Lightweight and Sustainable Materials in Engineered Fills

FINAL REPORT

October 1, 2021

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16. Abstract

Lightweight materials have been in production for decades as alternatives to traditional engineered fills using soils. Several options exist with varying ranges in material properties and necessary efforts related to fill construction/placement. When properly designed, there are a number of advantages to application of such lightweight fill materials, including reduction of loads on retaining systems and reduction of induced settlements on underlying soils. For certain lightweight fill materials, there are also potential advantages related to lower costs and accelerated construction times due to decreased reliance on staged construction. Consequently, utilization of a lightweight fill may be an appropriate approach to mitigate settlements at problematic sites and avoid more costly alternatives such as ground improvement measures or deep foundations to bypass surficial soils. Given the potential cost and time savings, there is a critical need to provide guidelines regarding appropriate implementation of lightweight fills to encourage their use, when appropriate, on PennDOT District 6-0 projects. These guidelines should consider factors such as application (e.g. foundation support, retaining structures, pavement subgrade, etc.), performance, ease of construction, and costs. Additionally, there is an increasing trend towards the use of sustainable or recycled products in lightweight fill materials. This has the added benefit of decreasing the carbon footprint of a project and promoting the application of sustainable materials for construction. Such benefits align well with PennDOT's Green Plan Policy. This final report summarizes research efforts to review current practices, case histories, and the state of the art technologies with respect to lightweight and sustainable engineered fills (e.g. lightweight concrete, foamed concrete, foamed glass, geofoam, tire derived aggregates, etc.), and provides recommendations regarding various aspects related to their use in lieu of other alternatives for problematic sites.

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Lightweight and Sustainable Materials in Engineered Fills

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Final Report

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1. INTRODUCTION

Lightweight materials with varying ranges in material properties have been in production for decades as alternatives to traditional engineered fills using soils. These materials can offer a number of advantages, including reduction of loads and induced settlements. Some projects may also realize lower costs and accelerated construction times when lightweight fill materials are used, particularly when compared to costly alternatives such as ground improvement techniques or the use of deep foundations. Additionally, the increasing use of sustainable or recycled products in lightweight fill materials has the added benefit of decreasing the carbon footprint of a project and promoting the application of sustainable materials for construction. The purpose of this research project (TEM WO 013) was to provide guidelines regarding appropriate implementation of lightweight and sustainable fill materials for use on PennDOT District 6-0 projects based on a comprehensive literature review and a review of recent case histories where such materials have been implemented.

1.1. Organization of the Report

As already noted, this report is based on a comprehensive literature review that explored various sources of information regarding lightweight/sustainable fill technologies, including projects reports, journal and conference papers, and published case histories. Based on these efforts, a set of guidelines was developed to aid engineers in selecting between various lightweight/sustainable fill technologies. The report is consequently organized into the following chapters:

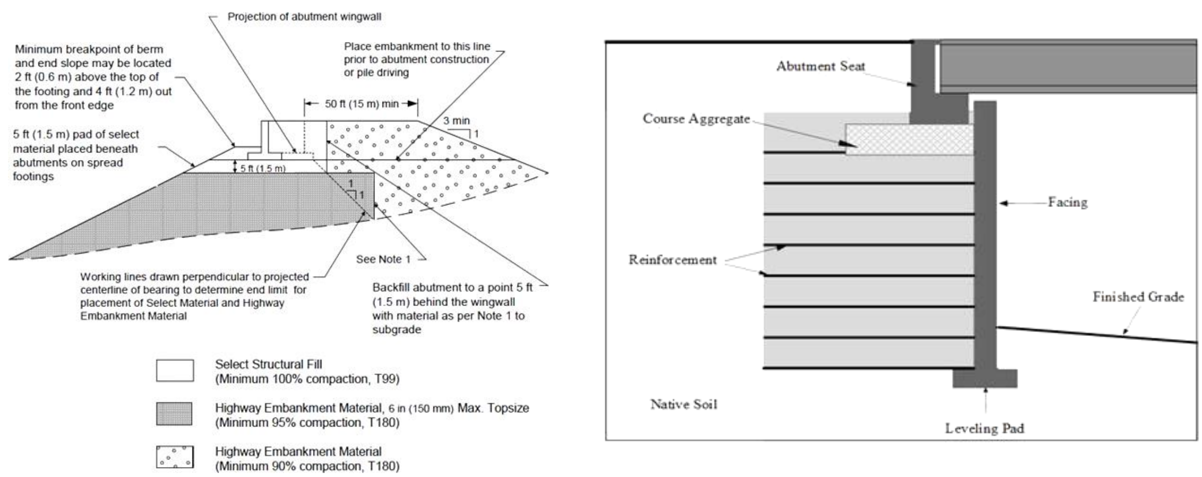
- Chapter 1: This chapter provides background information regarding the current standard of practice with respect to lightweight and sustainable fill technologies. Additionally, the current state of the art is explored to summarize new developments and any knowledge gaps that exist in the literature. This is accomplished by performing a comprehensive literature review on topics related to: (1) available lightweight and sustainable fill alternatives; (2) relevant engineering properties of lightweight and sustainable fill materials; (3) advantages and limitations of various lightweight and sustainable fill technologies; (4) applications of various lightweight and sustainable fill materials, including in conjunction with other geotechnical technologies such as ground improvement; and (5) recent developments related to materials and/or applications of lightweight and sustainable fill technologies.
- Chapter 2: This chapter specifically discussed projects where lightweight and/or sustainable fill materials were used on PennDOT projects. A total of three projects is presented. Included in the discussion for each project is detailed information regarding

project site conditions, geotechnical design, the specific lightweight/sustainable fill technology selected, and project costs.

- Chapter 3: During the literature review, several case histories where other departments of transportation constructed with lightweight/sustainable fill materials were identified. This chapter provides detailed discussion of these projects, again including information regarding the site conditions, geotechnical design, and selection of the lightweight and/or sustainable fill technology. Since these projects were not implemented by PennDOT, detailed cost information was not readily available.
- Chapter 4: This chapter takes the information summarized in the earlier sections of the report and synthesizes it into guidelines regarding appropriate selection of lightweight/sustainable fill technologies. Specific discussion is presented about what aspects of a lightweight/sustainable fill technology should be considered to establish the viability of such materials in highway construction.
- Chapter 5: This chapter briefly summarizes the major points emphasized throughout the report.

1.2. Engineered Fills

In many transportation projects, it is necessary for earthwork operations to provide grading at the site, including for placement of fills for foundation and retaining systems, and/or the construction of embankments (Figure 1.1). For the purposes of this report, “fill” will refer to a volume of earthen material that is placed and compacted in a hole or depression. Different terminology is used in the literature to describe screened earthen fill material specifically selected to help create a strong and stable support layer, including “structural fill”, “compacted structural fill”, “compacted granular fill”, and/or “engineered fills” (e.g., Chesner et al. 1998; Xiao et al. 2016). In roadway applications, such fills are often used in abutments or slabs, backfill for retaining structures, or filling of trenches and other excavations for roadway support. The term “embankment” refers to a volume of earthen material that is placed and compacted to raise the grade above the existing ground surface. One common example is the approach to a bridge, where the embankment raises the roadway so that it meets the same elevation as the bridge deck. Consequently, embankments are typically larger structures that may spatially span miles while fills are often smaller, discrete structures.



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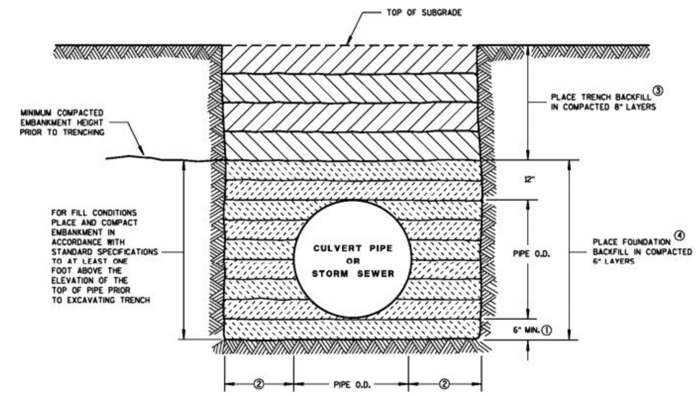


(c)

Figure 1.1. Examples of engineered fill applications: (a) structural approach road and bridge foundation (Xiao et al. 2016); (b) mechanically stabilized earth (MSE) wall (Xiao et al. 2016); and (c) pipe culvert (WisDOT 2017).

1.2.1. Types of Fill Materials

Both embankments or fills may be constructed of a relatively wide range of earthen material, including soils, aggregate, rock, and/or crushed paving material. Properly placed and compacted embankments and fill will be more rigid and uniform and have greater strength than most natural soils. However, care must be exercised in the selection of an appropriate fill material for the particular application in a given project. Broadly speaking, embankment and fill materials can be differentiated into two categories: (1) uncemented materials; and (2) cemented materials. The following sections briefly describe these fill materials and technologies.

1.2.1.1. Uncemented Fills

Uncemented backfills do not use any binding agents with the filling materials. The main uncemented fills include soil/aggregate/rock fills and hydraulic fills. Soil/aggregate/rock fills are constructed by compacting the earthen materials into place using grading equipment. In hydraulic fills, the earthen material is transported in suspension in water and is therefore placed by sedimentation. The sorting effect of flowing water can be utilized to control the gradation of the fill in select locations. Hydraulic fills are less common in transportation applications and are more commonly encountered in coastal reclamation projects (Choa 1994), earthen dam construction (Mejia et al. 2005), and dyke/levee systems (Sills et al. 2008), and backfilling underground mining stopes (Rankine et al. 2007).

Since engineered soil/aggregate/rock fills and embankments are typically constructed by compacting earthen materials in place, their performance is governed by the material properties that most affect compatibility, including gradation and moisture content. Additionally, given their application for load-carrying purposes, other material properties are also important, including their compressibility, shear strength, permeability, durability, plasticity, corrosion resistance, and frost resistance (Chesner et al. 1998). Generally, uncemented fill materials should be selected such that they meet a number of criteria:

- Provide adequate strength to support any overlying structures
- Appropriate gradation or use of filter fabric to prevent migration of native soils into the fill
- Adequate permeability to allow free drainage of any water accumulated in the fill
- Must not deteriorate in water
- Capable of being placed in a controlled and consistent manner for compaction
- Compatible with any overlying structures and/or native soils based on corrosion potential and chemical content

Consequently, the selection of appropriate engineered fill and embankment materials is governed by specifications developed by FHWA, AASHTO, and state departments of transportation. For example, Table 1.1 provides considerations for structural fills based on the FHWA *Soils and Foundations Reference Manual: Volume I* (Samtani and Nowatzki 2006). Additionally, PennDOT's Publication 408 highlights construction specifications for PennDOT projects and notes specific criteria for embankments and structural fills from various classes of excavations on a project (Table 1.2).

__Table 1.1. General considerations for structural fills (Samtani and Nowatzki 2006).__

<table>
<tr><th>Consideration</th><th>Comment</th></tr>
<tr><td>Lift Thickness</td><td>Limit to 6" to 8" (150 mm to 200 mm), so compaction is possible with small equipment.</td></tr>
<tr><td>Topsize (largest particle size)</td><td>Limit to less than ¾ of lift thickness.</td></tr>
<tr><td>Gradation/Percent Fines</td><td>Use well graded soil for ease of compaction. Typical gradation is as follows:
<table>
<tr><th>Sieve Size</th><th>Percent Passing (by weight)</th></tr>
<tr><td>4-in (100 mm)</td><td>100</td></tr>
<tr><td>No. 40 (0.425 mm)</td><td>0 to 70</td></tr>
<tr><td>No. 200 (0.075 mm)</td><td>0 to 15</td></tr>
</table>
The limitation on percent fines (particles smaller than No. 200 sieve) is to prevent piping and allow gravity drainage. For rapid drainage, consideration may be given to limiting the percent fines to 5%.</td></tr>
<tr><td>Plasticity Index</td><td>The plasticity index (PI) should not exceed 10 to control long-term deformation.</td></tr>
<tr><td>Durability</td><td>This consideration attempts to address breakdown of particles and resultant settlement. The material should be substantially free of shale or other soft, poor-durability particles. Where the agency elects to test for this requirement, a material with a magnesium sulfate soundness loss exceeding 30 should be rejected.</td></tr>
<tr><td>T99 Density Control</td><td>Small equipment cannot achieve AASHTO T180 densities. Minimum of 100 percent of standard Proctor maximum density is required.</td></tr>
<tr><td>Compatibility</td><td>Particles should not move into voids of adjacent fill or drain material</td></tr>
</table>

__Table 1.2. PennDOT material specifications for placement and compaction of embankment and fills from Common Borrow, Foreign Borrow, and Selected Borrow Excavations (PennDOT 2020).__

Embankment/Fill Material	Description
Soil	Material consisting of earth having 20% or more of the material passing the No. 200 sieve and having a minimum dry density of 95 pounds per cubic foot oven-dried mass determined according to PTM No. 106. Material must have a maximum liquid limit of 65, determined according to AASHTO T 89, and a plasticity index of not less than the liquid limit minus 30, determined according to AASHTO T 90 for soils with liquid limits of 41 to 65.
Granular Material, Type 1	Material consisting of natural or synthetic mineral aggregates having greater than 70% of the material passing the 3/8-inch sieve (less than 30% retained on the 3/8-inch sieve) and less than 20% passing the No. 200 sieve, except for AASHTO No. 8 coarse aggregate and select granular material (2RC).

Granular Material, Type 2	Material consisting of natural or synthetic mineral aggregates having less than or equal to 70% of the material passing the 3/8-inch sieve (greater than or equal 30% retained on the 3/8-inch sieve) and less than 20% passing the No. 200 sieve. Also includes AASHTO Nos. 8 or 57 coarse aggregate, or PennDOT Nos. 2A or OGS coarse aggregate meeting the requirements specified in Section 703.2, select granular material (2RC) meeting the requirements specified in Section 703.3, and structure backfill.
Rock	Includes natural material that cannot be excavated without blasting or using rippers; also boulders, detached stones, and concrete and masonry units of a size that cannot be readily incorporated into compacted 6-inch layers and having insufficient soil to fill the voids in each layer.
Shale	Includes rock-like material formed by natural consolidation of mud, clay, silt, and fine sand; usually thinly laminated, comparatively soft, and easily split.
Random Material	Includes Type 1 or Type 2 granular material combined with shale, concrete, brick, stone, or masonry units that can readily be incorporated into compacted 6-inch layers.

In addition to criteria regarding the selection of appropriate materials for soil/aggregate/rock fills, guidelines exist for their placement, compaction, and subsequent quality control. This includes method of compaction as well as moisture, density, and acceptance testing. Generally, the material is prescribed to be compacted in lifts not to exceed 6 inches to ensure adequate long-term performance of the engineered fill. Depending on the type of embankment/fill material, additional construction specifications exist regarding the spatial extent of their placement, acceptable compaction equipment, lift thickness, and frequency of quality control testing (e.g., PennDOT 2020).

1.2.1.2. Cemented Fills

Cemented fills use a small percentage of binder material, including Portland cement or a blend of Portland cement with other pozzolans such as fly-ash, gypsum, or blast furnace slag to help the binding and increasing the strength of mixtures. Examples of cemented fills for mining backfill purposes include cemented rock fills (CRF), cemented hydraulic fills (CHF), and cemented paste backfill (CPB) (Grice 2001; Kesimal et al. 2003; Fall et al. 2005; Rankine et al. 2007; Fall et al. 2008; Ercikdi et al. 2009; Pokharel and Fall 2013; Ghirian and Fall 2014). Generally, for transportation purposes, such cemented fills fall under the general category of flowable backfills [also referred to as controlled low-strength materials (CLSM) or controlled density fills] since they are typically placed by a tremie prior to hardening of the cementitious material (e.g., Janardhanatn et al. 1992; Peindl et al. 1992; Lee et al. 2001a,b).

