



## FIELD MONITORING OF THE COMPRESSIBILITY OF MUNICIPAL SOLID WASTE AND SOFT ALLUVIUM

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### ABSTRACT

*The paper presents the results of a settlement-monitoring program for a preloaded roadway embankment over a closed landfill underlain by three compressible units: municipal solid waste, alluvial peat, and organic silt. To monitor the progression of embankment settlement and assess the effectiveness of the preload treatment, a field instrumentation system was installed within the embankment footprint. The field instrumentation system, consisting of settlement plates, and vertical extensometer and vibrating wire piezometer stations, was monitored prior to construction, during construction, and for approximately 18 months thereafter. The extensometer and piezometer stations provided information on the individual response characteristics of the compressible units to load application. The paper discusses the field settlement and piezometric data, and provides time-settlement relationships for the compressible units. Back-calculated compressibility parameters for landfill refuse are compared with those reported by other researchers for such materials. Application of hyperbolic methods for consolidation analysis is evaluated with respect to reliability as a tool for settlement predictions for similar preload procedures.*

### INTRODUCTION

This paper describes a settlement monitoring program and evaluation for an approximately 600-foot long portion of new roadway, extending 41<sup>st</sup> Street SE over the old Everett landfill. The landfill is located along the west bank of the Snohomish River, in Everett, Washington, and was closed and capped with fill in 1974, after over 50 years of operation as a municipal solid waste (MSW) site.

The new roadway includes a bridge over Burlington Northern Santa Fe (BNSF) railroad right-of-way, bordering the west side of the landfill site, and embankment approach sections to both bridge abutments. The roadway and over-crossing replace an existing grade crossing of BNSF's mainline between Puget Sound ports and the Midwest. The project is a part of a regional effort to improve freight mobility throughout the Seattle, Tacoma, and Everett area. The east embankment (over the landfill) will vary in height from 28 feet at the east bridge abutment, descending to approximately 5 feet at the southeast terminus. The roadway will have 4 travel lanes, with sidewalks and bike lanes on both sides. The east embankment layout is shown on Figure 1.

#### Subsurface Conditions

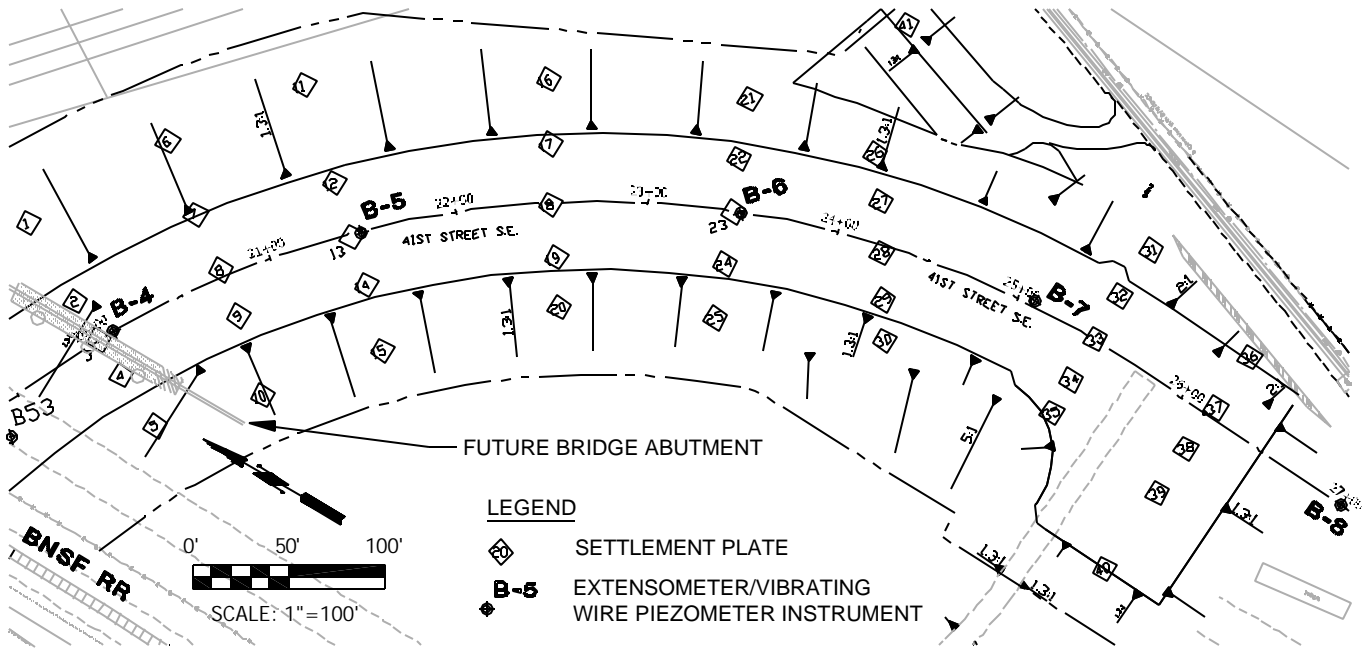
In addition to those advanced specifically for this project, dozens of borings have been drilled at the landfill for various purposes since it was closed, and subsurface conditions are well documented. Three compressible units underlie the roadway alignment: municipal solid waste (refuse), alluvial peat, and

alluvial organic silt. At existing ground surface, a 4- to 7-foot thick landfill cap, consisting of silty sand to very sandy silt, overlies the landfill refuse. The landfill refuse consists of 15 to 20 feet of a heterogeneous mixture of wood, organic matter, plastic, cloth, metal, and brick/concrete fragments, in a matrix of silt and sand. However, no laboratory tests were conducted on refuse samples for this project. The ground water (leachate) level is within the refuse at depths between 7 and 10 feet below ground surface.

Alluvial peat, varying in thickness from 10 to 20 feet, exists directly below the refuse. The peat is fibrous and woody, and contains logs. Laboratory tests on the peat indicate organic contents between 75 and 90 percent, and natural moisture contents between 100 and 425 percent. The average of 17 natural moisture content tests on the peat was 265 percent.

