South Portland Approach Preload Design, Instrumentation and Construction; Veteran's Memorial Bridge Design-Build Portland-South Portland, Maine

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ABSTRACT: The South Portland Approach for the Veteran's Memorial Bridge Replacement faced difficult constraints, including waterfront construction over the mudflats, permit-based limits on the allowable disturbance area, and a subsurface profile including up to 90 feet of moderate strength, compressible Presumpscot Marine Clay. The principal concerns for construction in this environment were stability and long-term settlement of the embankment. The design solution included a 25-foot high earthfill, preload embankment supported by a temporary, anchored, sheet pile cofferdam. The cofferdam provided lateral stability and limited the environmental impact on the mudflats, allowing construction of the permanent, pile-supported abutment and retaining wall. Geotechnical engineering evaluations included short-term and long-term global stability, long-term settlement, settlement mitigation using a preload and wick drains, and downdrag and lateral squeeze on the proposed pile-supported retaining wall. The team designed, installed and monitored geotechnical instrumentation to assess the performance of the preload, including settlement plates, piezometers, and inclinometers. Based on the measured performance, GZA recommended removal of the preload approximately five months ahead of schedule.

1 PROJECT BACKGROUND

1.1 *Maine DOT Design-Build RFP*

Replacement of the Veteran's Memorial Bridge (VMB) between Portland and South Portland, Maine was contracted by Maine Department of Transportation (MaineDOT) using the design-build procurement method. The process involved a Request for Proposal (RFP) phase where several teams develop competing technical proposals, and then final design and construction is awarded to the team selected on a best-value basis. GZA served as geotechnical engineer for the Reed & Reed / T.Y. Lin International design-build team. The team submitted the best-value proposal and was awarded the project.

The proposed bridge would replace a deteriorated, 2000-foot long circa-1950 steel bridge with a trapezoidal-box, segmental precast concrete structure that would span the Fore River. The proposal included an Alternative Technical Concept where the starting and ending points of the bridge were modified to improve traffic flow, as shown in **Figure 1**. In order to accommodate the new alignment, a triangular embankment widening was required over existing mud flats in South Portland. The South Portland embankment design and construction are the subjects of this paper.

1.2 *Historical Site Development*

Modern development of the site consisted of filling associated with transportation infrastructure. In the late 1800s railroad embankments and bridges were constructed on or near the current bridge alignment. In the early 1950s, the first VMB was constructed alongside a replacement of the railroad bridge. The new highway bridge and associated approach embankments were constructed on what was then

Figure 1 - Proposed Veteran's Memorial Bridge Alignment

a shallow mud flat. That construction involved the use of sand drains to promote consolidation of the underlying marine deposits.

2 SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

2.1 *Exploration and Lab Testing Programs*

A subsurface exploration program was completed in 1948 by the Maine State Highway Commission for design of the first VMB. That program included cased borings with split spoon, thin-wall tube sampling and bedrock coring, taken near the South Portland abutment and approach.

Those borings show that prior to construction of the VMB, the area was a shallow mud flat. Consolidation Testing completed on marine silty clay samples from those borings gave an unusual glimpse into the stress history of the deposits prior to development of the first bridge. Those pre-development data provided key information used by GZA to evaluate settlement mitigation design for the new VMB, particularly with respect to quasi-preconsolidation of the marine clay.

Three more recent subsurface exploration programs were undertaken to support the VMB design-build project. Results of explorations by MaineDOT's consultant completed prior to and concurrent with the RFP were provided in the March 27, 2009 Preliminary Geotechnical Data Report, and the July 31, 2009 Supplemental Geotechnical Data Report. GZA completed additional explorations for final design and instrumentation and submitted the results in various engineering reports prepared for the project.

The boring programs consisted of cased borings with split spoon sampling, field vane shear testing, thin wall tube sampling, rock coring and groundwater observation wells. Laboratory testing included index tests (Unit Weight, Atterberg Limits, Moisture Content, Sieve Analysis, and Organic Content), strength tests (CIUC Triaxial Tests on cohesive soils and Unconfined Compressive Strength and Modulus Tests of bedrock); and consolidation testing (Incremental and Constant Rate of Strain methods).

3 SUBSURFACE CONDITIONS

Soil units from the surface downward included: Fill, Outwash, Marine Sand, Marine Clay, and Glacial Till, overlying bedrock. Fill, primarily sand with lesser amounts of gravel and silt was encountered at the surface at the new South Portland abutments, as shown in **Figure 2**.

Marine Clay was found beneath the entire South Portland Abutment area. The marine clay was medium stiff to stiff with frequent seams and layers of Marine Sand. The marine sand was

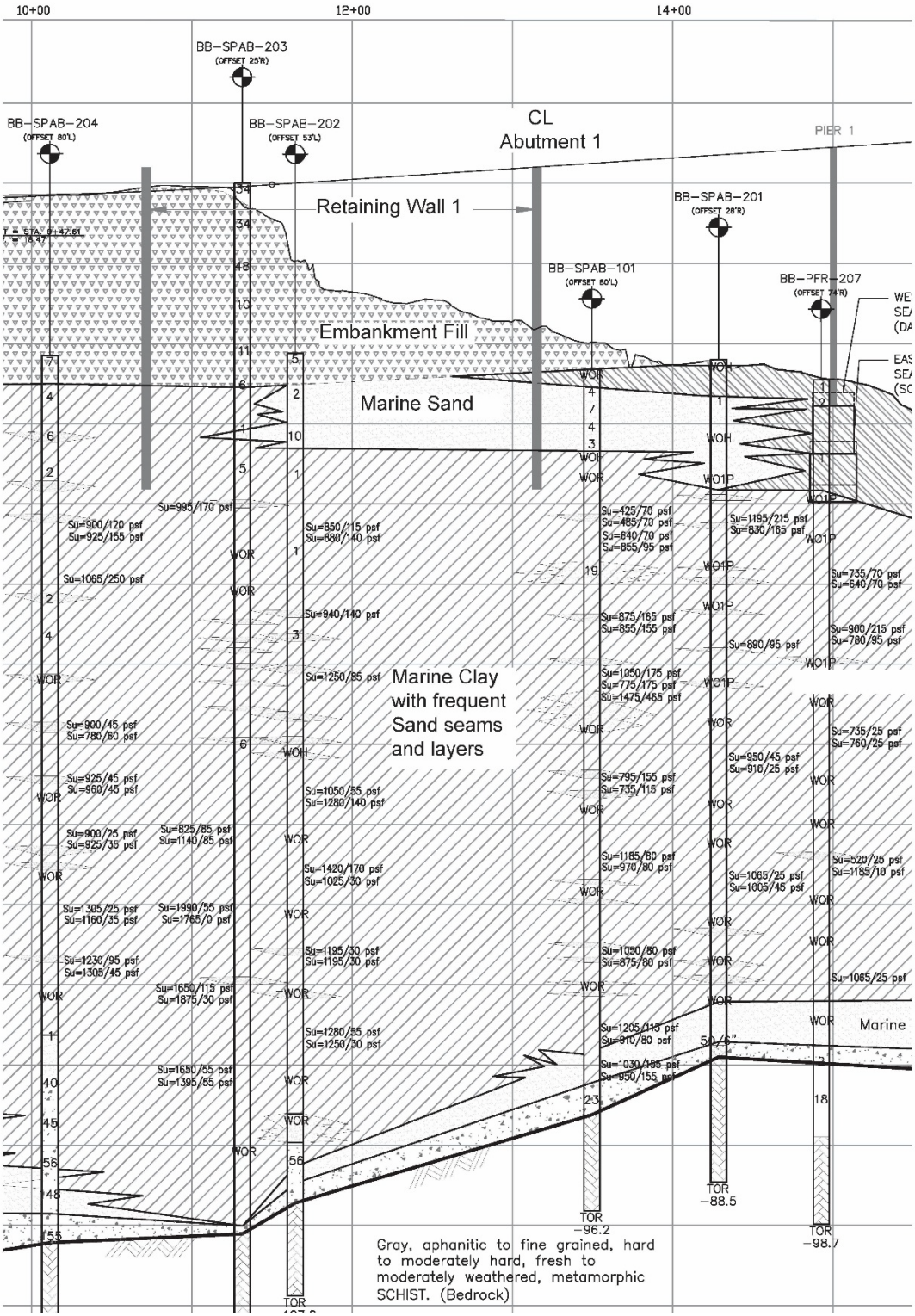


Figure 2 – Subsurface Profile at South Approach

present as discreet lenses and layers and in a thicker layer beneath the Marine Clay. Immediately above the bedrock was a discontinuous layer of very dense sand, silt and gravel, Glacial Till. Bedrock consisted of gray, fine-grained, hard, fresh to moderately weathered Schist or Phyllite.