As with engineered uncemented fills, the selection of appropriate materials and construction of cemented/flowable fills is governed by specifications developed by FHWA, AASHTO, and state departments of transportation. PennDOT's Publication 408 again notes specific criteria for flowable backfills based on application:

- Flowable Backfill, Type B: Future excavation of the backfill may be necessary such as at utility trenches, pipe trenches, bridge abutments, and around box or arch culverts.
- Flowable Backfill, Type C: Excavation of backfills not anticipated, including replacing unsuitable soils below structure foundations, filling abandoned conduits, tunnels and mines, and backfilling around pipe culverts where extra strength is required.
- Flowable Backfill, Type D: Construction in areas requiring low-density backfill material as in abutments over highly deformable soils, backfilling retaining walls, filling vaults, and backfilling on top of buried structures.

PennDOT Publication 408 also provides specifications for the allowable materials used in the mix design for the flowable backfill, including cement, fly ash, slag cement, fine aggregate, coarse aggregate, bottom ash, water, and any admixtures (Table 1.3). As with engineered uncemented fill materials, additional specifications are prescribed for the placement of cemented/flowable fills and subsequent quality control.

Table 1.3. PennDOT specifications for mix design of flowable backfill materials (PennDOT 2020).

Properties & Criteria	Type B	Type C	Type D
Mix Design (/CY) Cement (lbs)* Supplementary Cementitious Material (lbs)* Bottom ash (lbs)* or Coarse Aggregate or Fine Aggregate Air Generating Admixture*	 50 300 2600	 150-200 300 2600	 300-700 100-400 **
Slump (inches) AASHTO T 119, ASTM C 136	7 min ****	7 min ****	7 min ****
Density (pcf) AASHTO T 121, ASTM C 136	N/A	N/A	30-70 or as specified ***
Water Absorption of Aggregate AASHTO T 85	--	--	20% max
Compressive Strength (psi) PTM No. 604 28 Days	125 max	800 min	90-400

*Quantities may be varied or alternate designs submitted to adapt mix to conform to density and strength requirements or to adapt to specific site conditions.
**Requires using a suitable lightweight aggregate or air entraining admixture. Provide a mix design that achieves the specified strength and density requirements.
***Approximate Value. Use of air entraining agent may reduce these values.
****Some applications may require containing flowable backfill by constructing dikes from the mix by using less water to produce a 3-inch minimum slump, if approved by the Representative. Thickening of the mix in other areas is allowed if approved by the Representative.

1.2.2. Applications and Need for Lightweight and Sustainable Fill Materials

There are a wide range of applications where embankment and engineered fills are necessary for transportation projects. These applications can consist of back fills on foundations and retaining walls, bridge abutments, cut and bench fill construction for slope stability, void fills, utility and pipe trenches, culverts, annular pipe fills, development of working platforms, subgrade for pavements, soil remediation, and landscaping. As noted in the previous section, there are many available options for embankment and engineered fill materials. Determination of the most appropriate material for use on a particular application depends on a number of practical and site-specific considerations and factors, including the required support, site constraints, project proximity to fill material sources, material costs, and construction costs.

One particular aspect of material selection for engineered fills and embankments is the unit weight. Typical fills and embankments consist of earthen material with a unit weight generally in the range from about 115 to 140 pounds force per cubic foot (pcf). This unit density can result in significant settlement of any underlying soils or decreased stability on some projects. For example, the earth pressures behind a retaining wall backfilled with either uncemented or cemented fill materials may necessitate a larger cross section, cantilever, or increased reinforcements in the case of mechanically stabilized earth (MSE) walls. An earthen embankment may generate substantially high vertical stresses that lead to excessive amounts of short-term or long-term settlements. Consequently, the use of lighter weight materials for embankments or engineered fills is desirable. In this manner, lightweight fill and embankment materials can be categorized as a “ground improvement” technology since they address the issues associated with problematic native soils at a transportation project site (e.g., Schaefer et al. 2017a,b). Significant efforts have been undertaken over the last few decades to evaluate the use of such lightweight materials in transportation-related embankments and fills (e.g., Stoll and Holm 1985; Valsangkar and Holm 1990; Allen and Kilian 1993; Kilian and Ferry 1993; Horvath 1994; Stark et al. 2004; Arellano et al. 2011; Ahn and Cheng 2014; Xiao et al. 2015).

Another aspect affecting selection of embankment and fill materials for transportation projects is the increasing frequency with which recycled/reclaimed materials are being used due to increasing public awareness of CO_2 generation, diminishing nonrenewable natural resources, and the need for sustainable construction. Given the large amounts of material used in transportation projects, use of recycled/reclaimed materials can provide a useful avenue by which to “dispose” of otherwise waste materials. Much research has been performed over the last few decades to identify what waste recycling streams can prove valuable to transportation construction [e.g., see Stroup-Gardiner and Wattenberg-Komas (2013a) for a comprehensive discussion and references]. Consequently, as beneficial transportation material byproducts have been identified, local, state, and federal programs have encouraged their use in various highway-related applications. However, these materials exhibit considerable differences in costs, availability, engineering properties, sustainability, constructability, and performance.

The goal of the remainder of this chapter is to provide a thorough review of the application of lightweight and sustainable fill materials. Given the recent research developments and the wide range of applications, material properties, and performance of lightweight and sustainable fill materials, the goal was to synthesize the available literature and provide a snapshot of the current standard of practice and state of the art with respect to these materials. The subsequent discussion is divided into two sections, one that focuses on lightweight embankment/fill materials (section 1.4) and one that focuses on sustainable embankment/fill materials (section 1.5). Lightweight embankment/fill materials that are also sourced from recycled, waste, and/or by-product materials are discussed within the context of sustainable embankment/fill materials.

1.3. Lightweight Fill Materials

As noted previously, there are many situations where the use of typical uncemented or cemented embankment and fill materials can result in excessive overburden stresses on soils, lateral pressures behind retaining walls, and decreased stability of any number of transportation projects. The use of lightweight embankment and fill materials has long been recognized as a means to address these issues by increasing stability and reducing gravitational loads and settlements. Additionally, some lightweight fill technologies are more durable and allow for reduced construction time owing to simpler placement procedures, reductions in staging due to consolidation settlements, and ability to construct in adverse weather conditions. The subsequent decrease in project construction costs can offset the potential for increased material costs associated with some lightweight fill technologies. Given these benefits, lightweight fill materials have been developed and used for decades in transportation-related projects (PIARC 1997; Stroup-Gardiner and Wattenberg-Komas 2013a; Schaefer et al. 2017a). The following section provides some background information regarding lightweight fill technologies in general. Subsequent sections will focus on particular materials and expand on their use by discussing material properties (density, strength, compressibility), design concerns, and case histories. It should be noted that some lightweight fill materials are also derived from recycled materials, waste, and/or by-products of other industries. These particular materials are discussed in more detail in section 1.4 with sustainable fill technologies.

1.3.1. General Overview

The large variety of lightweight fill material types exhibits an extensive range in unit weights, ranging from less than 1 pcf to approximately 90 pcf (Table 1.4). For a general assessment, the behavior of lightweight fill materials is usually divided into two main categories: (1) "cohesive" materials, which have an inherent compressive strength and are treated as undrained when loaded; and (2) "granular" materials, which derive their strength from confinement and are treated as drained when loaded. Cohesive lightweight fill materials include expanded polystyrene (EPS) and extruded polystyrene (XPS) materials (i.e., Geofoam) as well as lightweight cellular concrete (LCC). Granular lightweight fill materials include the various slags, fly ash, wood fiber, expanded shale, clay and slate (ESCS), and recycled tire derived aggregates (TDA).

Potential benefits of lightweight fill materials have been previously mentioned. However, it should be noted that there are also some potential disadvantages associated with the various lightweight fill technologies. For example, the material properties of certain lightweight fill materials may not be compatible with natural soils/aggregates (Schaefer et al. 2017a). For example, shredded tires or wood fibers are more compressible than compacted natural soils. The stress-strain behavior of EPS geofoam blocks is generally linear to strain levels of about 0.5 percent, but it may exhibit yielding and subsequent long-term creep beyond that limit. Some lightweight fill materials exhibit such a low unit weight that they are buoyant in water. Consequently, they cannot be used below the ground water table. The availability of various lightweight fill materials is also regionally-dependent. For example, wood fiber is more readily available in lumber producing areas, fly ash and slag in heavily industrialized areas, and ESCS in regions where specific production plants are present. Specialty or additional equipment and extra care may be necessary for proper placement and compaction of some of the lightweight materials. For example, fly ash becomes quite spongy when wet but overly dusty when too dry. TDA require additional compaction energy due to their visco-elastic characteristics under dynamic loading. LCC requires specialized equipment to insert air and additives to the mixture. After placement, some of the materials require additional care to prevent any deterioration. For instance, EPS geofoam blocks require either a concrete slab or geomembrane to cover them and prevent damage from hydrocarbon spills (i.e., gasoline, diesel fuel, kerosene, etc.). Fly ash requires coverage by soil layers in order to prevent any erosion of side slopes. Some lightweight materials are sourced from industrial waste (e.g., various slag) that can cause some environmental concerns when placed in the subsurface and subjected to leachate. Management of these disadvantages is possible by considering appropriate design considerations. For example, environmental concerns can be mitigated by appropriate use of geomembrane systems to contain the lightweight fill materials.

Table 1.4. Range of unit density for common lightweight fill materials (Schaefer et al. 2017a)

Fill Type	Range in Density pcf	Range in Specific Gravity
Geofoam (RCPS)	0.70 to 3.00	0.01 to 0.05
Cellular Concrete	20 to 80	0.4 to 1.3
Wood Fiber	35 to 55	0.6 to 0.9
Tire Shreds	37 to 73	0.6 to 1.2
Expanded Shale, Clay, and Slate (ESCS)	37 to 65	0.6 to 1.0
Fly Ash	70 to 90	1.1 to 1.4
Boiler Slag	60 to 90	1.0 to 1.4
Expanded Air-Cooled Slag	69 to 94	1.1 to 1.5

With respect to costs, there is a wide range for lightweight fill technologies as multiple factors influence costs:

- Raw material costs: Generally, lightweight fill technologies that are based on the use of reclaimed or recycled materials will incur lower unit costs. For example, various slag materials are typically stockpiled as industrial byproducts and are cheaper per unit than a manufactured technology such as EPS geofoam block. However, if the reclaimed/recycled material requires additional processing such as crushing, shredding, and sieving, the per unit costs can increase considerably and compare with manufactured technologies.

- Transportation costs: The manner in which the material is transported to the site (e.g., barges versus long-haul trucks) can make a significant difference in overall costs. Additionally, the distance to the project site will affect the overall transportation costs.
- Material quantity: Some lightweight materials may be available at discounted unit costs as the amount of material needed for a project increases. Additionally, this can indirectly affect transportation costs depending on the mode of transport and the largest unit of material that can be transported.
- Material regional availability: Materials that are either not readily available or have low production rates in a given region will incur additional costs due to transportation as well as sourcing sufficient quantity of material from different manufacturers.
- Material placement/compaction costs: As noted previously, many lightweight fill materials require additional equipment or staging for placement and/or compaction. Some also require additional moisture control, use of concrete covers, geomembranes, or other technologies, to increase resiliency and ensure adequate long-term performance.

Tables 1.5 and 1.6 provide a general range of costs associated with the technologies listed in Table 1.4. Although the cost of granular materials in typical engineered embankment and fill applications is based on a per ton weight, there are a few issues with this approach for costing of lightweight fill materials. First, a ton of lightweight material will occupy larger volume than conventional soil/aggregate materials with higher unit weight. Second, there can be an incredible change in weight of certain lightweight materials in stockpile compared with the field. For example, the initial unit weight of stockpiled TDA can range from 25 pcf to 35 pcf. However, once compacted in place, the unit weight typically increases to somewhere between 45 pcf to 50 pcf (Cheng 2016). Estimating material costs based on the weight can therefore be inaccurate if based on a quoted per ton weight. This issue can be addressed by considering the in-place cost per cubic yard as noted in Table 1.5. Since estimates are typically provided by the supplier on a per ton weight basis FOB at the plant/processing facility, the in-place cost per cubic yard must be converted by estimating an in-place density and adding transportation charges. In some cases in Table 1.5, limited information is available regarding costs (noted with n/d). Additionally, the costs in both Tables 1.5 and 1.6 do not consider the effects of inflation since publication of those references. Consequently, it is always recommended that specific estimates are requested from local suppliers for a given project.

Table 1.5. Typical range of costs for common lightweight fill materials (Schaefer et al. 2017a)

Lightweight Fill	Material Cost/yd^3 FOB at source	Delivered Material Cost/yd^3 FOB at Project	In-Place Cost/yd^3
EPS-block geofoam	\$40 to \$60	\$60 to \$125	\$40 to \$100
Cellular concrete	n/d	\$70 to \$150	\$250 to \$340
Wood Fiber	n/d	\$6 to \$26	\$8 to \$30
Air-cooled blast furnace slag	\$6 to \$8	n/d	n/d
Expanded blast furnace slag	\$11 to \$15	n/d	n/d
Boiler slag	\$2 to \$3	n/d	n/d
Fly ash	\$12 to \$16	n/d	n/d
Expanded shale, clay, and slate (ESCS)	\$30 to \$45	n/d	n/d
Tire shreds	n/d	\$15 to \$30	n/d

Table 1.6. Typical range of costs for common lightweight fill materials (Stark et al. 2004)

Lightweight Fill Type	Range in Density/Unit Weight, kg/m^3 (lbf/ft^3)*(4)*	Range in Specific Gravity	Approximate Cost, \$/m^3 (\$/yd^3)	Source of Costs
EPS (expanded polystyrene)-block geofoam	12 to 32 (0.75 to 2.0)	0.01 to 0.03	35.00 - 65.00 (26.76 - 49.70)(2)	Supplier
Foamed portland-cement concrete geofoam	335 to 770 (21 to 48)	0.3 to 0.8	65.00 - 95.00 (49.70 - 72.63)(3)	Supplier, *(9)*
Wood Fiber	550 to 960 (34 to 60)	0.6 to 1.0	12.00 - 20.00 (9.17 - 15.29)(1)	*(11)*
Shredded tires	600 to 900 (38 to 56)	0.6 to 0.9	20.00 - 30.00 (15.29 - 22.94)(1)	*(10)*
Expanded shales and clays	600 to 1,040 (38 to 65)	0.6 to 1.0	40.00 - 55.00 (30.58 - 42.05)(2)	Supplier, *(9)*
Boiler slag	1,000 to 1,750 (62 to 109)	1.0 to 1.8	3.00 - 4.00 (2.29 - 3.06)(2)	Supplier
Air cooled blast furnace slag	1,100 to 1,500 (69 to 94)	1.1 to 1.5	7.50 - 9.00 (5.73 - 6.88)(2)	Supplier
Expanded blast furnace slag	Not provided	Not provided	15.00 - 20.00 (11.47 - 15.29)(2)	Supplier
Fly ash	1,120 to 1,440 (70 to 90)	1.1 to 1.4	15.00 - 21.00 (11.47 - 16.06)(2)	Supplier

Notes: These prices correspond to projects completed in 1993 - 1994. Current costs may differ due to inflation.

(1) Price includes transportation cost.

(2) FOB (freight on board) at the manufacturing site. Transportation costs should be added to this price.

(3) Mixed at job site using pumps to inject foaming agents into concrete grout mix.

(4) Lightweight fill materials typically are characterized in the U.S.A. using the quantity of unit weight with I-P units. Therefore, the dual unit system of density in SI units of kg/m^3 and unit weight in I-P units of lbf/ft^3 is used in this table.

Specific case histories that specifically discuss costs of multiple lightweight fill technology alternatives for the same project are limited in the literature, but Cheng (2016) does provide documentation of one embankment fill project in California with comprehensive cost information provided for the various lightweight materials considered for that project (Table 1.7). These costs highlight the considerable effects of transportation costs and regional availability. For example, despite a comparable material cost per cubic yard FOB at the source, there was a tremendous difference in the overall costs (not including installation costs or contractor's overhead/profit costs) between ESCS and EPS geofoam blocks. More specific case histories with cost information for particular technologies will be provided in their pertinent sections of this report where applicable.

Table 1.7. Example of lightweight fill project costs for a documented case history in Milpitas, California (Cheng 2016)

Material	Total Cost*
Traditional Soil Fill	$563K
Pumice Rock	$633K
EPS-block Geofoam	$1,145K
Expanded Shale Clay	$490K
Wood Chips	$545K
Tire-derived Aggregate	$334K

1.3.2. Discussion of Specific Lightweight Fill Technologies

The following sections highlight various aspects of the most common lightweight fill technologies currently used in transportation-related projects. Included in each discussion of a particular material is information about its generation, material properties (density, strength, compressibility), design/construction considerations, and case histories available in the literature.

