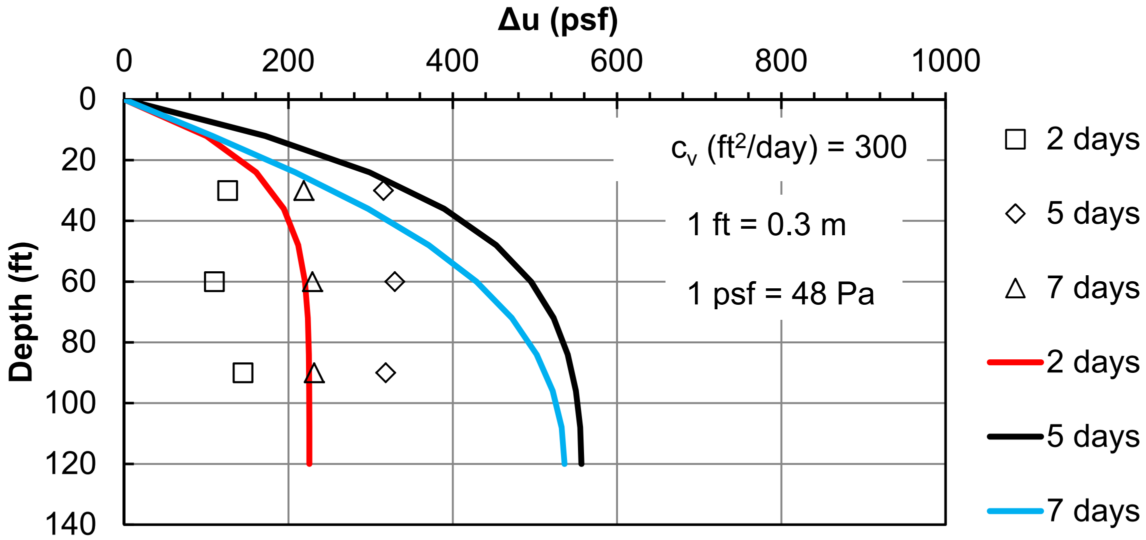
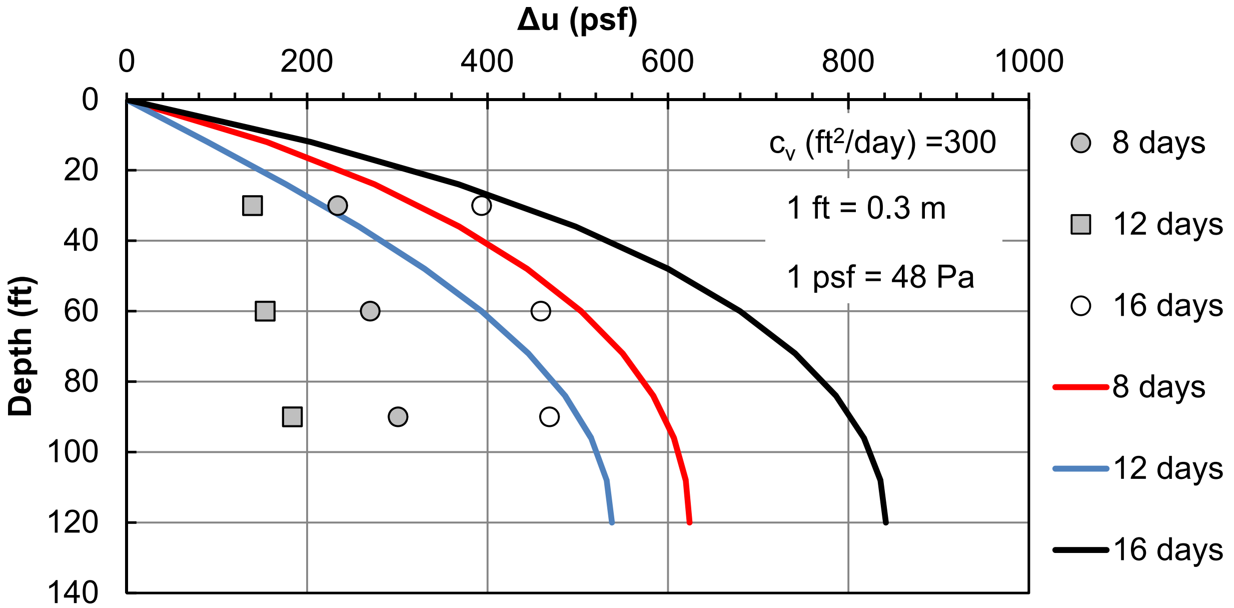


Figure 7. Comparison of Measured and Computed Excess Pore Pressures.



