

Instrumentation and Long-Term Monitoring of Geofoam Embankments, I-15 Reconstruction Project, Salt Lake City, Utah^{1,2,3,4}

Abstract

Expanded polystyrene (EPS) has been placed in several locations along the I-15 corridor in Salt Lake City, Utah. Geofoam has been used as lightweight backfill to alleviate potential primary consolidation settlement damage to underlying utilities, improve global stability for large embankments, and to expedite construction in some locales. The Utah Department of Transportation (UDOT) has installed geotechnical instrumentation at select locations along the I-15 corridor. The purpose of the instruments in the geofoam fill areas is to monitor the construction and long-term performance of geofoam embankments. The sensors consist of monitoring devices that measure vertical deformation and stress states in the geofoam / foundation soil / pavement system. This paper presents background information and performance data from instrumentation of a geofoam embankment at the 3300 South Street off ramp of the I-15 Reconstruction Project.

Introduction

The I-15 Reconstruction Project in Salt Lake City, Utah involved the widening and rebuilding of 27 kilometers of urban interstate on compressible stream and lake deposits. In some locales, these soils have relatively low shear strength and require up to 2 years to complete primary consolidation settlement. The aggressive 4 year construction schedule, which began in the summer of 1997 and ended in the summer of 2001, required much of the embankment and foundation work to be completed in the first 12 to 18 months of the project, including the time required to develop primary consolidation settlements. To meet demanding schedule constraints and overcome the foundation soil limitations, the geotechnical design and construction made extensive use of prefabricated vertical drains (PVD), lime cement columns, staged earth embankment construction, surcharging, mechanically stabilized earth (MSE) walls, light weight aggregates and geofoam. Table 1 shows the types of embankment and ground treatment used on the project with approximate quantities placed and their unit costs. Approximately 107,000 m³

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of geofoam has been placed at the various locations shown in Figure 1 making the I-15 Reconstruction project the largest application of geofoam to date in the United States.

Since the initial construction of I-15, Salt Lake City has experienced tremendous growth and adjacent areas along the alignment between 600 North and 10600 South Streets have been highly developed (Figure 1). UDOT, geotechnical consultants and the design-build contractor conducted extensive geotechnical investigations along the reconstruction alignment to supplement available information from the original construction and performance records maintained over the past 30 years. Much of the upper 5 m of the soil profile consists of Holocene alluvial sands, silts and clay transported by streams flowing from the nearby Wasatch Mountains and of fluvial deposits from the Jordan River. Groundwater is typically found at about 2 meters below surface. At depths between 5 to 20 m, Pleistocene lacustrine deposits from Lake Bonneville consist of interbedded low plasticity clays (CL) and silts (ML) with lesser amounts of plastic clays (CH) at the top of the layer and thin, silty sand seams (SM) near the middle of the unit. Typically, the Lake Bonneville sediments initiate primary consolidation settlement when approximately 2 to 3 meters of embankment is placed and require about 400 to 600 days to complete about 98 percent end-of-primary consolidation. Primary consolidation settlement can be as much as 10 to 15 percent of the embankment height, and up to 1 to 1.5 m of settlement has been observed previously.

Locations of critical buried utility lines that may be adversely affected by the highway widening were identified and alternative strategies to prevent service interruption and reduce settlements were considered. The primary application for which geofoam was first accepted on the I-15 Reconstruction Project was to prevent primary consolidation settlement damage to underground utilities. Many existing utility lines cross or run parallel to areas of new embankment. These utilities consist of high-pressure gas lines, water mains, and communication cables that had to remain in-service during construction. All options involving compacted soil or lightweight aggregates were predicted to result in primary consolidation settlements that exceeded strain tolerances for these utilities. Geofoam weighs about 100 times less than soil. When the pavement load was compensated by sub excavation and geofoam was replaced for the embankment soil, consolidation settlements predicted by standard geotechnical analysis became tolerable. The geofoam embankments over or near buried utilities were designed to produce a "zero net load" on the foundation soils. This application of geofoam enabled utility lines to remain in service and eliminated the possible need for interruption, relocation and or replacement of critical utilities.

Construction with geofoam occurred without need for staging, surcharging and compaction in thin lifts and took much less time than all other construction methods. Thus, geofoam was later used in time critical areas to accelerate construction and remain on schedule. For cases of high bridge approach embankments overlying soft soils, such application of geofoam eliminated global stability and settlement concerns and reduced the construction time from more than 1 year to 3 months or less.

Cost was another important consideration in deciding whether or not to deploy geofoam at a given site. At locations where geofoam was being considered as an alternative, the I-15 design-build contractor prepared an embankment construction estimate using conventional construction and an estimate using a geofoam system. Geofoam embankment was selected when justified by

schedule or cost advantage over other alternatives. Table 1 suggests that the material and placement costs of geofoam are about 6 times that of embankment fill. However, the embankment unit cost of \$9/m³ does not include additional construction costs associated with foundation treatments, surcharging and MSE wall construction that were required at many locales. For preliminary estimates, a unit cost of “geofoam wall” (which included the costs of the geofoam, load distribution slab and tilt-up panel fascia wall construction) of about \$700/m² of wall face was used. When this unit cost is compared with the overall construction cost for a two-stage MSE embankment; including PVD, surcharging, and face wall, a “geofoam system” costs about 1.2 to 1.5 times more. The cost comparisons should include a complete review of each construction situation and layout, including the subsurface conditions, construction geometry and utility locations. Also, the final cost-benefit analysis should consider less tangible costs or benefits before a comparison is meaningful. The potential for improved pavement life cycle costs, reduced construction time, continuing secondary settlements, and elimination of utility relocation costs must be included in the evaluation. Figure 3 presents approximate unit costs for the installation of the various components of a geofoam wall system on the I-15 Reconstruction Project. The cost summary includes all labor and materials and is averaged over all geofoam locations on the project. Where collateral settlements of adjacent developments and underlying utilities cannot be tolerated or potentially expensive, cost comparison can favor the application of geofoam in many instances.

Geofoam Monitoring for the I-15 Reconstruction Project

The widespread use of geofoam on the I-15 Reconstruction Project has generated broad interest in using EPS as a lightweight fill. An extensive program of instrumentation and field observation of geofoam embankments at 3300 South Street, I-80 connection with I-15 and 100 South Street was initiated (Figure 1). The objectives of the field instrumentation program are to: (1) monitor the construction and long-term settlements, (2) compare the settlement performance of geofoam and earthen embankments, (3) observe the vertical stress distribution in a pavement section underlain by geofoam, and (4) obtain data for calibrating/evaluating numerical models. Data collection and research regarding these arrays is on going and is a cooperative effort between the UDOT Research Division, Geofoam Research Center at Syracuse University and the University of Utah.

The remainder of this paper presents the instrumentation data gathered from a monitoring array placed in a geofoam embankment and wall constructed near the I-15 northbound off ramp to 3300 South Street between I-15 mainline stationing 25+207 to 25+417 m. At this location, a large geofoam embankment and wall were constructed as the north side approach to a Union Pacific Railroad overpass (Figure 4). The geofoam embankment shown in Figure 4 is approximately 210 m long and varies in full height from 8.3 to 5.8 m. Approximately 12,000 m³ of geofoam was placed at this location and is the largest usage of geofoam on the I-15 Reconstruction Project.

Subsurface Conditions at 3300 South Street Geofoam Wall

Figure 5 is a cone penetrometer (CPT) sounding that was completed in 1996 during the baseline geotechnical study near the face of the current geofoam embankment. The CPT data shows that

the alluvium is approximately 4 m thick and consists of silty sands, silts, and minor amounts of clayey soils. Groundwater is generally found about 2 m below surface. Underlying the alluvium, at a depth between 4 and 9 m, is clayey silt - silty clay layer that comprises the upper Bonneville Lake deposits. The upper Lake Bonneville deposits continue in the interval between 9 to 11.5 m as interbedded sand and silt layers. The lower Lake Bonneville deposits between a depth of 11.5 and 19 m predominately consist of clayey silt and silty clay deposits of lacustrine origin and are very similar to the upper Bonneville Lake deposits. Below about 19 m is a thick sequence of Pleistocene alluvium that predates the Lake Bonneville Deposits and is generally comprised of dense sand and fine gravel. Subsurface investigations and field performance monitoring in the northern Salt Lake Valley have shown that the upper and lower Lake Bonneville deposits are the most compressible layers in the profile. At this site, the upper Lake Bonneville deposits (Figure 5) have an average compression ratio of 0.21 and pre-consolidation stress of about 375 kPa. The lower Lake Bonneville deposits have an average compression ratio of about 0.24 and a pre-consolidation stress of about 450 kPa.