1.3.2.1. Cohesive Lightweight Fill Materials

Provided in this section is discussion of lightweight fill technologies that exhibit an inherent compressive strength. Consequently, they are treated as "cohesive" materials for design purposes, particularly with respect to load bearing capabilities and shear strength.

1.3.2.1.1. Expanded Polystyrene (EPS) and Extruded Polystyrene (XPS) Materials (Geofoam)

Geofoam is a generic term for cellular materials formed by either expansion or extrusion of polystyrene (EPS and XPS). These materials are also collectively referred to as "rigid cellular polystyrene" (RCPS) by the American Society for Testing and Materials (ASTM). Such materials were developed by the Norwegian Road Research Laboratory (NRRL) in the early 1960s initially for the function of thermal insulation to prevent frost from penetrating soils (Horvath 1994). Since that time, geofoam has been used in hundreds of project across the world, primarily in the form of EPS-block geofoam (Stark et al. 2004) (Figure 1.2). It is not exactly clear when the first EPS-block geofoam lightweight fill project in the United States was completed, though the technology dates back to at least the 1980's and at least two conceptual patents for the use of plastic foams as lightweight fill in earthworks are known to have been issued in the U.S.A. in the early 1970's (Monahan 1971; Monahan 1973). With respect to terminology, in current U.S. practice it is customary for the term "geofoam" to be synonymous with "EPS-block geofoam", which is the predominant geofoam material and product for transportation-related applications. However, there are many different geofoam materials and products, including one that uses a foamed Portland-cement concrete material [i.e., cellular Portland-cement concrete (CPCC)]. In fact, the term "geofoam" was also a U.S. registered trade mark for a now defunct proprietary commercial product that was marketed and used almost exclusively in Alaska in the 1970s (Horvath 2013). Consequently, the specific geofoam material and product should be specified for a project much like would be the case for a geosynthetic product (e.g., specify the polymer, manufacturing technique, desired weight, etc.).

Figure 1.2. EPS-block geofoam used for embankment construction (Schaefer et al. 2017a).

1.3.2.1.1.1 Manufacturing Process

The manufacturing process for geofoam depends on the type of polystyrene foam (EPS, XPS) used for the product. For EPS, the process is highlighted in Figure 1.3:

1. Raw material: EPS is composed initially of polystyrene solid resin beads. Styrene is a substance produced by copolymerization of polythene plus vesicant and stabilizing agent.
2. Foaming and maturing process: The polystyrene solid resin beads are placed in a foaming machine and expanded into a cellular sphere under steam. Usually, this is performed in a two-step process. The first foaming multiplies the polystyrene around 70 times (referred to as pre-expansion) and the second phase is around 50 to 55 times.
3. Molding process: In this step, a mold is preheated and the foamed styrene grade is placed inside the mold. Pressurized heating is applied to shape the EPS into a prismatic block shape. As the EPS cools, the operators can extract the EPS from the mold.
4. Drying process: The EPS block is allowed to fully cool at room temperature. Afterwards, it is inspected for shrinkage, cracks, and similar defects. Then the blocks are ready to use and for shipping (Lin et al. 2010).
5.

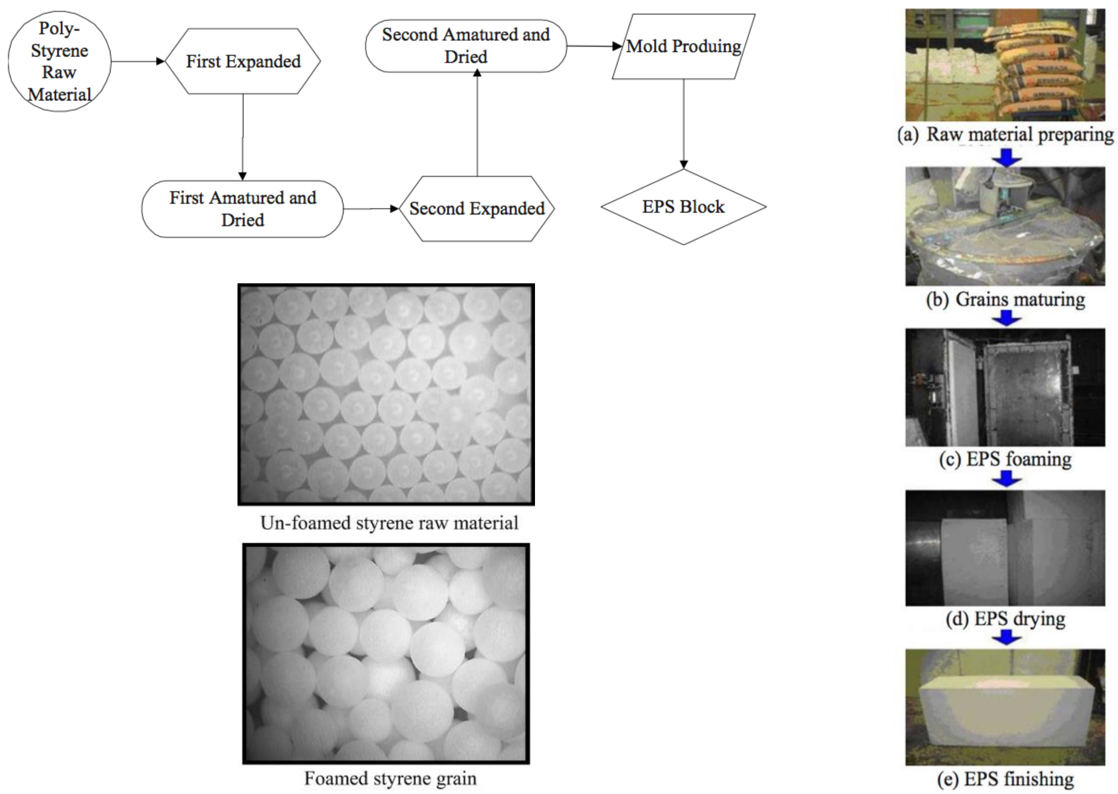


Figure 1.3. EPS-block geofoam manufacturing process (Lin et al. 2010).

The main difference for XPS products is that this manufacturing process is continuous (i.e., not divided into a multi-part foaming and molding process) with the raw material being liquefied and subsequently extruded as thin planks or panels. The resulting final appearance of XPS is a uniform texture of closed cells, while EPS appears as a collection of fused individual particles (Stark et al. 2004).

1.3.2.1.1.2 Engineering Properties & Design/Construction Considerations

As noted previously, EPS-block geofoam is the most commonly applied geofoam technology used in transportation-related projects (Stark et al. 2004). Of all the lightweight fill technologies, EPS-block geofoam is one of the most popular and has been used at least several hundreds of projects since its development in the 1960's. What sets EPS-block geofoam apart from other lightweight fill technologies is its capability of having such low unit weight (i.e., as low as 1% the unit weight of traditional earthen fills) while simultaneously exhibiting sufficient compressive strength to be used under high loading conditions such as high-volume roadways, lightly loaded buildings, and small bridge abutments. The compressive strength is a function of the density, which in turn is related to the density of the foamed polystyrene grain created during the first stage of manufacturing (the pre-expansion process) (Stark et al. 2004). The density of EPS-block geofoam can vary by as much as ±10% even within a single block due to inherent variabilities during the manufacturing process.

With respect to the strength of EPS-block geofoam, traditional design has been based on compressive strength, which implies an ultimate limit state type of failure involving material rupture. However, such a rupture does not necessarily occur and the actual stress-strain behavior of EPS-block geofoam is more complex as noted in Figure 1.4. Generally, the stress-strain behavior is approximately linear-elastic up to a compressive strain of 1%. The slope of the initial linear stress-strain curve (Zone 1 in Figure 1.4) is defined as the initial tangent Young's modulus, E_{ti}. This E_{ti} can be related to the density of the EPS-block geofoam as in Figure 1.5 and Equation 1.1 (Stark et al. 2004):

$$E_{ti} = 450\rho - 3000 \tag{1.1}$$

where ρ is the block density in kg/m^3 and E_{ti} is in units of kPa. There is evidence in the literature that E_{ti} increases with increases in dimensions (Stark et al. 2004), but additional research is necessary to examine whether this would cause an entire block to behave in a stiffer manner relative to laboratory tests on small specimens. Yielding occurs in Zone 2 of Figure 1.4 (i.e., strains between 3% - 5%) with the radius of curvature related to the density of the EPS-block geofoam (Stark et al. 2004). After yielding, the stress-strain behavior becomes approximately linear again.

Despite this complex stress-strain behavior, traditional EPS-block geofoam design is typically based on a compressive strength (σ_c) at some arbitrary strain level (typically 10% based on ASTM and most other world-wide standards organizations). This σ_c is in turn linearly related to block density:

$$\sigma_c = 8.82\rho - 61.7 \tag{1.2}$$

where ρ is the EPS density in kg/m^3 and σ_c is in units of kPa. Alternatively, a yield stress (σ_y) can be defined and used instead of σ_c to characterize the stress-strain performance of the EPS-block geofoam. This σ_y can be determined graphically by forward extrapolation of the initial linear stress-strain behavior (Zone 1) and backward extrapolation of the post-yield linear portion (Zone 3). A number of empirical relationships have been proposed that relate this σ_y to ρ based on the dataset in Figure 1.5:

$$\sigma_y = 6.41\rho - 35.2 \tag{1.3}$$

$$\sigma_y = 6.62\rho - 46.3 \tag{1.4}$$

$$\sigma_y = 6.83\rho - 48.4 \tag{1.5}$$

where ρ is the EPS density in kg/m^3 and σ_y is in units of kPa (Figure 1.6).

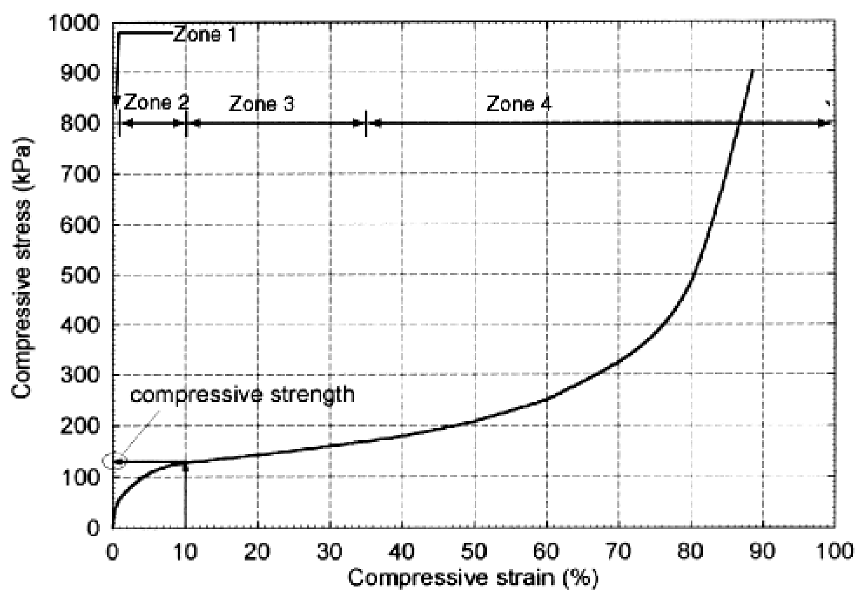


Figure 1.4. Example stress-strain behavior of EPS-block geofoam (Stark et al. 2004).

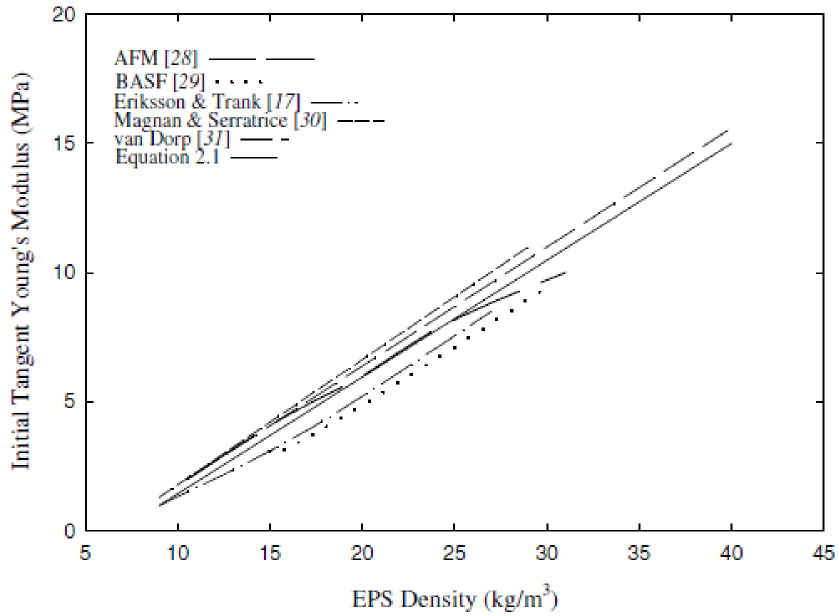


Figure 1.5. Correlation between ρ and E_{ti} for EPS-block geofoam (Stark et al. 2004). NOTE: Equation 2.1 refers to Equation 1.1 in this report.

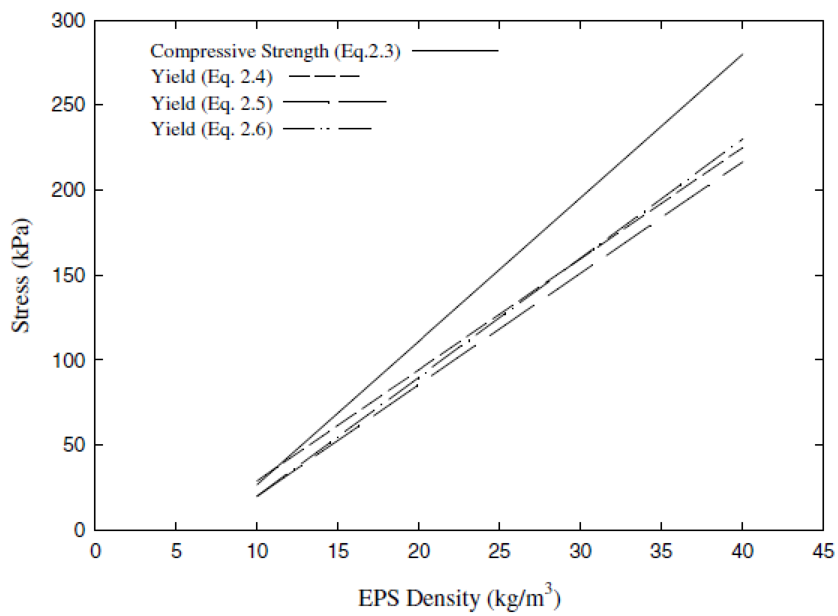


Figure 1.6. Correlation between ρ and σ_c and σ_y for EPS-block geofoam (Stark et al. 2004). Note: Equations 2.3 – 2.6 refer to Equations 1.2 – 1.5 in this report.

Limit-state stability analysis of a lightweight geofoam structure (e.g., embankment) includes both external and internal failure modes. This requires knowledge of the shear behavior of an individual geofoam block (external stability) in case the failure plane passes directly through a block. Internal stability analyses check the available shear resistance in between layers of geofoam blocks against horizontal forces such as hydrostatic sliding, wind, and earthquake loads. Various studies have therefore been conducted over the years on interface shear strength properties of EPS-block geofoam (e.g., Sheeley and Negussey 2000; Atmatzidis et al. 2001; Negussey et al. 2001; Barrett and Valsangkar 2009). Various design manuals, including the Norwegian Road Research Laboratory (1992), Stark et al. (2004), and Arellano et al. (2011) have summarized recommended interfacial friction angles (δ) for these analyses. For example, Stark et al. (2004) reported that most studies have found δ ranging between approximately 27° to 35°, with a recommended value of 30° for routine design. However, this δ range is valid when examining the EPS-EPS frictional resistance. There is less information available for δ between EPS block and other materials that may be encountered in lightweight fills (e.g., geotextiles, geomembranes, concrete, etc.), which represents an area of research need (Stark et al. 2004). Additionally, it may be necessary to interlock the EPS blocks due to excessive horizontal loads from seismic events. The most effective mechanism by which to accomplish this is still an active area of research (e.g., Özer and Akay 2016) with various approaches such as connector plates, polyurethane adhesives, shear keys, and interlocking block shapes discussed in the literature.