Very soft to stiff, organic, silt to silt/clay, about 8½ to 10 feet thick, exists below the peat along most of the embankment alignment, excluding the western end. Penetration tests in the alluvial silt indicated a typically soft to stiff consistency, suggestive of prior consolidation under the weight of the landfill.

Relatively incompressible alluvial and glacially-consolidated sands underlie the alluvial peat and organic silt. Hydrogeologic studies have found that the alluvial layers are an aquitard which separates the leachate from a deeper aquifer. A subsurface cross-section along the east embankment centerline is shown on Figure 2, and indicates a number of the project exploration



borings.

Fig. 1. Roadway embankment alignment

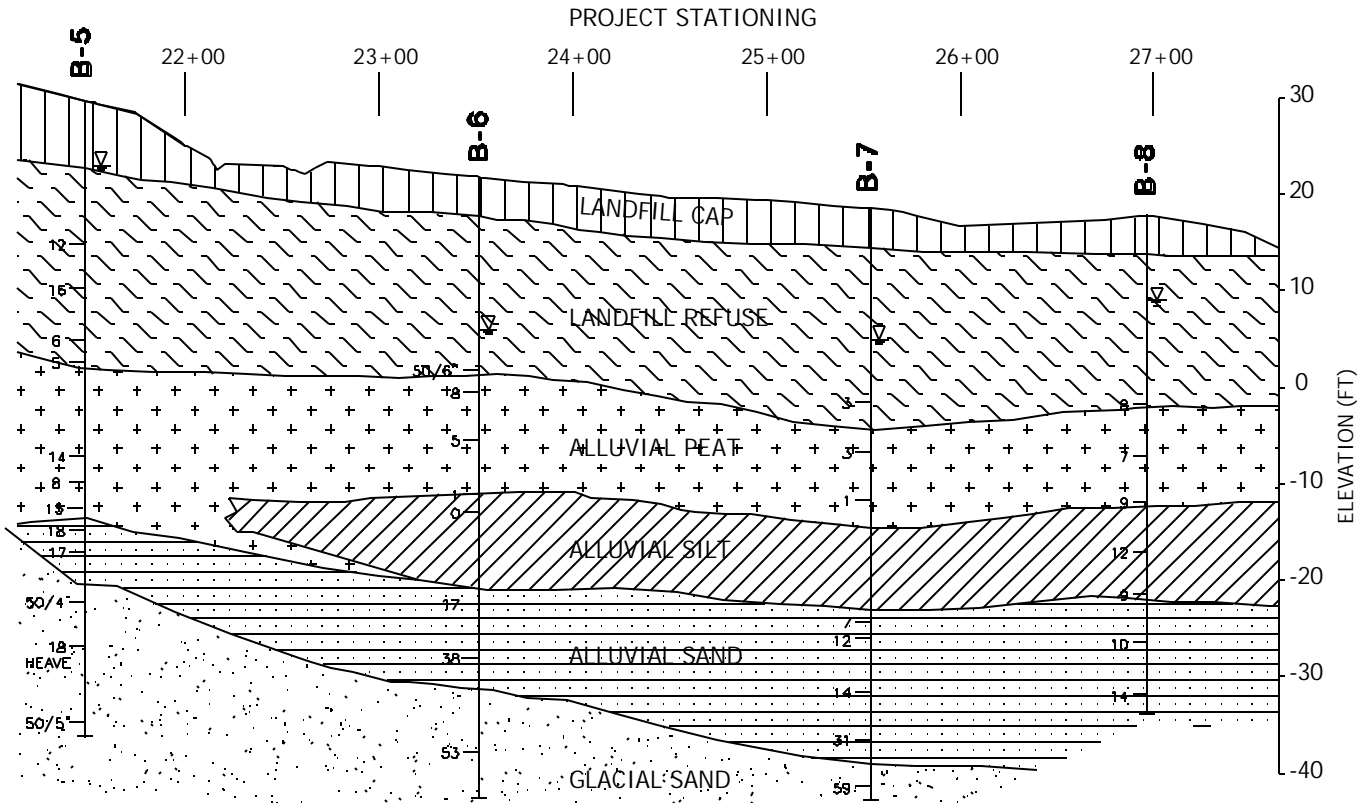


Fig. 2. Center-line subsurface cross-section.

### Preload Embankment

To mitigate effects of the significant settlements that were anticipated to occur from the weight of the embankment, a preload surcharge of 15 additional feet of soil was placed over the design embankment thickness. Based on limited available published compressibility parameters for MSW and peat, total (i.e., primary plus secondary) settlements of the preloaded embankment were estimated to be between about 8 and 10½ feet. The intent of the preloading was to generate settlement levels that would exceed the combined primary and most, if not all, of the secondary settlements that would occur from the design embankment thickness (i.e. without the preload surcharge).

### INSTRUMENTATION

To monitor the progression of settlement and assess the effectiveness of the preload treatment, an instrumentation system was installed in the embankment footprint. The instrumentation system included 41 settlement plates, and 3 deep instrumentation stations with vertical extensometers and vibrating wire piezometers (VWP's). The locations of the settlement plates and deep instrumentation stations are shown on Figure 1.

#### Settlement Plates

The settlement plates (SP's) consisted of 2-inch steel pipes attached to 2-foot square plywood plates, which were set on a sand bedding layer within the landfill cap. The steel pipes were progressively extended above the fill surface as fill was placed. Elevations of the tops of the steel pipes were recorded during fill placement and thereafter to monitor the settlement of the original ground surface. Of the 41 settlement plates, one (SP-3) was destroyed by construction equipment during fill placement. The remaining 40 settlement plates are still being monitored.