Design of the 3300 South Geofoam Wall

The I-15 Reconstruction Team specified geofoam with no more than five percent re-grind content. Although both Type VIII and Type II geofoam (ASTM C-578) were approved, only Type VIII geofoam (minimum 18 kg/m³ and nominal 20 kg/m³ density) was used as lightweight fill. The installed blocks were 0.83 m high by 1.26 m wide by 4.92 m long. The blocks were used, as manufactured without trimming. The design specifications called for a nominal compressive resistance of 90 kPa (at 10 percent strain) for Type VIII geofoam as per ASTM-C-578. Testing performed by Syracuse University at a strain rate of 10 percent per minute on a series of standard 50 mm side cube samples indicate the density consistently exceeded the minimum of 18 kg/m³ or 90 percent of nominal for Type VIII geofoam (Bartlett et al., 2000). Corrected initial Young's moduli from these tests were in the range of 2.9 to 5.1 MPa. The compressive strength at adjusted 5 and 10 percent strain were on average 97 and 111 kPa, respectively, with both exceeding the specification requirement.

Global stability improvement was the primary reason for geofoam use at 3300 South Street. Because of the height and extent of the embankment, slope stability calculations suggested that the safety factor against shear failure in the foundation soils was below the desired minimum value of 1.3. Thus, if this embankment were to be constructed conventionally, it would require PV drain installation, basal geotextile reinforcement, staged embankment construction, surcharging, and a long construction hiatus to complete primary settlement before the overpass structure could be constructed. The planned construction sequencing and schedule in this area could not accommodate lengthy waiting periods between embankment construction stages; therefore geofoam was selected at the 3300 South Street site to save time. Potential collateral settlements of the Union Pacific Railroad line and adjacent properties were additional considerations in favor of geofoam placement.

Geofoam can experience significant post-construction settlement (i.e., creep) if overstressed by long-term loads. The geofoam design for the I-15 project aimed to limit stresses due to dead load and live load to less than 30 and 10 percent, respectively, of the compressive strength. The geofoam compressive strength was determined on the basis of a corrected stress-strain curve for

a 50 mm cube sample loaded at 10 percent strain per minute. This criterion has been shown to be equivalent to an alternative criterion of a limit stress at 1 percent corrected strain also on the basis of a standard test on 50 mm cube samples. The I-15 design criteria generally emulated these past approaches for limiting long term creep settlement of geofoam to 1 percent or less.

Transition zones must also be carefully designed. Transition zones straddling geofoam and MSE embankments may experience different rates and amounts of construction and long-term settlements. For the 3300 South Street area, geofoam blocks were placed directly behind the pile-supported abutment of the overpass structure at the south end of the embankment. The pre-cast concrete fascia wall rests on a strip footing. The geofoam embankment transitions into an MSE embankment at its northern end at 3.5 horizontal to 1 vertical stepped inclination (Figure 4).

For the I-15 project, the MSE walls and embankments were surcharged (i.e., pre-loaded or pre-consolidated) to limit post-construction settlement. MSE wall/geofoam transition zones were also designed based on restricting the long-term applied loads to levels below the pre-consolidated stress in the foundation soils, so as not to trigger primary consolidation settlement in the transition zones. Some transition zones on the I-15 project had a significant increase of the roadway grade. For these cases, scoria (i.e., a pumice-like, crushed volcanic light weight rock) was used as backfill. The scoria density is about half of compacted earth fill. The placement of scoria in transition zones significantly decreased the embankment loading transferred to the foundation soils. However, scoria was not used in the transition zone shown in Figure 4 at the 3300 South Street geofoam wall. The MSE wall in this location was constructed in two stages with conventional granular backfill and surcharging.

The I-15 designers used 2-D settlement models based on consolidation properties obtained from laboratory tests to estimate the construction settlement of the foundation soils. Figure 6 shows the construction settlement estimated as a result of placement of the pavement structure (i.e., untreated base and concrete pavement) for a MSE wall cross-section immediately adjacent to the northern end of the 3300 South Street geofoam embankment. The estimated settlement profile takes into account that the foundation soils have been pre-load by surcharging. Thus, the calculated settlement is assumed to be solely due to recompression of the foundation soils. The predicted settlement at this locale is relatively small with about 20 mm of foundation settlement expected at the face of the MSE wall. No compression of the embankment material is included in this calculation.

Instrumentation for 3300 South Street Array

The I-15 Reconstruction Project offered an excellent opportunity to document the construction and long-term performance of geofoam embankment. This case history documents the background and performance to date of this large geofoam embankment. Available information on long-term settlement performance of geofoam embankments is very limited. The record for the geofoam embankment at the Lokkeberg bridge in Norway show an average strain of about 1 percent during a ten year period (Aabøe, 2000). Creep tests on small geofoam block samples indicate time-dependent deformations remain relatively small for stress levels corresponding to

about 1 percent strain for a standard test on 50 mm cube samples loaded at 10 percent strain per minute.

The 3300 South Street geofoam array consists of instrumentation and elevation surveys that measure the vertical deformation and pressure within the geofoam / foundation soil / pavement system. Two instrument arrays were installed at the 3300 South Street northbound off ramp, one array was installed at mainline stationing 25+347 m and the other at mainline stationing 25+315 m. At stationing 25+315, nine levels of geofoam were placed, making the total height of the geofoam approximately 7.3 m. Eight levels of geofoam were placed at stationing 25+347, making the total height of geofoam approximately 6.6 m. Figure 7 is a typical drawing of the instrument layout used in these array. Each array consists of magnet extensometers, vibrating wire (VW) total pressure cells and survey points. Magnetic extensometers were used to monitor settlements of the foundation soil and deformations of the geofoam at two blocks interval (Figure 7). Each extensometer stand-pipe (Figure 8) was placed 2.4 m from the wall face and the 1.25-cm thick base plates are supported in the sand-leveling course below the first layer of geofoam. (The thickness of extensometer plates has been reduced to minimize stress concentrations and local movement at other projects.) The road base thickness varies from 1 to 1.4 m. Steel monitoring well casing has been installed to provide a flush mounted access port to the PVC casing of each extensometer column. Settlement within the geofoam with time is detected by measuring changes in the depth to the magnets by a graduated tape and sounding probe that is inserted down the PVC standpipe.

Vibrating wire total pressure cells were installed in the base sand below the first level of geofoam block, approximately midway in the geofoam fill, at the top of the geofoam fill, above the load distribution slab and immediately below the concrete pavement (Figure 7). The allowable pressure range for the pressure cells is 0 to 170 kPa. The basal VW pressure cells were covered with approximately 25 mm of bedding sand. The pressure cells within the geofoam were placed between block layers and a groove was cut out within the geofoam to accommodate the pressure transducer and cable (Figure 9). A thin veneer of bedding sand was used to cover the pressure cells placed between geofoam blocks. A sheet of plastic was placed as a “bond breaker” for the top pressure cells just below the load distribution slab.

Survey points (lead plugs) were installed in the pavement to monitor settlements with time. Three rows of survey points were established, each paralleling mainline and placed relatively close to the face of the geofoam wall. One row was placed along the base of the concrete barrier, one along the inside edge of the moment slab, and one along the outside edge of the emergency lane. Surveying was done with a high precision self-reading level and rod. All surveys were established from a stationary off-site benchmark, checked for closure, and adjusted according to standard surveying practice. The baseline survey was completed about 3 months after the concrete pavement was placed. Construction operations in the area prohibited an earlier placement of the survey points.

