Light Rail on Geofoam

West Valley UTA TRAX Project

Ryan Snow
Project Manager
Stacy and Witbeck, Inc.
1320 Harbor Bay Parkway, Suite 240
Alameda, California 94502
510-748-1870
Fax 510-748-1205
Email: rsnow@stacywitbeck.com

James Webb P.E.
Senior Project Manager
Utah Transit Authority
669 West 200 South
Salt Lake City, Utah 84101
801-236-4733
Fax 801-741-8892
Email: jwebb@rideuta.com

Michael Sander
Project Engineer Structures
Stacy and Witbeck, Inc.
1320 Harbor Bay Parkway, Suite 240
Alameda, California 94502
510-748-1870
Fax 510-748-1205
Email: msander@stacywitbeck.com

2010 AREMA Conference and Exposition

Orlando, Florida

(Word Count: 6206)

Light Rail on Geofoam

ABSTRACT

In Salt Lake City, Utah, the public transit company Utah Transit Authority (UTA) used geofoam (an expanded polystyrene material) to construct embankments for four bridges on the new West Valley TRAX light rail extension line. A geotechnical report of the site during the design of the project determined that up to five feet of settlement over a three-year period could occur on the bridge embankments along the corridor due to localized Lake Bonneville deposits in the area. Not only did this time frame present a significant challenge to the project schedule, but it also could have had a huge cost impact in extending the duration of the contract. The solution to this problem was to place over 2.3 million cubic feet of geofoam fill in lieu of traditional aggregate fill for embankment. The existing subgrade was first excavated down approximately five feet where geofoam blocks were then stacked up to the new embankment height. Because of the removal of existing soil and the negligible weight of the geofoam, this procedure produced a zero net load on the existing soils under the embankment locations and did not induce any significant settlement. The geofoam was capped with a concrete load distribution slab and the sides were either covered with soil on the sloped embankments or encapsulated with tilt-up vertical concrete panels. The final configuration has the same appearance as any typical bridge embankment since the geofoam used in the embankment is not visible to the public eye. This project was second largest geofoam project in the United States and is the first known light rail geofoam application.

INTRODUCTION

The West Valley light rail TRAX project (WV LRT) consists of a 5.1 mile alignment with four stations that runs southwesterly from an existing light rail alignment (TRAX) running north/south (near 2100 South in Salt Lake City) to the intermodal hub near the West Valley city hall (figure 1). Utah Transit Authority (UTA) awarded this construction manager/general contractor (CM/GC) contract to Stacy and Witbeck/ Kiewit Western Co., a joint venture team (SWK) on April 10th, 2008. Substantial completion of the project is expected in spring of 2011. Project cost is estimated to be around $350 million and is under budget and ahead of schedule.

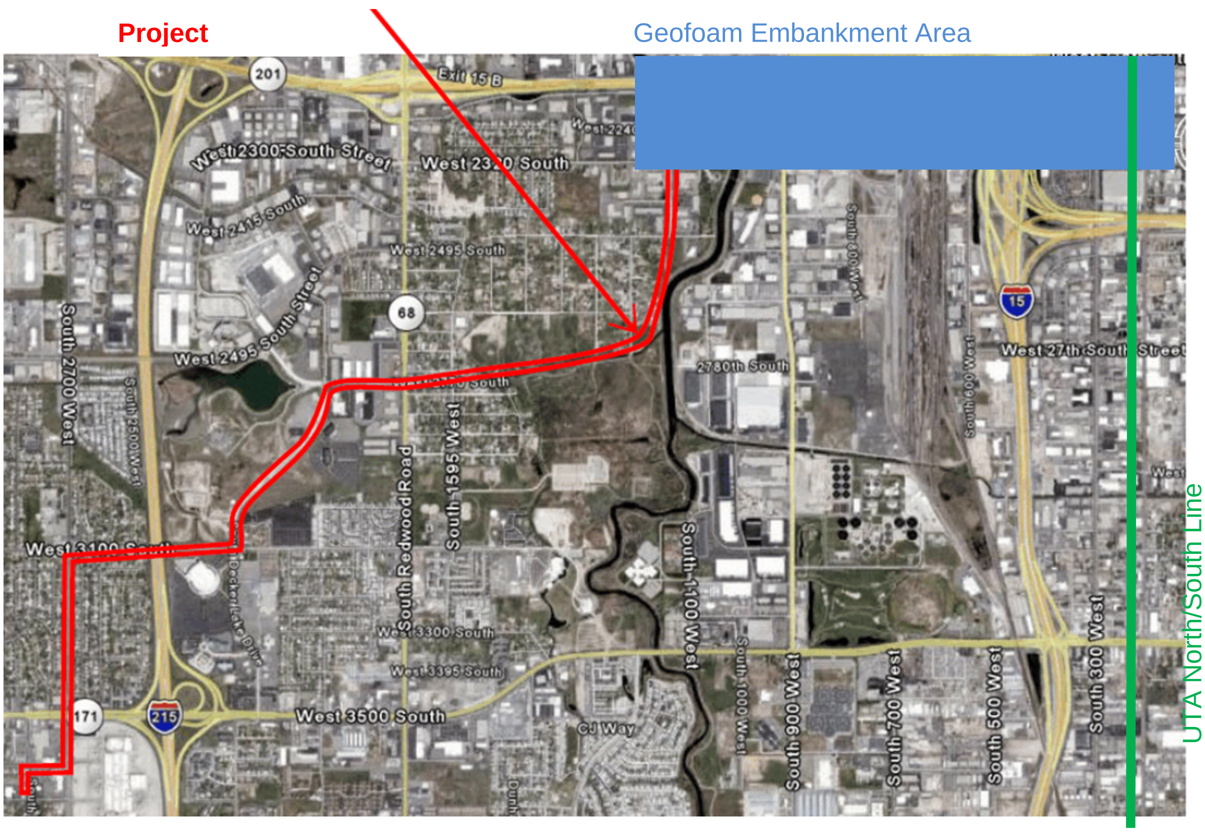

Figure 1: West Valley Project Alignment Map

Given the compressible soils and expected long-term soil settlement in vicinity of the WV LRT alignment, the project team considered many alternatives as solutions. One of the solutions considered was geofoam, which is a strong, low density (one to three pounds per cubic foot), cellular plastic material that has many advantages including light weight leading to reduced soil settlement. Geofoam has been used since 1972, where it was used for a roadway project in Norway. It has been used sparingly across the globe since then, but its use is now becoming more widespread.

EXISTING PROJECT CONDITIONS

Soil Conditions

The Salt Lake valley is plagued with highly compressible soils and seismic fault lines. These soils have low shear strength and require significant time to complete primary consolidation settlement. About 15,000 years ago the Salt Lake valley was located under Lake Bonneville, where the lake was at 5,200 feet above sea level at its highest point. The lake level then dropped severely over several thousand years to its current elevation, where it is now known as the Great Salt Lake. The lake size reduction deposited highly compressible clays throughout the valley. The Salt Lake valley is also located near the Intermountain Seismic Belt; which is a series of seismic fault zones that branch through the valley. This area has been classified as having a high potential for liquefaction. These conditions have created constructability challenges for most construction projects in the valley, including the West Valley LRT project.

There were seven different areas of concern along the alignment (as depicted in figures 2 and 3) where settlement mitigation was considered. The three eastern areas include the east and west fill approaches to 600 West and the fill approach west of Roper Yard (the Union Pacific

Railroad train yard). The Roper Yard bridge is a three-span bridge totaling 780 feet with the longest middle span being 320 feet long. It is located between expanded polystyrene (EPS) fills #1 and #2. The 600 West bridge is 100 feet long and located between EPS fills #2 and #3. The tallest of the fills is EPS #2 (otherwise known as the “island fill”) with walls on all four sides of the fill.

