Design of Geofoam Embankment for the I-15 Reconstruction

By

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Abstract

The Utah Department of Transportation, in conjunction with Wasatch Constructors, is in the process of reconstructing Interstate I-15 in Salt Lake City, Utah. As part of this $1.5 billion design-build project, innovative foundation treatments and embankment construction have been used to mitigate settlement and expedite construction on soft, clayey, foundation soils. Geofoam was placed at several locations along the I-15 corridor. Buried utility lines cross areas where new fill was required to raise grade or for embankment widening. Geofoam was used along utility corridors and widened areas to minimize consolidation settlements. This application of geofoam enabled critical utilities to remain in-service, without incurring expensive costs related to interruption, replacement, or relocation. Another use of geofoam was rapid construction of high embankments on soft soils. Geofoam was used to improve the global stability of some very tall embankment/wall systems. In some locales, relatively high MSE walls (ranging from 10 to 14 meters) were required at high bridge crossings. A typical MSE wall system, with its attendant time allowances for foundation preparation, construction, surcharging, and settlement, required approximately 9 months to one year before rigid pavement could be placed atop the system. In a handful of cases, more rapid construction was accomplished by replacing the MSE wall system with a "geofoam wall". This consisted of a geofoam embankment having a vertical outward face, which was covered by a concrete fascia wall. This system was successfully constructed in about one month, without any significant stability or settlement issues. The paper presents a background on geofoam use at the I-15 design build project and some of the design and installation considerations related to the various applications.

KEYWORDS: design-build, embankment, geofoam, settlement, stability

INTRODUCTION

Project Description

The Utah Department of Transportation (UDOT) in conjunction with Wasatch Constructors is in the process of reconstructing Interstate I-15 in Salt Lake City, Utah. The $1.5 billion design-build contract consists of modernizing I-15 from 600 North to 10600 South, which is approximately 27 kilometers of urban interstate (Figure 1). Construction began in May 1997 and will be completed by July, 2001 in time for the 2002 Winter Olympic Games.

The project essentially widens the existing I-15 corridor with an additional general-purpose lane, a high occupancy vehicle (HOV) lane, and an auxiliary lane between ramps on both north and southbound sides of the interstate. The project will replace all existing bridges with 144 new structures. Interchanges will be constructed at 400 South and 600 North for improved downtown access, and single point urban interchanges (SPUI) will reconfigure most remaining freeway/arterial intersections (Figure 1).

To accomplish the widening of the roadway within the limits of right of way, the reconstruction of the I-15 corridor will make use of approximately 160 mechanically stabilized earth (MSE) walls to construct "vertical fills." As part of this time critical project, several innovative foundation treatments and embankment construction methods have been used. These methods are being employed in areas where conventional solutions are costly or time consuming. The most innovative is the use of expanded polystrene (EPS) blocks, known commonly as "geofoam," for light-weight fill. Approximately 100,000 m^3 of geofoam has been placed, or will be placed, on the I-15 Reconstruction Project, making it the largest application of geofoam to-date in the United States.

Geofoam Manufacturing and Applications

Expanded polystrene is created during a two-stage process. In the first stage, expandable polystrene resin is pre-expanded by a hydrocarbon blowing agent that is contained within tiny resin beads. When the beads are exposed to steam, the polymer softens and the blowing agent expands, creating a cellular structure within the "pre-puff" beads. After a short stabilization period, the pre-puff is placed in a large rectangular block mold and steam is injected into the mold. Under this heat and pressure, the beads further expand and fuse to form a molded block. The result is a white, synthetic material that has a texture of closed, gas filled cells. Individual cells, or beads, are still visible after the molding process, but the beads have coalesced, to form a closed fabric, with essentially no void between the cells. Block molds are capable of producing rectangular block, that are typically 500 to 600 mm high, 1000 to 1200 mm wide and 2000 mm to 5000 mm long (Horvath, 1996). Extruded block can be cut by the manufacturer, or in the field, to various sizes and shapes for field installation. When left in full size blocks for installation as light-weight embankment, EPS is referred to in this paper as "geofoam." However, other types of polystrene have been used to manufacture geofoam (Horvath, 1995).

EPS was invented in the 1950s. The first below ground application was to insulate foundations of residential homes in Scandinavian countries in the 1960s. The placement of EPS underneath pavements to prevent seasonal freeze-thaw developed concurrently in Scandinavia, Canada, and the United States. In the 1970s, the use of EPS as lightweight embankment in highway and earthwork developed concurrently in the United States and Norway (Horvath, 1995). Most notably in 1972, the Norwegian Road Research Laboratory (NRRL) placed geofoam in the approach fill of the Flom Bridge (Aaboe, 2000). Since that time, the NRRL has carried out a research and monitoring program on this installation and others, which has greatly added to the knowledge of the long-term performance and material properties of geofoam. In the United States, geofoam has been used in highway construction projects in Colorado, Hawaii, Indiana, Michigan, New York, and Utah.

The primary application of geofoam on the I-15 Reconstruction Project is to minimize settlement of underground utilities. Many existing utility lines traverse areas of raised mainline or ramp embankments. These utilities consist of high-pressure gas lines, water mains, and communication cables, which must remain in-service during construction. MSE embankments were predicted to induce primary settlements of up to 1 meter, exceeding all strain tolerances for these buried utilities. However, when the soil mass of the MSE walls was replaced by low-density geofoam the predicted settlements became minimal. This placement of geofoam enabled buried utilities to remain in-place, eliminating possible expensive interruption, replacement, or relocation. Figure 2 shows a photo of a completed geofoam embankment, without the tilt-up fascia panel wall at the 100 South Utility Corridor.