#### Extensometer/Vibrating Wire Piezometer Instrument Stations

Vertical ring-type extensometers were installed in three borings (B-5, B-6, and B-7) located along the centerline of the roadway embankment. The extensometers consisted of a series of magnetic rings attached at nominal 3-foot spacings to a corrugated 3½-inch ABS pipe, loosely inserted over an inner 2-inch PVC casing pipe. Backfill in the annulus between the boring wall and the ABS pipe consisted of low-strength and compressible grout, proportioned of 9% Quick Gel (sodium bentonite), 17% Portland cement, and 74% water (by weight). The compressible grout and ABS pipe/magnetic-ring assembly deforms with the surrounding compressible units. A sensor was lowered into the PVC casing pipe periodically to determine elevations of the magnetic rings. Settlements for each compressible layer were determined from the measured elevation changes in the rings. Figure 3 is a photograph of the ABS extensometer pipe at instrument station B-6 prior to installation.



*Fig. 3 Corrugated ABS extensometer pipe at instrument station B-6, with attached settlement rings.*

At each of the three deep instrument stations, VWP's were installed in separate borings located within 10 feet of the adjacent extensometer installation. VWP's were installed in the alluvial peat unit at B-5, B-6, and B-7, and in the organic silt at B-6 and B-7, to monitor the generation and dissipation of excess pore water pressures from the weight of the preload embankment. The VWP installed in the peat at B-6 malfunctioned early during fill placement, but the others continue to operate. Figure 4 provides a typical boring and instrument log at instrument station B-7. The deep instrumentation was installed about 7 months prior to the start of fill placement, to permit establishment of baseline readings.

A summary of the initial subsurface conditions at each deep instrument station is presented in Table 1.

## EMBANKMENT AND PRELOAD CONSTRUCTION

Embankment construction commenced in early August 2001, and was substantially completed by mid-October 2001. The embankment was constructed using primarily imported sand and gravel borrow material. A low-permeability fill, consisting of very silty sand and gravel (glacial till derived from a nearby borrow source), was used as part of a landfill gas barrier/collection system installed within the lower portion of the embankment section. The preload embankment temporary sideslopes were inclined at approximately 1.3H:1V. A total of approximately 100,000 cubic yards of fill were placed. As the embankment was constructed, the deep instruments were extended upward and protected using 4-inch PVC outer casing pipes.

### FIELD MONITORING

During construction, settlement plates were surveyed daily, and extensometer and VWP surveys and readings were taken weekly. Instrument and settlement plate readings were taken monthly for the first 8 months after construction, and every two to three months thereafter. For this paper, only field data from settlement plates SP-13, SP-23, and SP-33, and the instrumentation data from adjacent instruments B-5, B-6, and B-7, are presented.

#### Settlement Plates

As of the most recent data set (June, 2003), measured total settlements along the embankment alignment vary from 1.7 feet at SP-10 to 8.5 feet at SP-19. The fill heights at SP-10 and SP-19 are approximately 17.0 and 36.1 feet, respectively.

Figures 5, 6, and 7 show the progression of fill placement and measured settlements, plotted against a log-time scale, at settlement plates SP-13, SP-23, and SP-33. Settlements occurred rapidly as the fill was placed, and the rates of settlement slowed significantly thereafter.

#### Extensometers

The settlement of an individual compressible layer was calculated as the change in distance between the upper- and lower-most rings within the particular layer, multiplied by the ratio of the original layer thickness to the original distance between the upper and lower rings for the layer. The multiplier ratio, which varies from 1.0 to 1.5, accounts for that portion of the compressible layer above the top ring and/or below the bottom ring for that layer. Figures 8, 9, and 10 show the calculated settlement of each compressible layer at extensometer stations B-5, B-6, and B-7.

As may be seen by comparing Figures 8-10 with Figures 5-7, the summation of settlements of each of the compressible layers determined by the extensometer data is within 5 percent of the settlements measured at the adjacent settlement plates.

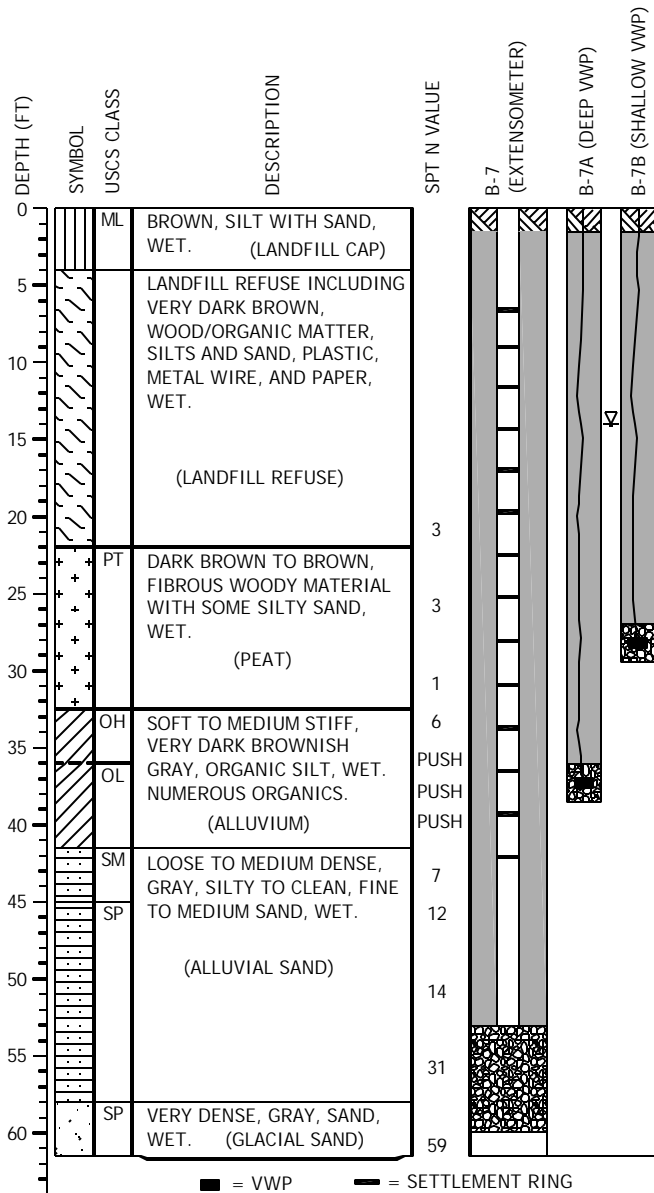


Fig. 4. Typical boring log (B-7) and instrument schematic

Table 1. Initial Subsurface Conditions and Preload Thicknesses

Instrument Location	B-5 / SP-13 STA 21+50	B-6 / SP-23 STA 23+50	B-7 / SP-33 STA 25+05/50
Original Cap and Compressible Layer Thicknesses	7 ft cap; 20½ ft MSW; and 15 ft Peat	4 ft cap; 16 ft MSW; 14 ft Peat; and 8½ ft Silt	4 ft cap; 18 ft MSW; 10½ ft Peat; and 9 ft Silt
Preload Fill Thickness	36 ft	28 ft	21 ft