Construction Related Settlement

Approximately 60 to 70 mm of construction related compression occurred within the geofoam mass during construction of the embankment at stations 25+347 and 25+315, respectively

(Figures 10 and 11). This compression occurred as the load distribution slab, untreated base course and Portland cement concrete pavement were placed. The compression is a result of elastic compression of the geofoam and seating. Seating compression is caused by the closing of a slight arch or crown that is characteristic of untrimmed geofoam blocks. Due to shrinkage during curing, blocks warp slightly and were placed concave down at all times. This practice allowed for a relatively close fit of the blocks along the side margins, and the slight horizontal gaps gradually closed under the weight of the overlying load distribution slab and pavement structure.

Unfortunately, the construction settlement was sufficient to damage fixed connections that tied the tilt-up panel wall to the load distribution slab. The tilt-up-panel-fascia wall was tied to the load distribution slab by threaded reinforcing bars that were held together by threaded couplers. At the 3300 South Street wall, this connection was made after the load distribution slab was placed, but before the pavement structure was added. The subsequent settlement proved to be too large for the relatively rigid connection, which was severed at several locations along the wall. The broken connections were repaired by drilling through the face of the panel wall at the elevation of the load distribution slab and setting epoxied anchor dowels into the load distribution slab. Subsequently, the connection was redesigned to accommodate differential movements between the geofoam and the precast concrete fascia wall

Post Construction Observations

Vertical stress measurements from the VW pressure cells are shown in Figure 12. The recorded stresses vary and diminish with depth. The average stress developed in the geofoam at two pressure cells placed near the middle of the geofoam mass atop levels 5 and 6 at stations 25+347 and 25+315 m, respectively, show measured stresses of 45 and 30 kPa, respectively and are relatively immune to the strong seasonal cycling seen in other cells positioned atop and above the geofoam. Vertical stress measurements from pressure cells placed at levels 8 and 9 at station 25+347 and 25+315, respectively, show strong seasonal cycling of pressure. These pressure cells are located above the geofoam embankment, immediately under the concrete load distribution slab. The cyclic behavior may be due to loading-unloading cycles resulting from thermal expansion and contraction of the overlying concrete. Similarly, measurements from pressure cells placed above the load distribution slab show this same cyclic pattern, but it is markedly more severe. The pressure cells placed within the bedding sand (level 0) are recording vertical stresses of about 5 and 15 kPa for stations 25+347 and 25+315 m, respectively. Data from the basal pressure indicate a notable decrease in vertical stress is occurring near the bottom of the geofoam embankment. This may be a consequence of the interaction of the relatively narrow geofoam base layer (4.9 m wide) with the adjacent sloped embankment and will be examined separately.

After 2 years of post-construction monitoring, the geofoam has compressed about 13 and 15 mm at stations 25+347 and 25+315, respectively (Figures 10 and 11). The settlement record also shows seasonal change as noted in the pressure cell observations. Geofoam is a better insulator than soil. Pavement sections in geofoam treated areas tend to be colder and hotter in winter and summer months, respectively, compared to pavements over earth embankments. The extent of seasonal deflection diminishes with depth but is up to 10 mm in the upper portions of the

geofoam embankment. Extrapolation of the magnet extensometer data suggests that about 0.25 percent strain may occur during the first 10-year post-construction period. Thus, based on these considerations, the creep settlement may be of the order of 25 mm by year 2008. This expected settlement is appreciably lower than the 75 mm expected in the same period for corresponding conventional embankments and MSE walls. Monitoring will continue over the coming years to verify the projected creep settlement.

Settlement measurements from three rows of survey points are shown in Figure 13. Because the survey points are set in the pavement surface, the reported settlements are a combination of compression within the geofoam and settlement of the foundation. These data show that the geofoam embankment and underlying foundation soils have settled up to 23 mm where the geofoam embankment is at full height (sta. 25+340 to 25+400). Settlement in the transition zone (sta. 25+400 to 25+420) ranges from about 11 to 21 mm. The adjacent MSE wall has settled less than 15 mm. The differential settlements that developed in the transition zone are consistent with observations in the geofoam and MSE wall end segments. The post-construction survey data also show that the face of the geofoam wall is settling less than the inside edge of the moment slab, suggesting a slight inward rotation of the moment slab relative to the face of the wall. This may be partly the result of the tilt-up panel wall resisting some downward movement at the face of the wall due to the rigid connection with the load distribution slab. Also, it may be partly due to a decrease in vertical stress near the face of the wall due to the 2-D edge effect of the free face. This in turn would tend to produce less compression of the underlying geofoam near the face and more transfer of vertical pressures towards the inclined embankment face. Such a mechanism may have contributed to the lower pressures registered by the base stress cells.

Conclusions

Geofoam fill has been successfully used on the I-15 Reconstruction Project to mitigate primary consolidation settlement near buried utilities, improve foundation stability for high approach embankments and to expedite construction in time critical areas. Field performance data gathered at the 3300 South Street geofoam arrays demonstrate adequate settlement performance to date. The data validate design criteria regarding allowable loads, creep deformation and differential settlement.

Magnet extensometer data gathered from two locations in the 3300 South Street area show that about 1 percent construction-related strain can occur in the geofoam mass during placement of the overlying load distribution slab and pavement structure. This initial strain mostly results from seating of the geofoam blocks and elastic compression. If not properly accounted for, this strain is sufficient to damage rigid horizontal connections that tie the fascia panel wall with the geofoam embankment.

In general, pressure cell data show that vertical stresses vary and diminish with depth. This effect is probably due to interaction with the adjacent sloping embankment and the presence of a freestanding face. Pressure cells above the geofoam indicate seasonal change that may be due to temperature inversion.

Projection of the creep deformations so far observed suggest that about 0.25 percent creep strain will develop over a 10-year post-construction period. Long-term settlements in the geofoam treated areas may be less than MSE segments.

The transition zone between geofoam and MSE segments was constructed on a 3.5 horizontal to 1 vertical slope. This degree of side slope appears to be adequate and has not produced significant differential settlements at the 3300 South Street array. However, care must be exercised in designing the height and extent of the surcharged MSE wall section so as not to trigger primary consolidation settlement in the transition zone. In some cases, this may require the placement of lightweight backfill, such as scoria, in the transition zone and in the immediately adjacent portions of the MSE wall.

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Embankment or Treatment Type	Approximate Quantity	Average In Place Unit Cost
Earthen Embankment	5,00,000 m ³	\$9 / m ³
Pre-fabricated Vertical Drains	7,400,000 m	\$1.50 / m (without pre-drilling) \$3.00 / m (with pre-drilling)
High Strength Geotextile	670,000 m ²	\$12 / m ²
Geofoam Embankment (Type VIII)	107,000 m ³	\$60 / m ³ (without wall) \$70 / m ³ (with wall)
Surcharge Fill Removal	500,000 m ³	\$6 / m ³
Slag Light Weight Aggregate	141,000 m ³	\$18 / m ³
Scoria Light Weight Aggregate	50,000 m ³	\$31 / m ³
Lime Cement Column Treatment	68,000 m	\$16 / m (0.6 m diameter) \$18 / m (0.8 m diameter)
Mechanically Stabilized Earth (MSE) Walls	160 walls	\$200 / m ² face of wall (one-stage) \$300 / m ² face of wall (two-stage)

Table 1. Foundation treatments and embankment used on the I-15 Reconstruction Project with approximate quantities and unit costs (adapted from Saye et al., 2001).