The four western areas include east and west fill approaches to 900 West and the Jordan River bridge. The 900 West bridge is 160 feet long and is located between the 900 West fills #1 and #2. The Jordan River bridge is a single span 200 feet long over the Jordan River and located between the Jordan River fills #1 and #2. The total of all seven fills is approximately 2.3 million cubic feet, with the largest fill height of these areas being just over 40 feet tall.

A geotechnical report, provided by Y^2 Geotechnical, P.C.(1), anticipated primary settlement in the Roper Yard ranging from 4 to 36 inches (tables 1 and 2) and for an estimated duration of 1½ months to 5½ years without soil remediation. The primary settlement at the Jordan River embankments (table 3) was estimated to range from 5 to 63 inches over the duration of 1 month to 1 year. The areas adjacent to 900 West also showed similar estimated settlement durations. This presented a large concern that the project had to overcome to meet the budget and schedule.

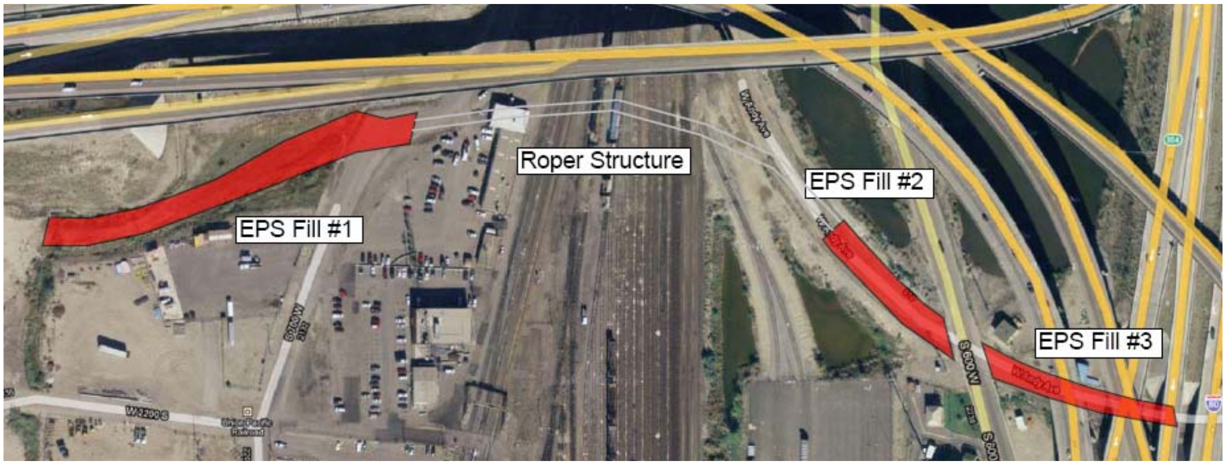

Figure 2: Roper Yard EPS Fills

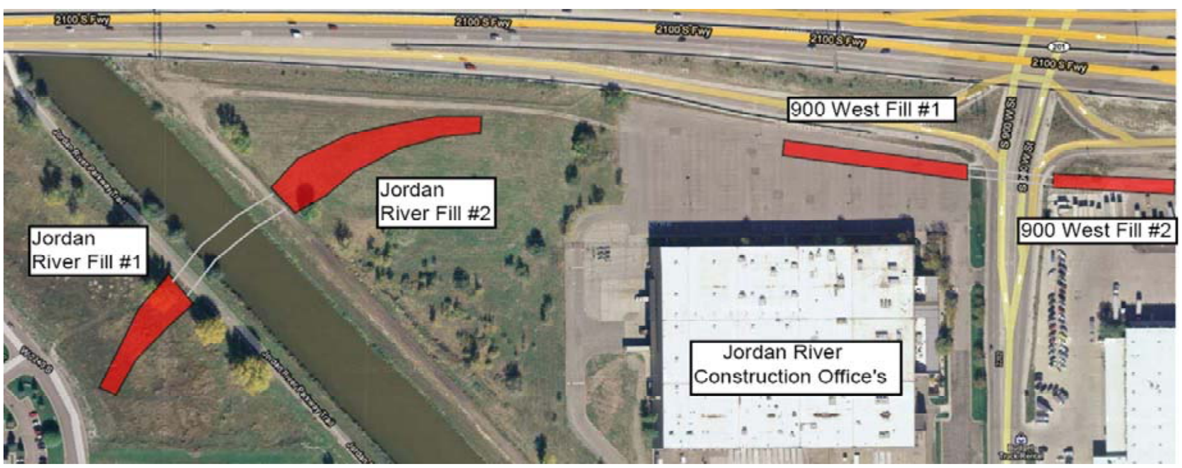

Figure 3: Jordan River and 900 West EPS Fills

TABLE 1: Roper Yard East Area Embankment-Induced Settlement

Embankment Location	Height of Embankment fill	Amount of Initial Settlement
Roper EPS Fill #1	5 feet	4 inches
Roper EPS Fill #1	10 feet	10 inches
Roper EPS Fill #1	15 feet	15 ¼ inches
Roper EPS Fill #1	20 feet	19 ¾ inches
Roper EPS Fill #1	25 feet	23 ¼ inches
Roper EPS Fill #1	30 feet	27 ½ inches
Roper EPS Fill #1	35 feet	30 ¾ inches
Roper EPS Fill #1	40 feet	34 inches
Roper EPS Fill #1	41 feet	34 ½ inches
Roper EPS Fill #2	5 feet	12 ½ inches
Roper EPS Fill #2	10 feet	22 ¾ inches
Roper EPS Fill #2	15 feet	30 inches
Roper EPS Fill #2	20 feet	35 ¾ inches
Roper EPS Fill #2	21 feet	36 ¾ inches

TABLE 2: Roper Yard West Area Embankment-Induced Settlement

Embankment Location	Height of Embankment fill	Amount of Initial Settlement
Roper EPS Fill #3	5 feet	2 ½ inches
Roper EPS Fill #3	10 feet	10 inches
Roper EPS Fill #3	15 feet	17 ¼ inches
Roper EPS Fill #3	20 feet	22 ¾ inches
Roper EPS Fill #3	25 feet	27 inches
Roper EPS Fill #3	30 feet	30 ¼ inches
Roper EPS Fill #3	31.2 feet	30 ¼ inches

TABLE 3: Jordan River Area Embankment-Induced Settlement

Embankment Location	Height of Embankment fill	Amount of Initial Settlement
Jordan River Fill #1	5 feet	17 ¼ inches
Jordan River Fill #1	10 feet	36 ¼ inches
Jordan River Fill #1	15 feet	48 ¾ inches
Jordan River Fill #1	20 feet	57 ¼ inches
Jordan River Fill #1	25 feet	63 inches
Jordan River Fill #2	5 feet	5 ½ inches
Jordan River Fill #2	10 feet	9 ¾ inches
Jordan River Fill #2	15 feet	12 ¼ inches
Jordan River Fill #2	20 feet	13 ¾ inches

Adjacent Structures

The proposed fills were in close proximity to several existing large highway structures (figure 1). This required the project team to investigate how to construct the original soil embankments without causing any impact to the nearby existing structures. Not only was the vertical pressure on the soil below a concern, but additional soil loads would also exert horizontal forces on existing bridge abutments, structures, and supporting walls. Soil typically creates about 40 pcf of lateral pressure, however, if a structure is found below ground level, that pressure can increase by a factor of ten. A conventional, 20-foot tall embankment on soft soil can cause settlement as much as 40 feet away. In addition to the soil loads, the West Valley LRT project had a unique concern east of 600 West where there were existing overhead structures. This created other challenges such as limitations on the ability to use wick drains or other methods to decrease settlement time. The number of nearby structures presented a significant concern for the project, leading to the decision to place lightweight loads near the adjacent structures.