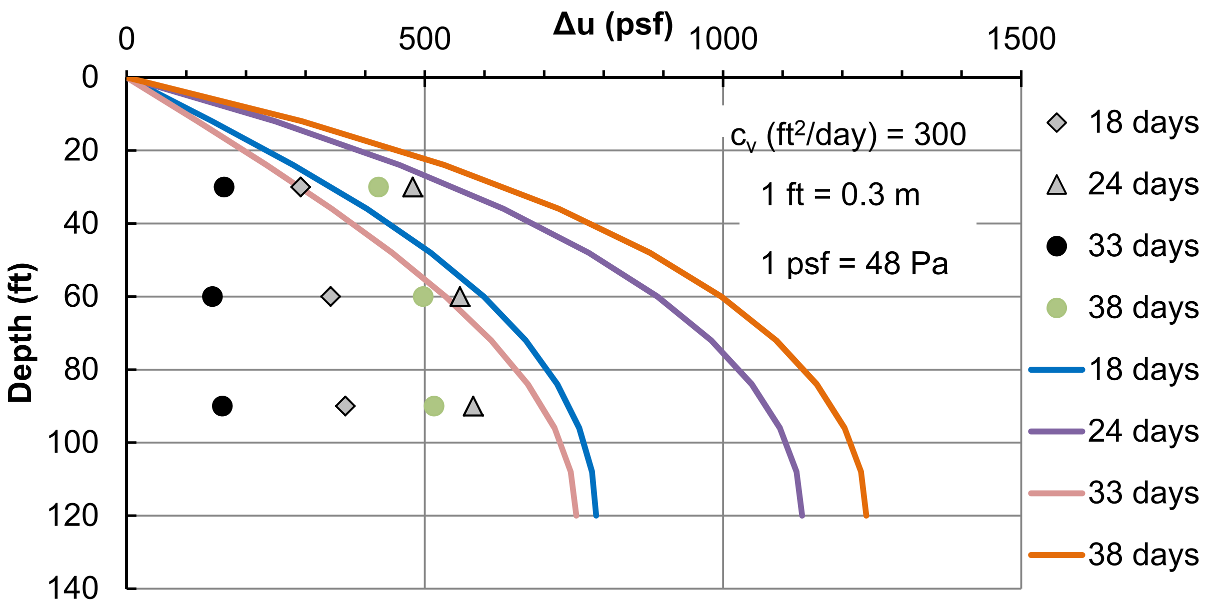
Note: Measured excess pore pressure data at 30 ft bgs is from PW-3, at 60 ft bgs is from PW-4, and at 90 ft bgs is from PW-5.

Figure 8. Comparison of Measured and Computed Excess Pore Pressures (Days 2, 5 and 7).



Note: Measured excess pore pressure data at 30 ft bgs is from PW-3, at 60 ft bgs is from PW-4, and at 90 ft bgs is from PW-5.

Figure 9. Comparison of Measured and Computed Excess Pore Pressures (Days 8, 12 and 16).



Note: Measured excess pore pressure data at 30 bgs is from PW-3, at 60 ft bgs is from PW-4, and at 90 ft bgs is from PW-5.

Figure 10. Comparison of Measured and Computed Excess Pore Pressures (Days 18, 24, 33 and 38).

Referring to Figure 4, it is noted that the best-fit c_v value of 3.2 cm^2/sec (300 ft^2/day) is, on average, greater than values obtained from the OSU lab testing on FAR-1 fly ash and the values for c_h for the Kingston fly ash near the failed dikes (i.e., the 200 series tests). This value, however, is within the computed range for c_h based on the piezocone dissipation tests conducted in the FAR-1 fly ash.