Another important use of geofoam on the I-15 project was to improve the stability of embankments. At some bridge locations, high embankments were required and the calculated safety factors against base failure were low. Such embankments are usually constructed with geotextile reinforcement and staged embankment construction that require several months of delay to allow excess pore pressure dissipation and subsequent shear strength gain. Construction of embankments with geofoam provided higher safety factors against instability and allowed the construction to proceed for critical path bridges. Figure 3 shows a typical bridge abutment with geofoam placed behind the abutment wall. This application of geofoam eliminated stability concerns at the bridge abutments and reduced the construction time by up to 75%. In addition, geofoam approach fills induce essentially no lateral pressure on retaining structures, provided that the soil-to-geofoam backslope transition is maintained at close to a self supporting repose angle, as shown in Figure 4.

DESIGN CONSIDERATIONS

Subsurface Conditions

Extensive geotechnical investigations were conducted along the I-15 corridor by UDOT and the design-build team. Much of the Salt Lake Valley is underlain by alluvium/colluvium from the nearby Wasatch Mountains that have interfingered with relatively thick deposits (5 to 10 m layers) of lacustrine silt and clay. The lacustrine deposits originate from the Great Salt Lake and its fresh water lake predecessors that were common in the Great Basin during Tertiary time. Cone penetrometer (CPT) logs and sampling from borings reveal interbedded sand layers within the lacustrine deposits, which mark numerous transgressions/regressions of ancestral lake shores, probably due to climatic changes. The lacustrine soils are generally low plasticity clays (CL) with some layers of low plasticity silts (ML) and high plasticity clays (CH).

Extensive deposits of compressible lacustrine clays and clayey silts are located in the northern segment of the I-15 in the downtown area. These deposits have a maximum thickness of approximately 25 meters and are saturated due to the shallow groundwater table (< 2 m). Typically, these lacustrine sediments begin consolidation on the virgin compression curve when approximately 2 to 3 meters of embankment is placed. MSE walls of 8 to 10 meters in height, typically experience about 1 m of settlement due to primary consolidation of these clayey soils. In order to expedite excess pore pressure dissipation and primary consolidation, prefabricated vertical (PV) drains were placed beneath many embankments. Without PV drains, the lacustrine deposits require about 400 to 600 days to complete primary consolidation. Consolidation times can be accelerated to about 100 to 200 days by the installation of PV drains, which were typically placed on 1.5-meter triangular spacing to a depth of about 25 meters. Surcharging was extensively used to minimize the amount of expected post-construction settlement. Typically, embankments were surcharged with fill that exceeded the design embankment height by 30 to 40 percent. This made the height of some of the larger embankments (fill + surcharge) about 14 meters above original ground, which triggered large settlements.

However, due to its extreme light weight, geofoam embankments do not trigger primary consolidation, nor result in excessive secondary consolidation settlements. Geofoam embankments were designed to produce "zero net load" on the foundation soils. This was accomplished by full load compensation, which meant subexcavating a volume of soil equal to the weight added by the new construction.

Material Properties

The I-15 Reconstruction Team specified geofoam with no more than five percent regrind content. Although both Type VIII and Type II geofoam (ASTM C-578) were approved, only Type VIII geofoam was used (Table 2). The blocks as installed were 0.8 m high by 1.2 m wide by 4.9 m long. The blocks, as manufactured, met the specified ± 0.5 percent dimensional and 5% flatness tolerances and trimming was not necessary. The overall design considered the nominal compressive resistance at 10 percent strain of 90 kPa for the specified Type VIII geofoam under ASTM-C-578-95. Actual tests performed at a strain rate of 10 percent per minute on a series of standard 50 mm side cube samples, Figure 5, indicate the density consistently exceeded the 18 kg/m^3 of the specification. The initial lag in the stress strain curves is due to uneven contact and must be adjusted. Corrected initial Young's moduli from these tests were in the range of 2.9 to 5.1 MPa. The compressive resistances at adjusted 5 and 10 percent strain were on average 97 and 111 kPa, respectively, with both exceeding the specification level for Type VIII geofoam in ASTM-C-578.

The range of densities and compression resistances at 5 percent strain represented in Figure 5 are shown in Figure. 6. The best fit line, equation (1), predicts compressive resistance for other densities of geofoam. A similar expression is given, equation (2), in the new European Standard (1998) for compression resistance at 10 percent strain.

$$\sigma_d = 7.3*D - 47 \qquad (1)$$

$$\sigma_d = 9.4*D - 76 \qquad (2)$$

Where σ_d is compressive resistance in kPa and D is density in kg/m^3.

The 5 percent criteria generally results in a compressive resistance that is about 10 percent lower than that for the 10 percent strain level. To limit long term creep deformation of the geofoam blocks, working stress levels due to dead load were limited to 30 percent of the compressive resistance for Type VIII geofoam with an additional of up to 10 percent allowed for live load due to traffic. Such criteria have been used widely before and are believed to result in no more than 2 percent creep strain in 50 years (European Standard, 1998). An alternative approach used in Japan is to limit working stress levels to compressive resistance at 1 percent strain (Miki, 1996). The two methods can be shown to be equivalent.

Corrected initial modulus values that are derived from standard tests as in Figure 5, are generally too low and over predict settlements when used in analyses (Frydenlund et al, 1996). Recent results on large block samples tested at Syracuse University now show that end effects unduly influence data from small specimens. Provided the imposed stresses are confined to induce predominantly elastic strains, the deformation that occurs in the geofoam will mostly take place during construction and post-construction deformation will be small. Thus the more meaningful modulus for practical purposes is the dynamic or resilient modulus. Because of the depth of pavement and load distribution of the concrete slab, stress increments that develop in the geofoam due to live loading are relatively small. Dynamic moduli from large block samples are of the order of more than double to triple the initial value obtained from conventional monotonic tests. Comparable initial moduli are also beginning to be observed in monotonic tests on full height samples obtained from laboratory testing and with local measurement of deformations.