Existing Utilities

Numerous utilities also existed in the areas being discussed. Some of these utilities crossed perpendicular to the track alignment, while others ran parallel to the alignment. Utilities in the area included wet utilities such as water, storm drain, and sewer as well as dry utilities such as communication and fiber optic lines. In addition, a unique seismograph station existed within ten feet of the alignment. There were concerns about the potential settlement of these devices which record seismographic occurrences and which were located in underground wells, one of which was about 400 feet deep. Similar loadings, as described in the section Existing Adjacent Structures, were of concern to the utility owners in the area. These settlement and load impacts were unacceptable to the stakeholders of the utilities and created a need for a solution that would mitigate the impacts from the project.

ALTERNATIVE SOLUTIONS

While considering the issues of predicted settlement and impacts to structures and utilities, various alternatives were considered. Solutions considered included traditional fill, geofoam as lightweight fill, fill surcharge including wick drains, soil mixing, and stone columns. As depicted in table 4, a matrix was developed by the project team to assist in the preliminary decision-making process for determining the best solution. Table 4 illustrates the section between Roper Yard and 600 West, which is only one of the seven settlement areas. The project team developed this type of matrix for the other areas of settlement concern with similar results. As shown in table 4, the cost of geofoam was reasonable in relation to its advantages and benefits. Geofoam allows much quicker construction, negligible settlement, and reduces cost escalation. Possible disadvantages of using geofoam included buoyancy of the geofoam in areas of high groundwater. Another disadvantage was the structural concern of resisting movement forces of overhead catenary pole foundations since geofoam does not resist lateral forces like traditional soil. These disadvantages and their solutions are discussed later in this report.

TABLE 4: Alternative Solution Matrix (Fill between 600 West and Roper Yard)

	Conceptual Cost (Based on 15,000 cy)	**Settlement Amount**	**Construction Duration**	**Settlement Duration**	**Pro/Con – Issues and Risk**	**Risk and Issue Mitigations**	**Comments**
Regular Fill Only Surcharge – 20'; multistage fill Liquefaction may need mitigation	Unit Cost – \$95/cy Assumes difficult island fill and fill access Total Cost - \$1,425,000	5' fill / 4" 41' fill / 34"	Longest settlement Add 3-6 month rest between stages (15' fill - 1st stage; 10' subsequent stages)	41 months	• Settlement time • Known performance • Cost • Surcharge removal • Differential settlement/utilities • Adjacent structure loadings	Liquefaction settlement up to 3". May not be acceptable. Mitigate possibly with 20' deep soil mixing	Settlement duration not feasible. Access drives up unit cost. Telebelt material into fill; Crane equipment onto fill and off fill; Removing burrito wrap/temp wall all drive up cost.

	Conceptual Cost (Based on 15,000 cy)	**Settlement Amount**	**Construction Duration**	**Settlement Duration**	**Pro/Con – Issues and Risk**	**Risk and Issue Mitigations**	**Comments**
Geofoam Liquefaction may need mitigation	Unit Cost – $110/cy Total Cost – $1,650,000 % Increase over Reg. Fill – 27%	Negligible with Net Zero Loading	Shortest construction No settlement period Material lead time	Negligible with Net Zero Loading	• Decreased construction time • No settlement • 100-yr flood plain concern • Cat pole foundations	Liquefaction settlement up to 3". May not be acceptable. Mitigate possibly with 20' deep soil mixing	Preferred option.
Wick Drain / Surcharge Surcharge – 20'+; multistage fill Liquefaction may need mitigation 4' wick drain grid	Unit Cost – $103/cy Total Cost – $1,545,000 % Increase over Reg. Fill – 12%	5' fill / 4" 41' fill / 34"	Decreased settlement period Add 2-3 week rest between stages (15' fill - 1st stage; 10' subsequent stages) Wick drain install ~ 3 weeks	3-6 months	• Reduced settlement time over reg. fill • Overhead Restrictions • No guaranteed time reduction • Haz-Mat plumes • Still settlement period • Constructability issue (access) • Adjacent structure loadings	Liquefaction settlement up to 3". May not be acceptable. Mitigate with possibly 20' deep soil mixing	Settlement issue on utilities unresolved. Ditto UPRR. Settlement duration most likely exceeds schedule float. Access drives up unit cost. Telebelt material into fill; Crane equipment onto fill and off fill; Removing burrito wrap/temp wall all drive up cost.
Soil Mixing 40% coverage – jet grout/drill 80' deep	Unit Cost – $390/cy Total Cost – $5,850,000 % Increase over Reg. Fill – 480%	Negligible	Long construction effort – 6 months No settlement period	Negligible	• Cures liquefaction concerns • No settlement • Excessive cost • Messy/clean-up disposal • Time for construction effort		Best for Soft Cohesive Soils – Better solutions for other soil types (HB)
Stone Columns 40% coverage – drilled mms 80' deep	Unit Cost – $400/cy Total Cost – $6,000,000 % Increase over Reg. Fill – 495%	Negligible	Long construction effort – 6 months No settlement period	Negligible	• Cures liquefaction concerns • No settlement • Excessive cost • Time for construction effort		

The project incurred approximately $2.3 million in additional costs to install geofoam in lieu of traditional fill on the project. Although this was a substantial increase in initial cost on the project, due to the site conditions geofoam ultimately cost less than the other approaches. For example, looking at a section of the alignment about six city blocks long (approximate construction cost of $45 million) with an average of two years in cost escalation (assume 3% escalation per year), and allowing for settlement time, the project would have incurred about

$2.75 million in additional project costs using traditional construction. Using geofoam enabled UTA to open the line earlier, reduced impacts to structures and utilities, and reduced other delay costs for overhead and personnel on the construction and project teams. For this project, geofoam was an excellent solution.

ZERO NET LOAD—GEOFOAM

To avoid settlement time and the associated project delays, the method chosen for settlement mitigation for these embankment areas was the net zero load design. This design method was accomplished by excavating the weight of existing soil which was equal to the dead weight to be added by anticipated new construction, and by replacing a portion of the existing soil with geofoam. This allowed for zero net load increase due to construction and operation. The dead load weights included the rail, ties, overhead catenary system, ballast, subballast, load distribution slab, geofoam, and other weights. This method allowed the project to assume no added net loads, ensuring that negligible settlement would occur.

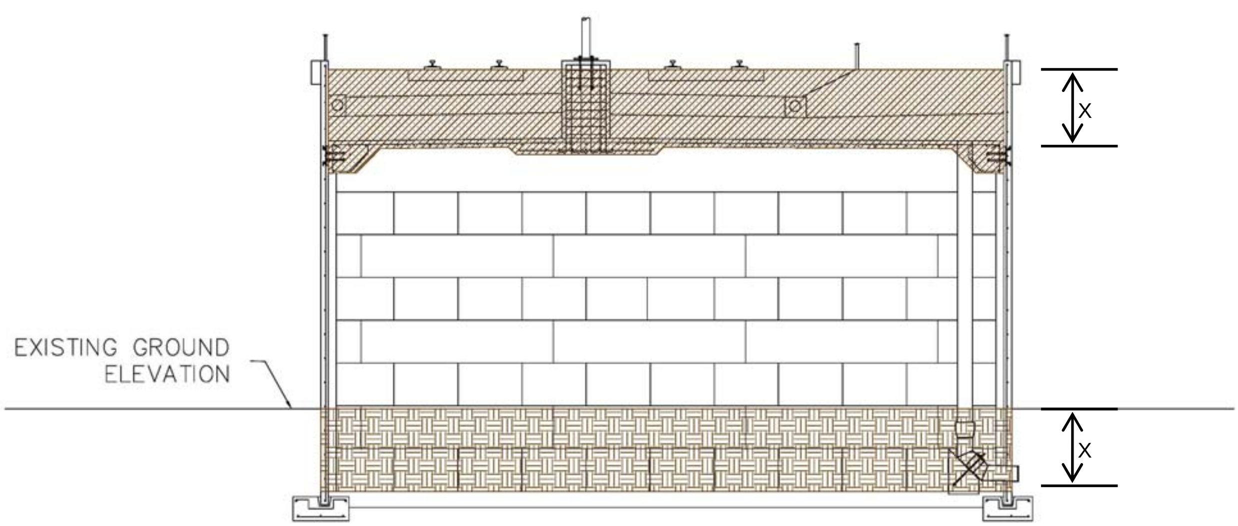

Figure 4: Zero Net Load

GEOFOAM (EPS) MATERIAL

Expanded polystyrene (EPS) geofoam is a low-density cellular plastic material that has 1% of the density of traditional fill materials. It is manufactured in block form, and the material meets ASTM D6817 (rigid, cellular polystyrene geofoam) standards. It is typically available in a range of densities and types to provide control of structural integrity and cost effectiveness for different situations. This same EPS material is used in standard coffee cups (with the exception of higher density, fire-retardant, and insect-repellant chemical properties).