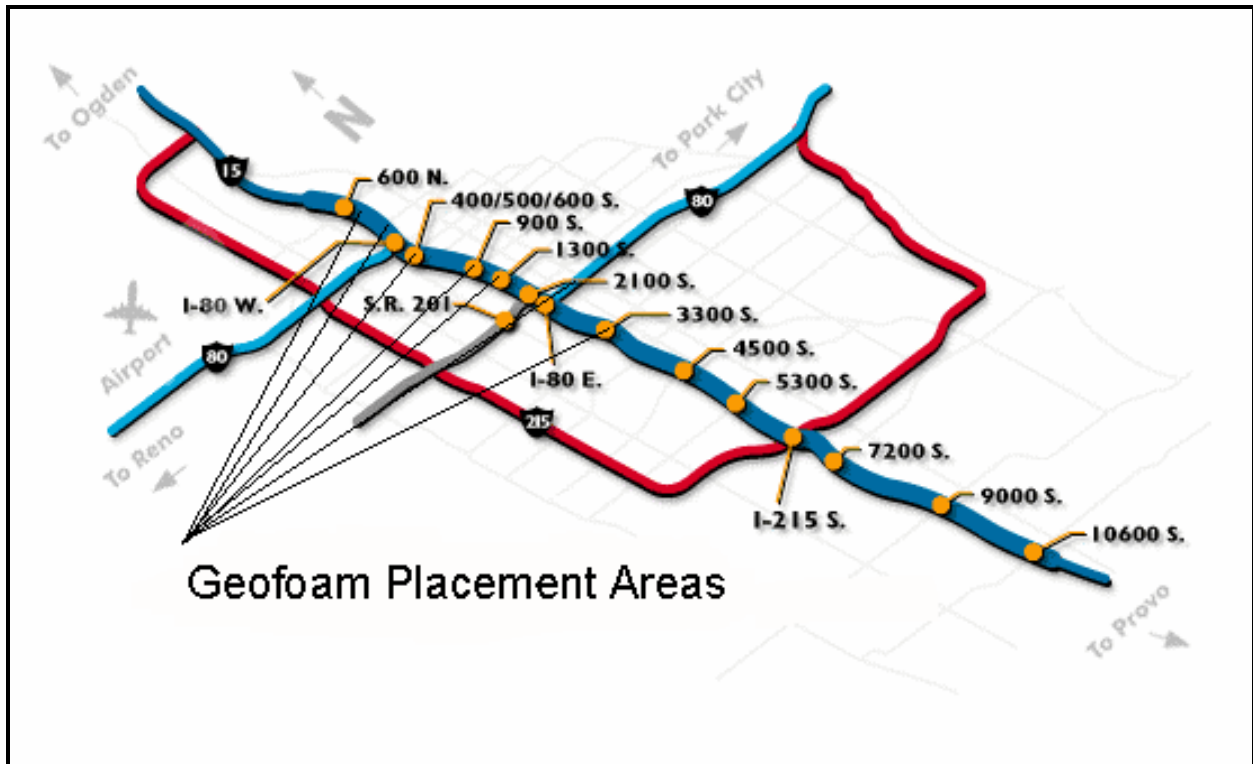


Figure 2. Geofoam placement areas on the I-15 Reconstruction Project.

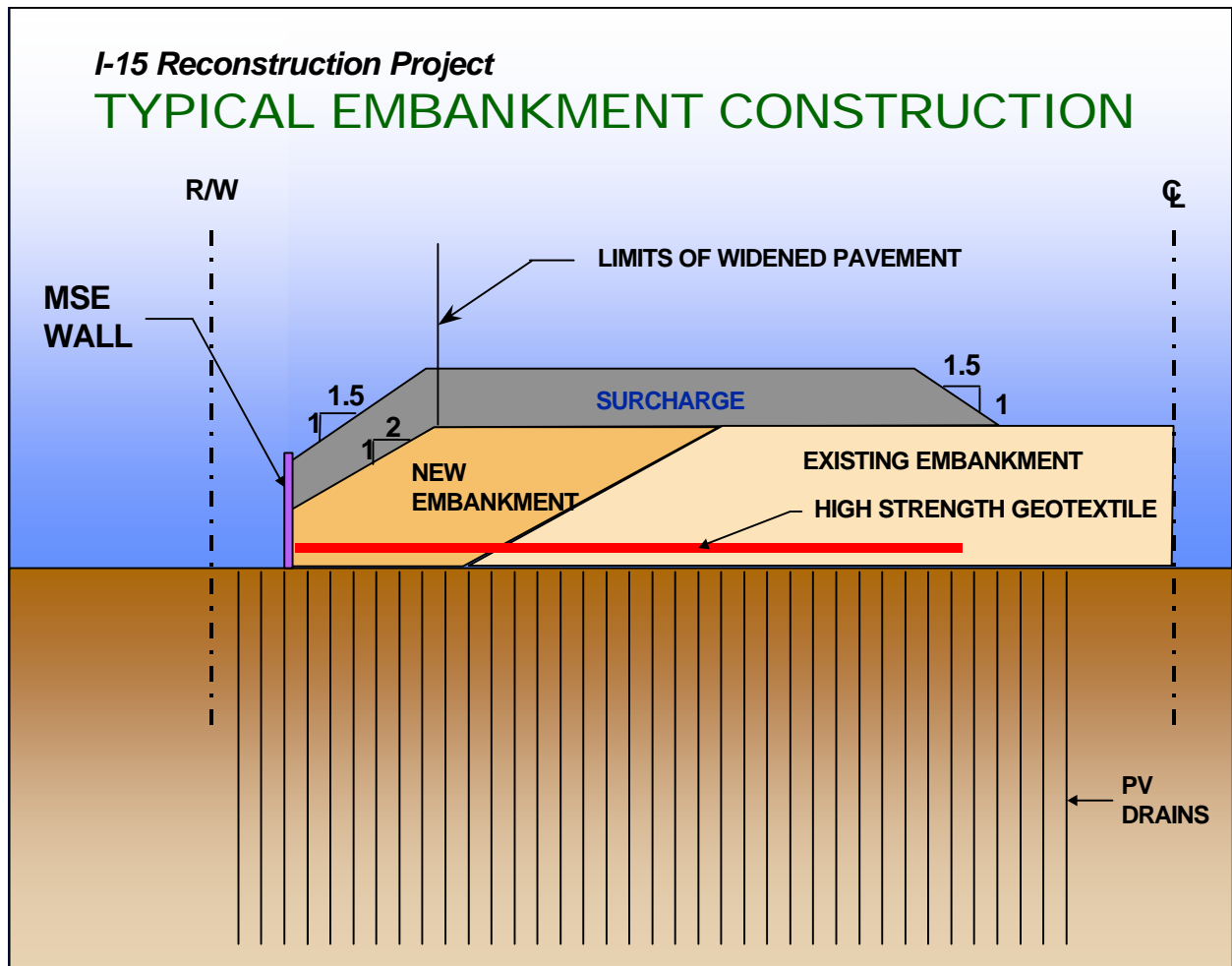


Figure 2. Typical mechanically stabilized earth (MSE) embankment construction, I-15 Reconstruction Project.