The behavior of EPS geofoam is strain rate dependent, particularly at higher strain levels. A lower value of compressive resistance develops with decreasing strain rate. Thus the value of specifying compressive resistance at set strain level of 5 or 10 percent and based on standard specimen sizes serves mainly as reference. There have been other projects that have been designed on the same basis and performed well. Perhaps more than confirming the validity of the methodology, the evidence that there have so far been no reported or documented cases of failed geofoam embankments suggests a reasonable degree of conservatism in current methods.

Interface shear strengths between geofoam blocks and between geofoam and bedding sand are shown in Figure 7. The test results are for a range of normal stresses due to the pavement load on the geofoam. Also shown as a lower bound envelope is the interface friction coefficient of 0.6 used in the I-15 design. The lower coefficients for the sand to foam interface imply failure at the interface would be localized to occur within the sand. Coefficients for both the foam-to-foam and foam-to-sand interfaces slightly decrease with increasing normal stress.

The load distribution concrete slab over the geofoam fill was cast in place. A relatively strong adhesion bond and a rough texture develops between poured in place concrete and geofoam surfaces resulting in a much higher interface strength than between foam to foam. In some cases, the scheduling of the load distribution slab construction fell behind the geofoam fill completion. The geofoam surface was exposed to prolonged duration of sunlight. Discoloration and dusting of the surface occurred due to UV degradation. The effect of surface degradation on interface strength between geofoam and cast in place concrete was investigated. Samples were subjected to accelerated UV exposure in a weatherometer and field samples exposed to the 90 days specification limit were recovered. Interface strengths determined for fresh foam, UV lab exposed surfaces and field degraded samples Figure 8. Also shown are results for field degraded but power washed geofoam to cast in place concrete interfaces. On the time scale, the 90 days of field exposure is approximated as being equivalent to 50 hours of UV exposure in the weatherometer. The design interface coefficient of 0.6 that was assumed for all interfaces involving foam is also shown as a lower bound for all of the test data. Interface strengths between geofoam and cast inplace concrete decrease with the level of UV exposure and surface degradation. Powers washing before concrete pouring was effective in removing the degraded surface and enabled full regain of interface strength to a value comparable for a fresh geofoam interface. Analyses indicate the interface strength demand due to braking or acceleration of trucks can be met by a friction coefficient of less than 50 percent of the design level of 0.6. The specification requirement for covering geofoam with plastic sheeting for exposure duration beyond 90 days can be relaxed. The sheeting was an additional expense and securing for protection against wind was necessary. If desired, reconditioning of UV degraded load bearing surfaces by power washing was a better alternative.

Barbed metal plates or binder plates were used with the intention of developing more interface shear resistance between geofoam blocks. However, test results performed for the I-15 Reconstruction Project indicate the plates do not provide significant resistance in one way loading and are even less effective on reverse loading. While the binder plates helped in maintaining the blocks in position during placement, the suppliers claimed value for enhancing shear resistance was minimal. This conclusion supports the previously expressed opinion of Sanders et al. (1996).

Solvent, Fire and Insect Protection

Geofoam must be protected from potential spills of petroleum based fuels and solvents (e.g., gasoline and diesel fuel) and from fire. The load distribution slab, pavement section, and fascia panel wall are the primary protection against spills. However, in applications where the geofoam was placed on a side slope, the addition of a geomembrane liner (28 mil minimum) was used. The geomembrane was specified as a tri-polymer consisting of polyvinyl chloride, ethylene interpolymer alloy, and polyurethane or a comparable polymer combination. A modified flame retardant resin was used for fire protection. Also, borate was added to prevent insect attack and boring intrusion. There has so far been no record of detrimental solvent or insect attack of geofoam fills for highway embankments anywhere. The extent and effectiveness of such pre-cautionary measures may need to be reviewed in future applications.

Connections

For the I-15 Reconstruction Project, the tilt-up-panel-facia wall is mechanically tied to the load distribution slab by threaded reinforcing bar placed in both elements and held together by threaded couplers. For one geofoam fill, which was 8 to 10 blocks high, this connection proved to be too rigid to accommodate some of the seating settlement within the geofoam mass and the connection was severed at a few locales. Seating settlement of approximately 3 to 4 cm, as measured by vertical extensometers, occurred during the placement of the untreated base coarse (UTBC) and Portland Cement Concrete Pavement (PCCP) above the geofoam block and load distribution slab. Seating settlement is partly caused by compression of a slight arch of individual geofoam blocks. This arch, or crown, in the geofoam blocks is visible prior to geofoam placement and is produced during ejection of the block from the mold and subsequent block cooling. Standard procedure by Wasatch Constructors' block installers is to place each block with the crown upward at all times. This practice allows for a relatively close fit of the block, but did not eliminate the presence of the crown, until the load of the overlying UTBC and PCCP was applied. Unfortunately, the connection between the tilt-up-panel-facia wall and the load distribution slab had been made prior to the occurrence of the seating settlement. This connection and construction sequencing is currently under reviewed and redesigned by the design-build team.

Water Absorption and Buoyancy

When placed underground, EPS will absorb water in two ways. First, water will enter voids between the beads, or cells, due to capillary rise. Second, water vapor may diffuse into the cells when a temperature gradient exists and later condense when the temperature decreases below the dew point. All geofoam placed on the I-15 project was placed at or near the existing surface grade, approximately 2 meters above high groundwater level, thus water absorption and buoyancy were not a great concern.