Table 1.8. Design and construction considerations for EPS-block geofoam (Schaefer et al. 2017a)

Item	Guideline
Design Parameters	**Density, Dry**: 0.75 to 2.0 pcf **Compressive and Flexural Strength**: Varies with density, 6 to14 psi **Modulus of Elasticity**: 580 to 1450 psi **California Bearing Ratio (CBR)**: 2 to 4 **Coefficient of Lateral Earth Pressure**: Lateral pressures from adjacent soil mass may be reduced to a ratio of 0.1 of horizontal to vertical pressure (PIARC 1997).
Environmental Considerations	There are no known environmental concerns. No decay of the material occurs when placed in the ground.
Design Considerations	EPS blocks will absorb water when placed in the ground. Blocks placed below water have resulted in densities of 4.8 to 6.4 pcf after 10 years. Blocks above the water had densities of 1.9 to 3.2 pcf after 10 years. For settlement and stability analyses, use the highest densities to account for water absorption. Buoyancy forces must be considered for blocks situated below the water table. Adequate cover should be provided to result in a minimum safety factor of 1.3 against uplift. Because petroleum products will dissolve geofoam, a geomembrane or a reinforced concrete slab is used to cover the blocks in roadways in case of accidental spills. Differential icing potential of pavement, due to a cooler pavement surface above the EPS versus pavement above a soil only subgrade. Differential icing can be minimized by providing a sufficient thickness of soil between the EPS and top of pavement surface. Use side slopes less than or equal to 2H:1V and a minimum cover thickness of 0.8 feet. If a vertical face is needed, cover exposed face of blocks with shotcrete or other material to provide long-term UV protection.
Construction Considerations	The subsoil should be leveled before placement of geofoam blocks. A layer of sand/gravel is frequently placed as a leveling course. When multiple layers of geofoam blocks are placed, the blocks should be placed at right angles to avoid continuous vertical joints and to promote interlocking. See Figure 3-9. A minimum of two layers of blocks must be used. Provide a mechanical connection between blocks using a galvanized barked plate for shear transfer. Place cover material over geofoam blocks as soon as possible to prevent displacement from wind or buoyancy. Avoid prolonged exposure to sunlight, which embrittles EPS.

Due to the low weight and the resiliency of EPS-block geofoam, it is very easy and quick to place at a project site, which decreases construction costs. In addition, geofoam is extremely easy and quick to place in all types of weather, which compensates for the larger per unit material costs relative to traditional earthen fills and other lightweight fill materials. However, EPS will dissolve in the presence of hydrocarbons (e.g., gasoline, diesel, etc.) and can become brittle when exposed to UV light (though only near the exposed surface and after long-term exposure). Consequently, EPS-block geofoam is typically placed with a cover material that protects it from the elements. Table 1.8 summarizes design and construction considerations for EPS-block geofoam.

1.3.2.1.1.3 Example Case Histories

Given the relative popularity of EPS-block geofoam, it is unsurprising that there exists quite a number of case histories documented in the literature where this technology was implemented [e.g., Arellano et al. (2018) provides a recent example of assorted case histories from an international conference specifically devoted to the use of EPS-block geofoam]. Often, this technology has been used on embankments, slope stabilization, and retaining wall projects. Generally, EPS-block geofoam has a longer history across Europe and in Japan as well as other countries across Asia (Stark et al. 2004). Example case histories from these countries include Vaslestad et al. (1993), Negussey and Sun (1996), Suzuki et al. (1996), Frydenlund and Aaboe (2001), Duškov and Nijhuis (2011), Herle (2011), and Kubota (2011). Though the technology was not widely available in the United States until sometime after, there is still a lot of case histories available in the literature that document its use in highway-related project (e.g., Yeh and Gilmore 1993; Gunalan et al. 1998; Jutkofsky 1998; Bartlett 1999; Zaheer 1999; Bartlett et al. 2000; Jutkofsky et al. 2000; Reuter and Rutz 2000; Reuter 2001; Stuedlein et al. 2004; Farnsworth et al. 2008; Stuedlein and Negussy 2013). Of the several EPS projects now completed in many parts of the world, only a few known failures have been reported (Frydenlund and Aaboe 2001; Horvath 2010). These failures have typically been associated with either buoyancy of the blocks after some unanticipated conditions led to a rise in the groundwater table or fires caused by construction activities (e.g., welding causing combustion of uncovered blocks).

1.3.2.1.1.4 Costs

Given the extensive case history database available in the literature, there is a significant amount of information regarding costs of EPS-block geofoam projects as noted in Table 1.9. However, the differences between projects make comprehensive comparisons difficult and lead to a broad range in contract values. Additionally, there are a number of factors that can drastically affect the overall costs associated with EPS-block geofoam projects, including unit costs and placement rate (Table 1.10). Stark et al. (2004) provides a significant amount of information regarding the cost effectiveness of EPS-block geofoam based on a number of case histories across the United States. The efforts from that study generally found that despite higher material costs, EPS-block geofoam is cost-competitive with other embankment and retaining wall options as well as ground improvement technologies such as prefabricated vertical drains (PVD), vibro-compaction, soil nailing, stone columns, and soil mixing.

Table 1.9. Summary of EPS-block geofoam costs for various roadway projects (Stark et al. 2004).

Date	Location of Project	Project Type	Quantity of EPS-Block m^3 (yd^3)	Unit Cost of EPS-Block $\$/m^3$ ($\$/yd^3$) (1)	Approximate Placement Rate m^3/day (yd^3/day)	Contract Value $
1993	Wyoming	Bridge Approach	377 (493)	39.00 - 72.00 (30.00-55.00) (2)	-	79,732
1993-1994	Hawaii	Embankment	13,470 (17,618)	-	175-250 (229-327)	-
1995	Indiana	Embankment	4,708 (6,157)	86.58 (66.20)	428 (560)	607,207
1995	New York	Slope	3,116 (4,075)	85.01 (65.00)	382 (500)	-
1995	Washington	Bridge Approach	1,835 (2,400)	72.00 (55.00)	-	-
1995 ±	Washington	Embankment	411 (537)	105.94 (81.00)	-	-
1997-1999 ±	Wyoming	Bridge Approach	146 (191)	104.00 (79.52)	-	30,326
1999 ±	Connecticut	Embankment	321 (420)	98.00 (75.00)	-	-
1999 ±	Maine	Embankment	-	57.21 (43.74) FOB Site		
1999 ±	Michigan	Embankment	1,052 (1,376)	52.50/ 43.00 (40.14/32.88) (3)	-	1,960,245/ 2,202,667
1999 ±	Michigan	Embankment	4,919 (6,434)	58.50/ 50.00 (44.73/38.23) (3)	-	5,696,732/ 5,970,269
1999 ±	Utah	Vertical Embankment	-	65.00 (50.00) (w/o facing wall) 75.00 (57.00) (with facing wall)	470 (615)	-
1999	Illinois	Embankment	15,291 (20,000)	-	313 (410)	-
1999	Wisconsin	Slope	-	61.50 (47.00)	-	-

Notes:
- Data not available.
(1) Unit cost of EPS blocks includes transportation and placement unless indicated otherwise.
(2)From usage questionnaire reply.
(3)The lowest two bid values are reported.

Table 1.10. Example of various costs associated with EPS-block geofoam projects (Stark et al. 2004).

MANUFACTURING COSTS:
1. Raw material price 1.1 Flame retardant chemicals 1.2 Use of low-VOC expandable polystyrene 1.3 Shipping from raw material supplier to molder 1.4 Subjective marketing factors 2. Density 2.1 Cost of blocks with increasing density. 2.2 Use of only one density versus using different product densities on the same project. 3. Manufacturer's cost 3.1 Direct purchase from molder 3.2 Purchase from a distributor 4. Shop drawings 5. Complexity of factory cut of blocks 6. Insecticide 7. Transportation from molder to job site 8. Overall project volume 9. Project schedule
DESIGN DETAIL COSTS:
1. Use of connector plates 2. Geometeric complexities of block layout 3. Wall facing system for vertical-faced embankment or soil cover for slope-sided embankment 4. Pavement system 4.1 Separation/stiffening material 5. Permanent drainage system 6. Other specialty items such as geotextiles and geomembranes
CONSTRUCTION COSTS:
1. On-site handling and storage 2. Subgrade preparation 2.1 Smooth, free of large objects, reasonably dry, leveling layer (if required) 3. Use of connector plates 4. Field cutting and block placement 5. Number of different density blocks 6. Season of year construction takes place 7. Misc. project constraints 7.1 Hours allowed 7.2 Days allowed 7.3 Relationship of geofoam work to other components 8. Temporary dewatering 9. Wall facing system for vertical-faced embankment or soil cover for sloped-sided embankment 10. Pavement system 10.1 Separation/stiffening material 11. Permanent drainage system 12. Other specialty items such as geotextiles and geomembranes

1.3.2.1.2. Lightweight Cellular Concrete (LCC)

Cellular concrete is a kind of lightweight concrete mix typically used as a backfill material (Figure 1.7). This material was first introduced in Sweden in the early 1900's as a way of improving the thermal insulating properties of concrete (Sutmoller 2020). Often, this technology falls under the category of flowable backfills [also referred to as controlled low-strength materials (CLSM) or controlled density fills] for transportation agencies, though the CLSM term can include mixtures that are not necessarily lightweight and that may differ slightly in behavior/performance. Consequently, the CLSM term is not interchangeable with LCC and clarity must be ensured with terminology when discussing cellular concrete products as design alternatives. The American Concrete Institute (ACI) also refers to this technology as low-density cellular concrete (LDCC) (ACI 2006). This technology is marketed under a broad range of trade names based on the different proprietary foaming agents available for manufacture (e.g., Elastizell, AERLITE, etc.).

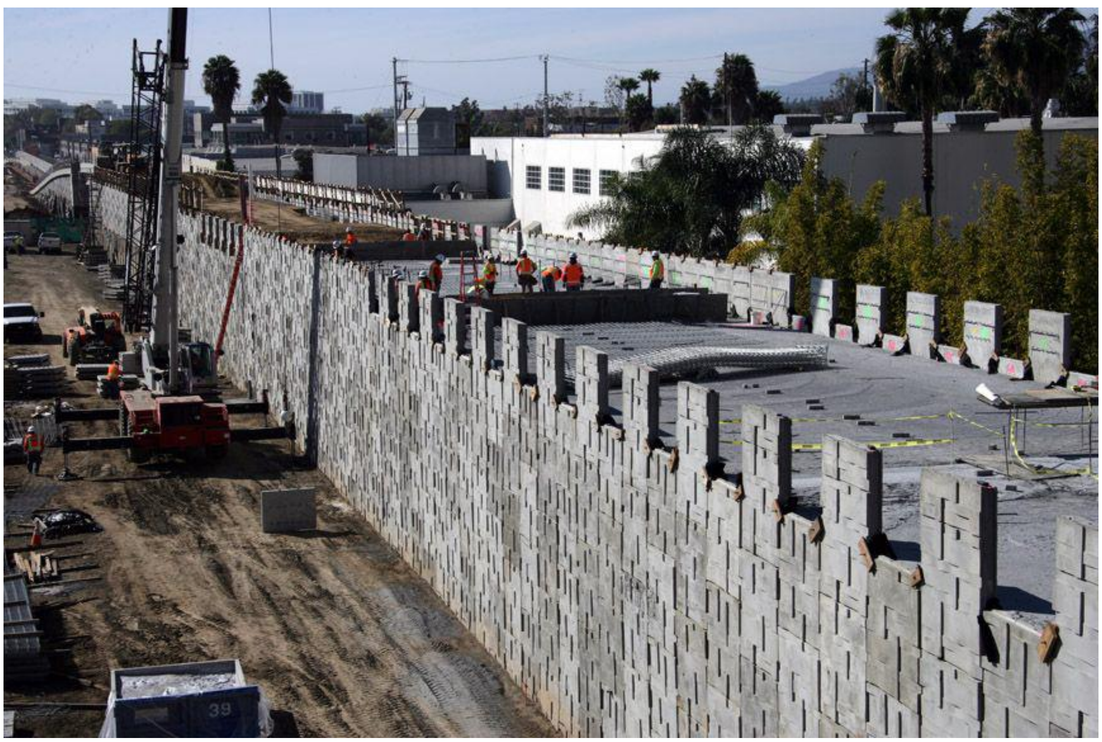


Figure 1.7. LCC backfill for an MSE wall light rail project in California (Schaefer et al. 2017a).

Depending on the mix design, different "types" of cellular concrete are possible, including some that are lightweight. For example, neat-cement cellular concrete is made from Portland cement, water, and a preformed foam with no solid aggregate. This limits the cast density (ρ) to approximately 800 kg/m^3 (Legatski 1994). Sanded cellular concrete contains fine aggregate in addition to cement, water and the performed foam resulting in a ρ range between 800 to 2080 kg/m^3 (Legatski 1994). Lightweight aggregate cellular concrete is similar to sanded cellular concrete, except for lightweight aggregates are used in place of all or a part of the sand. The strength of the mixture is directly related to the strength/density ratio obtained from the structural grades of aggregates. (Legatski 1994). Consequently, a range of possible ρ and compressive strengths (σ_c) are possible for LCC (Table 1.11).

Table 1.11. Example DOT classification of LCC based on Caltrans (Rollins et al. 2019).

Cellular Concrete Class	Cast Density kN/m^3 (lb/ft^3)	Minimum 28-day Compressive Strength kPa (psi)
I	3.8-4.6 (24-29)	69 (10)
II	4.7-5.5 (30-35)	276 (40)
III	5.7-6.4 (36-41)	550 (80)
IV	6.6-7.7 (42-49)	830(120)
V	7.9-12.4 (50-79)	1100 (160)
VI	(12.6-14.1) 80-90	2070 (300)

1.3.2.1.2.1 Manufacturing Process

There are multiple categories of LCC based on the manner in which pores are introduced into the concrete, production method, and how the concrete is cured (Figure 1.8). Typically, the composition of LCC consists of a proprietary foaming agent (similar in appearance to shaving cream) with cement slurry or cement grout (including aggregates and sometimes fly ash as a partial concrete replacement). The purpose of the foaming agent is to produce a porous structure by supplying air cells (Chica and Alzate 2019). Typically, two types of foaming agents are used in the production of LCC: (1) surfactant/synthetic; (2) and protein foaming agents (Panesar 2013). Both reduce the surface tension of the mixture, which in turn aids in the formation of stable air bubbles. Another category of LCC forgoes the use of a foaming agent and is instead produced by applying a highly diluted lime mortar into the mix that allows air to enter when the setting process starts (Chica and Alzate 2019). The water to cement (W/C) ratio typically varies between 0.4 – 1.25 and balances the rigidity of the final mixture with excessive segregation that can occur when too much water is present (Chica and Alzate 2019).

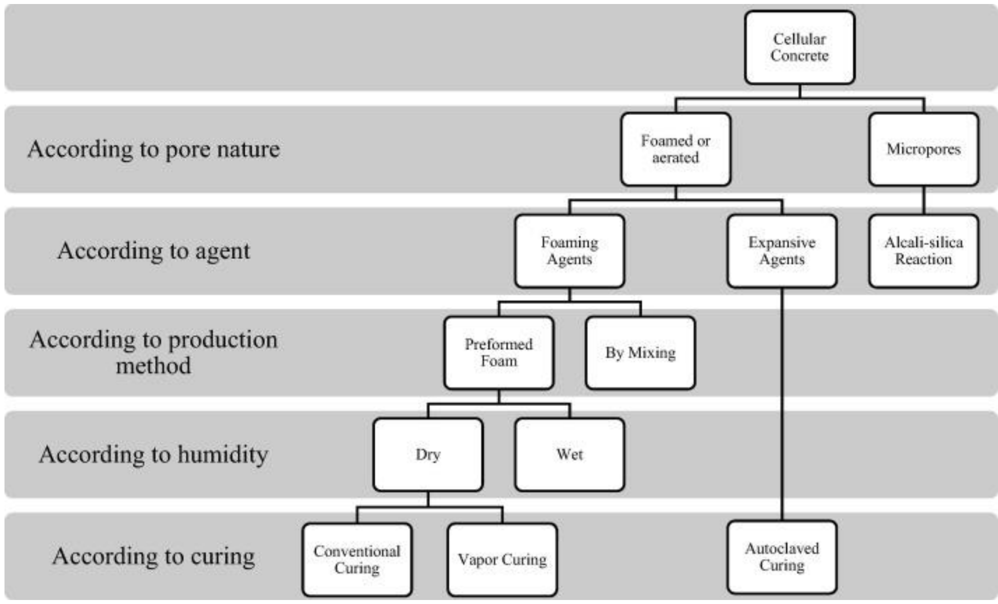


Figure 1.8. Types of LCC based on pore creation, production, and curing methods.