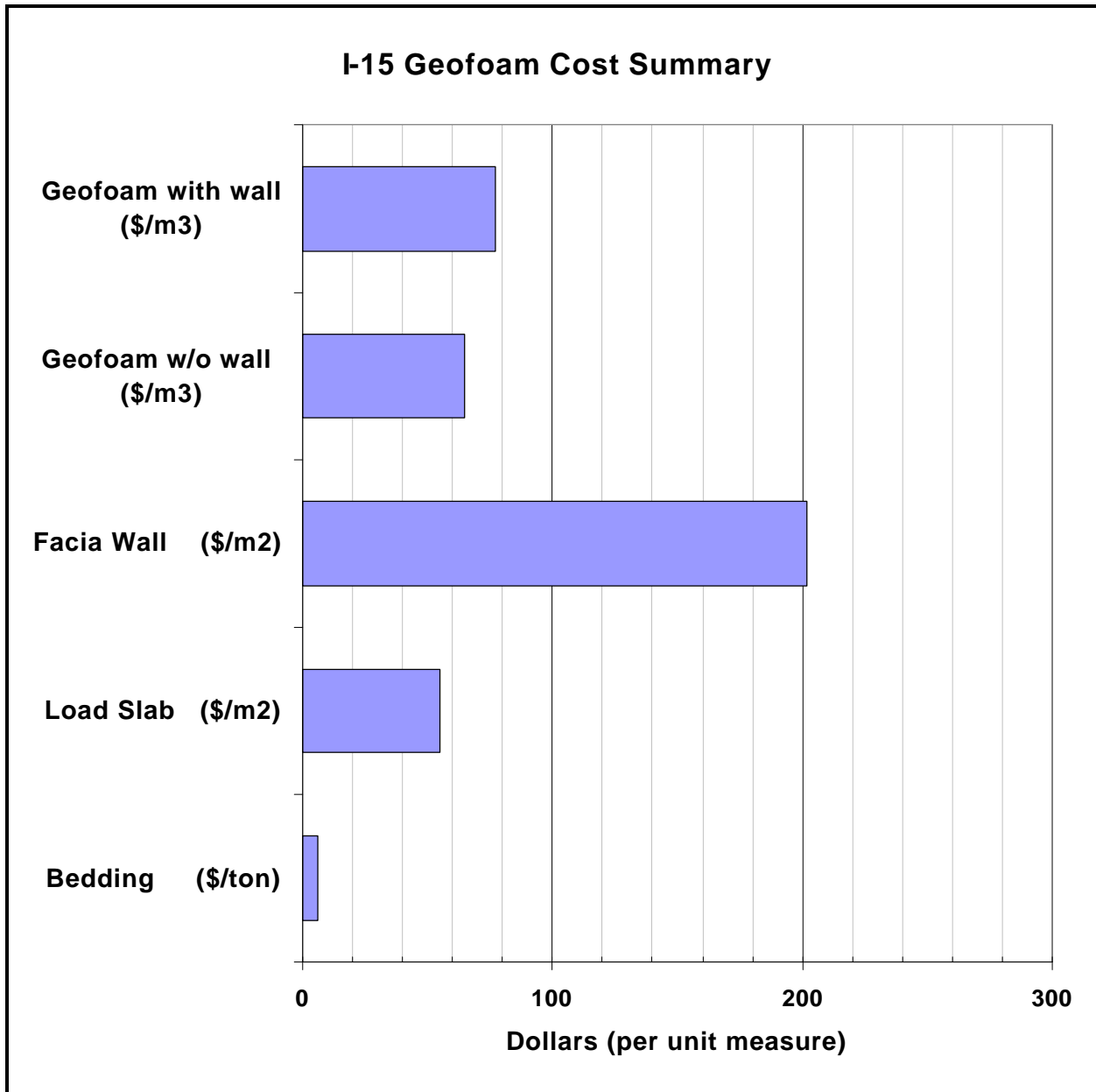


Figure 3. Geofoam Construction Cost Summary for the I-15 Reconstruction Project, Salt Lake City, Utah, 1996 - 2000.



Figure 4. Geoforembankment construction near the 3300 South Street Northbound off ramp. The far end (i.e., south) of the geoforembankment has been placed against the abutment of the Union Pacific Overpass. The closest end (i.e., north ends) transitions into a MSE wall. Tilt-up wall panel placement has begun on the south end of the embankment.

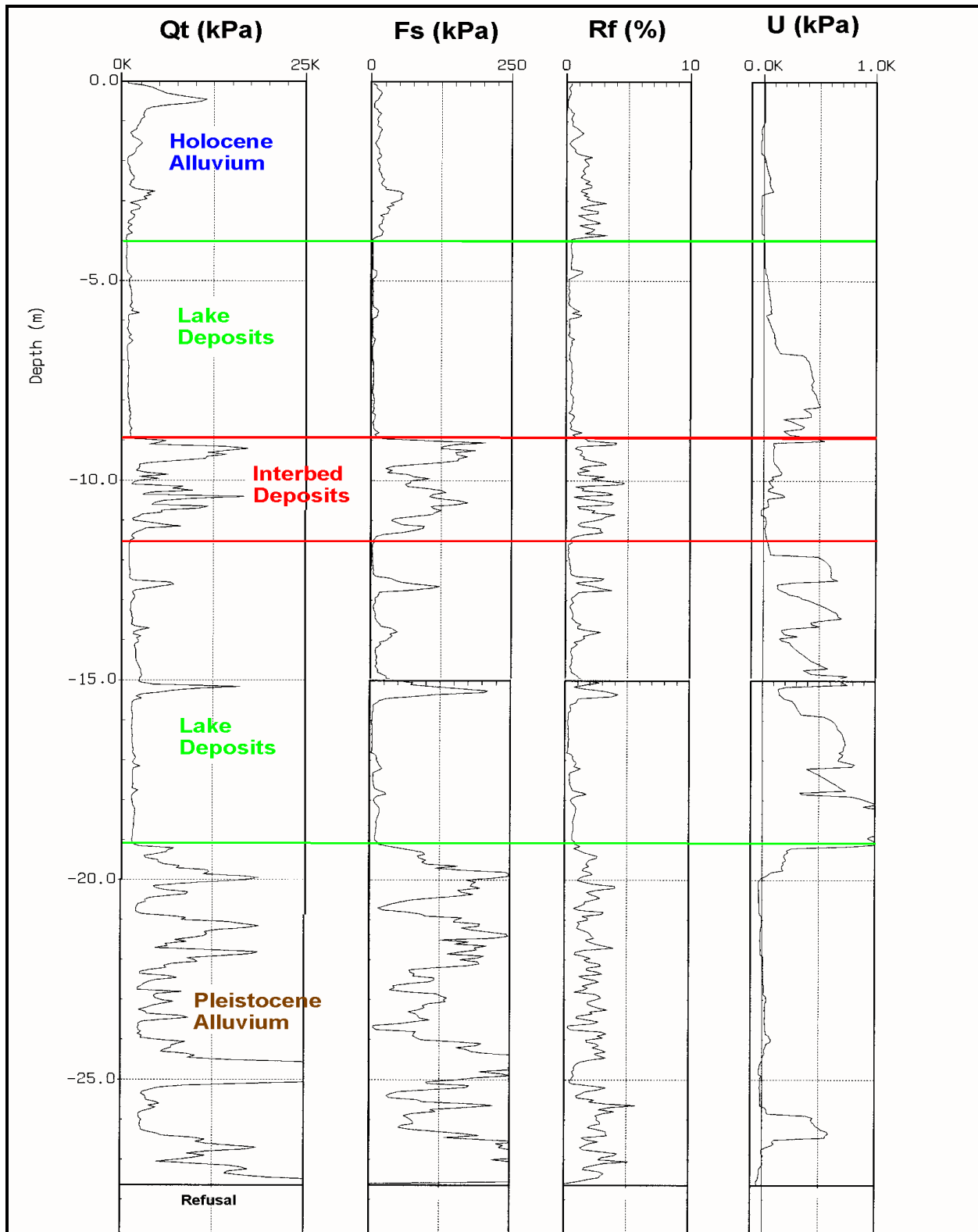


Figure 5 - CPT Sounding 33SC21 at 3300 South Street geofoam array showing layers in subsurface.