However, for other projects, where the geofoam will be placed at or near the water-table, long-term studies from Norway give valuable performance data and design guidance for water absorption (Aaboe, 2000). Test results from samples retrieved from EPS blocks placed in drained conditions (i.e., permanently installed above water-table) had water contents of 1 percent, or less. For blocks that were periodically submerged, water contents reached up to 4 percent by volume; and for blocks that were permanently submerged, water contents reached values approaching 10 percent by volume (Aaboe, 2000). Current Norwegian design practice is to use a design unit weight of 0.5 kN/m^3 for the drained case, and a design unit weight of 1.0 kN/m^3 for both the periodically and permanently submerged cases (Aaboe, 2000). This corresponds to an approximate design water content of 5 and 10 percent for the drained and submerged (e.g., periodic and permanent) cases, respectively, which appears to be reasonably conservative based on the retrieved samples.

Because of its light-weight nature, geofoam can also create large uplift forces when submerged. Norway has experienced a case where a geofoam supported highway literally floated, as a flood inundated the roadway (Aaboe, 2000). To counteract the buoyant force, the amount of material placed atop the geofoam must be increased. For the case of completely submerged geofoam, the design uplift force per unit submerged volume is about 9.6 kN/m^3. To counteract this buoyant force, Horvath (1995) recommends increasing the overburden weight atop the geofoam, until a factor of safety of 1.3 is obtained for water levels corresponding to a 100 year design flood event. If the depth of overburden is increased to counteract buoyancy, the designer should also ensure that the applied dead load does not exceed more than 30 percent of the compressive strength of the geofoam, in order to minimize long-term creep. Also, it is important to provide a well-drained sand layer behind and underneath geofoam embankments that are constructed into hillsides. Downslope groundwater flow should not be allowed to impound behind the geofoam mass, which could produce lateral pressures on the geofoam blocks and potentially result in lateral movement of the geofoam mass.

Long Term Performance

Based on current information, the material properties of geofoam do not appear to degrade significantly with time. Norwegian practice is requires a minimum compressive strength of 100 kPa for newly placed geofoam. To measure any potential degradation with time, Aaboe (2000) postulates that a significant loss of compressive strength would be an indicator of deterioration. To this end, the Norwegian Road Research Laboratory has exhumed and carried out strength testing on samples which have been underground for durations ranging from 4 to 24 years. Unconfined compressive strength for these samples ranged from 105 kPa to 130 kPa, with no distinct trend of decreasing compressive strength with age. Aaboe (2000) attributes most of the variation in geofoam strength to variations in geofoam quality at placement, and not to any degradation with time. Further, there was no sign of variation in compressive strength based on whether or not the retrieved samples were wet or dry. This suggests that water absorption has no affect on compressive strength.

Cost

Because of the nature of the design-build contract, some the itemized material and construction costs are not readily available. Further, making a blanket cost comparison between geofoam and earthen fills can be misleading. Each situation requires a complete review of the conditions and geometry before costs are compared. Direct costs of the geofoam, bedding, load slab, and facia wall must be compared to the costs of excavation, PV drains, geotextile, fill, surcharge and construction for each locale. Beyond the easily determined direct costs, less tangible costs must also be considered to make the comparison more meaningful. Potential improved life cycle costs to pavement, reduced construction time, elimination of utility relocation costs must be included in the evaluation. An approximation of costs for the installation of geofoam on the I-15 project is given in Table 3.. The cost summary includes all labor and materials and is averaged over all applications of geofoam on the project

Standard Drawings and Specifications

Standard Drawings and specifications were developed for geofoam applications on the I-15 corridor by Wasatch Constructor's Design-Build team. Figure 4 gives details of a typical section through a geofoam fill. The fascia panel, roadside barrier as well as details for a utility trench and pipe are also shown. Table 1 lists all the geofoam standard drawings that are currently available. Copies may be obtained by request from the Research Division, Utah Department of Transportation, 4501 S. 2700 W., Salt Lake City, Utah, 84114-8410.

CONCLUSIONS

Geofoam was successfully used as an alternative construction material for the I-15 reconstruction. Design and construction utilizing the very lightweight advantage of geofoam enabled settlement sensitive buried utilities to remain in service without need for relocation or disruption. Use of geofoam improved the base stability of high embankments. Primary consolidation settlements were not triggered and long term settlements are expected to be minimal for geofoam fill areas that were designed under no net load condition. Using geofoam at critical segments of the project has saved considerable time. Standard drawings have been developed and field monitoring is in progress. Experience gained at I-15 will benefit other like projects in the future.

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Drawing Number	I-15 Corridor Standard Plan Title
CS-42-1, CS-42-2	Catch Basin Down Drain in Geofoam
CS-43, CS-78	Elevation - Geofoam Walls
CS-44, CS-79	Geofoam Wall Panel Details
CS-45, CS-80	Geofoam Wall Restraint Details
CS-46, CS-81	Geofoam Wall Grade Beam Details
CS-47	Geofoam Wall Connection Details
CS-48-1	MSE Geofoam Conform Detail
CS-48-2	Load Distribution Slab Parapet Wall Detail
CS-49-1, CS-49-2, CS-49-3	Geofoam Coping at Bridges
CS-50	Geofoam Installation at Abutments
CS-51, CS-52, CS-77, CS-91, CS-92	Typical Geofoam Section
CS-53	Load Distribution Slab Drain

Table 1. Geofoam Standard Drawings used on the I-15 Reconstruction Project.