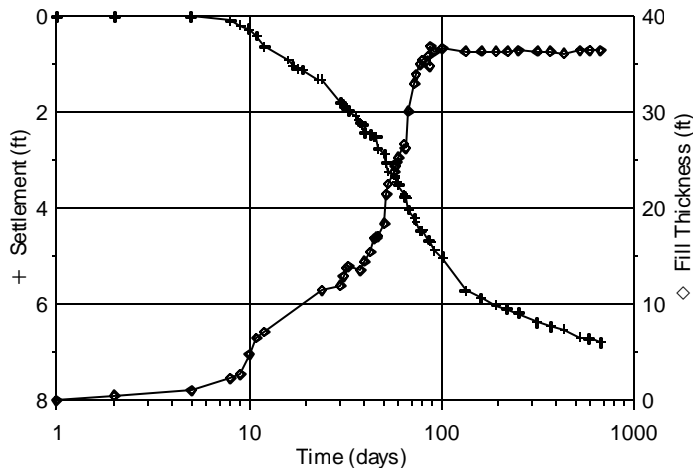


Fig. 5 Fill thickness and settlement at SP-13.

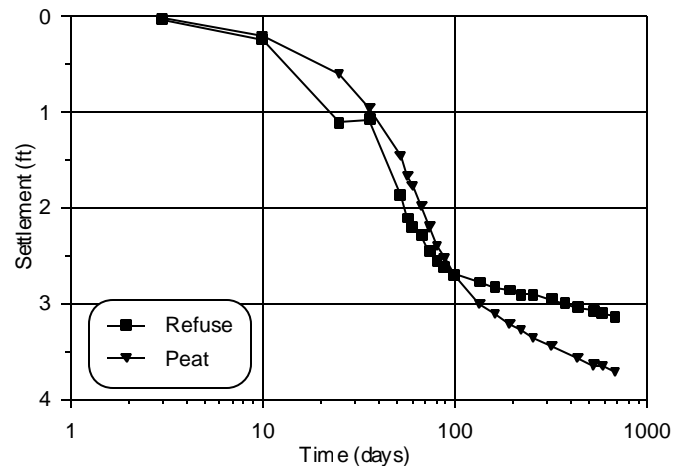


Fig. 8 Settlements at Extensometer Station B-5 (near SP-13).

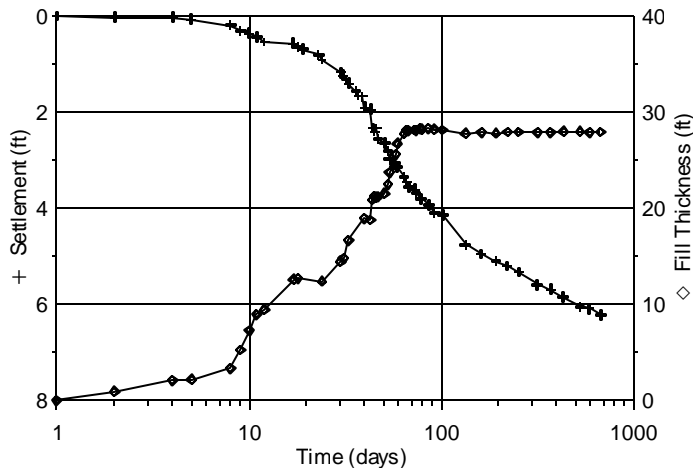


Fig. 6 Fill thickness and settlement at SP-23.

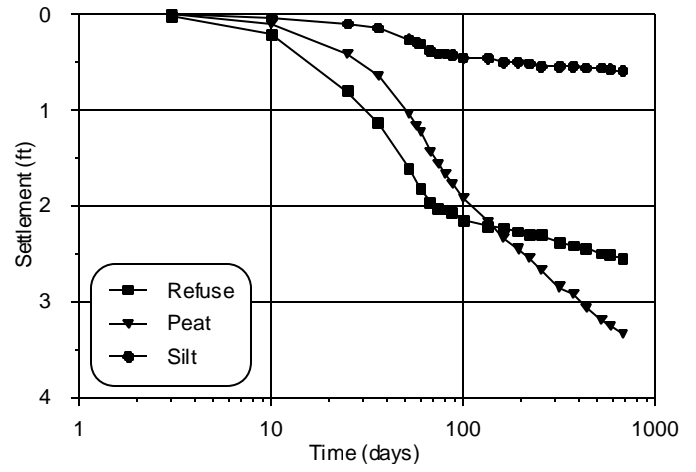


Fig. 9 Settlements at Extensometer Station B-6 (near SP-23).

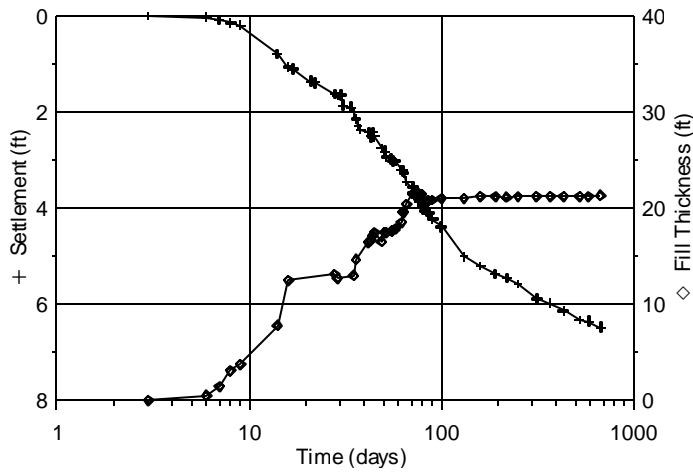


Fig. 7 Fill thickness and settlement at SP-33.