3.1 *Impact of Subsurface Conditions on Design*

The significant depth and lateral extent of the Marine Clay deposits at this site influenced the substructure and embankment designs throughout most of the alignment and particularly at the South Portland abutment. The compressibility and moderate strength of the clay make it unsuitable as a foundation bearing material and also make embankment stability a critical element of the design. Design features driven by the Marine Clay include:

- Deep foundations were required to penetrate the clay to suitable Glacial Till or Bedrock bearing materials.
- Conventional earth fills at the South Portland approach would suffer significant settlement due to consolidation of the clay, therefore settlement mitigation measures were needed. Settlement mitigation efforts were designed to provide the additional benefit of preventing downdrag loads from developing on deep foundations for the new abutment and retaining wall.

4 EMBANKMENT DESIGN SOLUTIONS

4.1 *Embankment and Preload Considerations*

The circa-1950 VMB South Portland approach embankment consisted of 25 to 40 feet of sandy fill and marine sand, overlying 60 to 90 feet of Marine Clay with fine sand seams, lenses and layers. The clay overlies higher permeability marine sand or glacial till, and bedrock. Given that the clay is moderately compressible, construction of new embankments could cause undesirable settlement of the approach roadway, and the settlement could induce downdrag on deep foundations.

The circa-1950 VMB South Portland approach embankment design included settlement mitigation measures including a grid of 18-inch sand drains installed beneath the fill with typical spacing of 10 to 20 feet on center. The replacement VMB alignment included widening of the existing embankment with a triangular fill up to 22 feet high, and 70 feet wide near the proposed abutment, as shown in **Figure 3**. Without mitigation measures, the new fill would induce significant settlement and large downdrag loads.

4.2 *Settlement Mitigation*

Settlement mitigation measures for the replacement bridge embankment included prefabricated vertical drains installed on a regular grid to accelerate consolidation of the clay. The drains extended beneath the shoulder of the circa-1950 embankment, adjacent to the original sand drain system.

A temporary surcharge fill was constructed and left in place. A system of piezometers and settlement platforms was used to monitor pore pressure dissipation in the clay, and the magnitude of settlement. The surcharge was removed once the embankment settlement under preload was equal to the estimated magnitude of primary consolidation plus secondary compression for the 100-year design life of the replacement bridge.

The design subsurface profile for compressibility was developed based on the subsurface data summarized above (Fig. 2). The design stress history profile is presented on **Figure 4**. Key assumptions related to the Marine Clay profile and properties are summarized below in the following paragraphs.

Soil between El. -2 and El. -20 consists primarily of sand; therefore, it is anticipated that measurable consolidation settlement will not occur.

GZA's interpretation of the data indicated that frequent horizontal layers of sand within the Marine Clay would reduce the vertical drainage distance during consolidation. It was judged that the Marine Clay layers between El. -20 and El. -50 are doubly drained at typical 10-foot thickness intervals (5-foot drainage path). Between El. -50 and El. -94 the interval between sand layers in the borings was on the order of 44 feet, resulting in a 22-foot drainage path.

Secondary compression of the 1950 embankment resulted in quasi-pre–consolidation of deeper portions of Marine Clay (in areas that

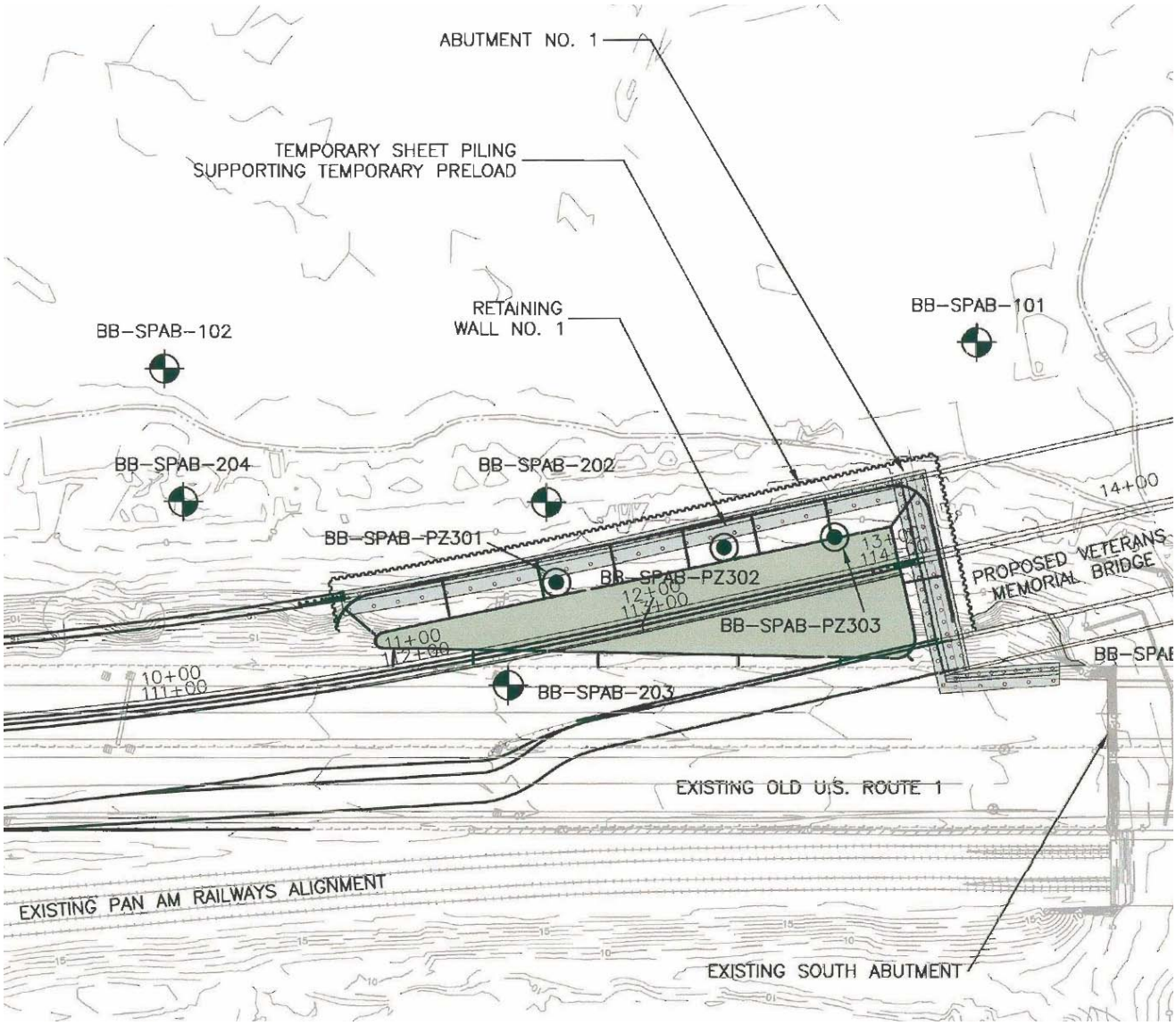


Figure 3 – Preload Instrumentation Area

experienced virgin compression). GZA judged that these deeper clay layers were slightly over consolidated, which reduced the estimated settlement.

Rates of consolidation and secondary compression were selected for the appropriate stress ratio (i.e., ratio of the effective stress over the maximum past pressure). Values relevant to the stress ratio for each sub–layer and loading increment were used for time-rate consolidation and secondary compression evaluations.

Settlement mitigation measures were evaluated by modeling the proposed temporary earth fill preload to estimate the post-construction settlement. The location considered to control the evaluation was at the back (south) edge of the pile cap supporting Retaining Wall 1, at the widest portion of the proposed approach embankment (approximately Sta. 13+05, 22' Lt.). This location represented the largest anticipated stress increase (and corresponding maximum consolidation settlement) adjacent to proposed piles. Therefore, settlement occurring after the preload at this location was considered to be the worst case scenario relative to potential downdrag loading on new piles.

Prefabricated vertical drains (wick drains) were modelled on a 5-foot rectangular grid beneath the preload area to accelerate consolidation during the preload. The wick drain grid extended to within a few feet of the sheeting (beyond the permanent embankment fill) on the outboard side and at to the edge of the existing sand drain installation on the inboard side.

GZA evaluated the settlement at the Sta. 13+05, 22' Lt. based on the temporary preload scheme.