Generally, LCC mixtures using foaming agents are the most popular due to their higher compressive strengths (Chica and Alzate 2019). This mixture is either produced at the project site or transferred in a ready-mix truck from a processing plant. When produced on-site, a mixing/calibrating unit, a cement truck with a hopper, and a water tanker are required (Figure 1.9). The foaming agent is first measured and placed in a dilution chamber. Then the foam is routed to a calibrating/mixing unit and the Portland cement is added to the flow. The slurry is subsequently pumped into place in lifts ranging from 1 ft – 4ft and left to cure for a minimum of 12 hours before additional lifts (Figure 1.9). Each lift surface is typically scarified and cleaned prior to the placement of another lift. Quality of the mixture is monitored through its cast unit weight, starting with the wet cast unit weight. The acceptable range of air voids for LCC material is between 10% to 70% which leads to unit weights as low as 20 pcf (Aberdeen Group 1963, Panesar 2013). Recent efforts have explored the use of recycled materials in the mix design of LCC to increase sustainability, ductility, toughness, and impact resistance (e.g., Benazzouk et al. 2006; Remadnia et al. 2009; Ruiz-Herrero et al. 2016).

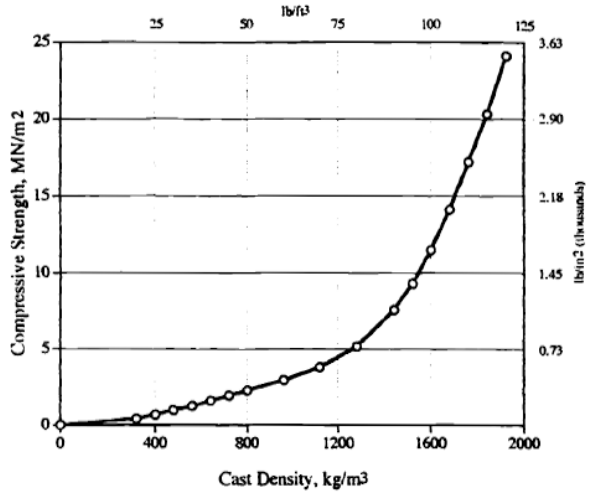
Figure 1.9. On-site production and placement of LCC (courtesy of Aerix Industries™).

1.3.2.1.2.2 Engineering Properties & Design/Construction Considerations

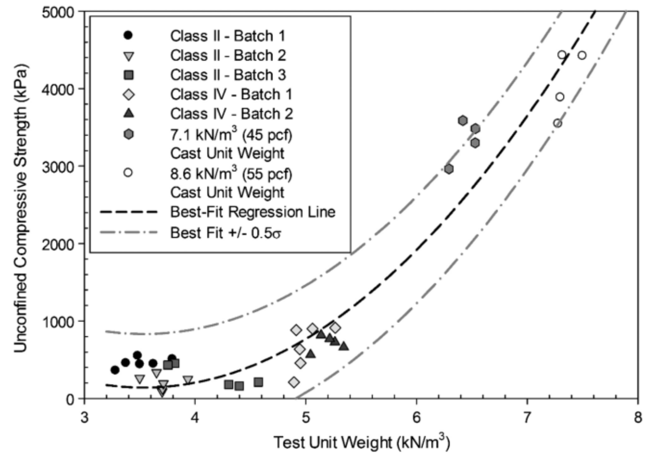
As with EPS-block geofoam, LCC is capable of very low unit weights in relation to its σ_c. Some of the factors that can make a difference in σ_c of LCC are ρ (or porosity since the number of voids is directly related to ρ), cement content, water/cement ratio, aggregate type and amount, special admixtures, and curing conditions. Figure 1.10 presents typical σ_c versus ρ (or porosity) relationships for LCC (Legatski 1994). In some research, the σ_c was found to decrease with an increase in moisture content (Narayanan and Ramamurthy 2000) and to vary based on the foaming agent (Aberdeen Group 1963). Wee et al. (2006) found that increasing the air content does not increase the size of internal voids but increases their number per unit volume. Consequently, it is possible to decrease the density without a substantial loss in compressive strength at a particular optimum air content. Since the modulus of elasticity (E) of concrete is generally based on density and compressive strength, it is rational that the modulus for LCC is less than regular concrete. Based on laboratory testing, the following equation has been proposed to represent this relationship between E, ρ, and σ_c for LCC (Legatski 1994):

$$E = (\rho - 5)^{1.5} \cdot 28.6 \cdot \sigma_c^{0.5} \tag{1.6}$$

where E and σ_c are in units of psi and ρ in units of pcf. As with typical concrete, σ_c is prescribed based on a 28-day curing time. Table 1.12 provides typical ranges of E, σ_c, and other engineering properties as a function of ρ. LCC does have some tensile strength, though it is quite low (i.e., approximately 10% - 15% of σ_c). Other properties of LCC are also typically dependent on density, including thermal conductivity, permeability, freeze-thaw resistance, and water absorption (Aberdeen Group 1963; Loudon 1979; Legatski 1994; Neville 2002; Garbalińska and Strzałkowski 2018). Amran et al. (2015) provides a comprehensive set of relationships for some of these properties, including strength and modulus parameters. Other factors include fly ash content, sand/aggregate type, and foaming agent (Nguyen et al. 2014; Sikora et al. 2016; Horszczaruk et al. 2017; Tiwari et al. 2017a,b). Research on fundamental cellular concrete behavior continues to grow, particularly with respect to density and compressive strength and how foam volume, type and proportion of additions, fillers and fibers affect these properties (e.g., see Chica and Alzate 2019 for a comprehensive review). However, there is still a tremendous need to study aspects related to rheological behavior (e.g., mixing, transport, storage and pumping) since these can affect foam stability and the internal structure of the LCC. This can have long-term implications on the mechanical performance once the LCC solidifies (Ramamurthy et al. 2009; Amran et al. 2015).



(a)



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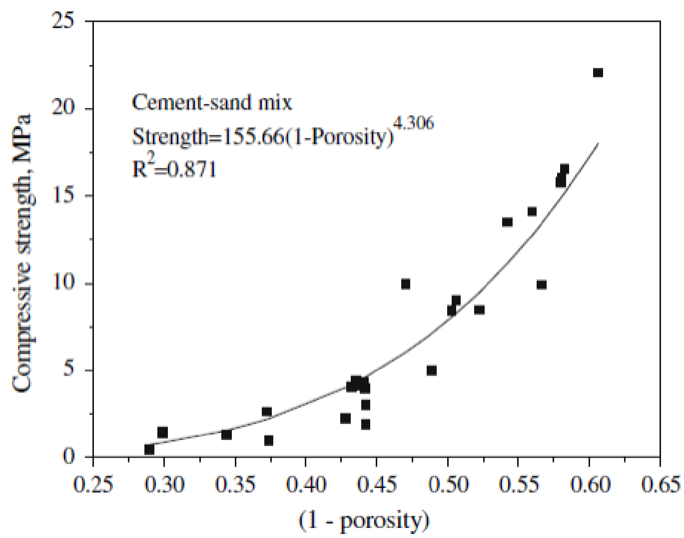


(c)

Figure 1.10. LCC compressive strength as a function of cast density: (a) Legatski 1994; and (b) Tiwari et al. (2017a).

Table 1.12. LCC properties with respect to density (Chica and Alzate 2019).

Dry Density [kg/m^3]	Compressive Strength [MPa]	Elastic modulus [GPa]	Thermal Conductivity [W/m K]	Volumetric Contraction [%]
400	0.5–1.0	0.8–1.0	0.07–0.11	0.30–0.35
500	1.0	1.24–1.84	0.08–0.13	–
600	1.0–1.5	2.0–2.5	0.11-0.17	0.22–0.25
800	1.5–2.0	20.-2.5	0.17–0.23	0.20–0.22
1000	2.5–3.0	2.5–3.0	0.23–0.30	0.15–0.18
1200	4.5–5.5	3.5–4.0	0.38–0.42	0.009–0.11
1400	6.0–8.0	5.0–6.0	0.50–0.55	0.07–0.009
1600	7.5–10.0	10.0–12.0	0.62–0.66	0.006–0.07

Cellular concrete has been used in a number of applications, including for sound and heat insulation, building panels, fire protection walls, energy-absorbing pads in roads, road sub-bases, and engineered fills (Tikalsky et al. 2004; Just and Middendorf 2009; Ramamurthy et al. 2009; Panesar 2013). It is easier to pump than traditional Portland cement concrete mixtures and does not need compacting, vibration or leveling (Chica and Alzate 2019). Its excellent flowability and ease of placement allows it to serve as a great backfill for either retaining walls or for voids under roadways (e.g., old sewers, storage tanks, etc.) (Amran et al. 2015). In this manner, it is superior to EPS-block geofoam since it does not have to be cut into a particular shape or placed in a particular pattern to conform to the dimensions of a void/backfill. Additionally, it can be mixed on-site and pumped long distances, which can reduce the required number of truckloads of raw materials to the site. However, placement of LCC requires more on-site equipment as well as specialty contractors familiar with the appropriate mixing procedures and proprietary admixtures/foaming agents. Additionally, care must be exercised when the LCC is to be placed below the water table due to buoyancy effects from its low density and its long-term absorption of water. Table 1.13 summarizes design and construction considerations for LCC.

Table 1.13. Design and construction considerations for LCC (Schaefer et al. 2017a).

Item	Guideline
Design Parameters	**Wet Density Range:** 20 to 80 pcf **Compressive Strength Range:** 10 to 300 psi, depending on density **Water Absorption:** 1.4 to 15 psf, depending on density **Freeze-thaw Resistance, 100 Cycles:** 92 to 98%, depending on density **Coefficient of Lateral Earth Pressure:** Negligible for vertical loads applied directly over the foamed concrete. Lateral pressures from adjacent soil mass may be transmitted undiminished.
Environmental Considerations	There are no known environmental concerns.
Design Considerations	Dry density values will be lower than wet density values. Buoyancy could be a problem if foamed concrete is placed below the water table and there is not sufficient vertical confinement. The lower compressive strength mixes are affected by freeze-thaw cycles. The product should be used below the zone of freezing or a higher compressive strength used. Densities greater than 37 pcf have reported excellent freeze-thaw resistance. There is some absorption of water into the voids, which could affect the density and compressive strength. Saturation by water should be prevented by construction of a drainage blanket and drains.
Construction Considerations	A staging area is required for batching, mixing, and placing on site. The foamed concrete is very fluid; formwork should be tight to avoid flow of the mix through joints or gaps in the forms. Polyethylene film may be used to prevent leakage. If the foamed concrete is placed in a confined area, forms are not necessary, as the fluid mix will flow to completely fill the void. The lift thickness should not exceed 4 feet, as the heat of hydration would have an adverse effect on the foam. Allow a minimum 12-hour waiting period between lifts. No special provisions for cold joints are necessary, although each lift surface should be scarified and clean. If shaping is required, the lift thickness should be limited to 2 feet to allow workers to shape the surface while wading in the fluid mix.

1.3.2.1.2.3 Example Case Histories

As with EPS-block geofoam, LCC has a relatively long history and extensive use on transportation-related projects. In fact, use of cellular concrete has been sufficient as to warrant entire conferences organized to discuss this technology in construction (e.g., Dhir et al. 2005). Common applications include tunnel/annular fills, bridge approaches and retaining wall backfills, underground tanks, pipelines, abandoned mines, and conduits (Sutmoller 2020). The literature contains many examples where LCC was used for retaining walls, including MSE walls with appropriately modified design parameters and installation practices with the LCC material (e.g., Harbuck 1993; Pradel and Tiwari 2015; Deni and Gladstone 2019) (Figure 1.11). LCC is also commonly used to reduce the vertical loads associated with embankments (e.g., Harbuck 1993; Lingwall and Anderson 2013; Ahmad et al. 2016; Veenland et al. 2019). However, case histories for LCC use extend into a broad array of (non-transportation) applications as well such as trench-fill foundations and ground slabs (Jones and Giannakou 2004).

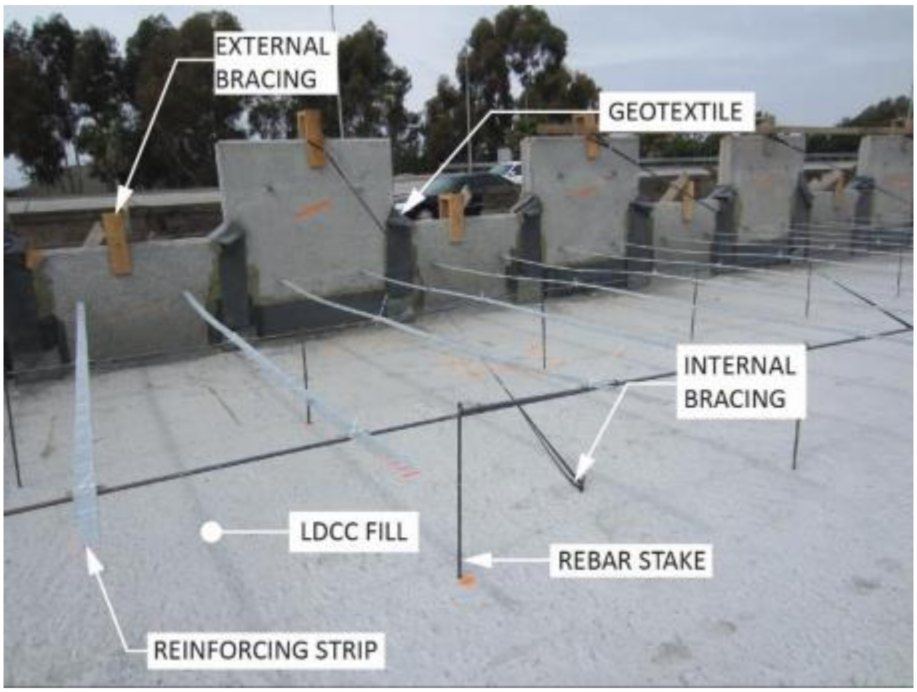


Figure 1.11. Example of support for steel strip reinforcements for an MSE wall constructed using LCC (Deni and Gladstone 2019).

1.3.2.1.2.4 Costs

As with other fill technologies, the overall costs associated with LCC will be dependent on the specific project needs, though the material costs are comparable to other lightweight fill technologies like EPS-block geofoam (Table 1.5). Generally speaking, the materials necessary to generate LCC are readily available. The mix design plays a large role in the overall costs, particularly the ratio of Portland cement and the total amount of foam agent necessary. Consequently, material costs can be readily estimated based on typical unit costs for Portland cement, foaming agent, and water. Online calculators exist that allow such an estimate (e.g., https://www.richway.com/construction-resources/mix-design-calculator.html), though they should be used cautiously, particularly if not periodically revised with updated unit material costs. Another aspect that affects overall costs is the manner in which the LCC is placed. The placement rate of LCC may be slower than for EPS-block geofoam since relatively thin 0.6 m (2 ft) lifts are necessary. This reduces the presence of large voids wherever the LCC is in contact with formwork or the structure (Stark et al. 2004). Additionally, this prevents excessive heat of concrete hydration, which can negatively affect the development of air cells necessary to reduce the unit weight of the LCC (Harbuck 1993). The 12-hour waiting period between lifts also negatively affects placement rate and overall costs.

1.3.2.2. Granular Lightweight Fill Materials

Provided in this section is discussion of lightweight fill technologies that behave similar to granular geomaterials whereby strength is derived from confinement. Consequently, they are treated in the same manner for design purposes, particularly with respect to load bearing capabilities and shear strength (i.e., friction angle).