Geofoam weighs about 1 to 3 pounds per cubic foot and is 50 to 100 times lighter than soil. It is 20 to 30 times lighter than alternative lightweight fill materials. Each typical (3' by 4' by 12') block weighs about 180 pounds. This allows geofoam to greatly reduce vertical pressure on soil by an approximate ratio of 120:1. The geofoam used on the WV LRT project will withstand vertical loads of over one thousand pounds per square foot. The specific material, densities used on the WV LRT project included EPS 19, EPS 22, and EPS 39. The EPS number refers to approximate density in Kg/m^3.

Exposure to water or water vapor does not cause swelling of the geofoam material. Although flame retardants are typically used in the manufacturing process, geofoam must still be considered combustible. Typically, a load distribution slab (6" to 8" concrete slab) caps the geofoam and acts as a barrier where petroleum spills can occur. Geofoam will not decompose and will not support mold or mildew growth.

GENERAL OVERVIEW OF GEOFOAM CONSTRUCTION

Geofoam embankments function similar to any other bridge or structure embankment in their final configuration. There are several different configurations (as with any bridge embankment). Depending on the surrounding circumstances, the geofoam may be installed as a sloped embankment where the geofoam will be covered with membrane and soil, and will present an outward appearance similar to a typical bridge structure embankment. If space or right-of-way constraints are an issue, precast concrete tilt-up panels can be used along the limits similar to any other retaining wall fill. Figure 5 illustrates a typical sloped fill on the left and a retained fill on the right.

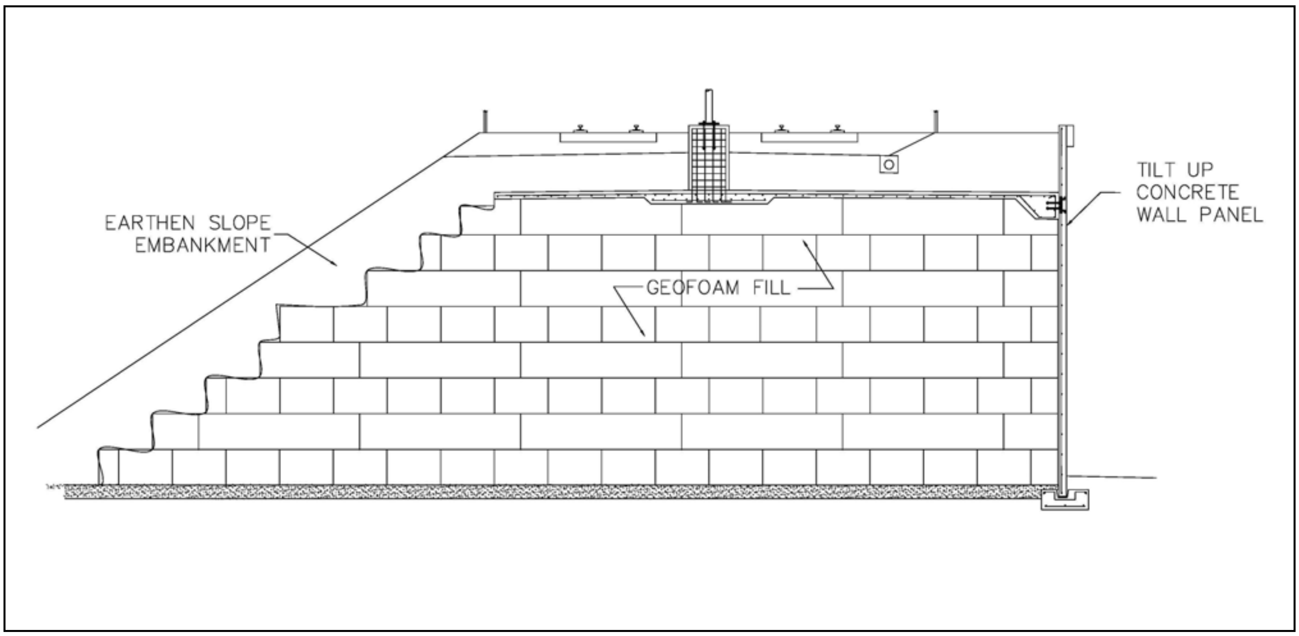

Figure 5: Geofoam Embankment

GEOFOAM CONSTRUCTION STEPS

The steps involved with geofoam construction are as follows:

1. Excavation
2. Grade beam for concrete tilt-up panel
3. Drainage blanket and leveling pad
4. Geofoam installation
5. Utilities and electrical ductbanks in geofoam
6. Load distribution slab (LDS)
7. Precast concrete tilt-up panels
8. LDS/tilt-up concrete panel closure pours
9. Concrete tilt-up panels connections
10. Backfill embankment

A detailed description of each of these steps is described in the following sections.

Excavation

Excavation for the geofoam follows the same process as any other excavation. The depth of the excavation is dependent upon the zero-net load determination. The total of the new induced loads is offset by the existing soil weight removed.

The sub-grade design elevation should be established depending on the manufacturer supplying the geofoam. Each manufacturer produces a slightly different dimension of block. The sub-grade should be set so that full blocks can be used up to the load distribution slab elevation. Cutting every block on the final layer to accommodate the load distribution slab elevation can be time consuming and costly. One inch of deviation in the block size over 15 blocks can result in 15” of discrepancy at the load distribution slab elevation. The supplier on the WV LRT project produced blocks 3’1” x 4’1” x 12’.

Grade Beam for Concrete Tilt-up Panels

As discussed in the excavation section, the grade beam can placed prior to geofoam, or after if the elevations do not coincide with each other. It is best to cast the grade beam before placing foam, if the situation allows, but it is not necessary. Having the grade beam defines the limits for placing foam, but the same can be accomplished with setting a string line.

The grade beam design on the WV LRT project varied based on the height of the wall panels, with the widths at either 2.5’ for panels shorter than 26’ and 3.5’ for panels taller than 26’. The grade beam was cast on structural backfill (figure 6).

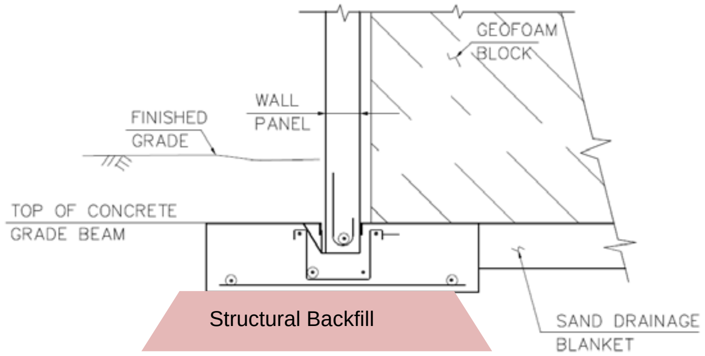

Figure 6: Grade Beam

Sand Drainage Blanket and Leveling Pad

A sand drain blanket / leveling pad was placed within the geofoam placement limits. The sand must be level as geofoam blocks are placed. Subsequent layers of geofoam are easier to install if the initial layer is level.

