Construction and Long-Term Performance of Transportation Infrastructure Constructed Using EPS Geofoam on Soft Soil Sites in Salt Lake Valley, Utah

Bartlett, Steven¹, Negussey, Dawit², Farnsworth, Clifton³, Stuedlein, Armin⁴

Abstract

The Utah Department of Transportation (UDOT) and the Utah Transit Authority (UTA) have made extensive use of EPS embankment for several major interstate, light rail, and commuter railway embankments in Salt Lake Valley, Utah. Constructed between 1998 and 2001, the Interstate 15 Reconstruction Project involved the widening of interstate embankments within a 27 km narrow corridor with limited right-of-way. Approximately, 100,000 m³ of EPS fill was placed at various locations to minimize postconstruction settlement of deep, compressible lake This paper discuss the construction and long-term performance of the EPS fill deposits. constructed for the I-15 Reconstruction Project and compares the performance of these fills with other geotechnologies. Geofoam embankments had the best overall settlement performance of the technologies monitored. Gap closure and deformation of the geofoam embankment due to placement of the load distribution slab and overlying roadway materials was about 1% of the embankment height. In addition, at 3300 South Street, the foundation soil settled about 15 mm due to the placement of the embankment and overlying loads and the face of the embankment settled an additional 25 mm in a 5-year period due to the placement of a 1.5-m toe berm placed at the outside base of the tilt-up wall. Total postconstruction settlement (foundation settlement and geofoam creep) is expected to be about 50 mm at the wall face for a 10-year postconstruction period. The trend of postconstruction settlements suggest that geofoam embankments are performing as designed and will meet the 50-year postconstruction deformation limit of 1% creep strain.

Overview of the I-15 Reconstruction Project

The Utah Department of Transportation (UDOT) together with Wasatch Constructors finished the \$1.6 billion reconstruction of Interstate 15 (I-15) that runs through Salt Lake City, Utah in 2001 just prior to the start of the 2002 Winter Olympics. The interstate embankment was widened from eight lanes to ten general-purpose lanes, plus two high occupancy vehicle lanes, within an existing north-south alignment that runs through the middle of the densely populated Salt Lake Valley. In this area, parts of the I-15 alignment are located atop extensive deposits of relatively compressible lake sediments. Collateral consolidation settlement along this widened

¹ Dept. of Civil and Environmental Engineering, University of Utah, ² Dept. of Civil and Environmental Engineering, Syracuse University, ³ Dept. of Civil and Environmental Engineering, University of Texas at Tyler, ⁴ Dept. of Civil and Environmental Engineering, Oregon State University

alignment had to be minimized to prevent differential settlement damage and subsequent disruption of critical buried utilities, buildings and businesses. The project had to be finished before the 2002 Winter Olympics and the available construction time was limited. Fast-paced and sometimes novel construction methods were employed to meet these constrains (i.e., lime cement columns, multi-staged embankment construction with pre-fabricated vertical drains, two-staged MSE walls and the use of light-weight embankment (e.g., EPS geofoam and crushed volcanic scoria) (Farnsworth, et al., 2008; Bartlett et al. 2000).

Long-Term Monitoring

Instrumentation was installed at several locations to monitor the construction and postconstruction settlement performance of these technologies and embankments (Bartlett and Farnsworth, 2004). This paper will focus on the long term monitoring of three EPS embankment locales: (1) 100 South Street, (2) 3300 South Street and (3) State Street.

100 South Street Site

The I-15 Reconstruction at 100 South Street required raising and widening of the existing embankment to the limits of the right-of-way. The geofoam fills in both the north and southbound directions were placed over a 406-mm high-pressure natural gas line and other buried utilities (Figures 1 and 2). The southbound portion of this embankment employed approximately 3,400 m³ of EPS 20, and the height of the embankment decreased southward to conform to the roadway elevation. The embankment height (not including the pavement thickness) decreased from 8.1 to 6.9 meters, corresponding to 10 to 8.5 layers of geofoam blocks, respectively (Figure 1). The geofoam embankment transitions to two-stage MSE walls on both the north and south sides. In this area, the top part of the existing embankment was sub excavated and replaced with scoria fill to raise the roadway grade within the utility corridor without causing primary consolidation in the underlying, compressible, foundation soils.