ASSESSMENT OF EXCESS PORE PRESSURES RESULTING FROM FUTURE PRELOAD CONSTRUCTION

Soil filling for the MPF was conducted over a 79-day time interval between 17 April 2007 and 5 July 2007. As the MPF settled between 0.5 m (1.5 ft) and 0.9 m (2.8 ft) during construction, the average filling rate is therefore considered to be approximately 10 m (33 ft) of fill over 79 days or 0.13 m/day (0.43 ft/day). It is noted that as part of actual construction for the proposed FGD waste landfill to be built over FAR-1, individual approximately 9-m (30-ft) high preload fills will be constructed (and then removed) before liner system construction to help control settlement of the liner system under service loading conditions. Actual preload fill heights (within each landfill cell) will be established in the field to achieve a target ground pressure (at the base of the preload fills) of approximately 182 kPa (3,800 psf). Therefore, the MPF program provides an excellent full-scale representation of the actual conditions that are expected during construction.

Time rate of consolidation analyses were conducted to evaluate pore pressures in the in-place fly ash resulting from future preload fills. It was assumed that a 9-m (30-ft) high preload would be placed at a rate of 0.15 m/day (0.5 ft/day), i.e., slightly faster than the average filling rate for the MPF resulting in a total construction duration of 60 days. It was also assumed that loading of the 9-m (30-ft) thick preload would occur as a continuous “ramp” loading (without specific quiet periods of no loading) and that the thickness of in-place fly ash beneath the MPF ranges from 30 to 37 m (100 to 120 ft).

Figure 11 depicts the computed excess pore pressure at Day 60 for multiple depths within the fly ash. It is noted that computed excess pore pressures at Day 60 are the maximum pore pressures for the analysis. Information provided on Figure 11 indicates a computed pore pressure at a depth of 9 m (30 ft) (at Day 60) of 26 kPa (534 psf) and 30 kPa (635 psf) for 30-m (100-ft) thick fly ash and 37-m (120-ft) thick fly ash, respectively. At this 9 m (30 ft) depth, for example, these computed pore pressures are 11 to 32 percent greater than that measured at this depth (i.e., PW-3) as part of the MPF.