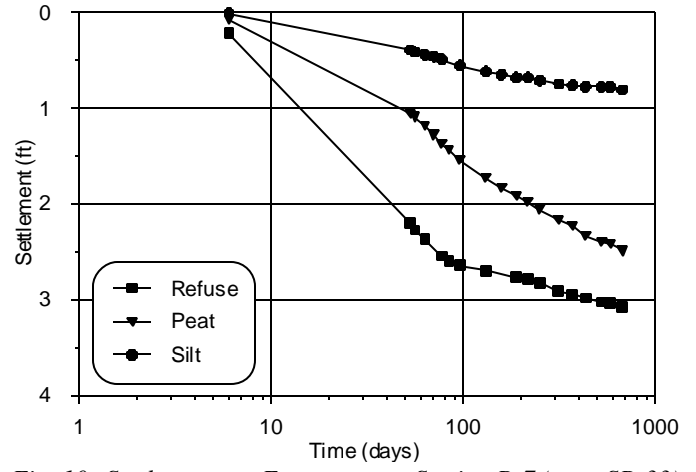


Fig. 10 Settlements at Extensometer Station B-7 (near SP-33).

Vibrating Wire Piezometers

Vibrating wire piezometers were installed in the alluvial peat and organic silt to monitor excess pore water pressures induced by the embankment filling. Figures 11, 12, and 13 show the piezometric surface elevation at instrument stations B-5, B-6, and B-7. As of the last reading set (June, 2003), the excess pore pressure head in the alluvial peat was about 3 feet at B-5, and

about 9 feet at B-7. The excess pore pressure head in the organic silt was about 2½ feet at B-6 and about 2 feet at B-7.

Although unlike B-5, the slow dissipation of excess pore pressure exhibited in the peat at station B-7 has been observed on other preloaded peat sites in the geographical region.

## ANALYSIS OF FIELD DATA

### Compressibility Parameters

The settlement data acquired from the three extensometer stations were used to back-calculate primary and secondary compression indices for each of the layers.

**Primary Modified Compression Index:** A conventional consolidation theory approach was used to calculate primary modified compression indices for the refuse, peat, and silt. This approach, despite inherent limitations (Edil et al., 1990), has been used by other researchers (e.g., Landva et al., 2000, Fassett, et al., 1994) to quantify the compressibility of MSW. The modified primary compression index,  $C_{ce}$ , is determined from Equation 1.

$$S_p = HC_{ce} \log \left( \frac{s_{vo}' + \Delta s}{s_{vo}'} \right) \quad (1)$$

$S_p$  is the primary settlement,  $H$  is the original layer thickness,  $s_{vo}'$  is the original vertical effective stress at the midpoint in the layer, and  $\Delta s$  is the increase in vertical stress due to the fill placement.

Primary settlements were determined using the Casagrande (1938) method, as the intersection of the two tangents on the extensometer S-log t plots (Figures 8-10). The average stress increase at the center of each compressible layer was estimated using Westergaard (1938) influence values for vertical stress under a very long strip load. In the analyses, the moist or saturated unit weights (as applicable) of the landfill cap, refuse, peat, and silt were assumed to be 110, 70, 75, 125 pounds per cubic foot, respectively. The baseline (pre-construction) ground water levels at each instrumentation station were used in determination of the original vertical effective stresses. The calculated results are presented in Table 2.

**Secondary Modified Compression Index:** The time-dependent, or secondary settlement parameter,  $C_{ae}$ , was determined using equation 2.

$$C_{ae} = \left( \frac{\Delta H}{\log \Delta t} \right) \frac{1}{H} \quad (2)$$

Where  $\Delta H$  is the long-term or secondary settlement,  $\Delta t$  is the time over which the long-term settlement occurs, and  $H$  is the initial height of the compressible layer. The first term in Equation 2 ( $\Delta H / \log \Delta t$ ) is the slope of the straight-line portion of the S-log t plot, after primary or rapid settlement is complete.

Table 2. Back-Calculated Compressibility Parameters

Unit	Primary $C_{ce}$	Secondary $C_{ae}$
Refuse	0.17-0.23	0.024-0.030
Alluvial Peat	0.31-0.33	0.055-0.108
Alluvial Silt	0.16-0.21	0.019-0.026

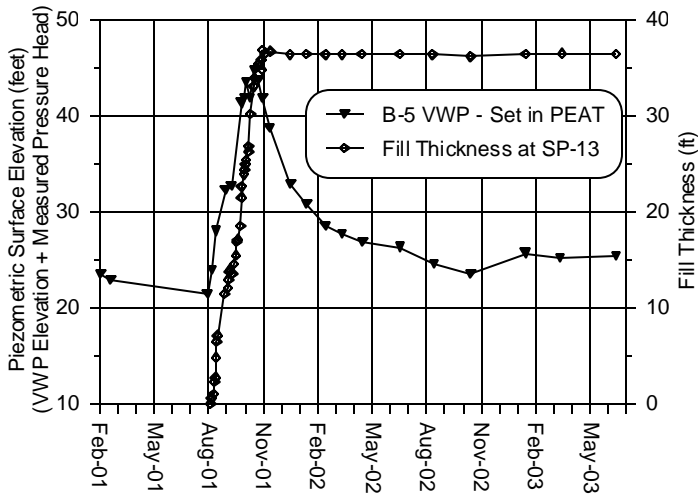


Fig. 11 Piezometric elevation versus date (linear time scale) in the alluvial peat at instrument B-5.