The instrumentation installed at this location consisted of: (1) basal vibrating wire (VW) total earth pressure cells placed in sand underneath the EPS, (2) horizontal inclinometers (one placed near the base and one near the top of embankment) and (3) two magnet extensometer placed within the geofoam fill (Figures 1 and 2) (Negussey and Stuedlein, 2003). The magnet plates for the extensometers were placed at EPS layers 0, 1.5, 3.5, 5.5, 7.5, 8.5 and 9.5 at the northern (i.e., left) location and at layers 0, 1.5, 3.5, 5.5, 7.5, 8.5, and 9 at the southern (i.e., right) location (Figure 1). All extensometer measurements were referenced to their respective base plate underlying the geofoam fill (and not the top of the riser pipe) hence, these data represent deformations of the geofoam fill with time and do not include any settlement of the foundation soils.

Figure 3 shows the construction and postconstruction strain time history of the southern location as calculated from the magnet extensioneter observations. The basal layers (0 to 1.5 m) underwent 1.8 percent vertical strain by end of construction. The total strain of the EPS

embankment (0 to 9 m) was about 1 percent at end of construction at this same location (Figure 3). Figure 4 shows the construction and postconstruction strain of the entire embankment (0 to 9 m). The vertical strain at the southern location is about 1.5 percent after 10 years of monitoring and is projected to be about 1.7 percent creep strain after 50 years.



Figure 1 Profile view of the EPS embankment and instrumentation at 100 South Street, Salt Lake City, Utah, I-15 Reconstruction Project (Negussey and Stuedlein, 2003).



Figure 2 Cross section view of the EPS embankment and instrumentation at 100 South Street, Salt Lake City, Utah, I-15 Reconstruction Project (Negussey and Stuedlein, 2003).



Figure 3 Construction and post construction strain in EPS measured in southern magnet extensometer, I-15 Reconstruction Project (Negussey and Stuedlein, 2003).



Figure 4 Construction and post construction global strain of entire thickness of EPS embankment, I-15 Reconstruction Project (Farnsworth et al., 2008).

The postconstruction settlement trend of Figure 4 is consistent with the limit 2 percent global strain in 50 years assumed in the I-15 design. Figure 3 shows that the lowest geofoam interval experienced more vertical strain when compared with the relatively uniform strain that occurred

in the overlying layers. It should be noted that the foundation footing for the adjacent panel wall laterally restrains the lowest geofoam layer. As a result, the mean normal stress in the lower geofoam layers is probably somewhat higher than the corresponding states of stress in the overlying geofoam layers. This effect would produce more vertical strain and also suggests that the influence of confinement may need to be considered in future design evaluations, as appropriate.

The loading history and total stress observations from the VW pressure cells indicate that the measured internal vertical stress levels are in reasonable agreement with the design criterion (Figure 5). (The factored working stress design criterion of about 30 kPa for EPS 20 was selected during the design phase to limit the construction compression to 1 percent vertical strain at the end of construction period and to 2 percent compressional strain over a 50 year period.) Thus, we conclude that this design criterion has been validated given the strain measurements presented in Figure 4. However, we note that the 1 percent vertical strain measured during the construction period (Figure 4) does not entirely consist of elastic compression of the EPS block. Undoubtedly, some of this vertical strain has also resulted from small gap closure of the slightly curved block that occurred upon initial loading of the geofoam embankment. Complete seating and gap closure probably did not occur in the geofoam embankment until the final load was in place, which consisted of the combined weights of the load distribution slab (LDS), roadway base materials and pavement. Undoubtedly, a portion of the initial 1 percent strain during loading can be attributed to seating and gap closure (Negussey et al. 2001). This issue and the numerical modeling of these effects and the compressional behavior of the geofoam arrays are further discussed in Newman et al. (2010).



Figure 5 Loading history and total pressure cell measurements at the 100 South Street site (Negussey and Stuedlein, 2003).

3300 South Street Site

The primary reason for use of geofoam at this site was the improvement of the global stability of a relatively high approach fill at a railroad crossing. Due to the height and extent of the embankment widening, design calculations suggested that the safety factor against shear failure in the foundation soils was lower than the required minimum value of 1.3 for short-term stability. 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 abutments for the overpass bridge could be constructed. The planned construction sequencing and schedule for this bridge could not be accommodated using conventional construction, so EPS embankment was used to shorten the construction schedule and meet the required factor of safety.