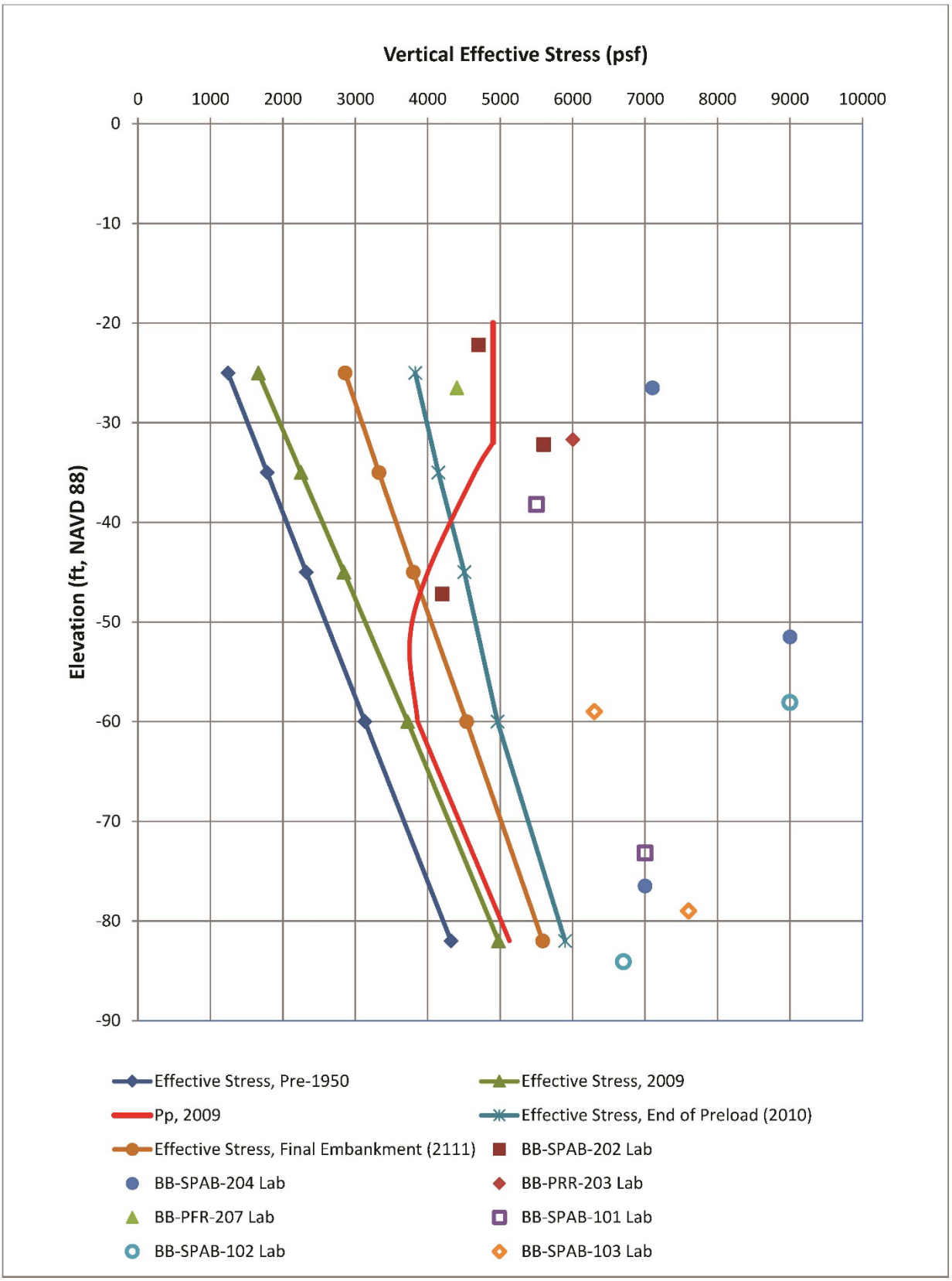


Figure 4 – Design Stress History Profile

The stress increase and consolidation settlement evaluations were conducted using the computer analytical software, "Foundation Stress and Settlement Analysis, FoSSA (2.0)," by ADAMA Engineering, Inc. The modeling capability of FoSSA includes evaluation of an "approximate three-dimensional configuration", which is based on a two-dimensional cross section with a finite length. The stress increase under embankment loading is calculated in FoSSA using the theory of elasticity, and consolidation settlement is evaluated at a given location based on the consolidation properties input for each sub–layer. Time-rate of settlement evaluations performed in FoSSA are based on the Federal Highway Administration's report entitled, "Prefabricated Vertical Drains, Vol. I, Engineering Guidelines," (Report No. FHWA/RD-86/168).

Three different embankment cases were evaluated: 1) Preconstruction embankment (1950-2009), 2) Preload embankment (2010-2011), and 3) Final Embankment (2011-2111). Consolidation settlement was evaluated in FoSSA for each case starting from the undeveloped site condition, but the stress history was updated for cases 2 and 3 to account for the quasi-pre–consolidation effects (i.e., increased maximum past pressure) from the 1950 embankment.

Preload embankment (case 2) settlement was evaluated as primary consolidation settlement over the duration of the preload, with design criteria of at least 90 percent average consolidation in each sub–layer at the end of the preload. The Marine Clay between El. -50 and El. -94 was anticipated to be the last layer to reach 90 percent consolidation.

Post-construction final embankment (case 3) settlement was evaluated as primary consolidation settlement and secondary compression over the 100-year design life of the bridge.

Coefficient of vertical consolidation (Cv) and coefficient of secondary compression (Cα) were selected for each sub–layer based on the stress ratio (the design bases for Cv and Cα are shown on **Figures 5 and 6**, respectively).

Based on FHWA/RD-86/168, and Haley & Aldrich (1969), the coefficient of horizontal consolidation (Ch) in each sub–layer was estimated to be approximately 1.3 times Cv. Soil disturbance (i.e., "smear" effect) and drain resistance were judged to reduce the effective rate of consolidation in accordance with FHWA/RD-86/168.

The results of GZA's settlement mitigation evaluations are summarized in the following paragraphs.

Total predicted settlement (primary consolidation plus secondary compression) during the preload period at Sta. 13+05, 22' Lt., was approximately 11 inches.

The deeper clay sub–layers were expected to be slowest to consolidate, but would achieve 90 percent consolidation under the preload in approximately 6 to 10 months. The preload would be removed once the instrumentation indicated that 90 percent consolidation has been achieved, minimal lateral movement was observed, and the project team concurred that 90 percent consolidation was achieved.

The predicted total settlement during the preload exceeded the anticipated primary consolidation settlement (approximately 7 inches) plus secondary compression (approximately 3 inches) anticipated under permanent loading at Sta. 13+05, 22' Lt.

To enhance consolidation, the preload was designed with a wider lateral footprint than the final embankment. A result of this in combination with the "shadowing" effect of the wall footings is that post-construction settlement in the vicinity of the piles was estimated to be less than 0.4 inches. FHWA guidance (Hannigan et al) indicates that downdrag forces are negligible below this threshold and are not to be considered in design.

The predicted post-construction pavement settlement was approximately 1/4-inch, and that settlement was only expected to be observed beyond the limits of the new wall footings. Therefore the new roadway would meet the post-construction pavement performance criteria associated with the five-year warranty period (less than 2 inches within 100 feet of abutment and less than 3 inches greater than 100 feet from abutment).

Sandford (1994) concluded that reducing post-construction settlement to less than 6 inches would result in "tolerable lateral movement" of abutments. Since less than ½-inch of post construction embankment settlement was anticipated, lateral squeeze, additional stresses on foundations, and post-construction wall rotation were not anticipated.