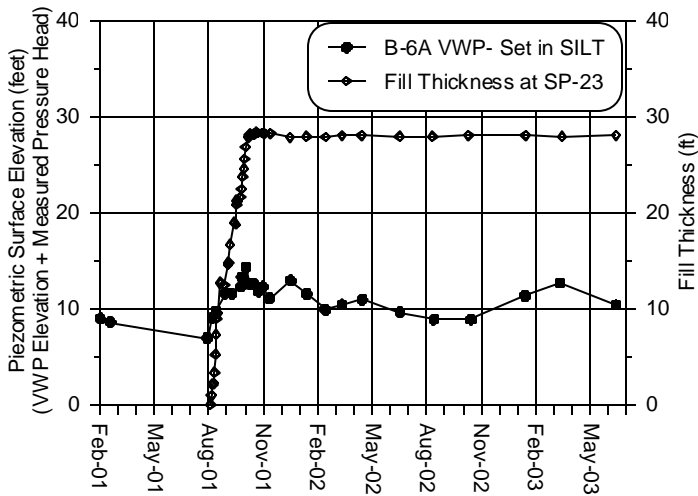


Fig. 12 Piezometric elevation versus date in the alluvial silt at instrument B-6.

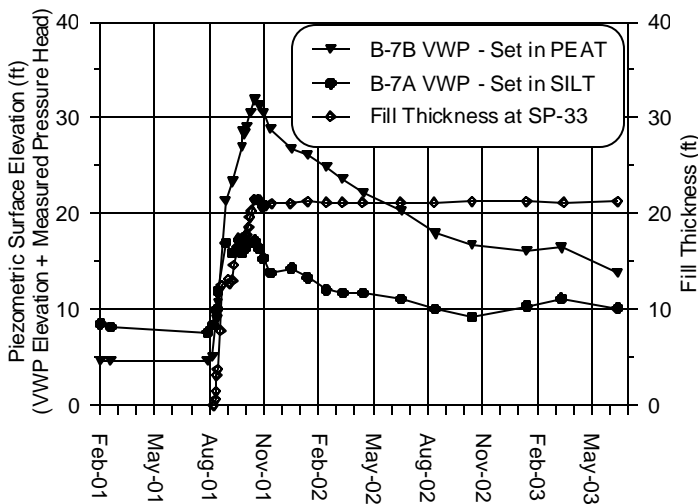


Fig. 13 Piezometric elevation versus date in the alluvial peat and silt at instrument B-7.

Table 3 presents other published refuse compressibility parameters, for comparison to those calculated for this study. As evident, our calculated values fall within a relatively narrow range, and are comparable with other published values.

Table 3. Published Refuse Compressibility Parameters

Reference <sup>(1)</sup>	Primary $C_{ce}$	Secondary $C_{ae}$
Sowers, 1973	0.1-0.41	0.02-0.07
Zoino, 1974	0.15-0.33	0.013-0.03
Converse, 1975	0.25-0.3	0.7
Chang and Hannon, 1976	--	0.013
Rao et al., 1977	0.16-0.235	0.012-0.046
York et al., 1977	0.08-0.21	0.02-0.04
Landva et al., 1984	0.2-0.5	0.0005-0.029
Burlingame, 1985	0.15-0.35	0.04
Oweis and Khera, 1986	0.08-0.217	--
Bjarngard & Edgers, 1990	--	0.004-0.04
Lukas, 1992	--	0.001-0.024
Wall and Zeiss, 1995	0.21-0.25	0.033-0.056
Gabr and Valero, 1995	0.2-0.23	0.015-0.023
Boutwell and Fiore, 1995	0.09-0.19	0.006-0.012
Stulgis et al., 1995	0.16	0.02
Green and Jamenjad, 1997	--	0.01-0.08
Landva et al., 2000	0.17-0.24	0.01-0.016
Present Study	0.17-0.23	0.024-0.029

(1) Compiled from Fassett et al., (1994), and Landva et al., (2000), and

For the peat, the calculated primary compression indices at B-5, B-6, and B-7 were 0.33, 0.31, and 0.32. The compressibility of peat is commonly correlated to moisture content. For the average initial moisture content of 265 percent, the average calculated modified primary compression index of 0.32 is within the range of published values for other peats (Hobbs, 1986). The calculated modified secondary compression indices at B-5, B-6, and B-7 were 0.055, 0.108, and 0.107. The latter values are high compared with other published values for peat, which range from about 0.015 to 0.05 (Hobbs, 1986).

For the alluvial silt, the average back-calculated modified primary compression indices at B-6 and B-7 were 0.16 and 0.21. These values lie within the range of observed modified primary compression indices for alluvial organic silts and clays in northwest Washington state.

### Hyperbolic Analysis of Settlements

The difficulties associated with conventional consolidation analyses for field conditions lie principally in the fact that soil properties are difficult to determine, and oftentimes the subsurface conditions consist of multi-layered soil deposits of differing characteristics. Soil deposits also are often non-homogeneous and transitional in nature, and distinct layered

units are rarely encountered throughout. Moreover, inherent difficulties exist in determination of initial effective stress conditions and incremental stresses at depth, induced by loading associated with structures and fills of variable geometry.

For these reasons, the hyperbolic method has found favor with many geotechnical engineers as a tool for prediction of ultimate settlements for a given set of field soil conditions and loadings. A number of researchers (e.g. Sridharan et al., 1987; Tan et al., 1991) have shown that the rectangular hyperbola method is well suited to the analysis of consolidation in the field. Tan et al. (1991) indicate that the method is popular with geotechnical engineers in Japan, as it is based on actual field settlement data that intrinsically includes the effects of secondary compression.

Simply stated, the hyperbolic method assumes that the relationship between settlement ( $S$ ) and time ( $t$ ) approaches a straight line in a linear plot of  $t/S$  versus  $t$ . The straight line describes a hyperbolic relationship and has been shown to be applicable for degrees of consolidation beyond about 40 percent (Sridharan et al., 1987). The inverse of the slope ( $m$ ) of this line yields the ultimate settlement (Tan et al., 1991). Hence, beyond the point in time that consolidation settlements have exceeded about 40 percent, the settlement data may be utilized to predict the future settlement for the loading condition generating the settlements.