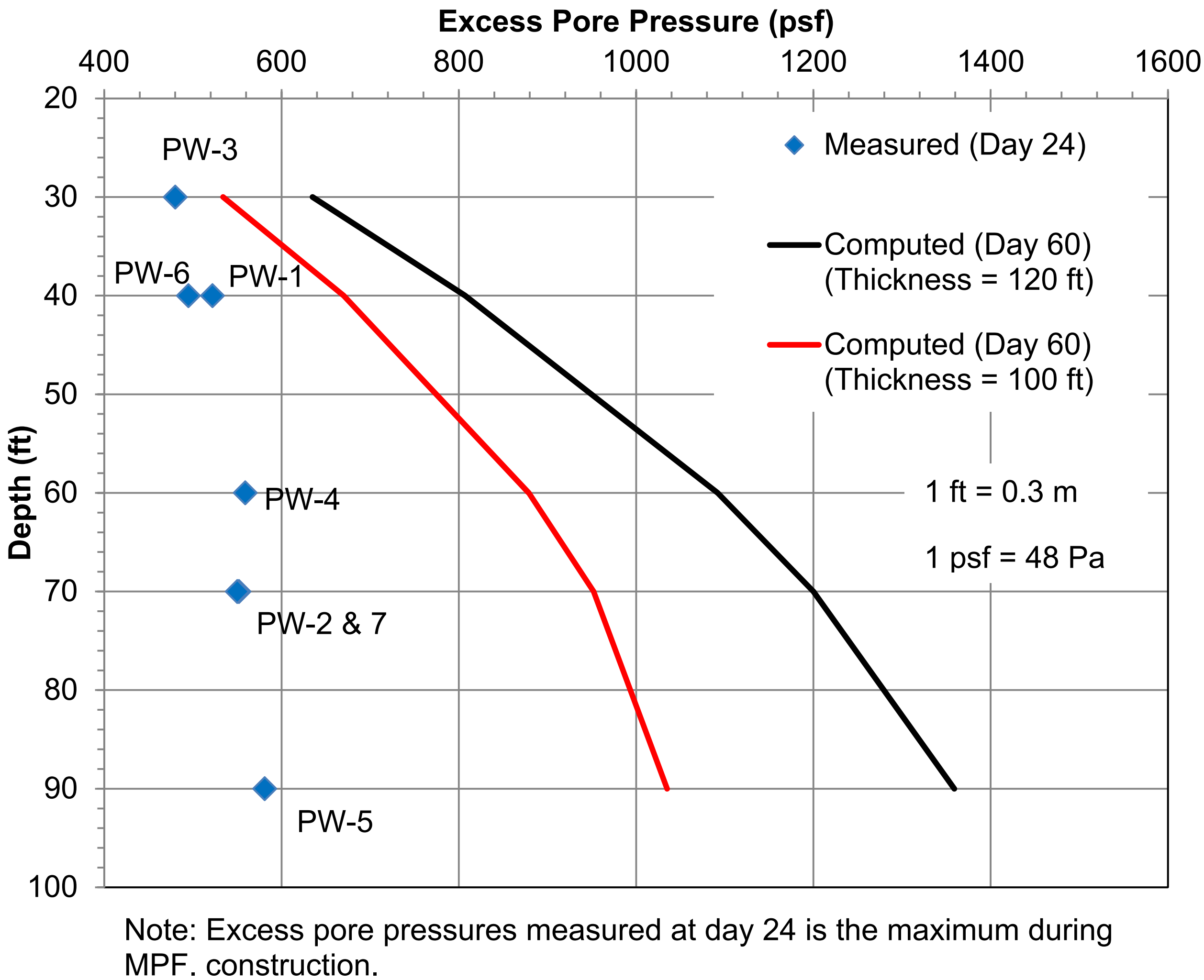


Figure 11. Computed Excess Pore Pressures for the Assumed Preload Fill Construction.

Results of this analysis indicated that the average degree of consolidation (for the 30-m (100-ft) thick in-place fly ash) is 82 percent at Day 60. This information is used in the next section to compute excess pore pressures in the in-place fly ash.

APPROACH TO ESTIMATE SHEAR-INDUCED EXCESS PORE PRESSURES

Herein, an analysis approach was developed to estimate excess pore pressures in the in-place fly ash at Day 60 (i.e., when the preload reaches its maximum height). As previously discussed, measured pore pressures from the MPF were primarily attributable to one-dimensional loading as the piezometers were located under the approximate center of the MPF; no piezometers were located near the edges of the

MPF. To address static liquefaction, it is necessary to develop a suite of estimated pore pressures that also considers the two-dimensional loading that will occur primarily towards the toe of the slope (i.e., shear-induced pore pressures).

The approach used to estimate excess pore pressures in the in-place fly ash is based on Henkel's approach whereby excess pore pressures (from undrained loading) are computed based on changes in total stresses (see Holtz and Kovacs, 1981[4]). To provide for a realistic assessment of actual anticipated conditions, it is noted that the in-place fly ash consolidates relatively rapidly (based on measured pore pressures from the MPF). The computed excess pore pressures (based on an undrained loading assumption) are, therefore, reduced to account for the estimated amount of consolidation that would occur by Day 60.

The following step-by-step approach was used to estimate excess pore pressures in the in-place fly ash:

- For an assumed 30 m (100 ft) thickness of in-place fly ash, compute the change in vertical stress ($\Delta\sigma_{zz}$), horizontal stress ($\Delta\sigma_{xx}$), and shear stress ($\Delta\sigma_{xz}$) resulting from the placement of a 30-ft high preload (with 3H:1V sideslopes) using the stress computation program "STRESS" (STRESS, 1993)[7]. For this analysis, the average ground pressure at the base of the preload (under the center line) is 182 kPa (3,800 psf). Compute the stresses at various locations in the subsurface.
- Compute the change in principal stresses (i.e., $\Delta\sigma_1$, $\Delta\sigma_2$, $\Delta\sigma_3$) at various locations according to:

$$\Delta\sigma_1 = \frac{1}{2}(\Delta\sigma_{xx} + \Delta\sigma_{zz}) + \sqrt{\left\{\frac{1}{2}(\Delta\sigma_{zz} - \Delta\sigma_{xx})\right\}^2 + \Delta\sigma_{xz}{}^2}$$

$$\Delta\sigma_3 = \frac{1}{2}(\Delta\sigma_{xx} + \Delta\sigma_{zz}) - \sqrt{\left\{\frac{1}{2}(\Delta\sigma_{zz} - \Delta\sigma_{xx})\right\}^2 + \Delta\sigma_{xz}{}^2}$$

$$\Delta\sigma_2 = \nu\ (\Delta\sigma_1 + \Delta\sigma_3)$$

  where v is the elastic Poisson's ratio assumed to be 0.5 for undrained loading.

- Compute the excess pore pressures (due to undrained loading) according to Henkel's equation

$$\Delta u_{(undrained\ )} = \frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) + \alpha\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2}$$

  where α is the pore pressure parameter (discussed subsequently).

- The SAF-TR analysis for the 9 m (30 ft) preload (see results in Figure 11) provides an estimate of the average degree of consolidation of the in-place fly for a given time. As previously noted, the average degree of consolidation at 60 days is 82 percent. Based on this, the corresponding consolidation time factor, T, can be computed as T = 1.781 – 0.933 log (100 – U_{avg}) (Holtz and Kovacs, 1981)[4], or T = 0.6.

- Compute the degree of consolidation (at t = 60 days) at various depths, U_z, in the in-place fly ash profile with T = 0.6.

- Excess pore pressures within the fly ash (at 60 days) can then be computed as:

$$\Delta u_{(60\ days)} = (1 - U_z) \times \Delta u_{(undrained)}$$

Henkel's parameter α can be related to Skempton's "A" parameter (for triaxial compression) as α = (3A-1)/√2. For triaxial compression, the A parameter is defined as the ratio of the excess pore pressure at failure divided by the shear stress at failure or

$$A = \frac{\Delta u_f}{(\sigma_1 - \sigma_3)_f}$$

Laboratory triaxial test data of fly ash specimens from FAR-1 indicate negative excess pore pressures at failure (implying a negative A parameter). For purposes of this analysis, however, a relatively high value for A of 1.0 was selected to evaluate the effects of high shear-induced pore pressures (which would be expected to be measured for a material undergoing static liquefaction). The corresponding value for α is, therefore, 1.4. As previously discussed, the MPF and fly ash foundation did not undergo static liquefaction, so this calculation approach is believed to be conservative.

SLOPE STABILITY ANALYSIS CONSIDERING EXCESS PORE PRESSURES DUE TO FUTURE PRELOAD CONSTRUCTION

Computed excess pore pressures, Δu(60 days), are added to the initial hydrostatic pore pressures (based on a water table at the top of the in-place fly ash) and these total pore pressures are input as a pore pressure grid in a slope stability analysis. Figure 12 shows a contour plot of the estimated excess pore pressures used for the analysis.

For the analysis, the in-place fly ash and a 0.6-m (2-ft) thick subsurface drainage layer were modeled considering a total unit weight of 16 kN/m^3 (100 pcf) and the preload fill was modeled considering a total unit weight of 20 kN/m^3 (127 pcf). These three materials were modeled assuming a shear strength represented by a drained friction angle of 30 degrees and a zero cohesion intercept.

Figure 13 presents the results of the stability analysis for the 9 m (30 ft) preload. The pore pressure grid used for the analysis is the sum of the hydrostatic pore pressure and the excess pore pressures (from the procedure described above). Even in consideration of relatively high shear-induced pore pressures (i.e., use of α = 1.4), the computed factor of safety (FS = 1.7) is relatively high.