Physical Property	ASTM Test Procedure	Type VIII Accepted Value	Type II Accepted Value	Tolerances
Density	D1622	18 kg/m^3	22 kg/m^3	± 10 %
Compressive Resistance	D1621	90 kN/m^2	104 kN/m^2	minimum @ yield or 10 percent axial deformation
Flexural Strength	C203	208 kN/m^2	276 kN/m^2	Minimum
Water Absorption	C272	3	3	< % by volume

Table 2. Properties of Type VIII Geofoam Specified for the Reconstruction I-15 Project.

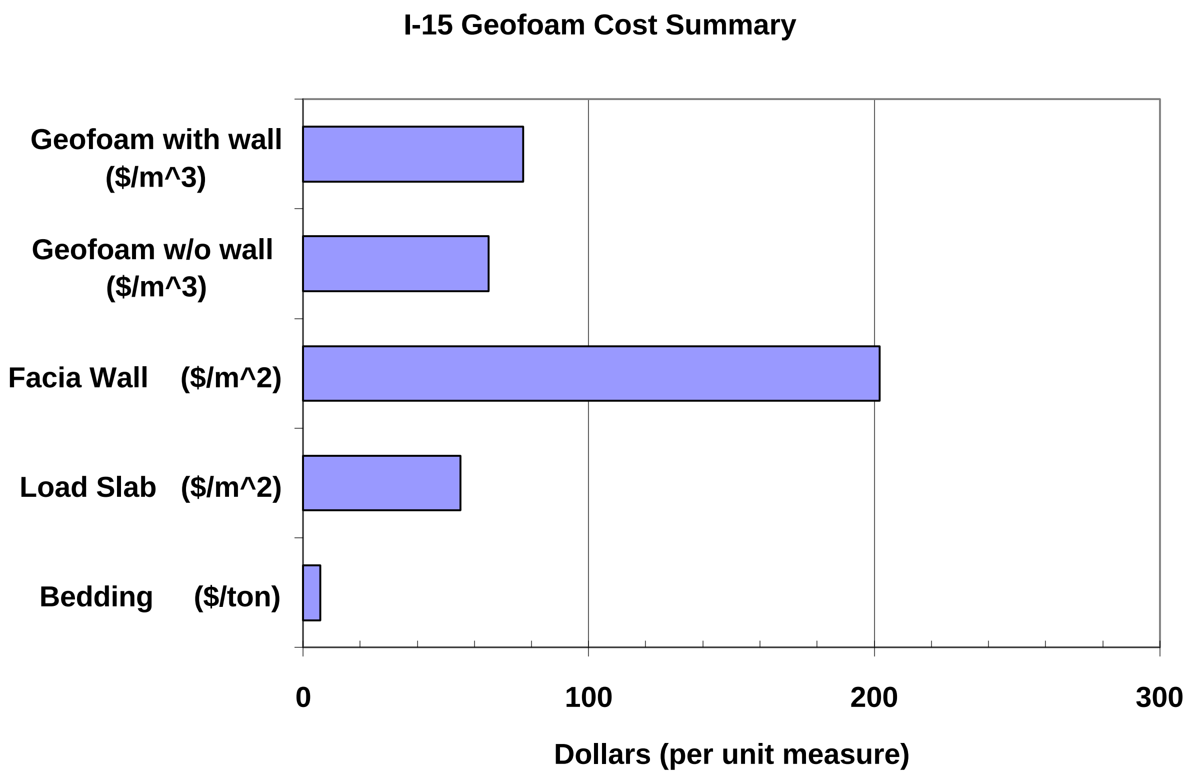

Table 3. Approximate Costs for Geofoam Installation at the Reconstruction I-15 Project.

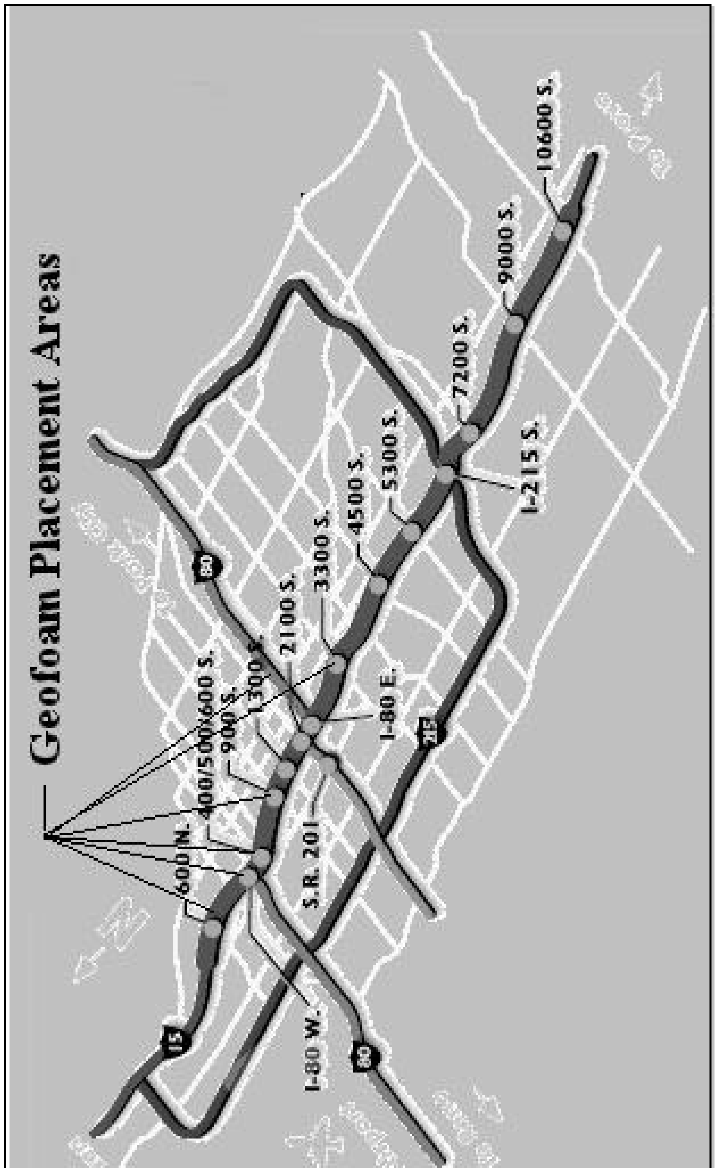

Figure 1. I-15 Alignment and Geofoam Placement Areas in Salt Lake City.