Geofoam Placement

Although the geofoam material is unique, the placement is a simple method of stacking blocks. Access to the site is a key component to effectively building the embankment. Generally, the manufacturer will not be able to supply sufficient geofoam to keep up with the placement of material. It is most cost efficient to have the material delivered and placed directly into the fill, but a temporary staging location is usually necessary.

On the WV LRT project the geofoam was transported from the staging area to the fill with an extendable boom forklift. Once the fill reached heights above the reach of the forklift, a rough terrain crane was utilized to place the geofoam blocks on top of the fills. Once blocks were placed on the fills the workers were able to move the blocks by hand into their final configuration.

Figure 7: Geofoam Placement

As discussed previously, geofoam is manufactured in various densities. The density of the geofoam is dependent on the design loads needed on the project. To reduce cost several different densities are usually used within one fill location. The material with highest density is used on the top of the fill and less dense material is used in lower levels of the fill.

On the WV LRT project, EPS 22 was used on the top layer and EPS 19 was used for all of the lower layers. EPS 39 was used underneath any high load bearing areas such as the overhead catenary foundations.

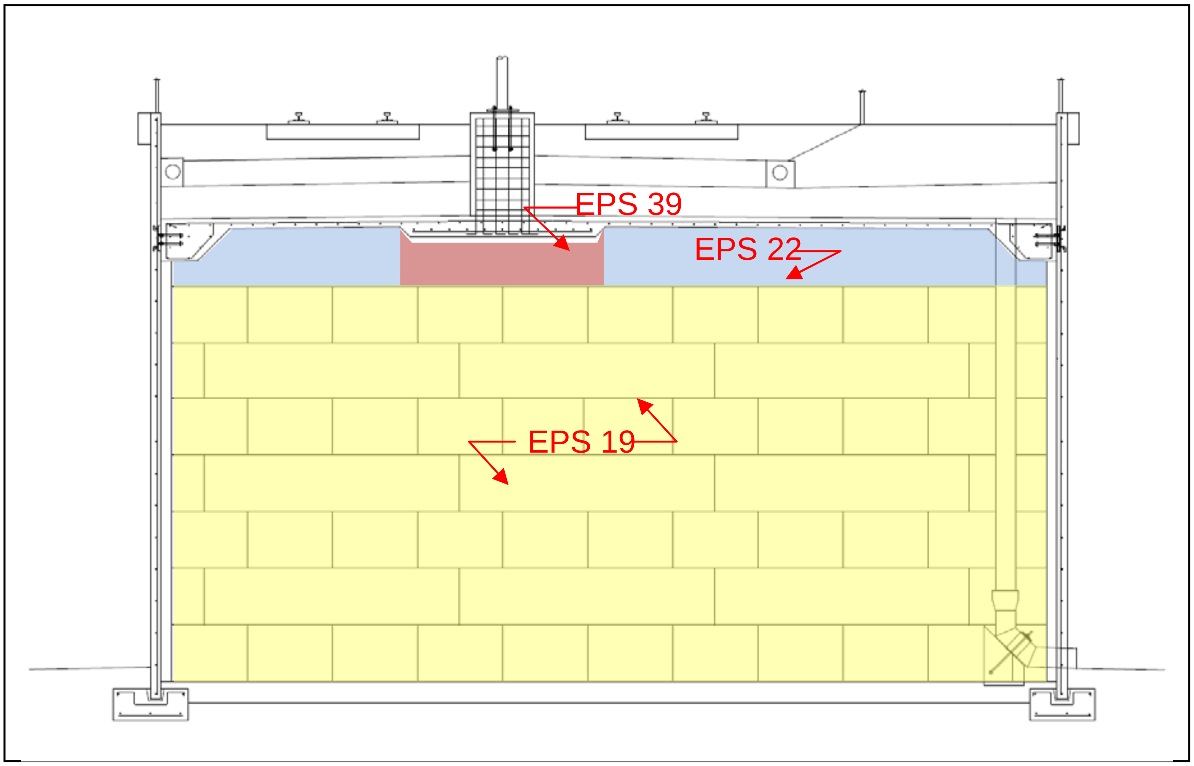

Figure 8: Typical Geofoam Section with Variable Densities

Each layer of geofoam is placed perpendicular to each other. The initial layer may be stacked longitudinally and the second layer will then be stacked transversely. The alternating direction will continue throughout the entire fill. This creates better stability as the fill continues upward.

In order to create lateral stability shear keys are placed throughout the geofoam embankments. The shear key is essentially a geofoam block that is installed within two layers and breaks the isolation plane between each layer (figure 9). The shear keys are placed periodically throughout the fill for lateral stability as recommended by the design engineer.

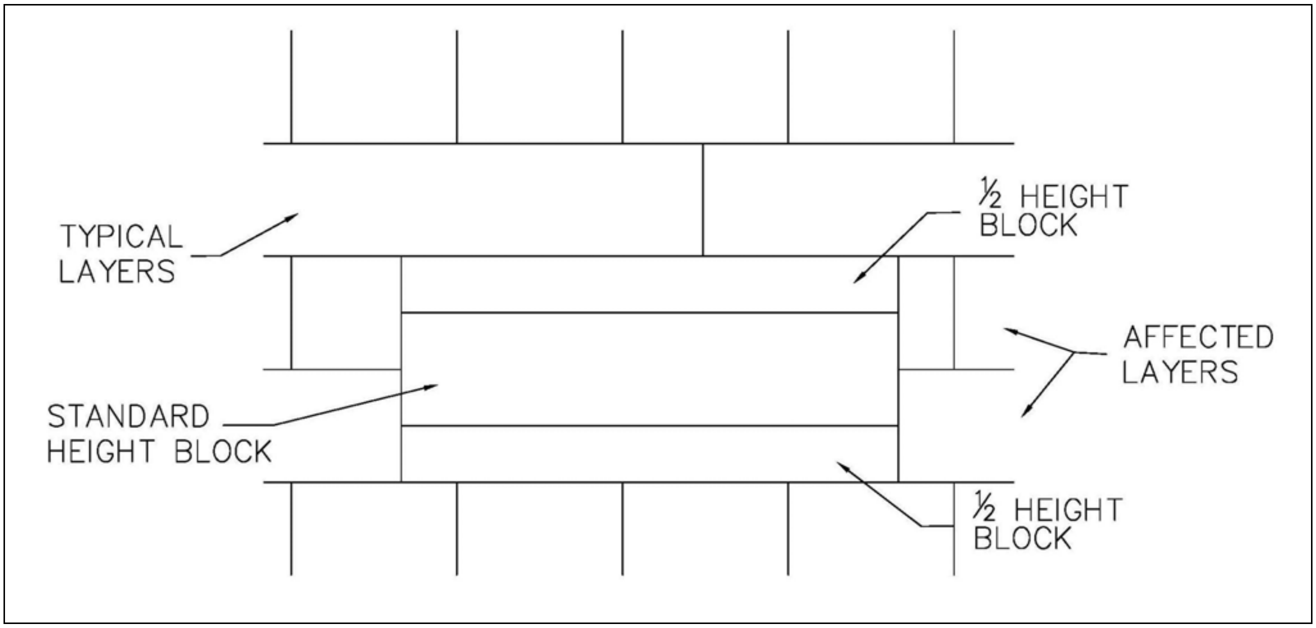

Figure 9: Typical Shear Key

As the geofoam is placed along the edges of the embankments or next to protrusions in the fill it is necessary to cut the material to a desired length. This is accomplished by using a “hot wire” cutter. Essentially, this a copper wire that is electrically charged. The wire cuts through the foam with precision and leaves a clean cut after completion.

Figure 10: Cutting Geofoam

As the foam approaches the load distribution slab elevation, the blocks are cut to produce longitudinal steps as the embankment increases in height. The elevation of the steps are set at half block increments to minimize the laborious cutting of each piece of geofoam. Therefore, a manufactured full and half blocks can be used rather than field cutting each block. At each step in half block the geofoam is cut on a 45 degree angle to provide a platform for the load distribution slab (figure 11).