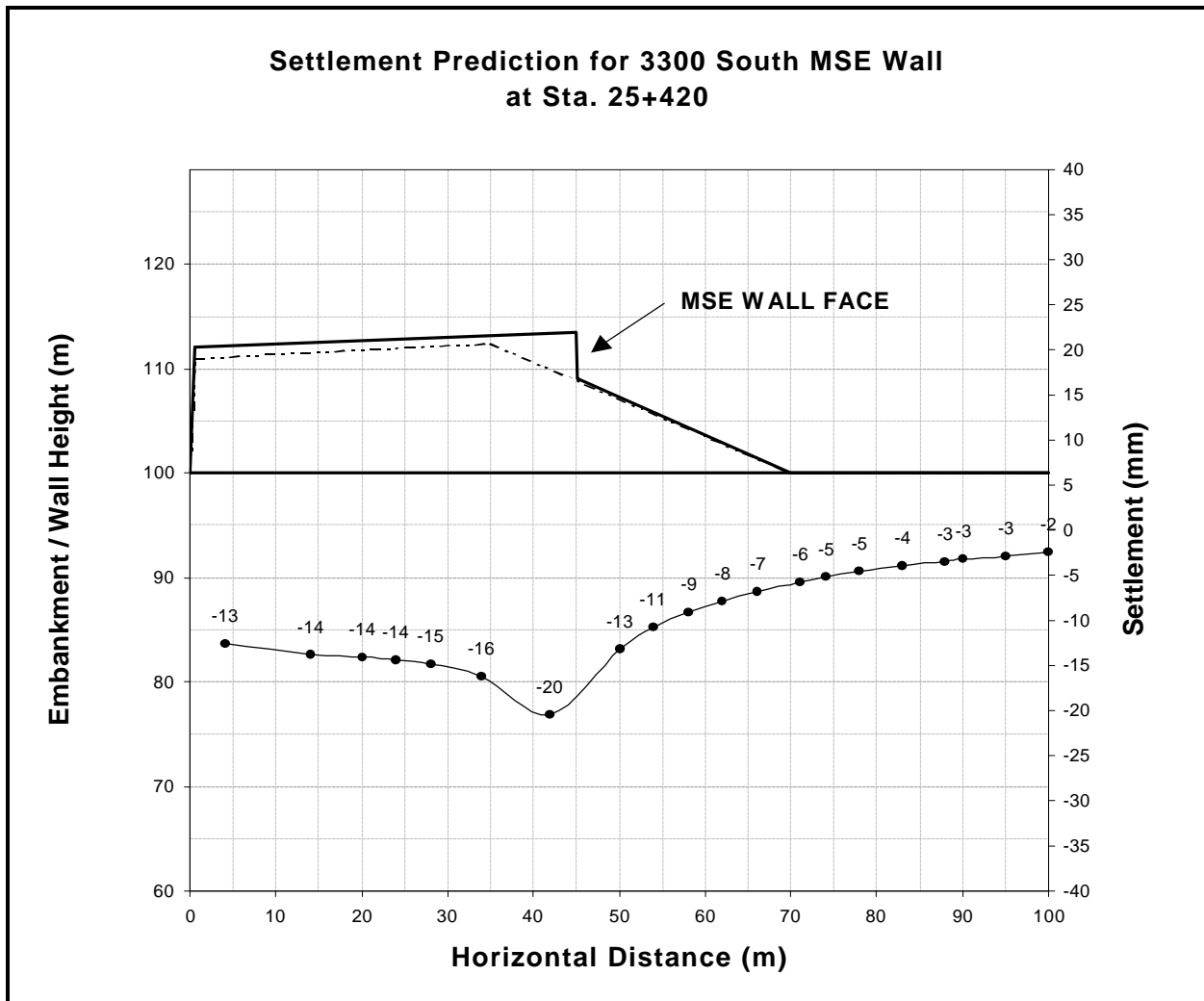


Figure 6. Construction (i.e., primary) settlement prediction for MSE wall at 3300 South Street at Station 25+420. Settlement calculation assumes no primary consolidation settlement (i.e., all settlement occurs in recompression).

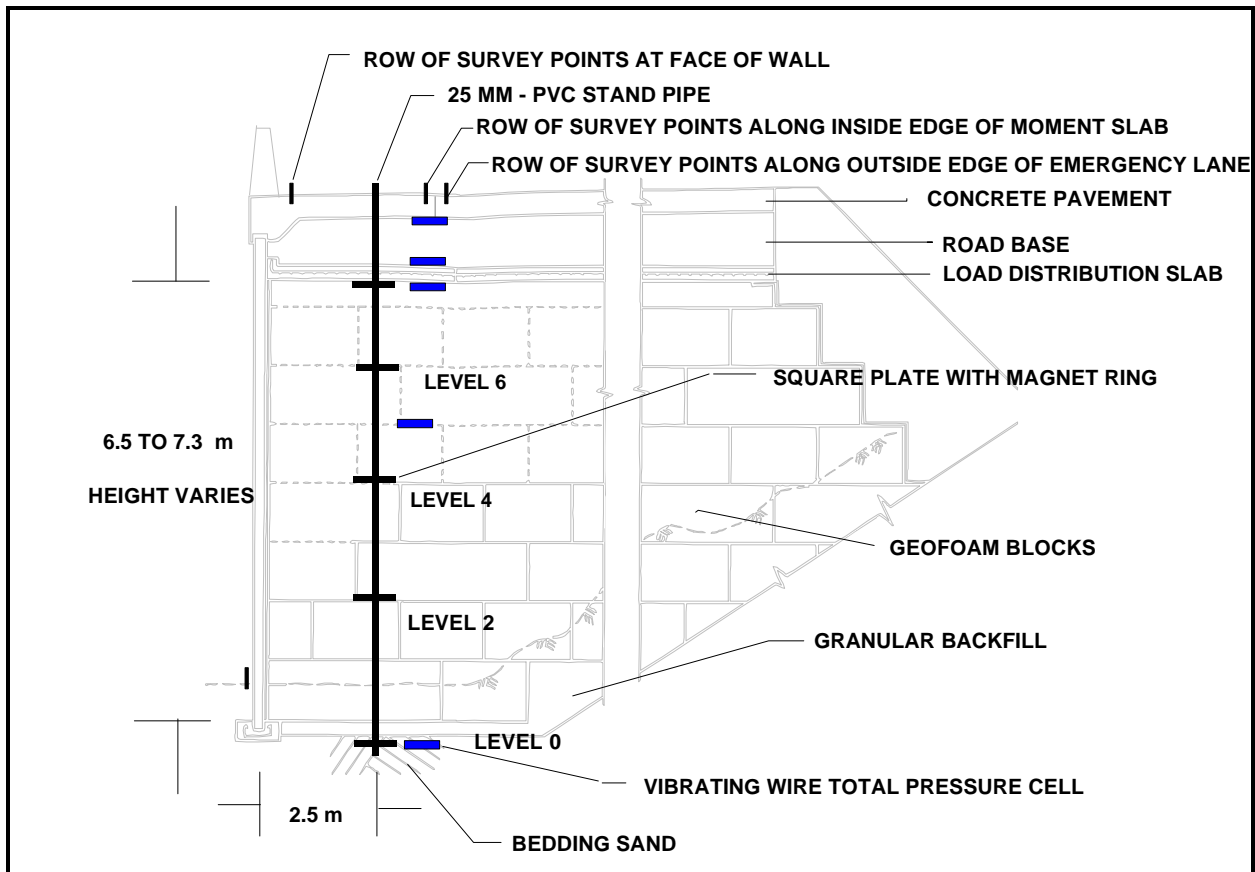


Figure 7 - Cross-sectional view of typical geofram construction and layout of magnet extensometer and survey points in geofram embankment at 3300 South Street Geofram Array.



Figure 8. Magnet extensometer installation within the geofabric embankment. Note the plastic plate with ring magnet has been placed atop the geofabric block. The plate is subsequently covered by additional geofabric layers.