Figure 2. A Geofoam Embankment at 100 South Utility Corridor Crossing of I-15.

Figure 3. Typical Bridge Abutment with Geofoam Backfill.

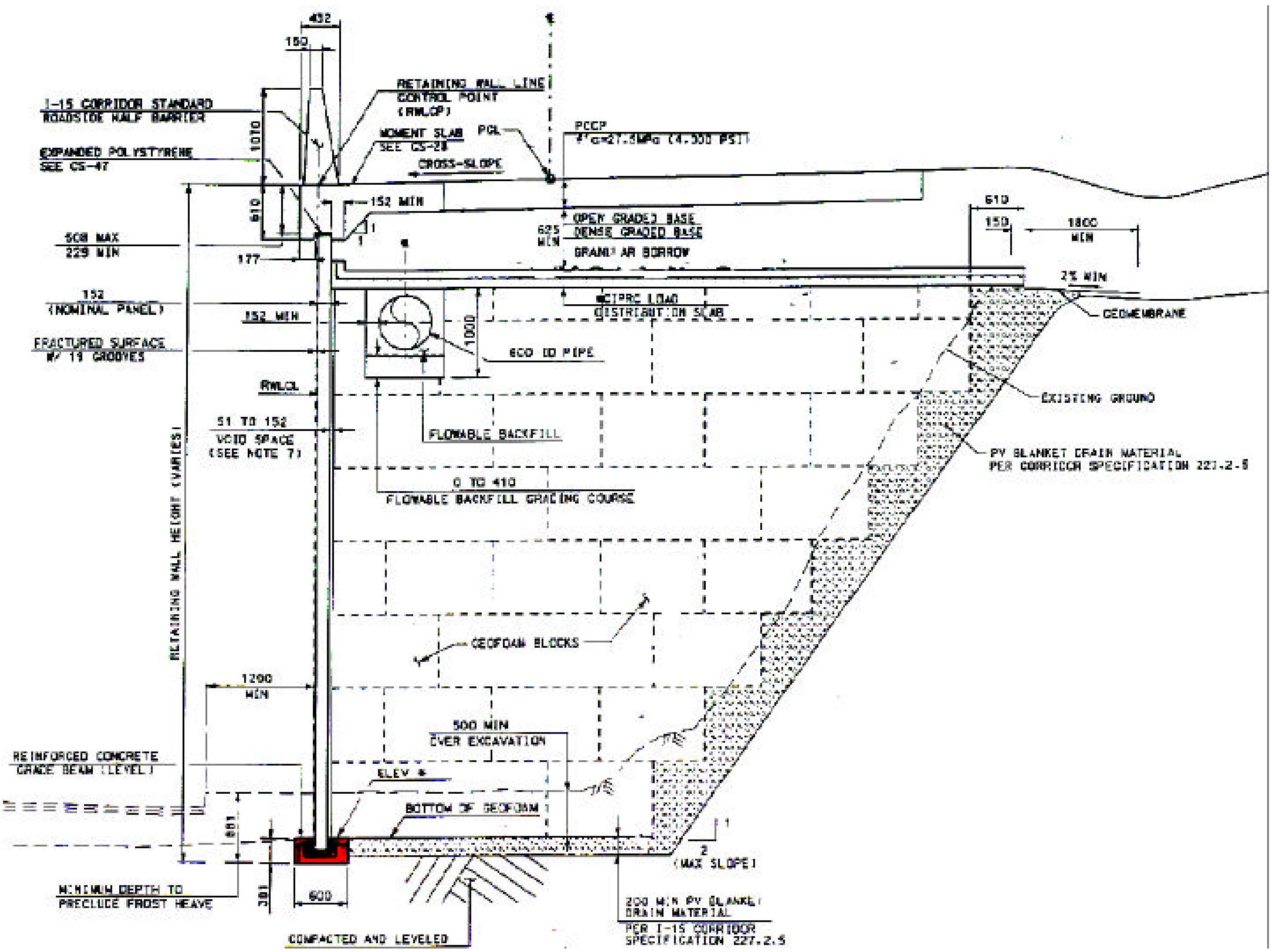

Figure 4. Details of a Typical I-15 Project Geofoam Fill.