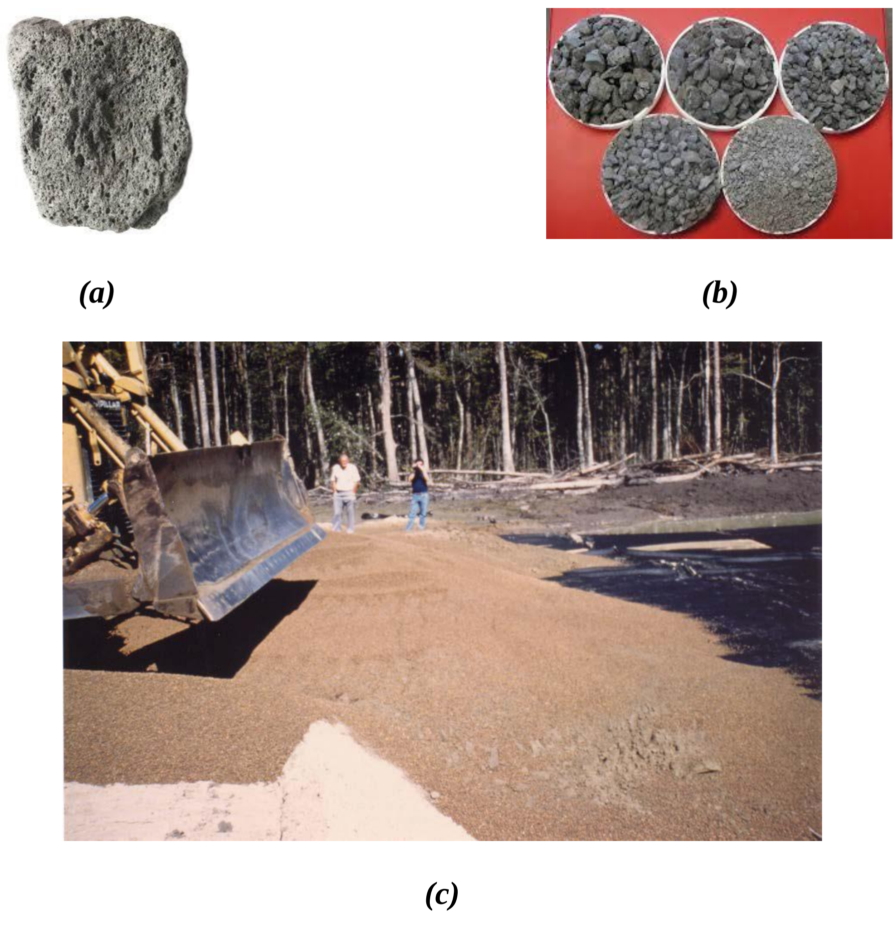


Figure 1.12. ESCS aggregates: (a) single particle (courtesy of ESCSI); (b) different gradations (Wall and Castrodale 2013); and (c) placement of ESCS fill (Schaefer et al. 2017a).

1.3.2.2.1. Expanded Shale, Clay, and Slate (ESCS)

ESCS refers to a range of synthetic aggregates produced from shale (Figure 1.12). These aggregates have been produced for more than a century and used in a number of structural applications, including for lightweight concrete masonry units (Harmathy 1970), construction of concrete ships (Holm and Bremner 2000), offshore platforms (Hoff 1992), high-rise buildings (ESCSI 2004), and bridges (Raithby and Lydon 1981), lightweight self-consolidating concrete (Wu et al. 2009), and in hot-mix asphalt applications (Khan and Mrawira 2010). It has been estimated that ESCS aggregates account for more than 95% of the structural lightweight concrete used in modern construction (ESCSI 2004). More recently, ESCS has been implemented in a wide range of geotechnical applications, including embankments on soft ground to reduce settlements (Saride et al. 2008), pile-supported bridge abutments/embankments to reduce down-drag loads (Popescu et al. 2011), municipal solid waste landfills as part of leachate collection systems (Bowders et al. 1997), and backfills behind retaining structures to decrease lateral earth pressures (Holm and Valsangkar 1993). ESCS is available by several aggregate providers across the United States under various trade names such as Solite, Stalite, Norlite, Utelite, and/or Haydite.

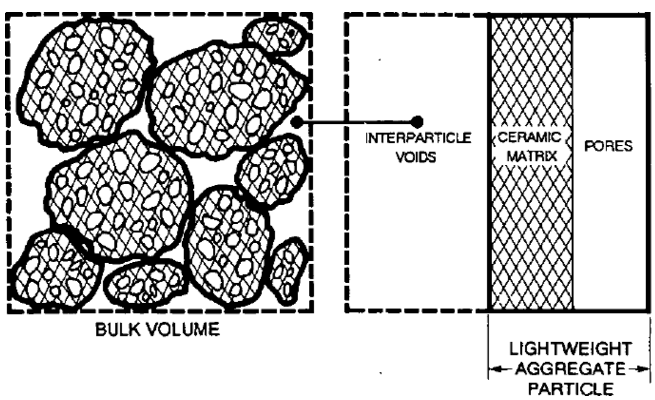


Figure 1.13. Cellular structure of ESCS aggregates (adapted from Holm and Valsangkar 1993).

1.3.2.2.1.1 Manufacturing Process

ESCS is manufactured by heating shale, clays, and/or slate in a rotary kiln through a controlled temperature process (in excess of 1100° C). This creates a cellular structure as the shale, clay, and/or slate expands, whereby spherical non-interconnecting pores are formed in a vitrified mass (Stoll and Holm 1985) (Figure 1.13). The next stage in manufacturing screens the aggregate in a careful manner to produce a desired gradation. The raw shale, clay, and/or slate material may be pre-sized before entering the kiln to limit the amount of crushing necessary to achieve the desired gradation or extruded/pelletized as a way to pre-size prior to placement in the kiln. Combinations of these approaches are found throughout the industry (ESCSI 2004). Typical gradations available from North American rotary kiln plans include 20 mm to 5 mm ($^3/_4$ - #4 sieves), 13 mm to 5 mm ($^1/_2$ - #4 sieves), or 10 mm to 2 mm ($^3/_8$ - #8 sieves) (Holm and Valsangkar 1993). The high volume of pores within the cellular structure coupled with high void contents caused by the controlled gradation leads to the lower density of ESCS aggregates. The manufacturing process also renders ESCS chemically inert and highly durable, with little long-term deterioration due to continuous submersion and/or free-thaw cycling exhibited in reclaimed samples from bridges (Holm and Valsangkar 1993). Additionally, the manufacturing process creates a cellular structure that enhances thermal properties and insulation capabilities (Holm and Valsangkar 1993).

1.3.2.2.1.2 Engineering Properties & Design/Construction Considerations

A number of researchers have explored various aspects of the engineering properties of ESCS, including its compactability, strength, durability, and hydraulic behavior (e.g., Stoll and Holm 1985; Valsangkar and Holm 1990; Bowders et al. 1997; Valsangkar and Holm 1997; Valsangkar and Holm 1999; DeMerchant et al. 2002; Saride et al. 2010; Mechleb et al. 2014). Tables 1.14 and 1.15 provide a brief summary of engineering properties and design/construction considerations when using ESCS.

The large number of vesicles present within the ESCS aggregate particles (Figure 1.14) leads to oven-dry specific gravity ranges between 1.25 to 1.40 depending on the source material placed in the kiln (Holm and Valsangkar 1993). This leads to bulk dry densities of approximately 40 lb/ft^3 – 50 lb/ft^3 and in-place compacted moist densities of less than 65 lb/ft^3 (Holm and Valsangkar 1993; Saride et al. 2010; Schaefer et al. 2017a). However, compaction behavior is slightly different than traditional well-graded soil fills. For example, ESCS will absorb some of the moisture added during compaction (e.g., Table 1.15, Figure 1.14), which means this water will not increase compactability. Additionally, the poor gradation of the ESCS aggregates prevents major increases in bulk density due to the limited amounts of fines that can pack into the interparticle void space (Holm and Valsangkar 1993). Finally, excessive passes by compaction equipment (particularly by those with steel-tracks) can cause particle crushing and degradation that can negatively affect stability (Schaefer et al. 2017a). Consequently, optimum field compaction placement is commonly achieved with limited passes (e.g., two to four) using rubber tire equipment where the goal is not to aim for maximum in-place density (Holm and Valsangkar 1993).

Table 1.14. Summary of engineering properties of ESCS based on Saride et al. (2010).

Aggregate property	Test method	Typical for ECS aggregates (ESCS 2004)	Present study	Typical design values for ordinary fills (ESCS 2004)
Compacted bulk density	ASTM D698	40–65 lb/ft^3	50 lb/ft^3 [a]	100–130 lb/ft^3
Frictional resistance (ϕ)	—	35°–45°	49.5°[a]	30°–38° (fine sand-sand and gravel)
Undrained cohesion (c_u)	ASTM D3080	—	75 kPa[a]	—
Loose bulk density	ASTM C29	Dry 30–50 lb/ft^3	—	89–105 lb/ft^3
Compressibility (C_c)	ASTM D2435	—	0.05[a]	0.27[a]
Free swell strain	ASTM D4546	—	0%[a]	2.5%[a] (varies with soil type)

[a]Present study.

Table 1.15. Design and construction considerations for ESCS (Schaefer et al. 2017a).

Item	Guideline
Design Parameters	**Dry Density, Compacted:** 50 to 65 pcf **Dry Density, Loose:** 40 to 54 pcf **Angle of Shearing Resistance:** 35° loose, 37° to 44° compacted **Grain Size Gradation:** 3/16 to 1 inch **Permeability:** High **Coefficient of Subgrade Reaction:** 33 to 37 pci loose, 140 to 155 pci compacted
Environmental Considerations	There are no known environmental concerns.
Design Considerations	The material will absorb some water after placement, when continually submerged. Samples compacted at a water content of 8.5 percent have been found after 1 year to have a water content of 28 percent. Over a longer period of time, the estimated long-term water content would be about 34 percent. Side slopes of embankments should be covered with a minimum of 2.5 feet of soil cover. Use side slopes of 1.5H to 1V or flatter to confine the material and provide internal stability. For calculating lateral earth pressures, use an angle of shearing resistance of 35°.
Construction Considerations	Particle degradation can occur from steel-tracked construction equipment. Use 2 to 4 passes with rubber-tired rollers and lift thickness of 3 feet or less. Fill should be unloaded at side of fill area, then distributed with lightweight equipment with a contact pressure of 4.5 psi or less. Field density may be approximated in the laboratory by conducting a modified one-point AASHTO T 272 density test.

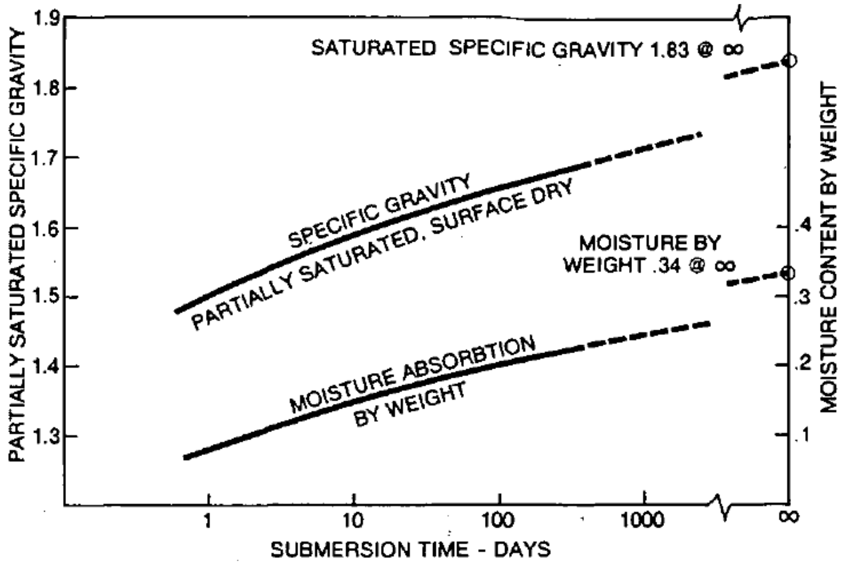


Figure 1.14. Moisture absorption by weight and subsequent changes in specific gravity for ESCS aggregates (Holm and Valsangkar 1993).

To evaluate the shear strength properties of ESCS, several studies have used direct shear and triaxial testing on modified equipment to allow for large samples typical of the aggregate size found in ESCS (Stoll and Holm 1985; Valsangkar and Holm 1990; Valsangkar and Holm 1999; Saride et al. 2008; Saride et al. 2010). These studies have generally confirmed that ESCS largely behaves as a granular material that derives its shear strength from frictional resistance and confinement. Angles of internal friction (ϕ) have typically ranged between 40° to 50°. For example, Stoll and Holm (1985) first explored shear strength of ESCS using triaxial compression tests (Figure 1.15) and noted ϕ ranged between 39.5° to 42° for uncompacted (i.e., "loose") ESCS and 44.5° to 48° for compacted ESCS. In another study by Valsangkar and Holm (1990), a large-scale shear box was used to explore the behavior of ESCS with and without geotextiles in paved and unpaved road structures constructed on peat subgrade. Again, ϕ values in the range of 40° to 48° were obtained similar to other studies. Moreover, Valsangkar and Holm (1990) highlighted that ESCS exhibited the same or superior shear strength performance as limestone aggregates when used with different geotextiles on roadways constructed on peats (Table 1.16). In the same study, Valsangkar and Holm (1990) also performed compressibility tests using a hydraulic jack system to apply load to the ESCS and limestone aggregates and found superior performance for the ESCS (Figure 1.16). Valsangkar and Holm (1997) and Valsangkar and Holm (1999) later used the same equipment to test for friction angles between ESCS and other construction materials (i.e., formed concrete, steel, and wool) and for changes in grain size due to particle breakage during shearing (Table 1.17). Again, they noted performance similar to other aggregates used in construction and found that less than 5% by weight of fines were generated in ESCS aggregates during shearing.

Finally, Saride et al. (2008) and Saride et al. (2010) conducted a direct shear test according to ASTM D 3080 in a 2.5-inch shear box and found that ϕ = 49.5° for the ESCS used to construct the embankment in their study.

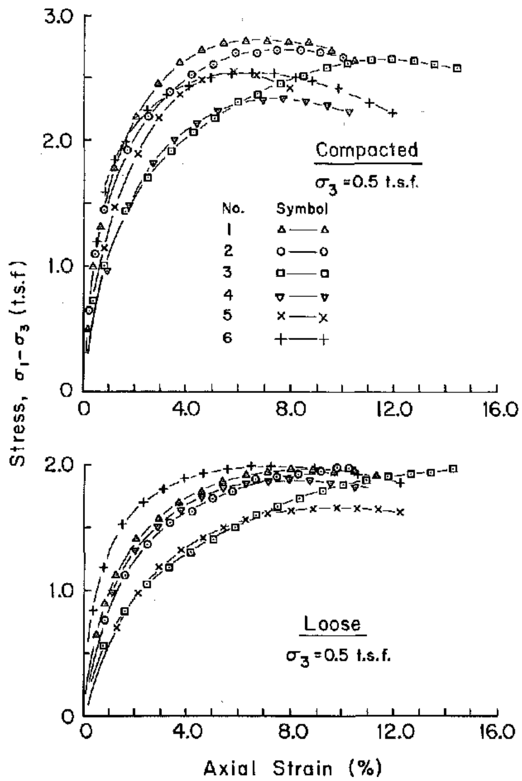


Figure 1.15. Stress-strain curves for triaxial compression tests performed by Stoll and Holm (1985) on ESCS aggregates sourced from six different locations across the United States.

Table 1.16. Friction angle between geotextiles and coarse aggregates based on Valsangkar and Holm (1990).

Material in Lower Box	Material in Upper Box	Fabric	Friction Angle, Degrees
Limestone	Limestone	Woven	41.0
Limestone	Limestone	Nonwoven	42.0
Minto	Minto	Woven	47.0
Minto	Minto	Nonwoven	47.0
Peat	Peat	. . .	31.0
Limestone	Peat	Woven	32.0
Limestone	Peat	Nonwoven	32.0
Minto	Peat	Woven	32.0
Minto	Peat	Nonwoven	32.0
Peat	Peat	Woven	31.0
Peat	Peat	Nonwoven	30.0

NOTE: Water content of peat = 600%; Unit weight of limestone aggregate = 13.5 kN/m³; Unit weight of Minto aggregate = 8.5 kN/m³.