The 3300 South Street geofoam array consisted of geotechnical instrumentation and highprecision digital level elevation surveys to measure the vertical compression and stresses that developed within the geofoam / foundation soil / pavement system. Two instrument arrays were installed at this site at mainline stationing 25+347 m, and stationing 25+315 m, respectively. 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 6 is a typical cross-section showing the instrument layout used at the 3300 South Street arrays.

Each array consisted of magnet extensometers, VW total pressure cells and survey points. Magnetic extensometers were used to monitor settlements of the foundation soil and deformations of the geofoam at various height intervals. Each extensometer stand-pipe was placed 2.4 m from the wall face and the 1.25-cm thick base plates were supported in the sand leveling course below the first layer of geofoam. 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, immediately above the load distribution slab and immediately below the concrete pavement (Figure 6).

Approximately 70 mm of construction related compression was measured within the geofoam mass during construction of the embankment at station 25+315 (Figure 7), which corresponds to about 1 percent global vertical strain within the entire EPS mass. Similar strains were measured at stationing 25+347 (Bartlett et al., 2001). The compression was a result of elastic compression of the geofoam, and seating and gap closure resulting from the placement of the load distribution slab, untreated base course and Portland cement concrete pavement. Unfortunately, the 1 percent vertical strain that occurred during load placement was sufficiently large to damage some of the fixed connections that secured the tilt-up panel wall to the load distribution slab (LDS). The damaged connections were subsequently repaired by drilling through the face of the panel wall at the elevation of the LDS and setting epoxy anchored dowels into the LDS.

The vertical stress histories measured by the VW pressure cells are shown in Figure 8. The pressure cells located near the roadway surface in the untreated base course (UTBC) and just underneath the LDS show a strong seasonal cycling of vertical stress. This cycling is attributed to thermal expansion and contraction of the geofoam, LDS and/or PCCP, and potential interaction of the LDS and the tilt-up panel wall at their connection (Figure 6). Potential thermal loading/unloading was most pronounced for pressure cells placed at the top or above cells placed just above and below the LDS (Figure 8). In addition because the granular borrow and road base were compacted against the back of the adjacent panel wall, this can also cause vertical stress transfer due frictional stress that exists along this contact surface. Also, although not intended by the design, the top of the panel wall may be interacting with the cantilevered pavement slab. (The design typical drawing required a gap to be constructed between the pavement slab and the top of the tilt-up panel wall and the overlying pavement system; however, there probably is some interaction that is ongoing, as suggested by the cycling of the VW pressure cell measurements (Newman et al., 2010).

The pressure cells located just above the LDS recorded positive values for several months and then began to record slightly negative values (Figure 8). This behavior may be due to malfunctioning of the pressure cells, but it may also represent complex interactions and potential unloading that is occurring near the connection with the tilt-up panel wall. The LDS has a somewhat rigid connection with this wall, thus any expansion and contraction of the geofoam mass or overlying pavement section could transfer vertical loads to the wall and subsequently change the vertical pressure recorded near the LDS.

The measured stresses at locations deeper into the geofoam mass show less thermal cycling and are easier to interpret (Figure 8). To verify these measurements, simple one-dimensional calculations and two-dimension numerical modeling predict that the vertical stress should be about 27 kPa in the top of the geofoam at level 9 and about 25 kPa at level 6 (Figure 9) (Newman et al., 2010). These VW pressure cell measurements were model with a 2D finite difference program using a bilinear elastic constitutive relation for the EPS (Figure 10) with a Poisson's ratio of 0.103 (Newman et al., 2010). These calculations neglected any thermal effects (and hence could not predict the complex interaction with the LDS, which has been labeled as a "suspect data point" in Figure 9. For VW pressure cell measurement with the EPS, the average stress level at level 9 is about 24 kPa, with a cycling variation that varies between about 19 to 29 kPa (Figure 8). The measured vertical stress at level 6 varied from about 27 to 30 kPa, until this pressure cell went out of service (Figure 8).