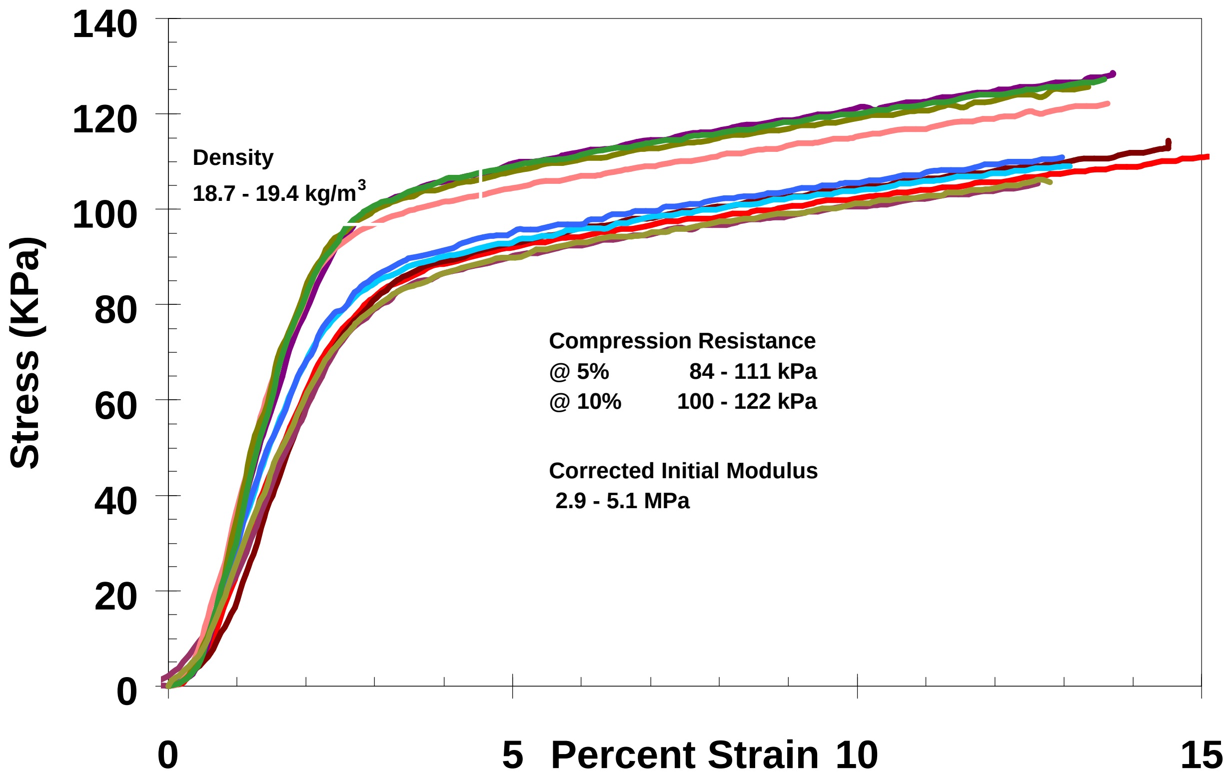

Figure 5. Stress-Strain Curves for Type VIII Geofoam, 50-mm Samples at 10% Strain Rate.

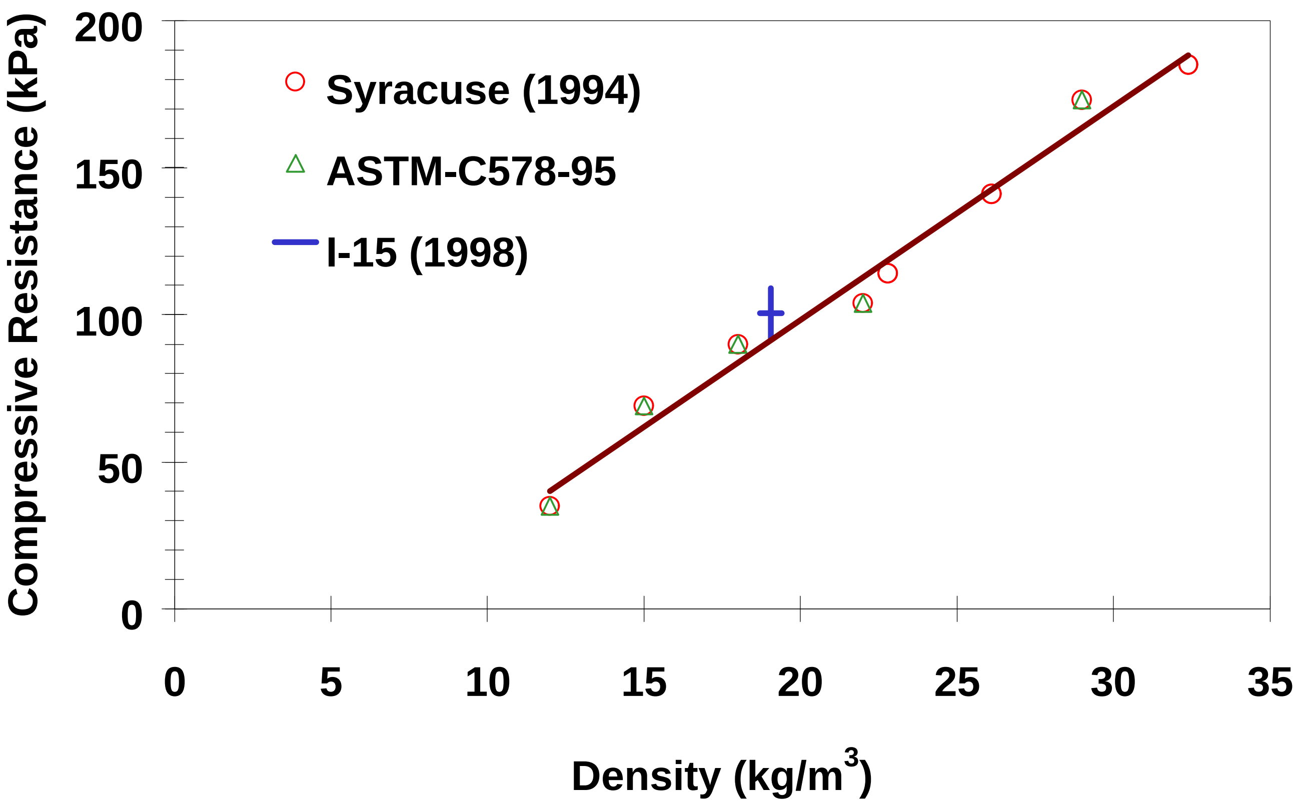

Figure 6. Compressive Resistance versus Geofoam Density.