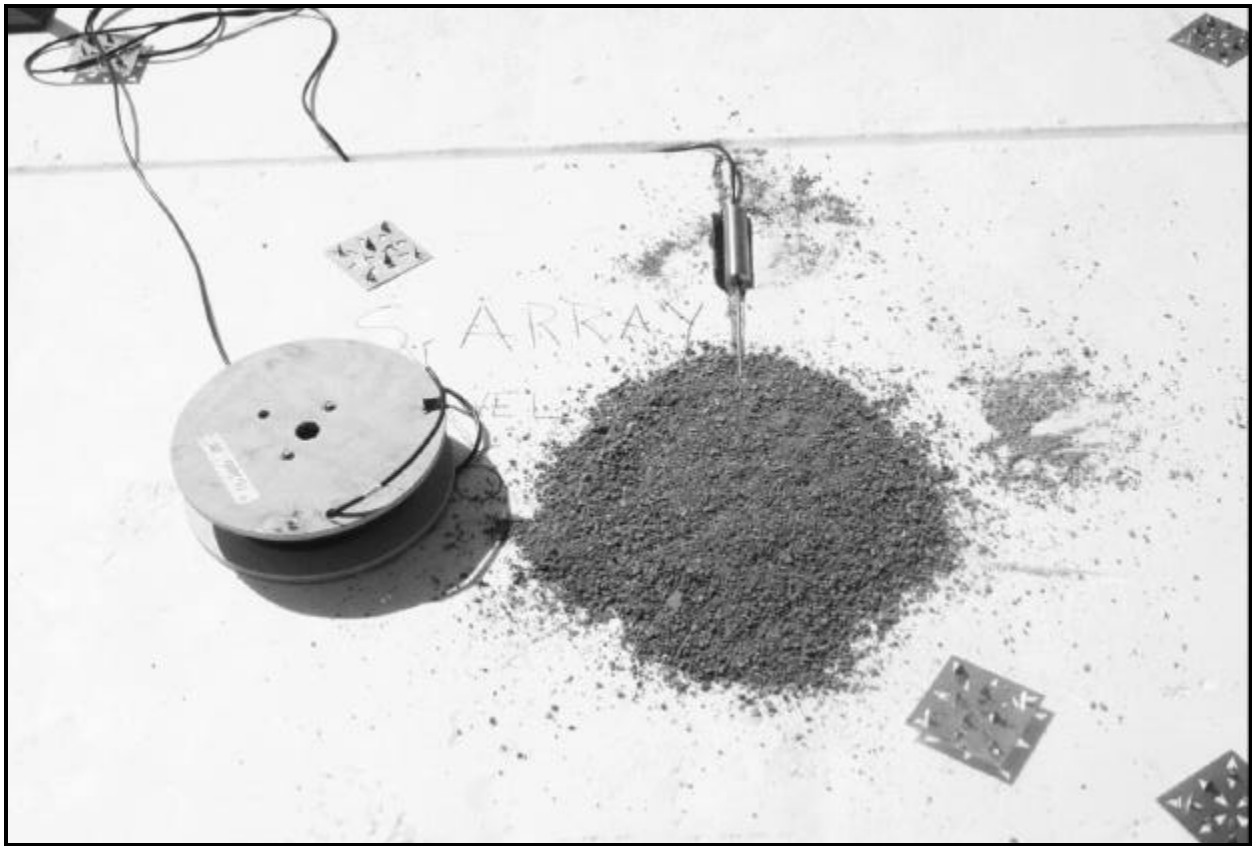


Figure 9. Installation of VW pressure cell in geofam embankment at station 25+315, level 6. Bedding sand was used with a thin veneer of soil atop the pressure to better distribute the pressure of the overlying block. Also a narrow groove was cut to house the pressure transducer and cable so that these items did not interfere with load transfer to the cell face.

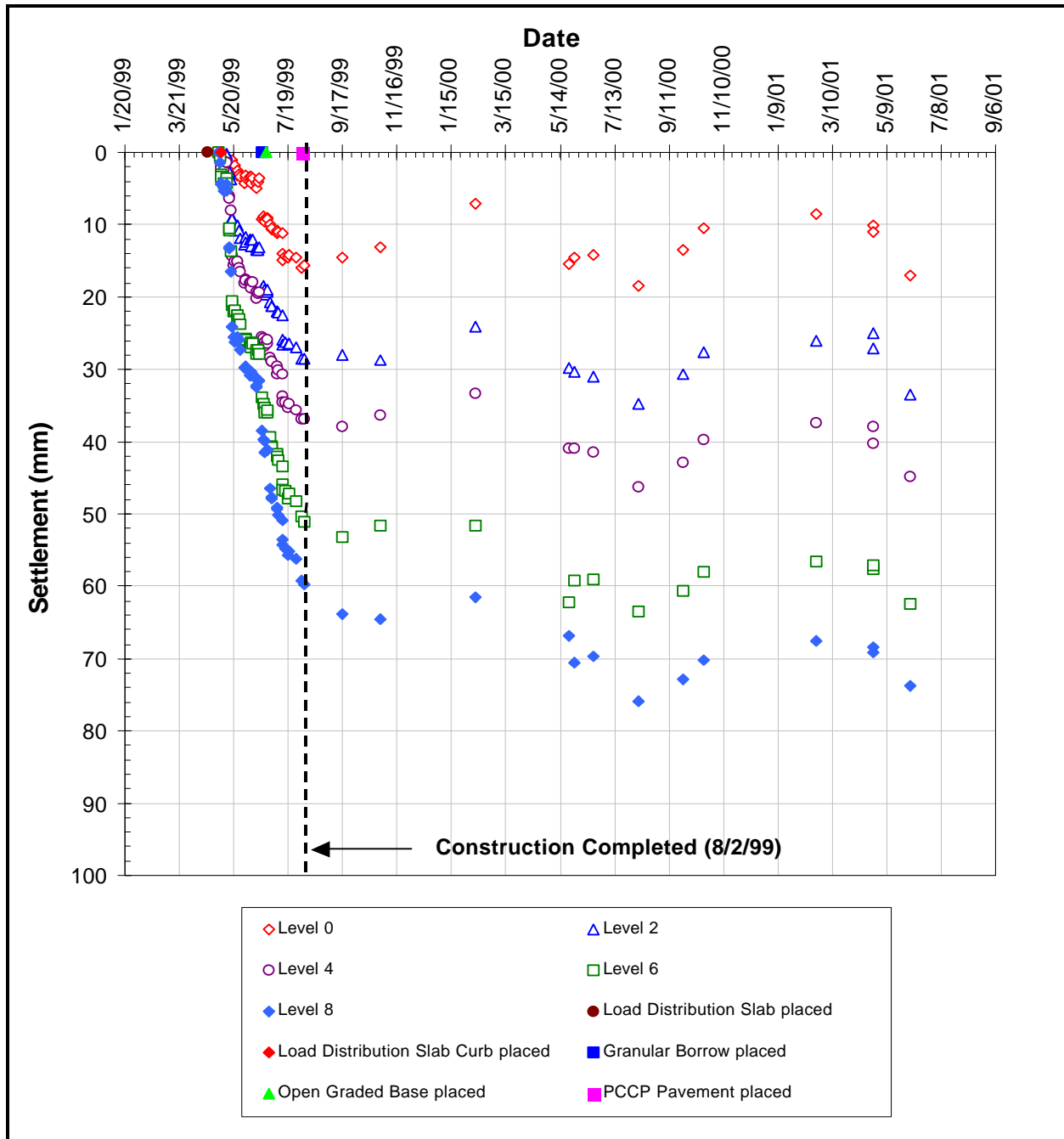


Figure 10. Settlement versus time measurements for 3300 South Street Geofam Array located at Station 25+347 m, right 36.5 m.