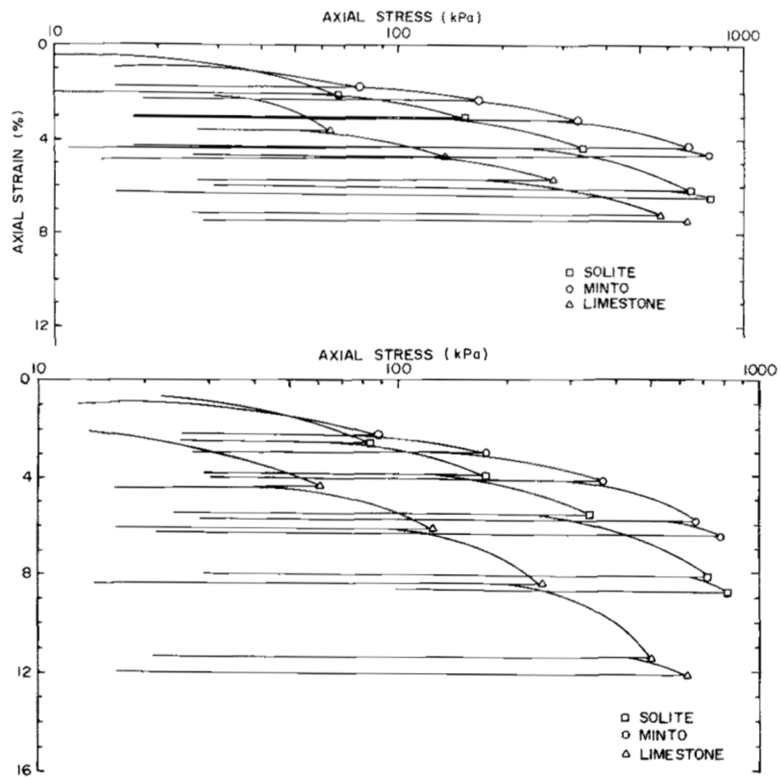


Figure 1.16. Compression tests on compacted (top) and "loose" (bottom) ESCS and limestone aggregates (Valsangkar and Holm 1990).

Table 1.17. Mobilized friction angle between ESCS and other construction materials based on Valsangkar and Holm (1997).

Interface Material	Maximum Friction Angle, degrees	Friction Angle at 0.5% Strain, degrees
Formed concrete	32°	25.2°
Steel	21°	16.5°
Wood (parallel to grain)	36°	24.5°
Wood (perpendicular to grain)	36.4°	24.0°

In certain applications (e.g., landfill leachate collection systems), the hydraulic properties of ESCS become quite important. To that effect, a few studies exist in the literature where the permeability of ESCS was directly measured. Bowders et al. (1997) compared the permeability of four different ESCS aggregates (two expanded shales and two expanded clays) to a leachate collection system (LCS) sand collected from a stockpile of a local municipal solid-waste (MSW) landfill in Texas. They found that the permeability of the ESCS aggregates generally outperformed the LCS sand and were well above the typical minimum threshold of 0.01 cm/s specified for LCS applications (Table 1.18). Mechleb et al. (2014) examined the effects of ESCS on the hydraulic performance of clays. The volume of ESCS aggregate mixed into the clay was varied between 0% to 50% and compaction was performed at 60% and 100% of the standard proctor compaction effort. The results demonstrated that the permeability of the amended ESCS-clay mixture was orders of magnitude higher than the original clay. Both studies point to the high permeability of ESCS, which is advantageous in several geotechnical applications where a free-draining fill material is desirable.

Table 1.18. Permeability of ESCS and leachate collection system (LCS) sand (Bowders et al. 1997).

Permeability (1)	Expanded Shale: 19 to 5 mm (3/4 in. to no. 4) (2)	Expanded Shale: 10 to 2 mm (3/8 in. to no. 10) (3)	Expanded Clay: 9 to 5 mm (3/4 in. to no. 4) (4)	Expanded Clay: 13 to 5 mm (1/2 in. to no. 4) (5)	LCS Sand (6)
(*a*) Hazen Formula					
k (cm/s)	25	6	25	25	0.06
(*b*) Constant-Head Tests					
k (cm/s) $1 \le i \le 2$	9 to 6	3 to 2	11 to 6	8 to 6	—
k (cm/s) $0.2 \le i \le 0.5$	—	6 to 4	19 to 12	—	—
(*c*) Constant Rate-of-Flow Tests, $\Delta h_{(measured)} \sim 0.7$ to 0.3 cm					
k (cm/s)	44	10	39	40	0.02
(*d*) Constant-Head Tests, Postleachate Immersion					
k (cm/s) $0.2 \le i \le 0.5$	—	2 to 0.2	13 to 8	—	—
(*e*) Constant-Head Tests, under 350 kPa Normal Stress					
k (cm/s) $0.2 \le i \le 0.5$	7 to 3	0.6 to 0.1	12 to 9	6 to 2	0.4 to 0.2[a]

Note: i = hydraulic gradient; Δh = head loss.
[a]Due to sidewall leakage during the test, these values are not representative; lower hydraulic conductivities are to be expected.

1.3.2.2.1.3 Example Case Histories

As previously noted, ESCS has been increasingly utilized in a wide range of geotechnical applications (e.g., Holm and Valsangkar 1993; Bowders et al. 1997; Saride et al. 2008; Popescu et al. 2011). In terms of documented case histories in the literature, a few notable studies stand out that highlight the benefits of ESCS. Wall and Castrodale (2013) presented at the NCDOT GEO^{3}T^2 conference a number of successful case histories with ESCS fills. Included in the accompanying paper and presentation is discussion of the following ESCS projects: (1) Tranters Creek Bridge Approach, which was an embankment widening project on low blow-count alluvial soils in Washington, NC; (2) 11th Street Bridges Design-Build, which was a bridge replacement project in Washington, D.C., where ESCS was used as fill material over storm water drainage outfall structures constructed in the 1850's; (3) Blackburn Road Over Neabsco Creek, which was an emergency bridge repair project in Woodbridge, VA, where ESCS was used behind the repaired bridge abutments to decrease lateral loads on their drilled shaft foundations; (4) US 17 Bypass Interchange, which was a rapid embankment construction project over soft compressible soils in Myrtle Beach, SC where ESCS was used to decrease settlements and their associated waiting periods during construction; (5) CATS South Boulevard Project, which was a new parking deck at Interstate 485 and South Boulevard in Charlotte, NC, where ESCS was used to reduce backfill pressures on the parking deck walls. In terms of instrumented case studies, Saride et al. (2008) and Saride et al. (2010) documented the use of ESCS fill for an embankment on SH 360 in Arlington, TX (Figure 1.17). Two inclinometers were installed at a depth of 40 ft. at the centerline of the median (labeled VI 1 in Figure 1.17) and at the outer slope of the embankment (VI 2 in Figure 1.17). After monitoring both inclinometers for periods of time corresponding to construction loads and then the onset of traffic loads, they reported that the behavior of VI 1 was within the permissible limit of 1 inch while VI 2 showed that there was a motion beyond 1 inch. Saride et al. (2010) commented that the possible reason for the additional movement measured in VI 2 was erosion caused by the heavy rainfall during monitoring operations. The results of this study generally supported that ESCS is a suitable alternative as an embankment fill material.

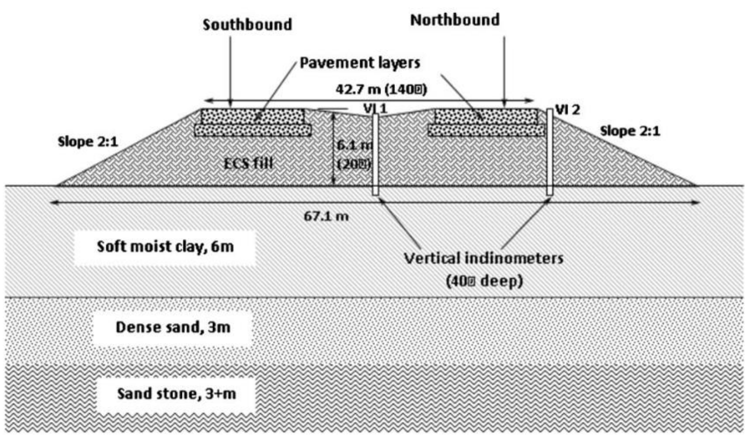


Figure 1.17. Typical cross section and construction stages of the ESCS embankment with subsoil strata from the Saride et al. (2008) and Saride et al. (2010) case history.

1.3.2.2.1.4 Costs

Table 1.5 and Table 1.6 provide typical ranges in estimated unit costs for ESCS along with other lightweight fill technologies. Immediately apparent in these Tables is the relatively high unit cost of ESCS. Its unit cost compares with other specialty lightweight products such as EPS-block geofoam or LCC. This is not surprising given that ESCS is a specialty product manufactured under controlled conditions with raw materials that are relatively expensive, similar to those other products. Lightweight fill materials that tend to be produced from recycled byproducts or “waste” materials from other industries tend to have lower unit costs. However, in the Table 1.7 case history from Milpitas, California, ESCS is actually one of the cheaper alternatives. As noted previously, other factors (e.g., availability, shipping costs, ease of placement, etc.) can play a role in the overall construction costs associated with a particular lightweight technology. It should be noted that given the large range of potential applications for ESCS, there are a large number of suppliers present across the United States (a map can be located at https://www.escsi.org/memberlist/). Furthermore, ESCS can be sourced in rather small quantities for smaller-scale projects such as green roofs and horticultural applications.

1.3.3. Summary

The lightweight fill technologies presented in this chapter of the report highlight the diverse range of materials, manufacturing processes, engineering properties, and applications available to geotechnical engineers. Case histories were described that demonstrated superior performance of lightweight materials with respect to compressibility, durability, strength, and drainage. In certain situations, lightweight fill materials therefore present advantages over traditional engineered fill materials. The technologies discussed in this chapter were some of the most expensive lightweight materials available on the market with respect to unit cost since they are specialty products manufactured for particular applications. Nevertheless, these technologies can be cost competitive, particularly when compared to the savings associated with the more efficient designs they allow for a given project. Absent in this discussion has been many of the lightweight technologies that are sourced from waste products from other industries. These technologies can sometimes present options with unit costs much lower than those presented in this chapter. Given that their chief components are typically recycled/reused byproducts, these particular lightweight fill materials are presented in section 1.5 that reviews sustainable fill alternatives.

1.4. Sustainable Fill Materials

The selection of the appropriate fill material for a given transportation project must consider many attributes, including the suitability of engineering properties, ease of placement, construction costs, and similar concerns. Increasingly, local, state, and federal programs have promoted "green" construction and reduction of CO_2 generation across highway networks, which has driven sustainability as another factor for consideration in transportation projects (Stroup-Gardiner and Wattenberg-Komas 2013a). Sustainability is a broad-encompassing term, under which many materials can be classified that exhibit considerable differences in costs, availability, engineering properties, sustainability, constructability, and performance. The following chapter provides some background information regarding sustainability in general. Subsequent sections will focus on particular sustainable materials for use in transportation-related construction and discuss their engineering properties (density, strength, compressibility), design concerns, and case histories. Some of these materials share characteristics of lightweight fill technologies discussed in Chapter 3, though not all sustainable materials discussed in this chapter will be lightweight. Those that are both sustainable and lightweight will receive additional consideration as pertinent to the goals of TEM WO 013.

1.4.1. Sustainable Engineering

Sustainability can be simply defined as the ability of a system to survive and retain its functionality over time (Basu et al. 2013). The system in question is often our planet, which is a highly complex environment with a delicate ecological balance. Engineered systems are developed to serve humanity and are therefore inextricably linked with social, environmental, and economic systems. Within the context of engineering, sustainability addresses several aspects of design and construction and can be described as an interaction between four E's – engineering design, economy, environment, and equity (Figure 1.18). Sustainability poses additional challenges because a traditional engineering solution may not be acceptable from an environmental or societal point of view, and even if acceptable, may have unknown and unforeseeable consequences (Basu 2013). Sustainability is therefore a multi-dimensional concept that promotes a balanced pathway of human activities so that the natural environment is not degraded and natural resources are not depleted beyond acceptable limits. Sustainability within civil engineering, in particular, can have a major impact because it is estimated that the construction industry accounts for about 40% of the global energy consumption, depletes large amounts of sand, gravel and stone reserves every year, and contributes to desertification, deforestation, soil erosion, and land, water and air pollution (Dixit et al. 2010; Kibert 2016).

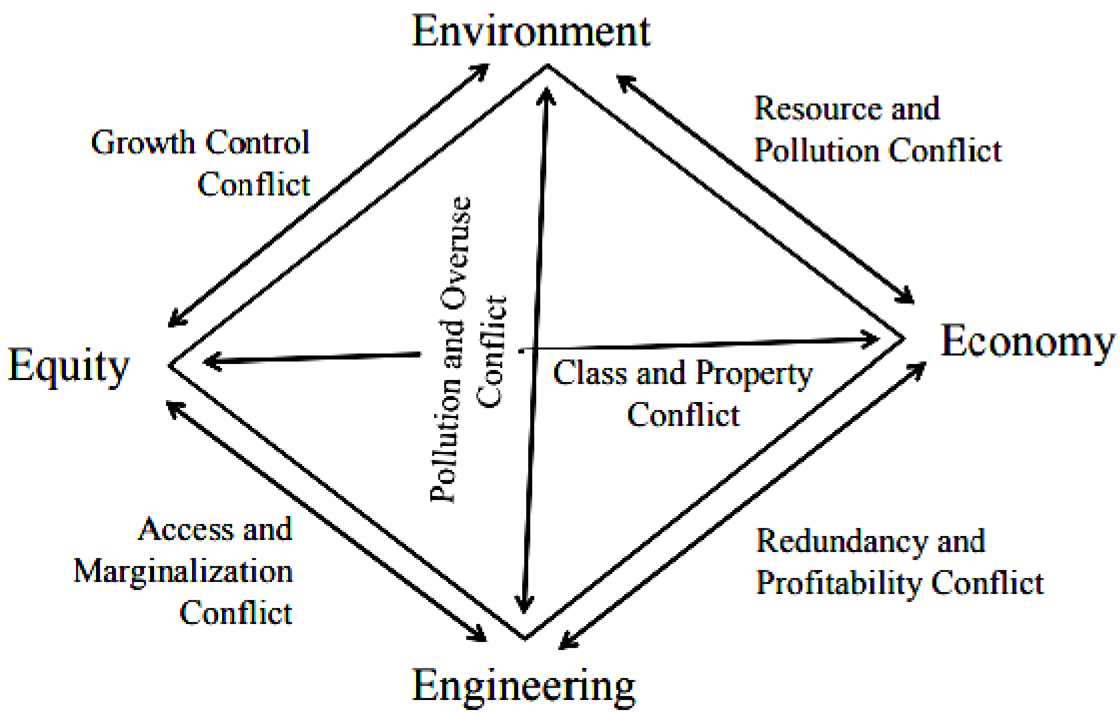


Figure 1.18. Concept of the four E's of sustainability in engineering projects (Basu et al. 2013)

Sustainability within the broader context of civil engineering have led to developments like green buildings that aim for energy and resource-efficient design and construction practices (Kibert 2016). For geotechnical engineering, sustainable practices can take on many forms: design and construction in a manner that minimize the cost of recycling and pulverizing waste material; minimization of the use of resources and energy in planning, design, construction and maintenance of geotechnical facilities; avoidance of materials and methods that may cause negative impacts on the ecology and environment; and reuse of existing geotechnical facilities whenever possible to minimize waste (Basu 2011). Consequently, a multicriteria framework is often proposed to assess the impact of geotechnical designs (Figure 1.19). Sustainable geotechnical practices within the context of transportation projects have been most commonly pursued in the following three areas: sustainable ground improvement methods; sustainable earthworks (including the reuse of natural geomaterials and recycled aggregates); and the development of alternative materials, foundation reuse, and improved rehabilitation and maintenance practices for transportation geo-systems (Correia et al. 2016). Sustainable earthworks represent the broad area of focus for TEM WO 013 in which advances in the reuse of recycled materials, by-products, and waste materials have contributed to sustainable transportation geotechnics.