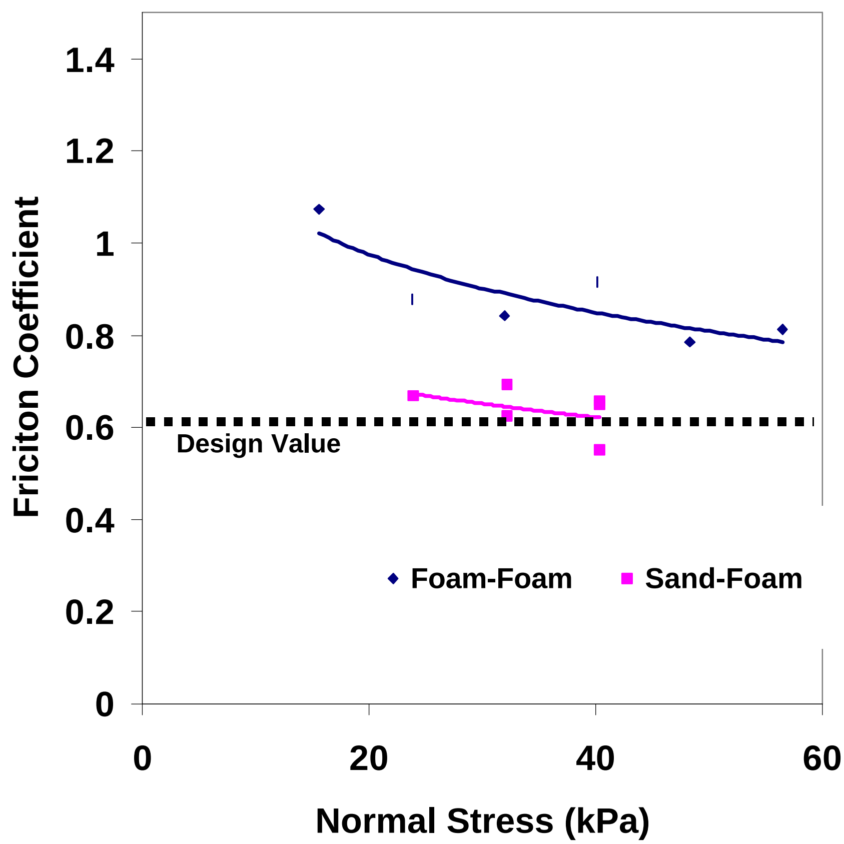

Figure 7. Interface Coefficients for Type VIII Geofoam.

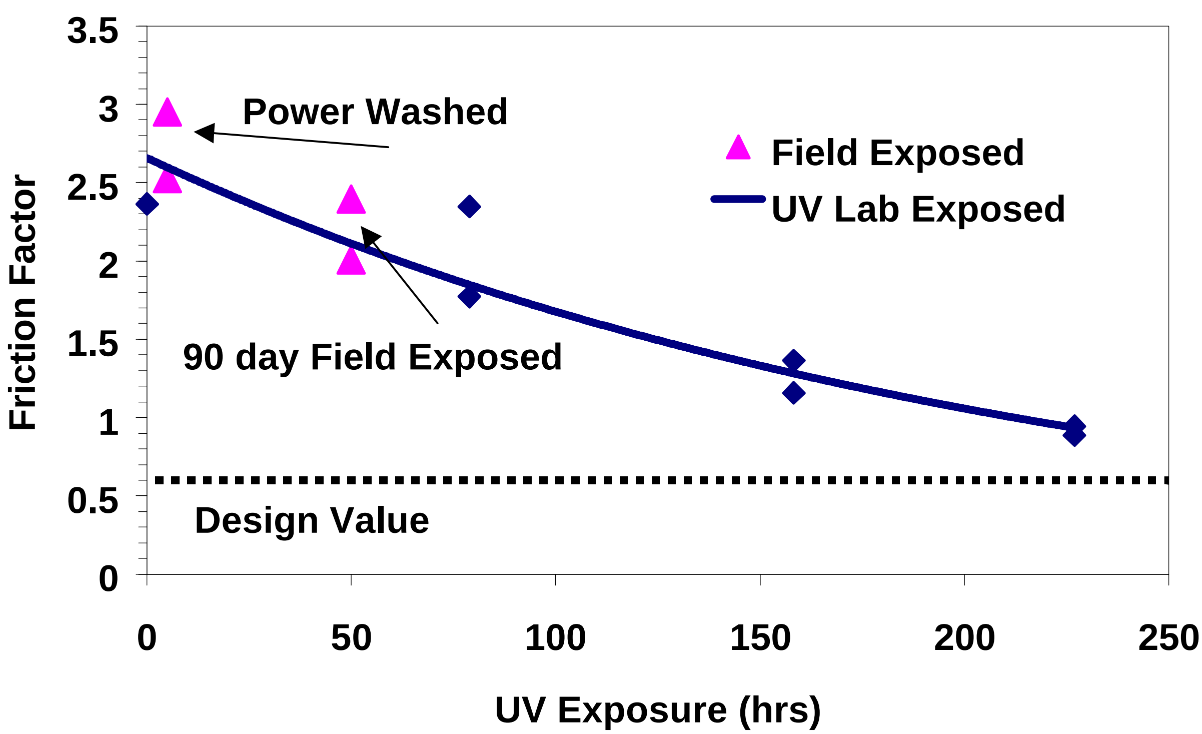

Figure 8. Interface Coefficients for Geofoam – Cast in Place Concrete with UV Exposure Duration.