Figure 11: Load Distribution Slab Steps

Utilities and Electrical Ductbanks in Geofoam

As with any embankment, utilities and electrical ductbanks occur within the geofoam embankment. Any protrusion in the foam can be accommodated by simply cutting the foam at each infrastructure location.

On the WV LRT project there were several items that were required to penetrate the geofoam. Drainage pipes, overhead catenary electrical grounding, and communication and signaling ductbanks were all cut through the geofoam fill.

To accommodate electrical grounding, the lines were run out laterally to the limits of the geofoam and extended down between the geofoam and the concrete tilt-up panels. The drainage pipe was designed to run through the layers of geofoam vertically and then daylight out through the concrete tilt-up panel. The electrical ductbank was placed on the layer below the final layer of geofoam. The ductbank was then encapsulated with concrete. A field-cut geofoam piece was placed on top of the encasement. The electrical vaults were installed such that they were set within the geofoam and protruded up through the load distribution slab.

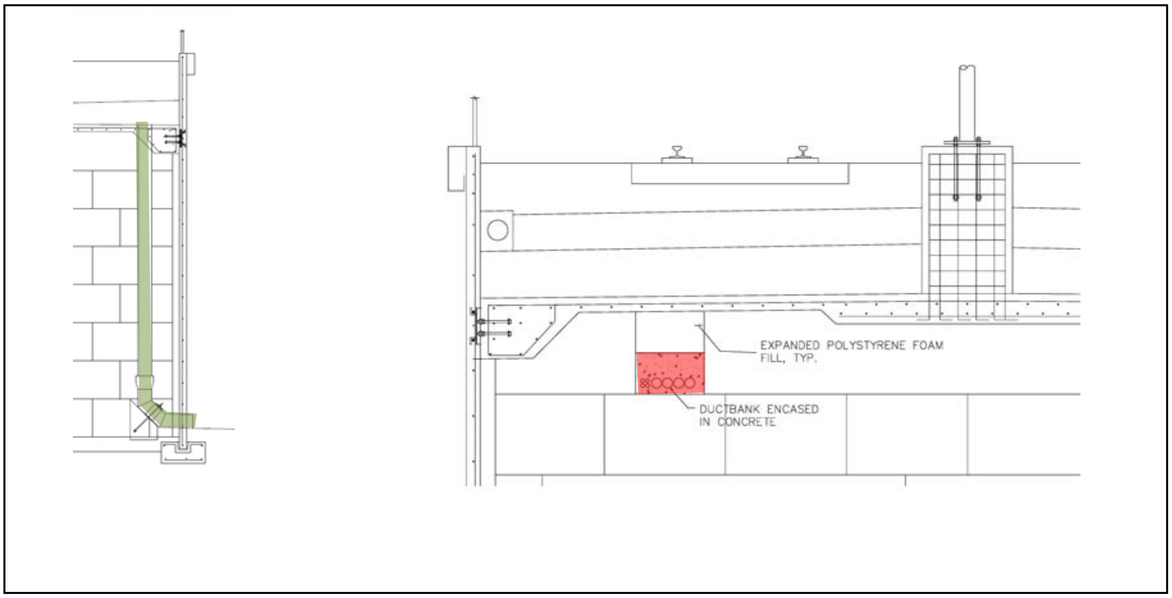

Figure 12: Typical Drainage and Electrical Ductbank Section

Load Distribution Slab

The load distribution slab (LDS) is usually an eight-inch thick reinforced concrete slab that is placed directly on top of the geofoam. There is no membrane between the geofoam and the LDS since the bond or friction forces are part of design considerations.

The LDS was approximately 40 feet wide on the WV LRT project and extended the length of the geofoam.

There are two different methods for finishing load slabs: 1) tilt-up wall panels are connected to the LDS at the interface point or 2) sloped embankment fill is backfilled to the edge of the LDS. Geofoam fills surrounded with tilt-up wall panels have the center portion of the LDS poured leaving a second six-foot wide closure pour between the tilt-up wall panels and the main load distribution slab as shown in figure 13. The second “closure” pour takes place after the wall panels are set and connection components are installed (see Installing Tilt-Up Panel Hardware).

Figure 13: Initial Load Distribution Slab Pour

During the design phase on the WV LRT project, the integration of the overhead catenary system (OCS) foundations into the geofoam fill presented some challenges due to the lateral resistance within the geofoam. The geofoam would not support the lateral loads for a typical shaft foundation. The designers developed an integrated foundation in the load distribution slab. This solution is detailed later in this report (Complications on West Valley TRAX Project: Overhead Catenary Foundations). In order to ground the OCS foundations, an HDPE conduit was run up the center of the rebar cage and a small trough was cut in the geofoam to the edge of the fill where it extended to the ground.

Precast Concrete Tilt-Up Panels

In areas where right-of-way and access was limited, concrete tilt-up panels were used at the perimeters of the geofoam fills. This work was challenging and required additional engineering in order to place the panels (which were as large as 43 feet tall and 8 feet wide). Not only was the handling difficult, but the placement of the panels presented challenges due to the limited space available along the corridor.

Casting Wall Panels

Wall panels on the WV LRT project varied in height from 4 feet to 43 feet with a width of 8 feet. The architectural finish was a fracture fin finish with horizontal joints staggered between panels at 8-foot centers. A casting bed was set up to cast the precast panel for the project. A gantry crane was used to move the panels from the casting beds, to storage, and then onto the flatbed trucks for delivery. The panels were steam cured. This accelerated the concrete curing process, allowing sufficient strength in the panels to move them the following day. The panels were cast in two different thicknesses to support the structural loads. Panels under 25 feet tall were 8 inches thick, while those over 25 feet tall were 10 inches thick. The key joint was designed so that if 8-inch or 10-inch panels were connected together the exposed face would be uniform.

Installing Precast Wall Panels

The need to move these large panels required additional engineering to ensure that the panels would not be damaged during the transport and installation processes. Embedded pick points were designed in the wall panels to mitigate the bending forces in the panel during placement. Using a crane with multiple hoist lines, the panel was rigged to the back and top of the wall panel (figure 14) and rotated into a vertical position. The bottom of the panel was rested on the ground to control the rotation of the panel.