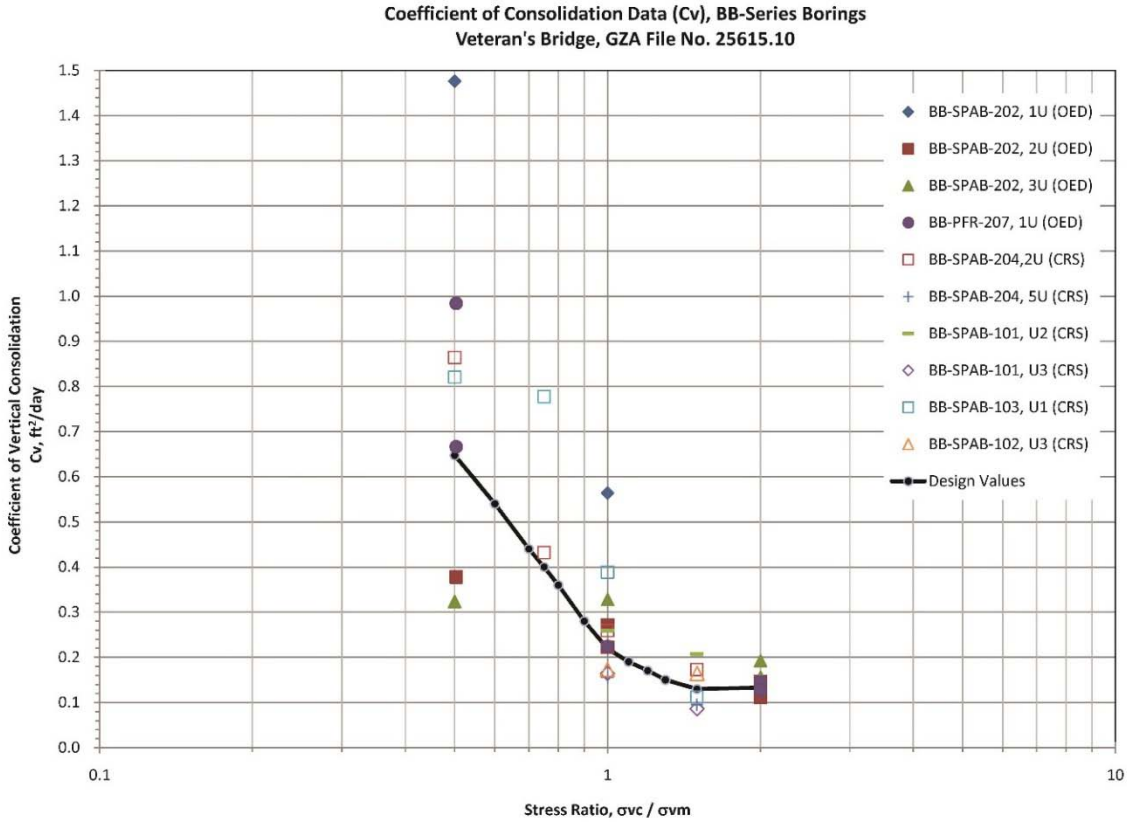


Figure 5 – Coefficient of Consolidation

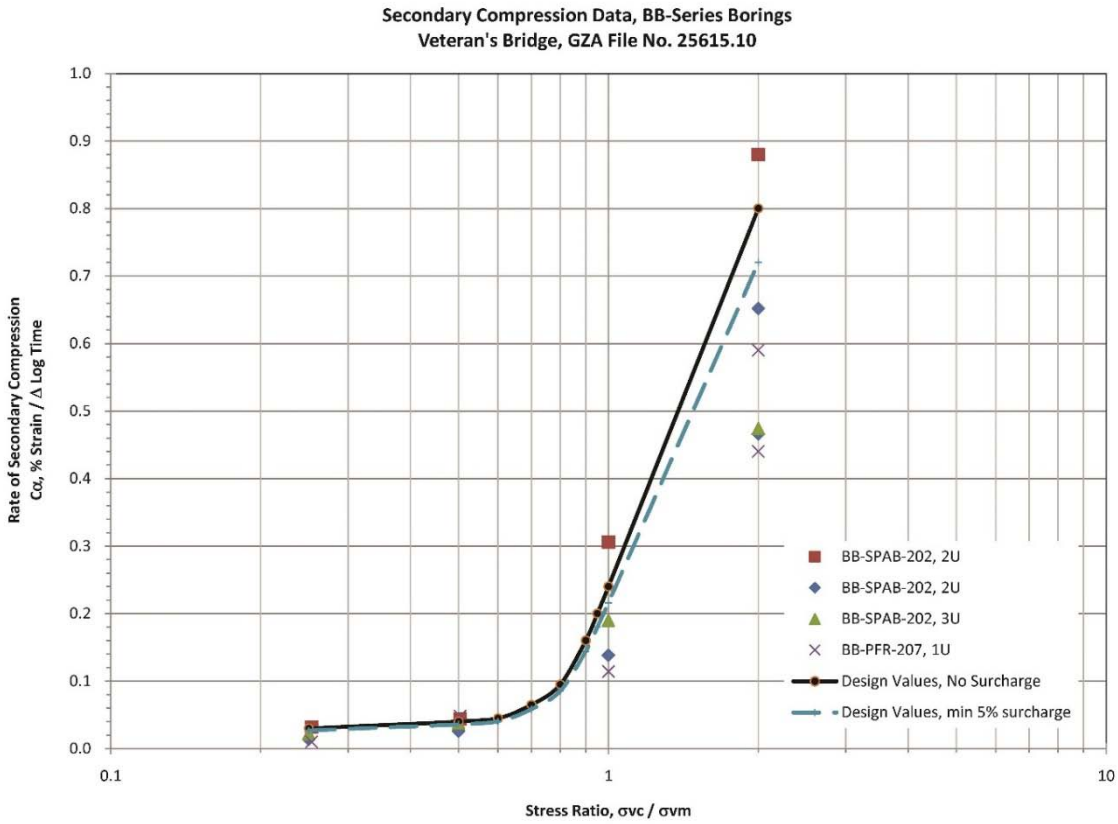


Figure 6 – Coefficient of Secondary Compression

As a design-build project, the allowable limits to impact the aquatic resource zone were established before the design was developed. The wall helped meet the impact limits specified in the RFP, and provided structural support for the relatively high fill placed on the unimproved marine sand and clay deposits. Without the wall, the soils could not support the embankment preload.

In order to improve soils beneath the new retaining wall and abutment, sheet piling was installed approximately 5 feet beyond the outside edge of the proposed South Abutment and Retaining Wall 1 pile caps. The preload fill was constructed with tiered fill elevations: full height (El. 25) on the inboard half where the permanent embankment would sit and lower (El. 14) adjacent to the sheeting in order to preserve short-term global stability.

The construction sequence at the South Portland abutment was as follows:

- Install outboard temporary sheet pile wall;
- Install inboard temporary inner wall to support active roadway;
- Install settlement platforms, piezometers and inclinometers to monitor preload;
- Fill to create a drainage blanket and a level work area;
- Install prefabricated vertical drains to the bottom of the clay;
- Construct remaining fill and temporary preload;
- As fill is placed, install sheet pile tieback system connecting to recycled precast concrete deadman anchor slabs;
- Monitor lateral deformation using the inclinometers;
- Monitor consolidation settlement using the piezometers and settlement platforms until the preload settlement objectives are achieved (originally estimated to require 6 to 10 months);
- Remove preload and excavate fill to base of abutment and retaining wall level;
- Drive piling;
- Construct new walls;
- Construct finished slopes between new wall and temporary wall;
- Remove temporary wall system;
- Complete final grading and paving.

4.3 *Impacts on Adjacent Facilities*

GZA evaluated the potential for settlement that could result in serviceability loss or damage of adjacent facilities due to the preload. Facilities assessed for possible impacts included: the original VMB South Abutment which would remain in service until the replacement bridge was completed; the Pan Am Railways Rail Line on the opposite side of the original embankment; and the existing pavement in the nearest travel lane.

To evaluate settlement at the first three locations, the stress increase imposed by the preload (representing 100 percent consolidation) was calculated at each station and compared to the pre–consolidation pressure (accounting for quasi-pre–consolidation in layers that were loaded into virgin compression under the current embankment). The results showed:

- The maximum past pressure in deeper clay layers was greater than or equal to the final stress under the preload, indicating there would be minimal settlement under the new fill loads at these distances.
- Settlement at the rail line, existing utilities, and the existing South Abutment would consist of recompression under a small load increase and will be less than about ½ inch.
- Since the stress levels did not exceed the maximum past pressure, settlement due to secondary compression was anticipated to be negligible at these locations.

The stress increase was anticipated to cause virgin consolidation at the edge of the existing pavement/edge of new pavement. The evaluation focused on serviceability of the pavement during the preload and post construction pavement settlement, and showed:

- Settlement at the existing roadway shoulder (Sta. 12+80, 45’ Rt.) during the preload period was estimated to be less than about 3-½ inches.
- Post-construction settlement during the warranty period was estimated to be approximately 1-½ inches. Since this is less than 2 inches, it met the project pavement warranty criteria.
- Maximum settlement of the existing roadway was expected along the northern edge, and would decrease across the width of the embankment toward the south.