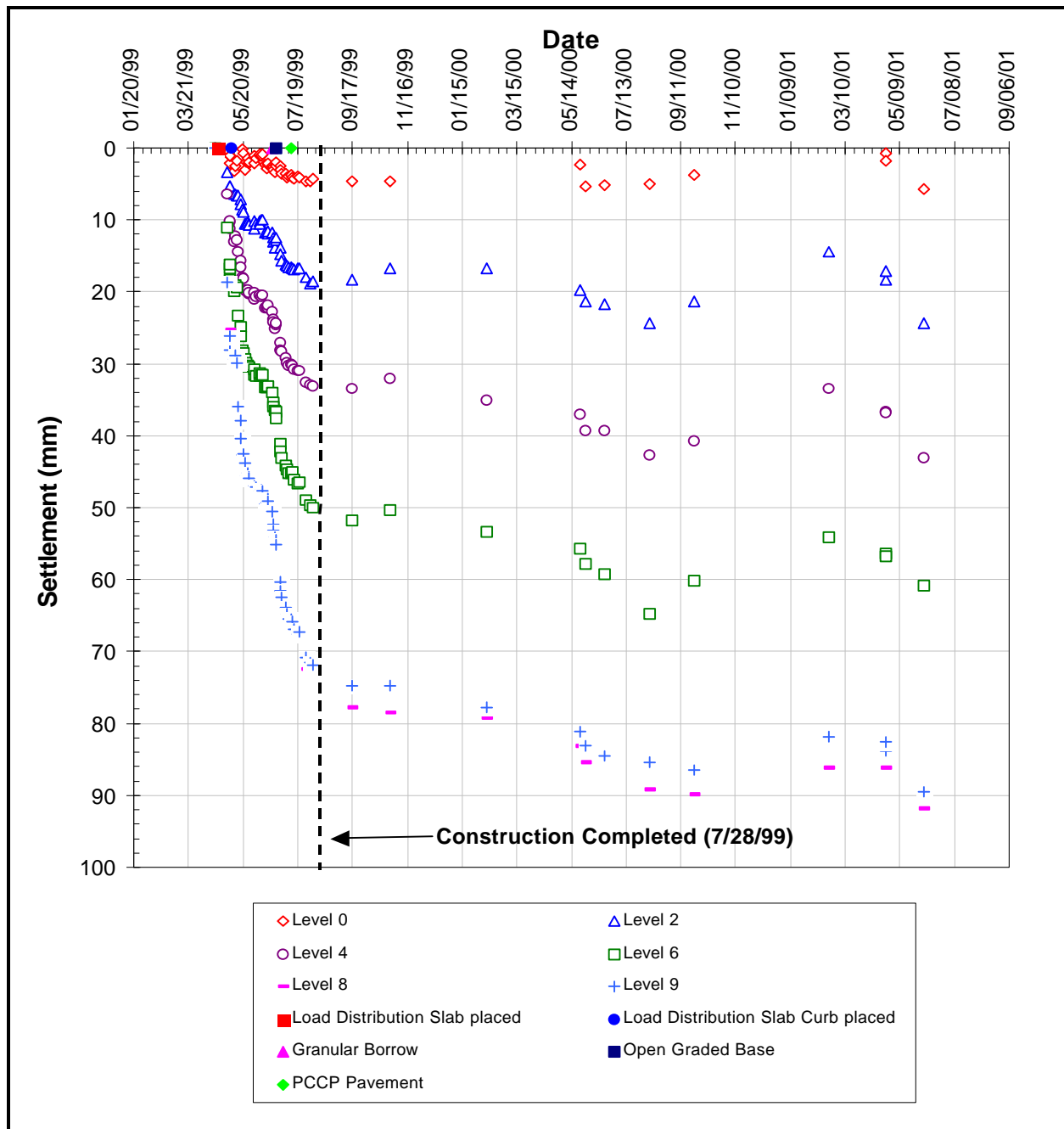


Figure 11. Settlement versus time measurements for 3300 South Street Geofom Array located at Station 25+315 m, right 35 m).

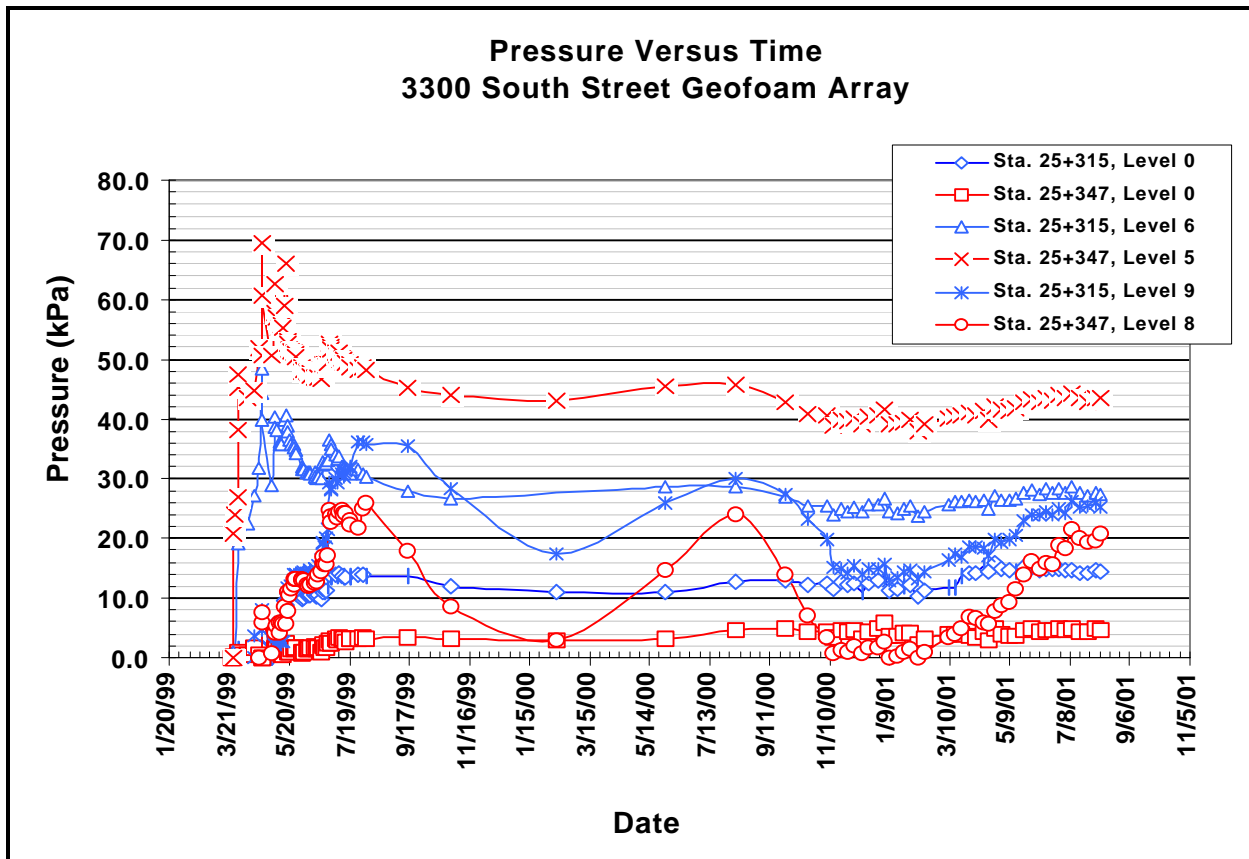


Figure 12. Vertical stress measurements for the VW pressure cells at the 3300 South Street Array.

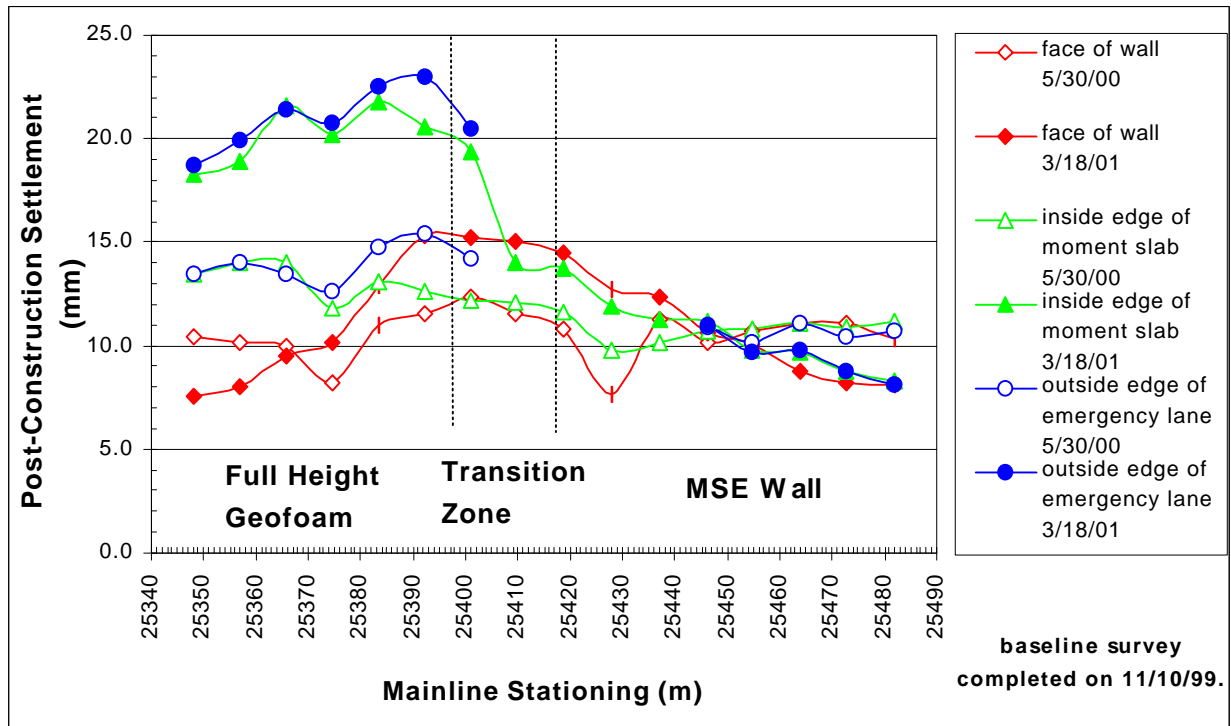


Figure 13. Settlement profiles of post-construction pavement settlement atop the 3300 South Street Geofoam Array. Baseline pavement survey was completed on 11/10/99. This is approximately 3 months after construction had been completed on 8/2/99. See Figure 7 for the approximate location of the rows of survey points.