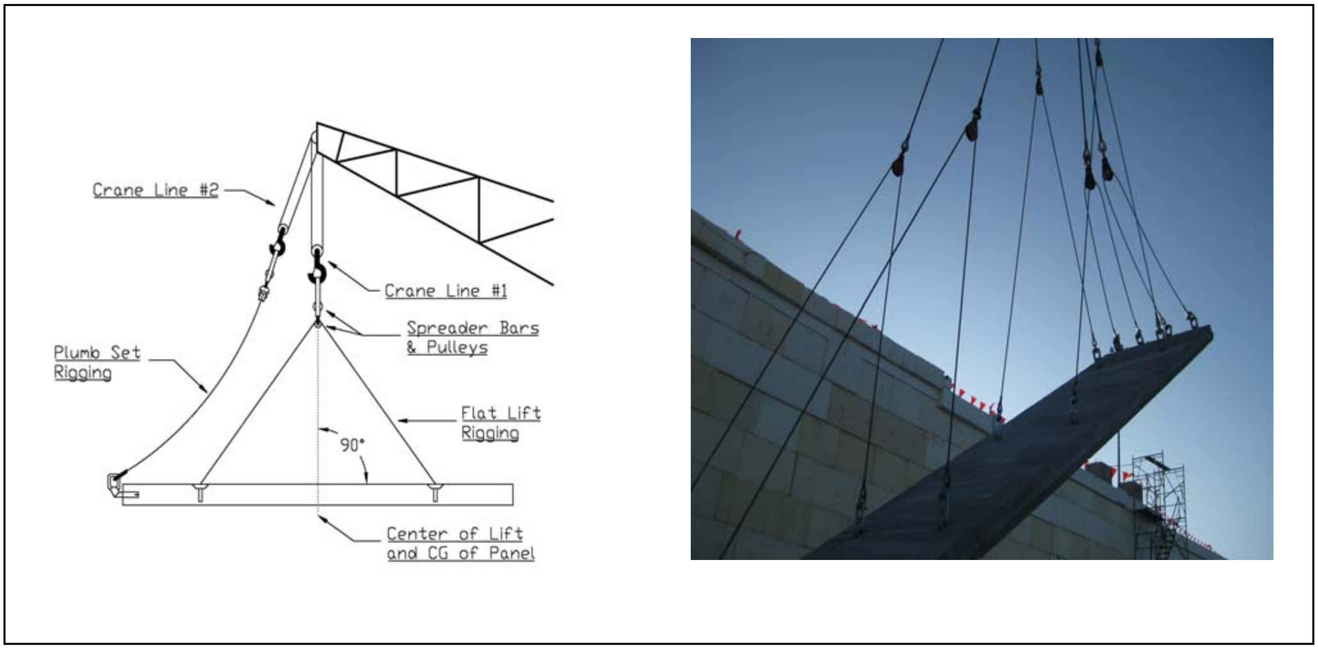

Figure 14: Tilt-Up Panel Lifting Rigging

A man-lift was used to unhook the rigging on the back of the panels while the crane held the weight of the panel from the top. The panel was then placed in the keyway of the grade beam where it was shimmed and set plumb. Pole bracing was used to secure the wall panels to the load distribution slab. Pole braces were secured to the back of wall panel and drilled into the previously cast LDS to secure the wall panels until the closure pour occurred.

Figure 15: Rotation to Install Precast Wall Panel

LDS/Tilt-up Panel Closure Pours

After the wall panels have been placed and then bolted to the LDS with pole braces, the hardware is installed to connect the panels to the load distribution slab.

On the WV LRT project, due to additional dead load being imposed on the load distribution slab, it was important to ensure that this load would not be transferred to the tilt-up panels as they were not designed to hold the vertical load structurally. To do this a piece of uni-strut was embedded in the precast tilt-up panel at the elevation of the load distribution slab. With the panel in position, a sheet of 30-mil plastic was secured to the back of wall panel. Two bolts were then threaded into a spring washer that was installed in each uni-strut. These bolts served as connection anchors from the wall panels to the LDS after completing the closure pour. The bolts were able to move within the uni-strut to allow for settling after the dead load was applied. The plastic barrier and uni-strut allowed the LDS to settle up to five inches under the weight of the dead loads placed on the LDS, without creating additional stresses for the wall panels.

It is important to ensure that the uni-struts are installed at the right location in the tilt-up panel corresponding to the steps in the load distribution slab. Once the connections are in place the concrete can be placed to complete the load distribution slab to the limits of the fill.

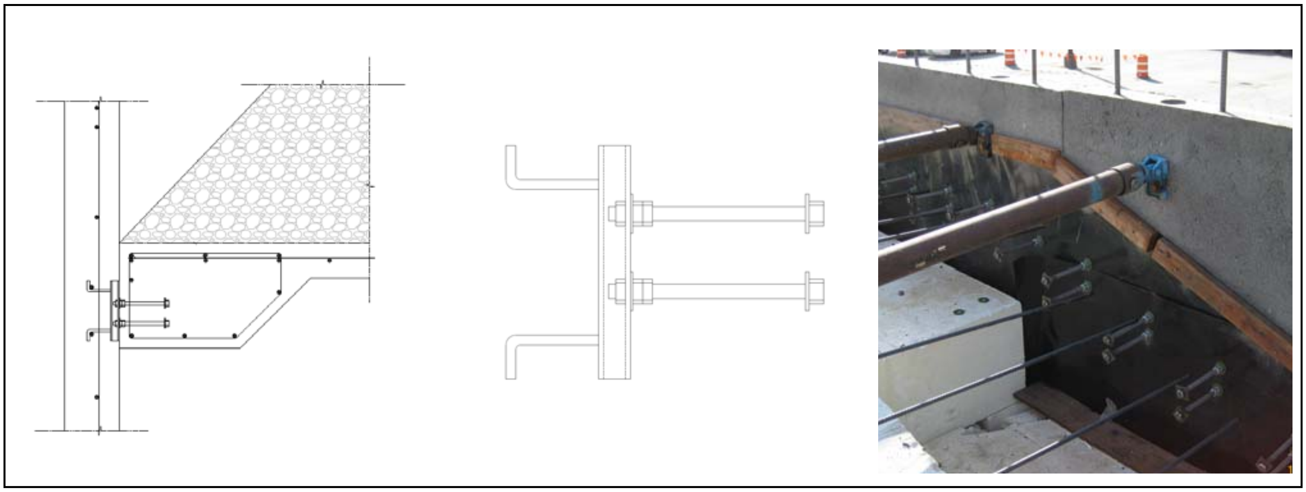

Figure 16: Uni-Strut Connection

Concrete Tilt-Up Panels Connections at Abutments

Each of the abutment locations required a different connection than those used at the load distribution slab locations. Across the face of the abutments there was enough room to allow a person to drill and place epoxy-threaded rods into the abutment. A turnbuckle connection was used to connect the wall panels to the abutment; shown below in figure 17.

Figure 17: Tilt-Up Panel Connection at Abutments

At the sides of the abutments there was only 2 to 12 inches of space between the abutment and the back of the wall panel. A special bracket had to be fabricated because the space given was not enough to use the same method as at the face of the abutment. The bracket was attached to the panel on the ground and the panel lifted into position. A ramp was mounted to the footing to slide the panel under the wing walls of the abutment and then a chain ratchet was used to pull the panel into position. A person in a man-lift then drilled and installed the mechanical anchors to secure the wall panels (figure 18).

Figure 18: Modified Connection at Side Abutment Locations

Once all of the panels were in place and connected to the load distribution slab a concrete coping was formed and poured along the top of the panels.

Backfilling the Embankment

Where the tilt-up panels were not used as part of the geofoam embankment a standard aggregate fill was used to encapsulate the foam. A membrane was placed on the geofoam to protect it from any surface penetrations and two feet of backfill material was placed over the foam.

Once the concrete work was completed the load distribution slab was backfilled with subballast. The subballast was spread so that a level constant grade occurred over the various steps in the LDS. With the subballast in place the embankment fills were ready for track construction.

SAFETY AND QUALITY GUIDELINES

Safety / Hazards

Potential hazards identified on the job included the following:

- Placement of expanded polystyrene (EPS or geofoam) should NOT be conducted with winds exceeding 20 mph. Due to their surface area and light weight the wind will move the blocks quite easily.
- Hot wire cutting – chance of electrocution; GFCI required.
- Water / Flooding – geofoam blocks are buoyant.
- Tie down / secure foam – make sure everything is secure in areas not being worked on, as well as at the end of shift. On the WV LRT project, concrete blocks were placed along the perimeters and mule tape was draped across the foam.
- The foam is extremely slick when wet, snow covered, or where frost has built up.
- Make sure all holes are leveled out at the end of shift so it is not a hazard the next day (holes for shear keys are difficult to distinguish in the snow).
- Secure all loads when transporting onsite during inclement weather.

Quality Standards

To meet quality standards when building with geofoam, you need to know the following specifications:

- Slight damage (<0.12 cubic feet of damage over a length less than one foot) may be left in place as-is.
- Moderate damage (<0.35 cubic feet of damage over a length less than 3.3ft) shall be filled with sand.
- Excessive damage – cut to eliminate the excessive damage.
- Surface variation – 0.05 foot in any 10-foot interval.
- Maximum of three inch gap between blocks.
- Gap between EPS and wall panels: 2 to 12 inches.
- Testing samples – 10 each per 500 CY (90 blocks).

    a. ASTM D1621 – Test Method for Compressive Resistance
    b. ASTM D1622 – Test Method for Density
    c. ASTM C203 – Test Method for Flexural Strength
    d. ASTM D1623 – Test Method for Tensile Strength
    e. ASTM C272 – Test Method for Water Absorption

- Shear keys shall be placed at the locations designated by the engineer.
- Each subsequent layer is staggered and placed perpendicular to previous layer.
- Finish grade +/- two inches from design.