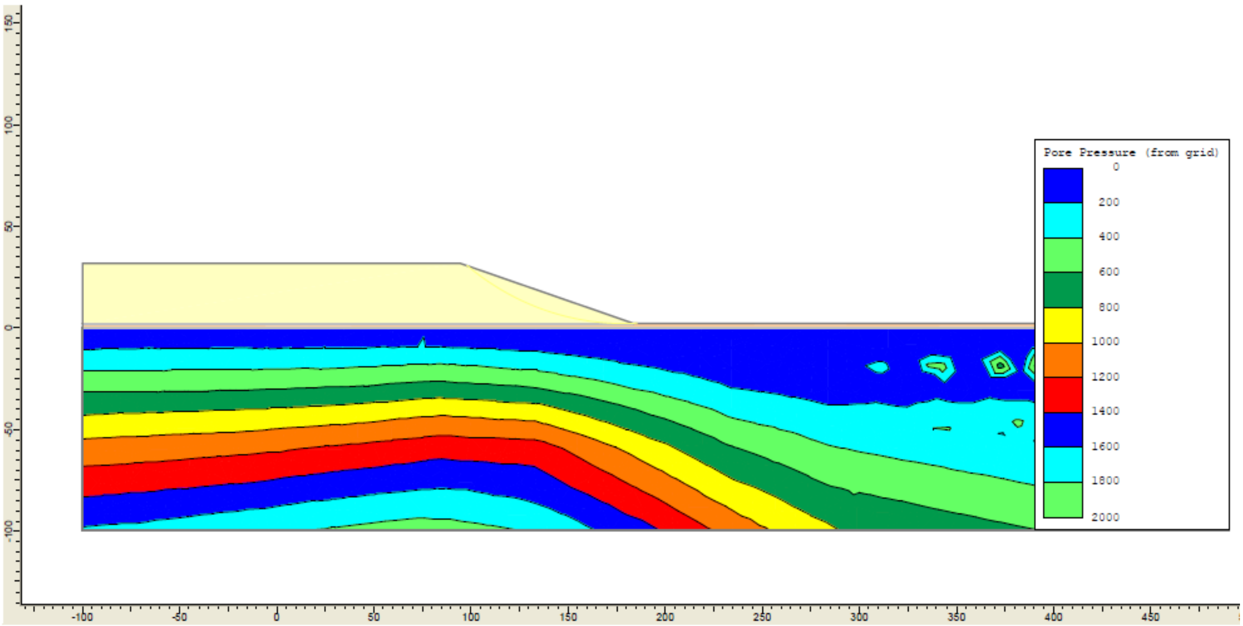
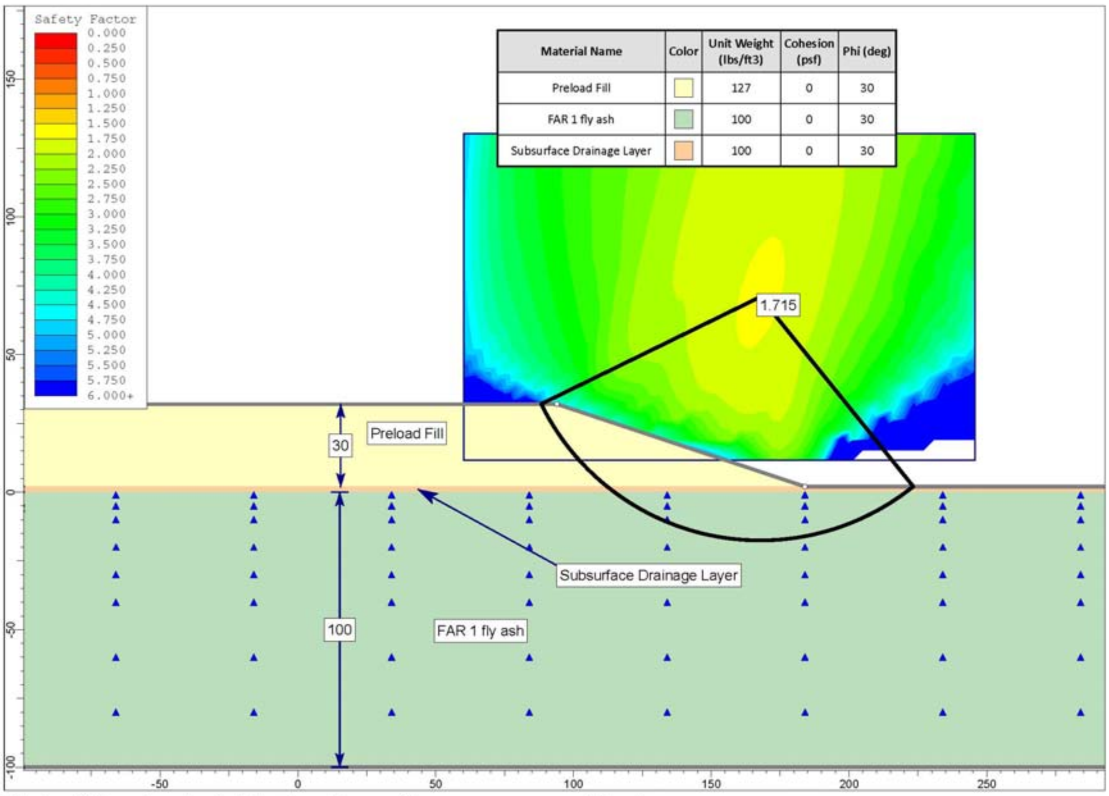