The linear portion of the  $t/S$  versus  $t$  plot can be defined by Equation 3, following:

$$\frac{t}{S} = mt + c \quad (3)$$

In equation 3,  $c$  is the intercept of the straight line on the  $t/s$  axis, and the equation may be rewritten as:

$$S = \frac{1}{\frac{c}{t} + m} \quad (4)$$

As  $t$  approaches infinity; the limiting value of settlement becomes  $1/m$ . Hence, the ultimate settlement is theoretically equal to the reciprocal of the slope  $m$ , of the linear portion of the  $t/S$  versus  $t$  plot. However, Sridharan et al. (1987) have shown for experimental consolidation data that the dimensionless plot of the ratio of deflection at 100 percent consolidation,  $d_{100}$ , to the initial specimen height,  $H_i$ , (i.e.  $d_{100}/H_i$ ) versus  $1/mH_i$  is a straight line also. Based on laboratory tests on mineral soils, the straight-line relationship has been further shown by Sridharan et al (1987) to be satisfactorily approximated by the relationship:

$$\frac{d_{100}}{H_i} = \frac{0.859}{mH_i} \quad (5)$$

In this paper, we have employed the correlation coefficient of 0.859 indicated in Equation 5, utilizing the data obtained during

the monitoring program for the various instrumentation stations, to determine how the data lends itself to prediction of ultimate settlements. Predicted ultimate settlements have been determined for both the individual layers and the composite profiles at each location that surface settlement plate and deep extensometer gauges exist.

Figures 14, 15, and 16 provide plots of settlement, fill thickness,  $t/S$ , and predicted settlement, for settlement plates SP-13, SP-23, and SP-33. It is to be noted that the  $t/S$  plots have been divided by a scaling factor of 3 to fit the plot within the scale selected for fill thickness. This scaling factor is accounted for in the predicted settlements.

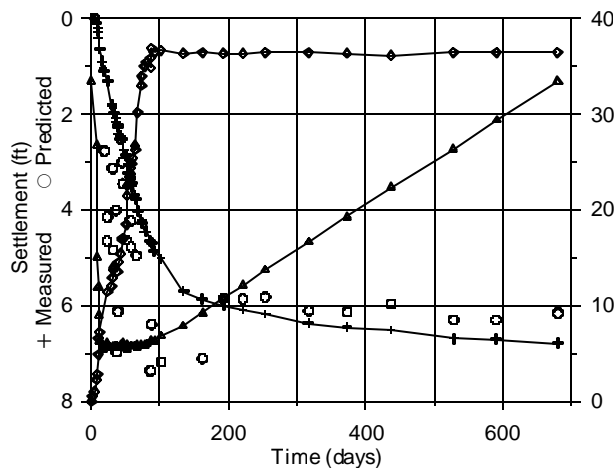


Fig. 14. Measured settlement, fill thickness,  $t/S$ , and predicted settlement at SP-13.

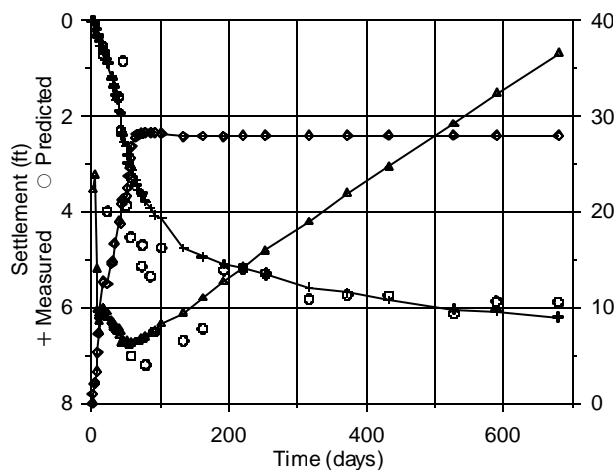


Fig. 15. Measured settlement, fill thickness,  $t/S$ , and predicted settlement at SP-23.

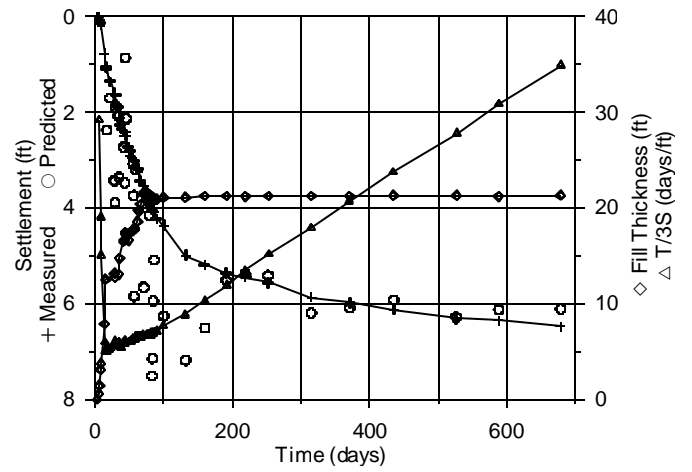


Fig. 16. Measured settlement, fill thickness,  $t/S$ , and predicted settlement at SP-33.

As evident from these figures, the predicted total ground settlements are approximately 6.3, 5.9 and 6.1 feet, at the locations of settlement plates SP-13, SP-23, and SP-33, compared with actual measured settlements to date of 6.8, 6.2, and 6.5 feet. The settlements predicted by the hyperbolic method agree fairly closely with the actual settlements measured to date. It is further evident that the predicted settlements after approximately 300 days are essentially the same as those at the last reading set (about 680 days). Hence, in the case of the conditions at this site, it can be seen that the ultimate settlements could have been predicted about 1-year previously; other circumstances, however, cause the preload to remain in place currently.

The hyperbolic method was also applied to the settlement data for the individual compressible layers (refuse, peat, and silt) acquired from the extensometer stations. For brevity, the measured settlements,  $t/S$ , and predicted settlement plots are provided for only for station B-7 (near SP-33), on figures 17, 18, and 19. The plots for stations B-5 and B-6 are similar to those for station B-7.