4.4 *Embankment Stability*

4.4.1 *Soil Properties*

GZA developed design soil properties for the South Portland approach embankment based on the available subsurface data. Granular soils (including Fill, River Bottom Deposit, and Marine Sand) were modeled using effective stress parameters (i.e., drained friction angles) estimated based on SPT N-values and typical properties of new fill materials. Presumpscot clay was modeled using undrained shear strengths estimated using the in-situ vane shear tests in the borings nearest to the proposed embankment and laboratory isotropically consolidated undrained triaxial shear strength (CIUC) tests conducted on samples from borings. The strength parameters assigned to each soil layer are summarized in **Table 1**.

The undrained shear strength data for the Marine Clay were assembled in a plot of elevation versus undrained shear strength, as presented in **Figure 7**. It is based on in-situ vane shear testing (93 tests from eight test borings) and CIUC laboratory testing with pore pressure measurements (7 tests from three test borings). The plotted shear strength values were interpreted to provide input soil parameters for the Marine Clay layers for use in global stability analyses. It is recognized that the existing undrained shear strength of the Marine Clay is dependent on the existing overburden and preconsolidation stress, which varies beneath the existing and proposed embankment.

Since the majority of the data was collected from borings drilled beneath or outside of the toe of the existing embankment, GZA judged that the interpreted design profile was representative of pre-construction shear strengths in the vicinity.

4.4.2 *Design Section*

Short-term stability was initially considered in both the longitudinal and transverse directions relative to the proposed centerline. The soil layering was similar for both profiles, so the controlling parameter was the embankment geometry. Since the outboard mudline level was lower in the transverse direction there would be less resisting mass than in the longitudinal direction, the transverse direction was considered the controlling case.

4.4.3 *Short Term Stability*

GZA performed a series of analyses to assess rotational stability of the preload embankment. The analyses were conducted using the computer analytical software, "Slope/W 2007," developed by Geo-Slope International, based on the Modified Bishop method.

Analyses showed that without sheet piling, a shallow rotational failure would be anticipated.

The solution was to drive sheet piling to a toe El. -35 and maintain it there during preloading, forcing the slip surfaces below the bottom of the sheeting. Groundwater was interpreted to be tidally-controlled. Weep holes in the sheet pile wall provided drainage. The design water levels accounted for a 2-foot "lag" in drainage of the embankment soil during ebb tide.

The end-of-construction case was considered with the full preload embankment and a 250 psf equipment surcharge load in place using pre-construction undrained shear strength parameters in the Marine Clay (i.e., no strength gain was assumed).

The results of GZA's evaluation indicate that the calculated minimum factor of safety against rotational failure exceeds 1.3, as shown on **Figure 8**. Current engineering practice considers a factor of safety of 1.3 to be acceptable for temporary conditions, such as during construction.

Therefore, GZA concluded that the preload fill could be constructed in its entirety in a single stage. However from a practical standpoint, maximum fill rates were anticipated to be on the order of 4 feet per day due to access and trucking issues in the constricted work area.

Instrumentation was installed and monitored to assess the stability of the preload fill during the fill placement and preload period, as described in subsequent sections.

4.5 *Long Term Stability*

Long term stability of the embankment was considered as part of the design of the permanent abutment and retaining wall. These structures were designed by conventional means. Since they were to be constructed on piles extending though marine clay soils, analyses were made to assess potential lateral squeeze, and downdrag, as previously described.

Table 1 - Summary of Soil Properties for Stability Evaluation			
Unit	**El. Range (ft), NAVD 88**	**Existing In-situ Strength Parameter**	**Unit Weight**
Fill (Existing)	El. 22 to El. -2	Φ = 34°	125 pcf
Fill (Proposed)	El. 25 to El. -2	Φ = 32°	125 pcf
River Bottom Sand/Upper Marine Sand	El. -2 to El. -15	Φ = 30°	118 pcf
Marine Clay:			
-Upper	El. -15 to El. -22	Su = 700 psf	118 pcf
-Middle 1	El. -22 to El. -40	Su = 850 psf	118 pcf
-Middle 2	El. -40 to El. -70	Su = 950 psf	118 pcf
-Lower	El. -70 to El. -90	Su = 1,150 psf	118 pcf