Figure 12. Computed Excess Pore Pressure Distribution in the Fly Ash Foundation.



Note: Triangles depict the locations of pore pressure grid values.

Figure 13. Slope Stability Analysis Result for Day 60.

EVALUATION OF STATIC LIQUEFACTION RESULTING FROM FGD WASTE FILLING

As part of this study, the client provided information on anticipated future FGD waste filling rates at the site. Information provided includes: (1) typical maximum open cell size is 4 hectares (10 acres); and (2) average waste filling rate is expected to be on the order of 30,000 tons/month. Assuming that the unit weight of the residual waste is 16 kN/m^3 (100 pcf), this indicates that 17,000 m^3 (600,000 ft^3) of waste would be placed each month. For a 4 hectares (10 acres) open cell, this corresponds to a filling rate of 0.42 m/month (1.38 ft/month) or 0.02 m/day (0.05 ft/day).

FGD waste filling rates are therefore significantly slower than anticipated preload filling rates. It can be concluded that factor of safety values for analyses which consider excess pore pressure development resulting from FGD waste filling would be higher than factor of safety values reported above for the preload filling and therefore the potential for static liquefaction during waste filling is considered to be negligible.

CONCLUSIONS

The primary goal of this study was to assess the potential for static liquefaction of the fly ash in FAR-1. The analysis results presented herein and the physical evidence obtained from the MPF demonstrate that the potential for static liquefaction of the FAR-1 fly ash is highly unlikely. Principal findings that support this conclusion include the relatively high permeability of the in-place ash and the expected relatively slow rate of loading. These two findings indicate that there is negligible potential for undrained loading of the ash that would lead to the large excess pore pressures necessary to trigger static liquefaction.

RECOMMENDATIONS

The results of the analyses provided herein indicate that the development of excess pore pressures in the FAR-1 fly ash leading to static liquefaction is unlikely. Specific engineering measures, however, will be also be implemented as part of the FGD waste landfill design which, while not developed to mitigate the potential for static liquefaction, provide an added measure of redundancy. A description of these measures is provided below.

- The permitted design for the FGD waste landfill includes a continuous 0.6-m (2-ft) thick (minimum) subsurface drainage layer with a minimum laboratory-measured hydraulic conductivity of 1×10^{-3} cm/s. To enhance drainage, this design also includes perforated HDPE collector pipes placed at maximum 61-m (200-ft) spacing that drain collected water from the drainage layer.
- It is recommended that piezometers be installed for each landfill cell over FAR-1 to monitor pore pressures in the in-place fly ash resulting from preload filling for each cell. Piezometer readings will be taken to evaluate the rate at which excess pore pressures in the in-place fly ash reduce with time. If measured pore

pressures and rates of pore pressure reduction are deemed unacceptable, filling rates will be reduced (as needed).

- The results provided herein for the MPF indicate that average filling rates on the order of 0.15 m/day (0.5 ft/day) (or less) would be sufficiently slow to minimize the potential for instability of the preload fills and the underlying in-place fly ash foundation. Pore pressure measurements (as described above) can be used to modify this fill placement rate (if needed). It is noted, however, that inevitable localized failures resulting from initial lift construction will likely still occur.

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