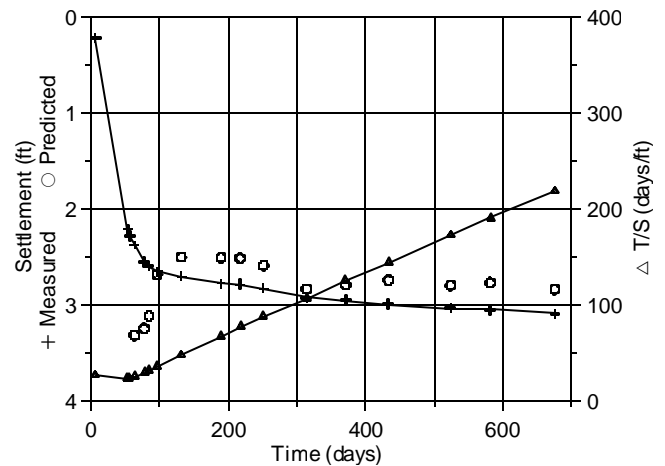


Fig. 17. Measured settlement,  $t/S$ , and predicted settlement for the refuse at extensometer station B-7 (near SP-33).

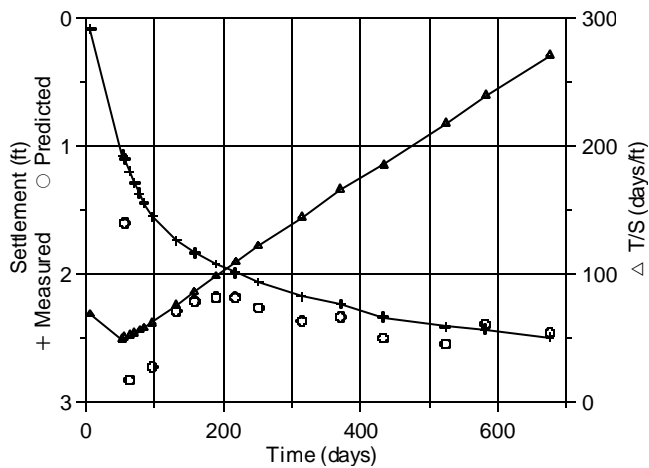


Fig. 18. Measured settlement,  $t/S$ , and predicted settlement for the alluvial peat at extensometer station B-7 (near SP-33).

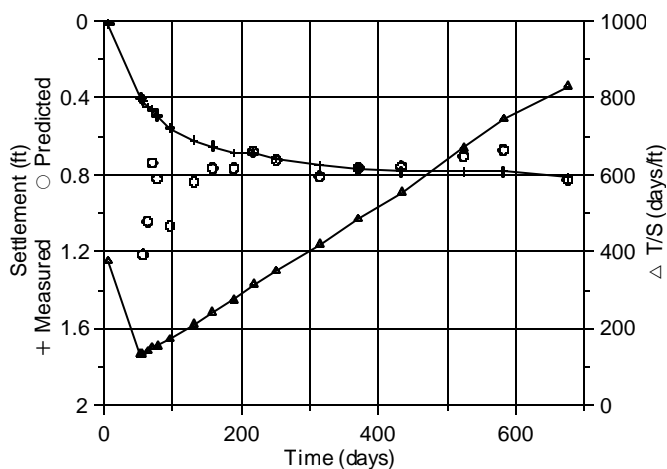


Fig. 19. Measured settlement,  $t/S$ , and predicted settlement for the alluvial silt at extensometer station B-7 (near SP-33).

The figures indicate that the predicted settlement of the refuse, peat, and silt at B-7 were approximately 2.8, 2.5, and 0.8 feet, and the measured settlements were approximately 3.1, 2.5 and 0.8 feet.

Layer settlements predicted by the hyperbolic method at all three instrument stations ranged from 92 to 103 percent of the values measured by the extensometers. The cumulative totals of the individual predicted settlements ranged from 94 to 104 percent of the settlements measured at the surface plates. Use of the correlation coefficient 0.859 consistently under-predicted the refuse settlement by 7 to 8 percent. Increasing the correlation coefficient to 0.93 resulted in very good agreement between measured and predicted results for the refuse. Use of the coefficient 0.859 resulted in predicted peat settlements that varied from 93 to 103 percent of measured, and predicted silt settlements that varied from 103 to 101 percent of measured.

## CONCLUSIONS

Comparison of monitoring results from the deep extensometers with those of the surface settlement plates indicates that, for practical purposes, surface monitoring stations are sufficient for assessment of total settlements of multi-layered soil systems. However, extensometer and VWP instrumentation is useful for monitoring the progression of settlement in individual layers.

The hyperbolic method is a good tool for prediction of long-term or ultimate settlements, as it incorporates the effects of secondary compression. Adoption of this method for ultimate settlement prediction can be effective in limiting the duration for which a site preload is required to remain in place. The correlation coefficient 0.859 developed by Sridharan (1987) was satisfactory for prediction of the peat and silt settlement at this site. For hyperbolic predictions in refuse, a correlation coefficient of about 0.93 is suggested. For hyperbolic prediction in a layered system based on settlement plate monitoring data, an intermediate value should be selected.

The extensometer data enabled back-calculation of compression parameters for the MSW, peat, and silt. The calculated modified primary compression and secondary compression indices for the refuse fall within a relatively narrow range, and are comparable to other published results. Back-calculated modified primary compression indices for the alluvial peat and organic silt are within the range of expected values for such soils.

The calculated secondary compression index for the peat at instrument stations B-6 and B-7 (0.11) is high compared with other published values. The VWP monitoring data in the peat at B-7 (Fig. 13) revealed a slow dissipation of excess pore water pressure. As of the last reading set (June, 2003), the excess pore pressure in the peat at B-7 was about 27 percent of the calculated stress increase at the midpoint of the peat layer. However, at instrument station B-5, the excess pore pressure in the peat was only approximately 5 percent of the calculated stress increase at the midpoint of the peat. Unlike B-5, the S-log t plots for the peat at stations B-6 and B-7 (Figures 9 and 10) do not exhibit a significant change in slope after 100 days. It is possible that the peat may be undergoing some retarded primary consolidation effects. A suggested explanation for this is that the peat is underlain by relatively impervious alluvial organic silt and, thus, single drainage conditions may be operating at these locations.

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