We note that the vertical stress measured in the base sand of about 15 kPa is less than the predicted values from the numerical modeling (Figure 9). Based on this analysis, we believe that the base sand (level 0) and overlying geofoam block have partially lost contact pressure in some manner leading to an incomplete transfer of vertical stress to the VW cell positioned in the base sand. This led us to the conclusion that perhaps some other phenomenon, not accounted for in the

numerical model, has affected the measurement of vertical stress in the base sand. For example, it is possible that the curved contact surface of the untrimmed geofoam blocks are creating a partial arch which, in turn, is not allowing for a uniform contact surface and equal distribution of vertical stress in the underlying sand at this location (Newman et al., 2010).

These vertical stress measurements presented in Figure 8 indicate that the stress through most of the geofoam upper part mass is about 25 to 30 kPa, which is approximately equal to the working stress design criterion of 30 kPa. The creep displacement has been about 20 mm (Figure 7), which corresponds to 0.27 percent over a two-year period and is consistent with the rate measured at 100 South Street for this same post construction period (Figure 4). Subsequent monitoring of the magnet extensometer at this location was terminated due to safety concerns as the interstate became operational.

The long-term consolidation settlement of the foundation soils underlying the 3300 South Street embankment is continuing (Figure 11). This data show about 40 mm of post construction foundation settlement has occurred in a 5.5-year post construction period. About 15 mm of this settlement occurred from the placement of the fill and pavement materials atop the EPS. The additional 25 mm of settlement occurred due to the placement of the toe berm, which was constructed at the base of the wall. The projected foundation settlement is about 50 mm in a 10-year post construction period. These data show that the usage of geofoam embankment at this location has met the I-15 Reconstruction period (Farnsworth et al., 2008), despite the additional load produced by the toe berm.



Figure 6 Typical instrumentation layout used at the 3300 South Street site (Newman et al. 2010).



Figure 7 Vertical displacement versus time for geofoam array at station 25+347, 3300 South Street, I-15 Reconstruction Project (Bartlett et al., 2001).



Figure 8 Measured vertical stresses within the embankment at various levels for station 25+347, 3300 South Street, I-15 Reconstruction Project (Newman et al. 2010).



Figure 9 Measured and calculated vertical stresses within the embankment at various levels for station 25+347, 3300 South Street, I-15 Reconstruction Project (Newman et al., 2010).



Figure 10. Bilinear constitutive relation used to model EPS embankment for the I-15 Reconstruction Project (Newman et al., 2010).



Figure 11 Foundation settlement versus time for the geofoam embankment at 3300 South Street, I-15 Reconstruction Project (Farnsworth et al., 2008).

Figure 12 presents a summary plot of postconstruction settlements measured at the various locations along the I-15 alignment and represent the various geotechnologies employed on the project (i.e., sloped earthen embankment with PV drains, MSE wall with PV drains, lime cement column (LCC) treatment of foundation soils and geofoam embankment) (Bartlett and Farnsworth, 2004; Farnsworth et al. 2008). The rate of postconstruction settlement at the 3300 South Street geofoam embankment with the placement of the 1.5-m toe berm and a projected rate had the toe berm not been placed are also shown. Figure 11 also shows the rate of secondary settlement for foundation settlements of the lime cement column (LCC) array at two locations: One set of data shows the rate of settlement at the wall face and the other shows the rate occurring 13.5 m inside the wall using horizontal inclinometer observations (Bartlett and Farnsworth, 2002). These data suggest that the LCC soil mixing technology will meet the 75 mm 10-year postconstruction settlement goal.

However, the two-stage MSE wall with PV drains installed in the foundation soils for the 200 South Street array is projected to exceed the 10-year postconstruction settlement goal at the wall face, where the most pronounced settlement is occurring (Figure 12). (Unfortunately, the horizontal inclinometer within the wall was damaged at the end of construction, thus postconstruction settlements within the wall footprint are not available.) In addition, postconstruction settlement trends at 2400 South, 900 West, and 400 South Street arrays show these large, sloped, earthen embankments have exceeded the 75 mm in 10-year postconstruction settlement goal (Figure 12). These earthen embankments were primarily constructed in areas of new alignment where preexisting embankment had not significantly preloaded the foundation

soils. PV drains were deployed in the foundation soils at these sites to accelerate consolidation, and the embankments were surcharged to reduce the amount of secondary settlement.