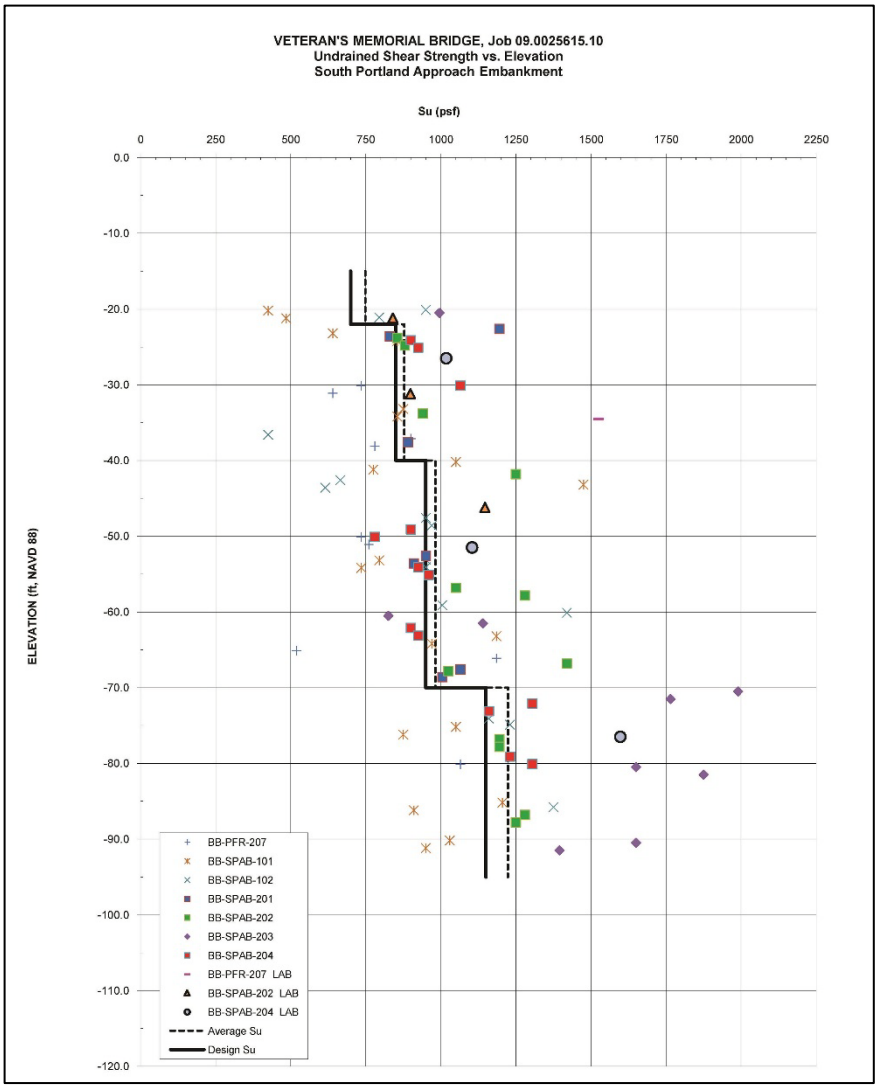


Figure 7 – Undrained Shear Strength Profile

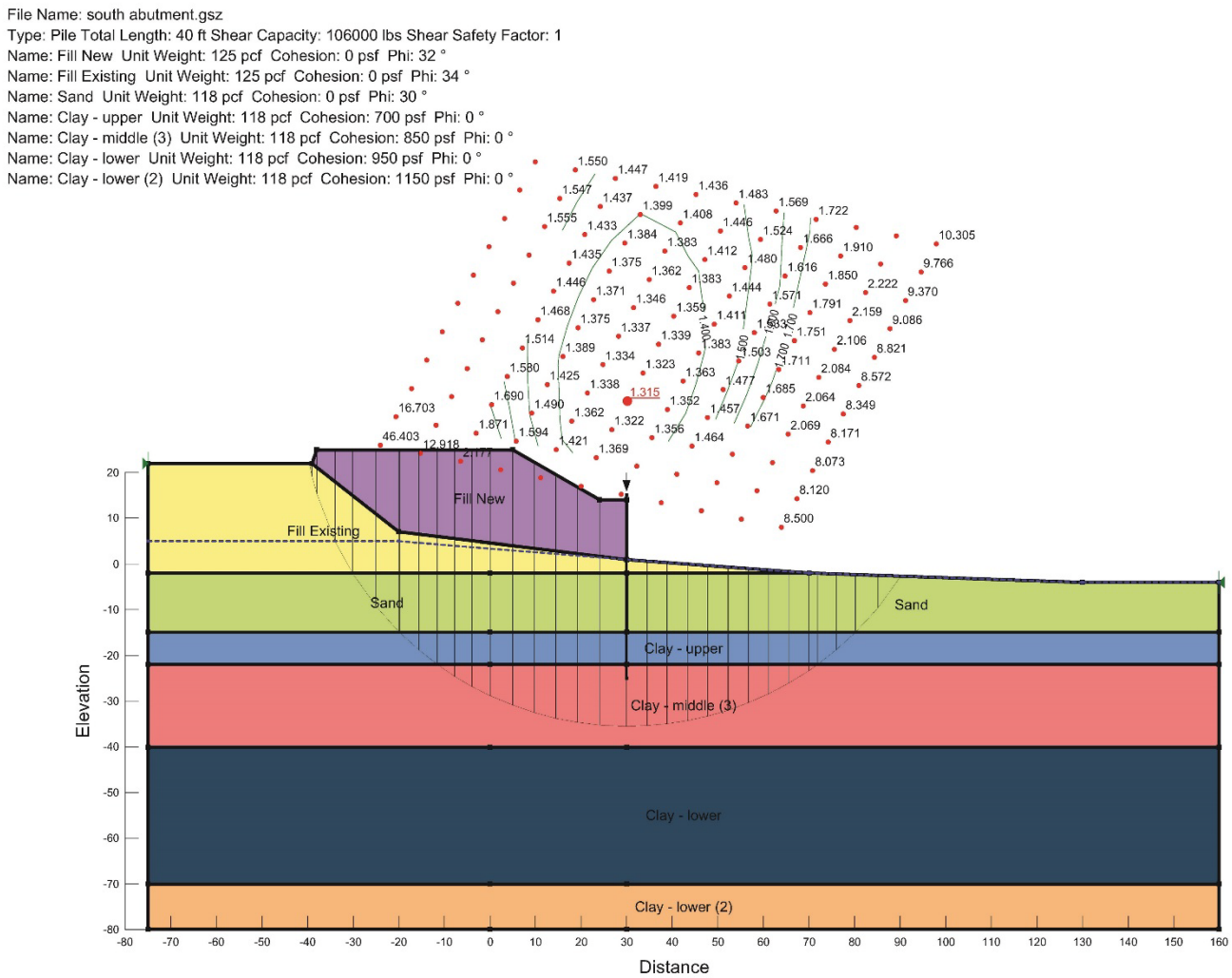


Figure 8 – Global Stability at End of Fill Placement

4.6 *Temporary Sheet Pile Wall*

The temporary sheet pile wall was analyzed in two stages. The first stage considered a cantilever condition with fill to El. 10 plus a 2-foot construction equipment surcharge, representing the worst case before the tie rods were installed. The second stage was assessed using the free earth support method, assuming the tie rods were installed at El. 10 and the remainder of the preload fill was placed thereafter, including a 2-foot construction equipment surcharge. Both cases used the initial undrained shear strengths in the cohesive soils. Therefore, construction of the second stage required that the deadman and tie rods be installed, but did not require strength gain due to the consolidation of the cohesive soils.

The sheet pile analyses showed that wall equilibrium would occur with tip embedment to approximately El. -23.7 for the cantilever case and El. -17.3 for the anchored case. Consistent with the 1995 AASHTO Guide Design Specifications for Bridge Temporary Works, the embedment was increased to provide a margin of safety against instability. Given the critical nature of the embankment and with the desire to limit lateral deformation, the tip elevation was established at El.-35 (a 50-plus percent embedment increase).

Due to the proximity of the temporary wall to the batter piles of the new foundations, it was necessary to raise selected sheet piles approximately 12 feet prior to pile driving. As a precaution against creating an unstable condition, the surcharge was removed and the fill level lowered prior to raising the sheets.

4.7 *Instrumentation Design*

Instrumentation was used to monitor settlement beneath the preload and potential lateral movement near its edge. The ability to meet the settlement criteria and mitigate downdrag was directly dependent on the percentage of consolidation completed. In order to assess when the required amount of consolidation had been achieved, GZA monitored consolidation via direct measurements of excess pore water pressures and settlement. In addition, lateral soil movements were observed to assess possible lateral deformation.

Settlement measurements were made with conventional settlement platforms in fill areas, and by optical survey points set at key locations on existing pavements and structures. The optical survey points were located to assess settlement of the existing bridge abutment and pavement along the approach roadway adjacent to the preload. Top-of-rail monitoring points were to assess potential settlement of the railroad. Monitoring points were established on both rails to allow measurement of total and cross-tie settlement.

Three optical monitoring points were installed on the sheeting adjacent to settlement platform locations. A baseline elevation measurement was taken at each monitoring point at the time of tie rod installation. The monitoring points were available to collect subsequent readings in the event that differential settlement of the tie rod, sheet pile, or anchorage system was suspected. Since no distortion of the tie back system was observed, subsequent survey was not required.

Pore pressures were monitored with vibrating-wire piezometers installed beneath each settlement platform, denoted as PZ-101 through PZ-103 on **Figure 3**. The piezometers were installed at three depths per instrument cluster location (in the upper third, middle third, and lower third of the Marine Clay). The piezometers were read electronically and excess pore pressure was plotted against time and fill height. Wiring was extended up along the settlement platform piping for protection and to minimize interference with the filling operations. Having the piezometer locations adjacent to settlement platforms allowed direct comparison of settlement and pore pressure response.

Lateral soil movements were monitored using a series of inclinometers. The inclinometer locations were chosen in the areas judged by GZA to have the least resistance to lateral movement, typically at the center of longer straight sections. Inclinometers were not planned near the corner of the temporary and permanent walls because the three-dimensional effect at the corner makes it less susceptible to movement. Inclinometer casings were installed and grouted into boreholes oriented so that one set of grooves was aligned with the expected direction of movement (perpendicular to the wall). Angle readings were taken at regular intervals and integrated from the tip upward to develop displacement curves for the casings. Subsequent reading sets were compared to baseline data to evaluate lateral deformation of the soil.

The instrumentation was installed prior to any filling and concurrent with installation of the outside sheet pile wall. The drilling equipment for

piezometer and inclinometer installation was set on temporary, pile-supported platforms to allow work above tide level before the fill was placed. This allowed collection of baseline data and instrumentation stabilization prior to fill placement. Piezometer baseline data was collected at regular intervals through several tide cycles, and compared to river levels measured on a tide board in the river. These data provided a basis for assessing the relative impact of tidal variations on specific piezometers.

4.8 *Post Construction Instrumentation*

Upon completion of backfilling, it was planned to turn over the majority of the instrumentation to MaineDOT.

Permanent survey monuments were cast in Retaining Wall 1 and the South Abutment.

The inclinometer systems were maintained during construction, cast into the permanent wall footings, and turned over to Maine DOT when the permanent walls were completely backfilled. Piezometers were also turned over to Maine DOT after construction. Upon completion of the preload, the piezometer readout boxes were relocated adjacent to Retaining Wall 1.

The settlement platforms were decommissioned just prior to final paving, since they were located in the travel way. Optical survey points were maintained until just prior to removal of existing pavement.

5 INSTRUMENTATION IMPLEMENTATION

5.1 *Program Summary*

The instrumentation clusters installed to assess completion of the preload included three-level piezometer installations, an inclinometer and a settlement plate at three locations. The instrumentation cluster locations correspond to the three boring locations, BB-SPAB-PZ301 through BB-SPAB-PZ303, shown in **Figure 3**.

The team installed the instrumentation, collected and submitted data in accordance with Geotechnical Instrumentation Plan. The planned schedule for recording instrumentation data included one to two sets of readings per day during fill placement, decreasing to one set of readings per month after approximately one month.

A representative set of monitoring data is presented in **Figures 9** through **11**. The data were recorded from piezometers, inclinometers, and settlement plates at BB-SPAB-PZ303.

5.2 *Preload Completion Criteria*

The settlement mitigation design required that 90 percent consolidation be completed prior to removal of the preload. The Geotechnical Instrumentation Plan established the following general criteria for evaluating completion of the preload.

- At least 90 percent of excess pore pressure is dissipated;
- Inclinometers show no significant persistent horizontal movement in successive readings; and
- Slope of settlement vs. time plots indicate completion of primary consolidation.

5.3 *Monitoring Results*

The monitoring data indicate that the criteria for completion of the preload were met. Specific results and evaluations are presented for each Instrument set in the sections that follow.