COMPLICATIONS ON THE WEST VALLEY TRAX PROJECT

Overhead Catenary Foundations

During the project design phase for the geofoam, a conflict arose with implementation of overhead catenary foundations in the geofoam. Overhead catenary foundations support the poles and catenary wire that power the light rail vehicles. Due to the high tension placed on the catenary wire the foundations needed to withstand an overturn moment force. Geofoam did not provide the lateral strength to resist this moment. The design team reviewed several different scenarios, but determined that a spread footing incorporated in the load distribution slab would be the best option to support the lateral loads. By using a thickened concrete section and connecting the entire load distribution slab as the foundation for the overhead catenary poles, designers were able to meet the design parameters. As indicated in figure 20, the OCS foundations were cast into the load distribution slab. The rebar cage had a 10-foot square spread footing placed under the LDS rebar mat. Additionally, EPS 39 material was required beneath the foundation to withstand the moment load on the foundations.

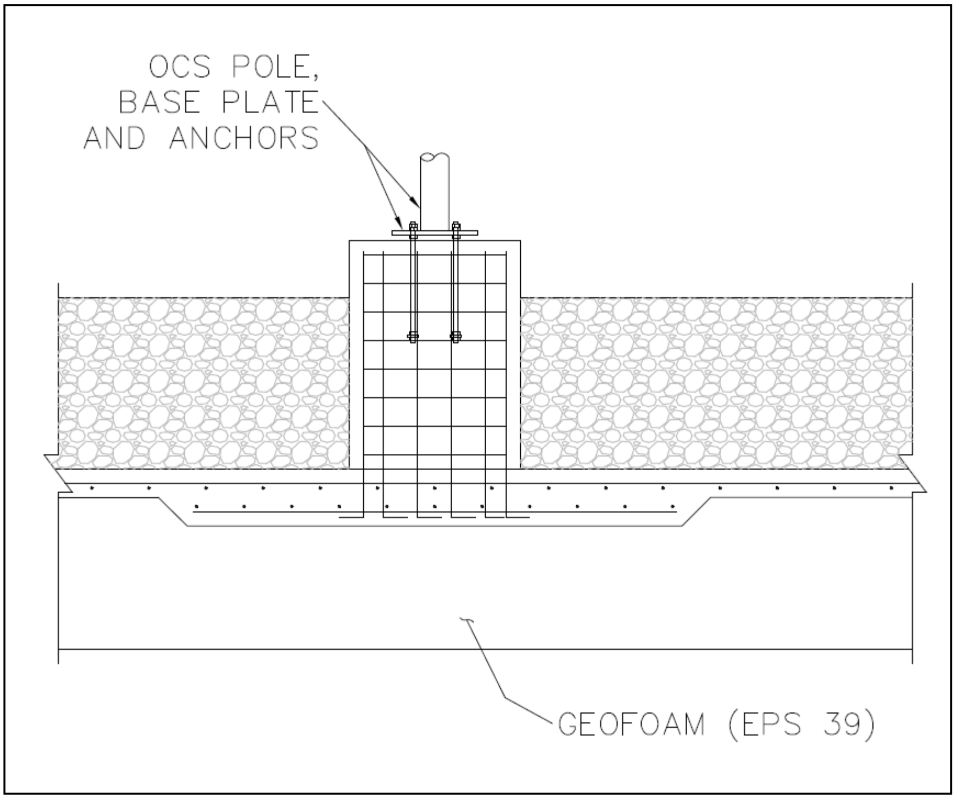

Figure 20: Design for Modified OCS Foundations on Geofoam

Figure 21: OCS Foundation in Load Distribution Slab

Lateral Loads at the Bridge Abutments

The original design concept had the abutments placed on the geofoam fills, but designers realized early in the design that the geofoam could not support the design lateral spread forces of the driven piles for the perched abutments. The solution was to change to a light-weight fill using a two-stage wire wall system in the abutment vicinity. The abutment locations were over-excavated eight feet from native ground level to minimize the settlement duration. Mechanically stabilized earth (MSE) 2-stage walls were constructed and backfilled with a pumice rock that weighed approximately 70 pcf. The limits of the light-weight fill were designed to resist the anticipated lateral loads from the abutments extending back on a 45 degree angle from the bottom of the abutment. Settlement monitoring probes were installed in the fill to measure the settlement. The fills were loaded with concrete ecology blocks to obtain the initial settlement. Once the settlement had been obtained the blocks were removed and the geofoam was benched into the light-weight fill. Although the initial settlement had occurred the secondary settlement would still occur. This settlement was determined to be less than 5 inches. Because the track in this section was ballasted track, the settlement in the track could be corrected by re-surfacing the track at a later date if the secondary settlement did occur.

Buoyancy of Geofoam

On the WV LRT Project the effect of ground water on geofoam was quickly realized. If the ground water level is above the sub-grade elevation for the geofoam, then dewatering pumps are required as the geofoam is buoyant and will not maintain its position if the ground water level is higher than the geofoam subgrade. Continual monitoring is required until the load distribution slab is placed. During the design phase the water table level should be researched to ensure that the water table will not rise to a level that would jeopardize the placement of the fill.

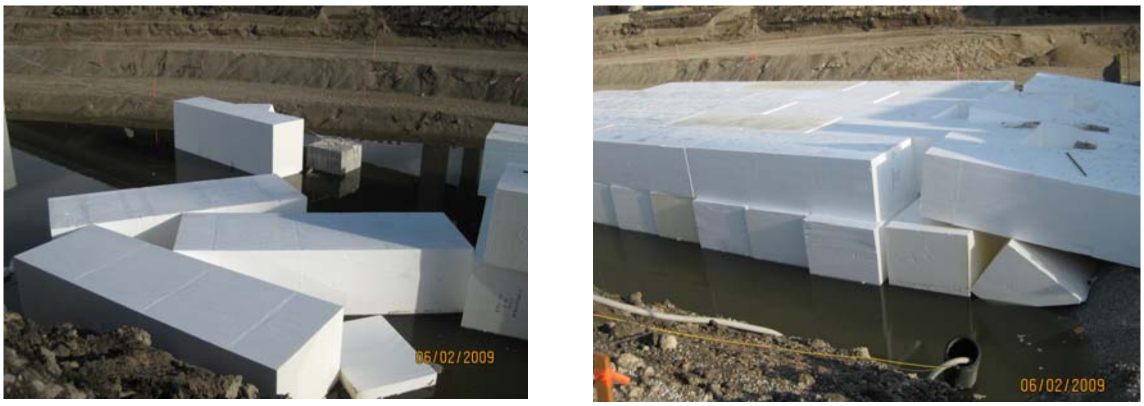

Figure 22: Buoyant Geofoam after Groundwater Level Raised

CONCLUSION

On the West Valley TRAX project it was determined that geofoam was the best solution for the problems presented near four of the five structures on the alignment. The existing soils presented a condition that would jeopardize the ability to construct the project on schedule, adjacent to other structures, over existing utilities, and for the most cost efficient price. Although geofoam is not the answer for all settlement situations, it does provide a good alternative for undesirable soil conditions. On the West Valley TRAX project geofoam was used and returned the desired cost and schedule benefits.

Reference List

(1) Y[2] Geotechnical, P.C. Geotechnical Study—West Valley LRT Project (South Salt Lake City to West Valley City, Utah), Volume 1 of 5 Report, May 2008. (UTA Project Number: 101584/Phase 3/Task 8) CD-ROM.

List of Tables

List of Figures