Figure 12 Rate of foundation creep extrapolated to 10 years post construction (dashed lines) compared to the design criteria of 75 mm of post construction settlement over 10 years for lime cement columns, geofoam fill, two-stage MSE wall at wall face, and large earth embankments with PV drains at full height, I-15 Reconstruction Project (Farnsworth et al., 2008).

State Street Site

EPS was also placed at the State Street exit ramp to prevent settlement damage to buried utility lines that paralleled the interstate widening. At this location, a full-height geofoam embankment abuts against a pile-supported bridge on the west end of the State Street off ramp. Geofoam embankment supports the bridge approach slab and the adjacent pavement section. VW pressure cells were installed to monitor the stress that developed in the EPS adjacent to the bridge abutment (Figure 13).

At the bridge abutment, total pressure cells were oriented horizontally and vertically to measure the vertical and horizontal stresses that develop at the face of the concrete abutment and in the adjacent geofoam block. A VW pressure cell was cast in the face of the concrete abutment to measure horizontal stresses that developed at this contact. Two additional cells, one oriented vertically and the other oriented horizontally, were inserted in the adjacent geofoam block to measure the horizontal and vertical stresses, respectively (Figure 13). For the cell placed in the geofoam, a precision cut was made in the block by the block molder using a computerized hotwire cutter in order to produce an exact fit for the shape of the pressure cells positioned within the block. The instrumented blocks were subsequently placed within the embankment during construction.



Figure 13 VW pressure cells installed adjacent to bridge abutment at State Street site, I-15 Reconstruction Project (Newman et al., 2010).

Error! Reference source not found.14 shows the horizontal and vertical stresses measured at the abutment face and in the adjacent geofoam block. The first data point, labeled "abutment horizontal," represents the measured and predicted horizontal stresses corresponding to the pressure cell that was cast flush with the concrete abutment face. The points labeled "geofoam horizontal" and "geofoam vertical" represent the measured and predicted horizontal and vertical stresses, from a vertically and a horizontally oriented pressure cell, respectively, inserted in the adjacent geofoam block.

Because of the complexity of the stresses that developed at this location, 2D numerical modeling was used to interpret and verify the field measurements in the same manner as was done for other arrays (Newman et al., 2010) using the bi-linear constitutive relationship shown in Figure 10 and a Poisson's ratio of 0.103. Figure 14 shows that the modeling approach reasonably estimates the measured stresses at and near the bridge abutment and that the measured vertical stress within the EPS is approximately 25 kPa at this location.

No long term monitoring of EPS compression and settlement were performed at this array due to operational and safety considerations that limited access to this array as the interstate became operational.



Figure 14 VW pressure cell measurements at bridge abutment at State Street site, I-15 Reconstruction Project (Newman et al., 2010).

Conclusions

The I-15 Reconstruction Project provides a good case history illustrating the challenges of rapid construction of large embankments over soft soil sites in an urban setting. One of the key technologies employed was EPS geofoam, which was primarily used to prevent possible settlement damage to buried utilities that crossed or paralleled the new interstate alignment. Of all the technologies evaluated, EPS geofoam embankments had the best overall settlement performance (Farnsworth et al., 2008). Field measurements show that elastic compression and gap closure of the EPS block produced about 1 percent vertical strain as the load distribution slab and overlying pavement materials were placed atop the EPS. In addition, the placement of these loads produced about 15 mm settlement in the foundation soils, as measured at the 3300 South Street site. Subsequently, the EPS embankment has undergone about 0.2 to 0.4 percent creep strain in a 10-year post construction period under a working stress level of about 25 to 30 kPa, as measured at two sites. In addition, the current creep trends and vertical stress measurements from the VW total pressure cells indicate that the design performance goals have been met. The measured internal vertical stress levels from the VW instruments are in reasonable agreement with the design limit of 30 kPa. This design criterion was selected to limit the construction compression of EPS 20 to about 1 percent vertical strain at the end of construction period and to about 2 percent compressional strain over a 50-year period. Subsequent long-term monitoring of the creep deformation shows that this 50-year criterion will most likely be met with the maximum projected compressional strain of about 1.5 to 1.7 percent at the 3300 South and 100 South Street sites.

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