5.3.1 *Piezometers*

Baseline data were recorded after completion of sheet pile driving and prior to filling activities. The data were intended to cover two cycles of tidal fluctuation. Baseline data were recorded in mid-July, 2010. During the two-day baseline data collection period the tidal head variation in the piezometers was on the order 2.5 to 5.6 feet, while the tidal range measured in a temporary piezometer in the river was about 9.4 feet. These data suggest some level of communication between the tide variation and the piezometers. The baseline reading program was designed to as a means to make corrections to subsequent readings based on tide stage. All piezometer readings were presented in feet of fresh water.

Embankment and preload filling occurred between late July and late September, 2010. During active filling in piezometers P-1A/B/C, P-2A and P-3A/B, the recorded pore pressure response ranged from about 2 to 5.9 feet above the long term readings at low tide. The measured pore pressure response was typically similar to the tidal head variation, plus or minus about 1 foot. The

subsequent readings were scheduled to be taken on a consistent tide stage to eliminate the need for tidal correction. It is likely that larger pore pressure head variations occurred due to filling, but that the drainage occurred so rapidly that they were not measured.

In piezometers P-2B/C and P-3B, the recorded pore pressure response during active filling ranged from about 5.9 to 8.3 feet above the long term readings at low tide. These data indicate that the measured pore pressure response exceeded the tidal head variation by as much as about 4.5 feet. It is likely that larger pore pressure head variations occurred due to filling, but that the drainage occurred so rapidly that the full magnitude was not measured.

The peak predicted pore pressure response shown on **Figure 9** was approximately 20 feet of water, even though there was a 25 foot fill height. For an infinitely large surface load, the stress increase would be constant with depth. However, since the piezometer is at significant depth beneath the base of the narrow strip load, the stress increase and predicted pore pressure response is much less.

Placement of fill and preload was completed on September 27, 2010. After October 1, 2010, the piezometers were read at low tide (+/-30 minutes) to minimize the influence of tidal fluctuation. The data indicate that during the period after October 7, the head had decreased in all the piezometers to approximately the long term low tide level, and remained there through December 2010. Therefore, GZA concluded that the piezometer data clearly showed the dissipation of excess pore pressure, and hence, primary consolidation, was completed by approximately October 7.

5.3.2 *Inclinometers*

Inclinometer baseline data were gathered prior to the start of filling, then daily during active filling and less frequently thereafter. The cumulative displacement plots show the cumulative displacement in inches at each location for the A-Axis and the B-Axis. The data of primary interest are the A-Axis because these movements are perpendicular to the wall in the direction of primary movement and bending of the sheet piling.

The maximum cumulative horizontal

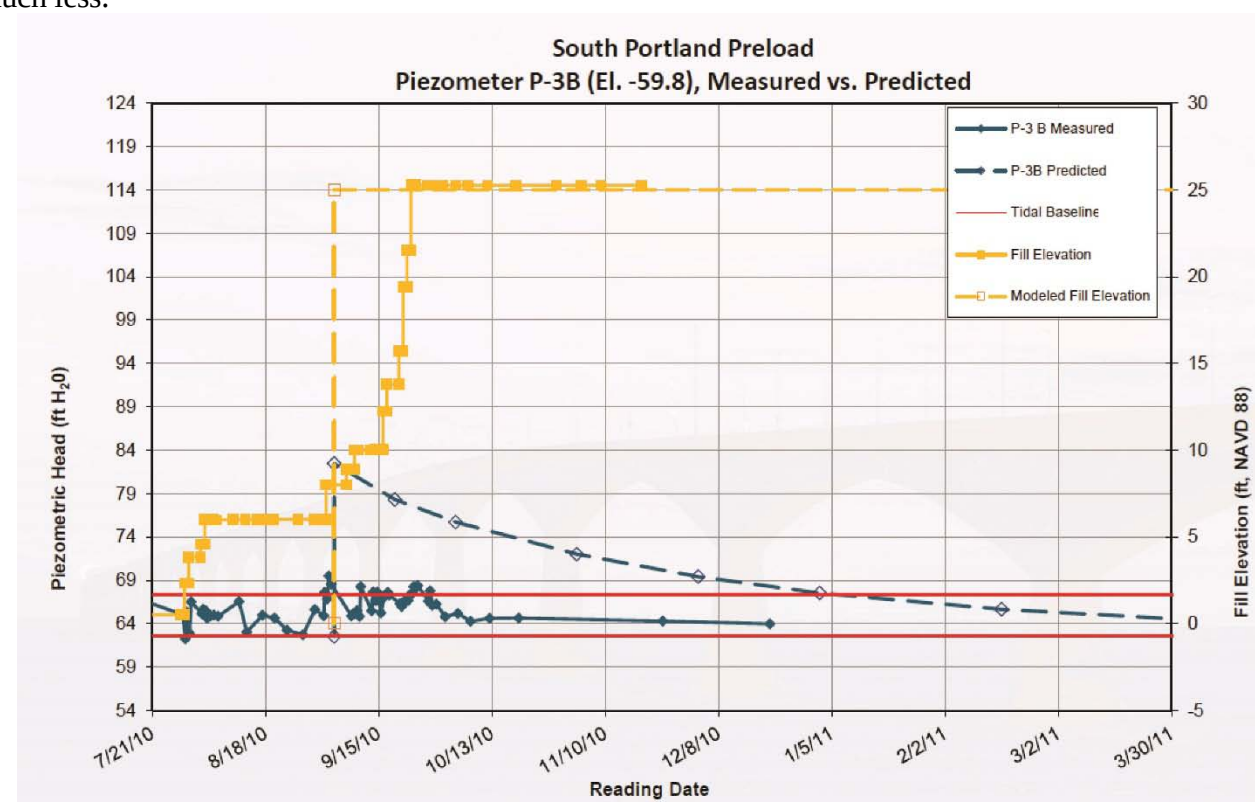


Figure 9 – Pore Pressure Response at Mid-Depth of Clay

deflections (measured at and below the original mudline) ranged between 1.4 inches (INC-1) and 2.5 inches (INC-2). Maximum deflections estimated in the design calculations ranged from about 2.6 inches at mudline, to about 6 inches at the top of the sheets and were anticipated to result from the cantilever condition, prior to tieback installation. Since the measured deflection at mudline agreed well with the estimated deflection, the fill and wall behavior were consistent with expectations. It was not practical to measure cumulative deflection at the top of sheet pile level since the inclinometer casing was not installed there until after the baseline data were recorded.

Cumulative displacement and incremental strain were plotted versus time to assess long-term lateral movement. Displacement and strain were tracked versus time at two levels for each inclinometer; the maximum value within the sheeted depth (typically within 0 to 15 feet of the original mudline) and the maximum value below the sheeting (typically 1 to 18 feet below the tip of the sheets).

The data show that incremental lateral strain peaked around October 10, 2010 and decreased thereafter.

The cumulative displacements were seen to peak around November 24, 2010, and have shown no significant increase thereafter. An anomaly was noted in the December 20, 2010 data set for INC-1. Subsequent readings were taken to assess the potential for ongoing movement. However based on the January 11, 2011 data, GZA judged that there was an error in the December data at INC-1 only. GZA concluded that persistent horizontal movement was completed at the time of the November 24, 2010 readings. The cumulative lateral displacements are plotted versus time in **Figure 10**.

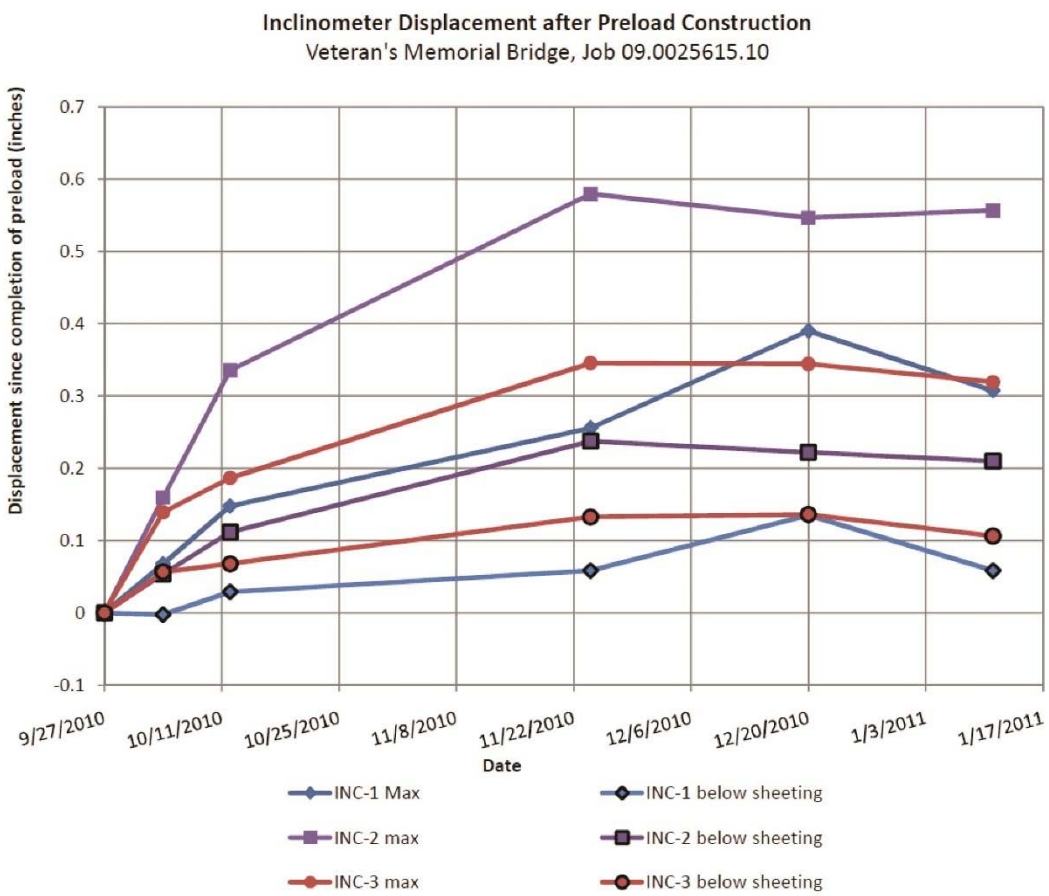
5.3.3 *Settlement Plates*

The settlement plates were installed and baseline data were gathered prior to the start of filling. Readings were taken daily during active filling and less frequently thereafter. **Figure 11** shows the predicted and measured settlement versus fill elevation at settlement plate SP-3.

The measured settlement ranged from approximately 4-¾ inches at SP-1, to approximately 6-½ inches at SP-3. As anticipated, the magnitude of settlement was greater at SP-3 because the new embankment fill was widest there. Since the embankment is narrowest at SP-1, the measured settlements were lowest there.

GZA predicted 11 inches of settlement during the preload period, and observed a maximum of 6-1/2 inches.

GZA's design estimated the time to reach 90 percent consolidation between 6 and 10 months after completion of filling. The settlement plate



data show that settlement occurred very quickly in response to fill placement, and that a significant portion of the total settlement had occurred within a few weeks after fill placement.

The Casagrande Logarithm (Log) of Time Fitting Method was applied to assess the completion of primary consolidation of clay beneath the preload. GZA's use of the Casagrande method consisted of plotting the settlement versus log of time and performing a graphical construction to estimate the time to reach "end of primary" consolidation (t_{100}). The method was modified for combined vertical and horizontal drainage in accordance with 1986 FHWA to account for the wick drains. The horizontal drainage distance was well defined by the engineered wick drain spacing. The vertical drainage distance was estimated based on test boring data as previously described.

Graphical construction for settlement plate SP3 is shown in **Figure 12**. The results indicate the end of primary consolidation occurred on approximately September 26, 2010 for all three settlement plates. These results show very rapid completion of primary consolidation, within one week after completion of filling. Based on GZA's

interpretation of the settlement plate data set, primary consolidation was completed by the end of September.

6 DISCUSSION OF RESULTS AND CONCLUSIONS

The design evaluations provided conservative estimates of the total magnitude of settlement and the time to complete consolidation of the preload.

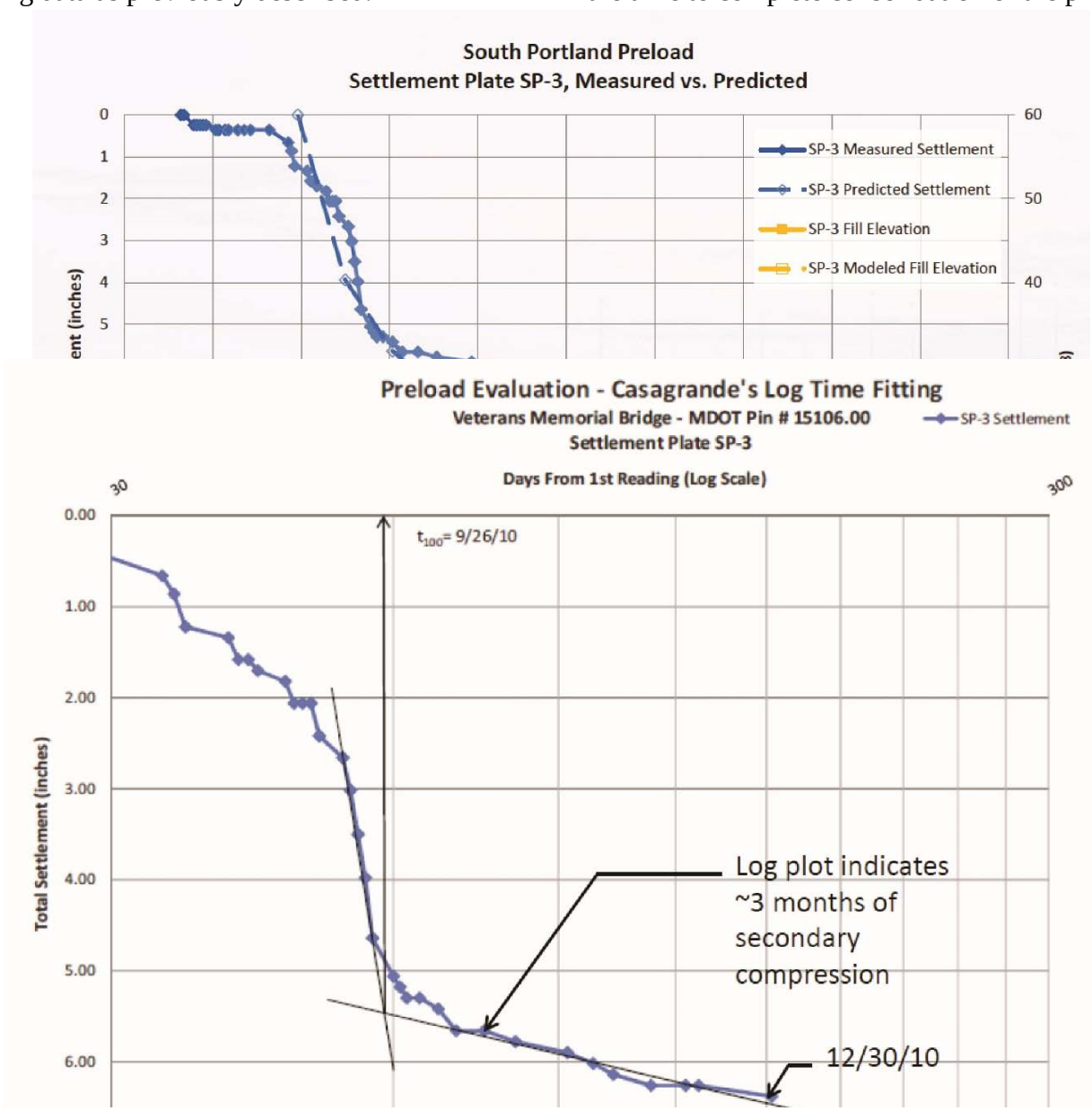


Figure 12 – Preload Completion

It is our opinion that the presence of significantly more sand seams than were delineated via traditional test borings effectively lowered the compression index and reduced the vertical drainage path in the clay deposit. The effect of these changes in material properties of the deposit were manifested in lower total settlement and more rapid consolidation. It is also possible that the ratio of horizontal to vertical coefficient of consolidation may have been higher than the value of 1.3 used in the analyses.

Another factor that may have affected the embankment performance is the estimated maximum past pressure. It is possible that the quasi-preconsolidation that had occurred due to secondary compression of the circa-1950 embankment was underestimated. Since the maximum past pressure defines the stress at which consolidation goes from recompression to virgin compression, underestimation of the maximum past pressure would show more of the settlement to occur in virgin compression. The result would be greater predicted total settlement and slower consolidation, as were observed.

The analyses provided a practical basis for embankment design that allowed embankment construction to meet a schedule that had been developed during preliminary design. The instrumentation data from the South Portland preload showed that the desired effect of the preload had been achieved and that the preload could be removed in January 2010; and that the desired improvements to the marine clay had been achieved in 3 to 4 months, rather than the predicted 6 to 10 months.

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