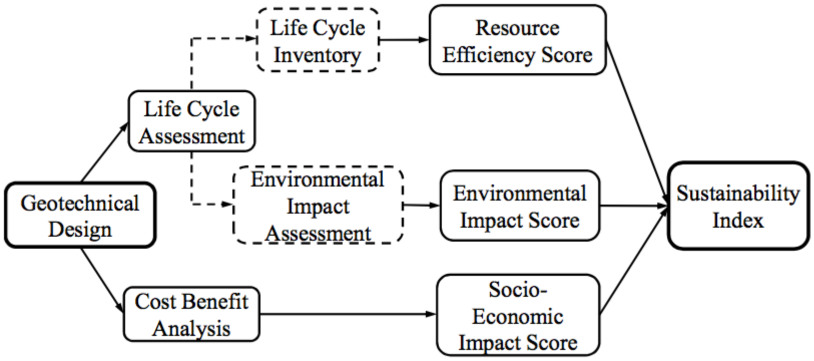


Figure 1.19. Example of a multicriteria sustainability assessment framework (Basu 2013).

1.4.2. Discussion of Specific Sustainable Fill Technologies

The following sections highlight various aspects of the most common sustainable fill technologies used in transportation geotechnical projects. As with the lightweight fill technologies introduced in Chapter 3, a broad overview is provided for each sustainable fill technology where manufacturing, material properties (density, strength, compressibility), design/construction considerations, and case histories are discussed. The sustainable fill materials are divided into those that exhibit lightweight characteristics when compared to natural earthen fill materials and those that do not.

1.4.2.1. Lightweight Materials

The following sustainable technologies are also lightweight when their compacted in-place densities are compared to typical earthen materials used to construct fills. In many cases, these lightweight and sustainable materials can exhibit some cost savings relative to those lightweight technologies introduced in Chapter 3. This is because they utilize a stream of waste by-products from the production of other resources and are therefore often plentiful in availability. However, as will be noted in the particular discussions for the available technologies, the low unit costs may be offset by additional design and/or construction concerns related to substandard engineering properties and performance in certain applications.

1.4.2.1.1. Tire Derived Aggregate (TDA)

According to RMA (2017), approximately 230 – 315 million scrap tires have been generated per year between 2007 – 2015. Though a small percentage of these scrap tires are stockpiled and placed in landfills, the vast majority are recycled or reused (Figure 1.20). There are several markets and applications for recycled rubber derived from discarded tries (Figure 1.21). Non-civil engineering applications of recycled tries include breakwaters, artificial reefs, and reclaiming of rubber and other ingredients. The most common use of scrap tires is for the development of tire-derived fuels for power plants, cement plants, and other industrial applications (Figure 1.20). Civil engineering applications include the use of crumb rubber additive in asphalt pavements, use of tires and their products for soil reinforcement and slope protection, and use of shredded tires as a fill material [i.e., tire derived aggregates (TDA)]. When used as a fill material, the unit weight of TDA is roughly half of typical aggregates depending on compaction efforts (Table 1.19).

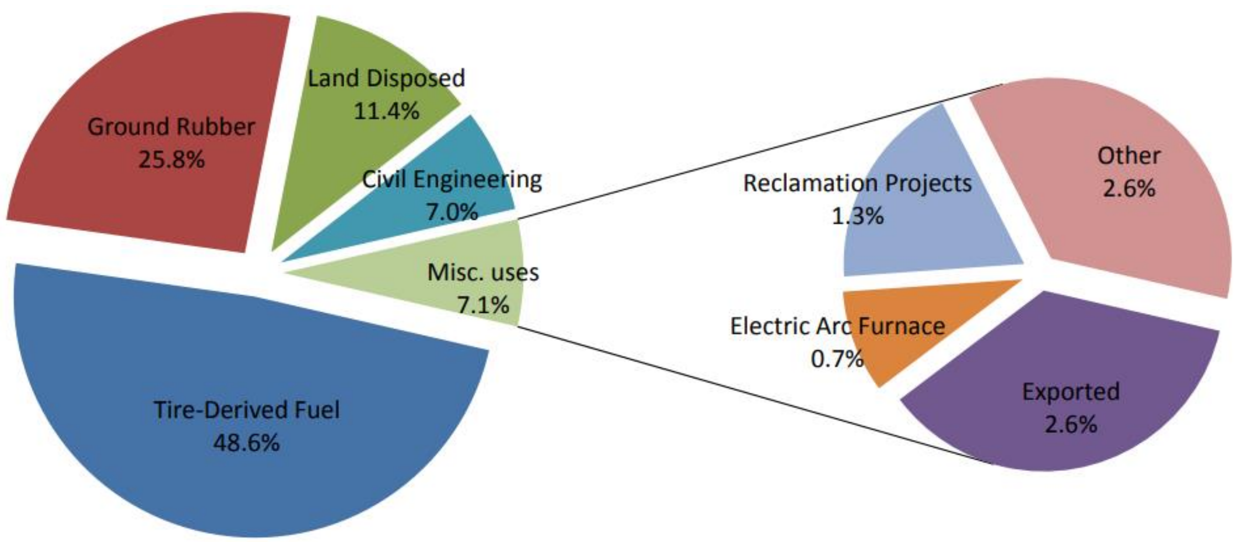


Figure 1.20. Disposition of United States scrap tires in 2015 (RMA 2017). Note: Numbers may not add due to rounding.

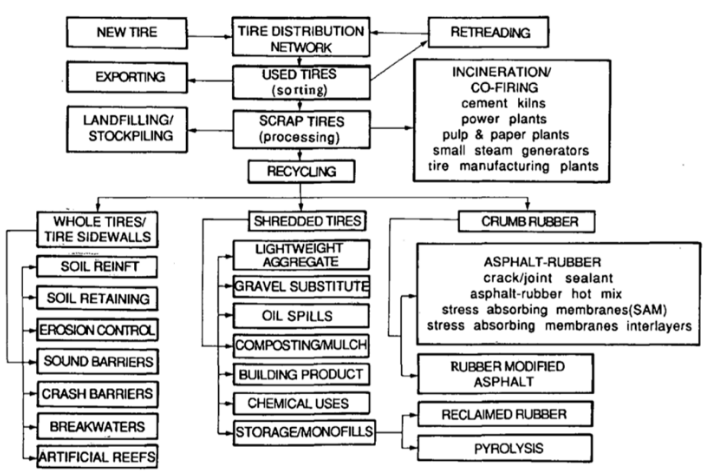


Figure 1.21. Summary of various end uses of rubber tires (Ahmed and Lovell 1993).

Table 1.19. Shredded tire densities (Upton and Machan 1993).

Condition	Density
Loose Density (as loaded in trucks)	390 - 485 kg/m^3 (24 - 30 pcf)
Loose Density (after 64 km haul in trucks)	535 kg/m^3 (33 pcf)
Compacted Density (after three dozer passes)	730 kg/m^3 (45 pcf)
Surcharged Density (after final pavement lift)	845 kg/m^3 (52 pcf)
Final Density (after 1 year of compression)	860 kg/m^3 (53 pcf)

1.4.2.1.1.1 Manufacturing Process

Scrap tires can be processed into different shapes and sizes depending on application. The most common scrap tire products for civil engineering include slit tires, tire shreds, tire chips, and ground/crumb rubber in order of decreasing size of the material. Slit tires are created by specialized tire cutting machines that split the tire into two halves or separate the tread from the sidewall. Tire shreds and tire chips are typically manufactured using the same kind of shredding machines. In this case the waste tires are fed through sequences of rotating blades, where they are cut into smaller pieces. These pieces will have exposed fragments of the steel belt along their edges since the belt is not removed prior to shredding. The resulting pieces are sorted by size, which can vary from as large as 300 to 460 mm (12 to 18 in) long by 100 to 230 mm (4 to 9 in) wide to as small as 100 to 150 mm (4 to 6 in) in length, depending on the machine used. To produce the smaller sized tire chips, more than one shredder device is used to further cut the particles and create finer gradation. Furthermore, a classifier can be used to separate fine and coarse particles from each other. This way, the final product can range in size from 76 mm (3 in) down to 13 mm (1/2 in), based on primary or secondary shredding operations.

1.4.2.1.1.2 Engineering Properties & Design/Construction Considerations

TDA properties investigated by researchers include thermal conductivity, shear strength, lateral earth pressure, and permeability (Kersten 1949; Bressette 1984; Blumenthal and Zelibor 1993; Ahmed and Lovell 1993; Humphrey et al. 1993; Foose et al. 1996; Tweedie et al. 1998; Lee et al. 1999; Shalaby and Khan 2002; Moo-Young et al. 2003; Jeremić et al. 2004; Humphrey and Helstrom 2009; Garcia-Theran et al. 2014; Ahn et al. 2015; Xiao et al. 2015). For thermal conductivity, TDA offers additional insulation over typical soil fills. Previous studies have demonstrated thermal conductivity approximately five to eight times smaller than the thermal conductivity of conventional clays and gravels (Shalaby and Khan 2002). Consequently, TDA fills are better able to resist frost penetration relative to conventional fills.

Testing of TDA for shear strength properties has necessitated specialty equipment due to the large size of the individual pieces. For example, Foose et al. (1996) developed a direct shear test for TDA particle sizes ranging from 100 to 150 mm. Moo-Young et al. (2003) developed a large direct shear device (shear box dimensions of 610 mm length by 610 mm width by 305 mm height) to evaluate the TDA with particle sizes from 50 to 300mm. In other experiments conducted by Xiao et al. (2015), direct shear testing was performed on type B TDA, with the largest pieces removed, using a large direct shear device (shear box dimensions of 800 mm length by 787 mm width by 1,219 mm height). The size of the tires used was between 5 to 150 mm, and the rate of the test was 22 mm/min. Table 1.20 and Figure 1.22 summarize the results from these studies.

Table 1.20. Summary shear strength of previous studies for TDA (Ahn et al. 2015)

TDA size (mm)	Unit weight (kN/m³)	Normal stress (kPa)	Shear stress (kPa)	Shear box size[a] (mm)	Failure criteria	Reference
100–150	4.41[b]	7	6	279(D) × 314(H)	Lateral displacement of 25.4 mm	Foose et al. (1996)
		42	19			
		55	36			
		76	44			
100–200	6.30[c]	4	14	610(L) × 610(W) × 305(H)	Shear force decrease or lateral displacement of 61 mm	Moo-Young et al. (2003)
		13	19			
		19	22			
200–300	6.00[c]	10	18			
		16	20			
		19	21			
5–150	6.60[c]	24	31	800(L) × 787(W) × 1,219(H)	Shear force decrease or lateral displacement of 152 mm	Xiao et al. (2013)
		24	33			
		48	48			
		48	51			
		96	82			
		96	86			
		144	88			
		144	94			
5–150	7.27[c]	26	23	3,048(L) × 1,219(W) × 1,829(H)	Shear force decrease	Fox (2013a)

[a]D, L, W, and H = diameter, length, width, and height, respectively.
[b]Uncompacted unit weight.
[c]Compacted unit weight.

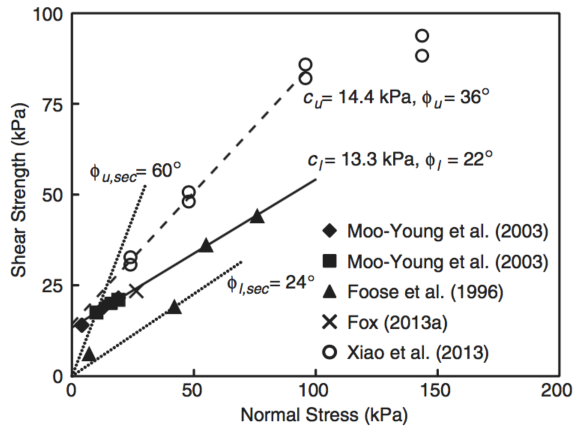


Figure 1.22. Cohesion and friction angle of TDA (Ahn et al. 2015).

Based on studies by Humphrey et al. (1993) and Jeremić et al. (2004), the coefficient of lateral earth pressure for the at-rest condition of TDA samples ranges from 0.26 – 0.32. According to Tweedie et al. (1998), as long as active conditions need a substantial amount of movement, it is impossible to occur for walls with TDA backfill. Thus, a specific amount of at-rest lateral earth pressure coefficient equals (K_o = 0.3) was recommended for designs involving TDA backfill (Humphrey and Helstrom 2009). Garcia-Theran et al. (2014) measured the lateral pressures in geotechnical centrifuge models using tactile pressure sensors for the at-rest condition. The tactile sensors' embedded length was 18.7 cm in the model, which corresponds to a prototype depth of about 7.5 m when the centrifuge box is spinning at 40*g*. As a result, at-rest coefficients of lateral pressure (K_o) were estimated to be 0.401 and 0.235 for the Nevada sand and the 100% TDA backfills, respectively. Table 1.21 shows the range of lateral earth pressure based on TDA size.

Table 1.21. Lateral Earth Pressure (Ahn et al. 2015)

TDA size (mm)	Compacted unit weight (kN/m^3)	Lateral earth pressure coefficient, K	TDA layer thickness (m)	Vertical load	Remarks	Reference
≤76	6.77	0.22–0.23	4	35.9-kPa surcharge	Univ. of Maine wall test,	Tweedie
≤76	6.97	0.22–0.25			0.01-H rotation in the retaining wall[a]	et al. (1998a)
≤200[b] and ≤450[d]	7.90[c]	0.21–0.50	4.2	1.3-m-thick soil	Merrymeeting Bridge, pile foundation	Helstrom et al. (2010)
≤450[d]		0.22–0.64	3	0.9 to 2.3 m thick soil	Limestone Run Bridge, pile foundation	
≤450[d]		0.33–2.30	3	0.5 to 0.7 m thick soil	Wall 119	
≤450[d]		0.20–0.62	2.23–3.05	1.16 to 1.22 m thick soil	Wall 207	

[a]Active pressure not mobilized; lateral earth pressure coefficient between at-rest and active conditions.
[b]Type A TDA.
[c]TDA compacted unit weight assumed for calculation of vertical stress.
[d]Type B TDA.

Various studies have explored the permeability properties of TDA. Typically, laboratory testing using a modified constant head permeameter that can accommodate larger particle sizes has been performed. The results of such efforts have yielded permeability values that range based on particle size from 10^{-1} to 10^{1} cm/s (Bressette 1984; Blumenthal and Zelibor 1993; Ahmed and Lovell 1993; Schaefer et al. 2017a). Generally, this range in permeability is similar to clean sand and gravels and represent excellent drainage characteristics.

Generally, TDA exhibits some excellent qualities as a fill material. As a lightweight material, stresses are reduced, which can limit induced settlements. Additionally, the lateral pressures are reduced due to the low weight and comparable earth pressure coefficients to conventional fills. And as noted previously, TDA is freely draining similar to coarse-grained soils. It also a very durable material that maintains its engineering properties over time given its limited biodegradation. However, TDA does exhibit quite high compressibility and relatively low strength and designers should anticipate significant volume reduction of the fill after placement. Additionally, several TDA fills have self-combusted during the early applications of this material in backfills, resulting in guidelines from the ASTM aimed at avoiding this problem (Arroyo et al. 2011). Table 1.22 summaries design and construction guidelines for the use of TDA as fill materials in geotechnical applications.

Table 1.22. Design and construction considerations for TDA (Schaefer et al. 2017a).

Item	Guideline
Design Parameters	**Dry Density:** 21 to 53 pcf loose and 30-73 pcf compacted, with various of gradations and reporting sources **Angle of Shearing Resistance:** 19° to 30° **Cohesion Intercept: 100** to 230 psf, use 0 for design **Compressibility:** 5 to 40 percent vertical strain over a range of 200 to 4,200 psf vertical stress **Permeability:** 0.5 to 60 cm/sec **Type A Gradation (ASTM D6270)**: 8-inch maximum dimension; 100% passing 4-inch, a minimum of 95% passing 3-inch, a maximum of 50% passing the 1.5-inch, and a maximum of 5% passing the 0.2-inch sieve **Coefficient of Lateral Earth Pressure:** 0.25 to 0.47
Environmental Considerations	The design considerations listed below address minimizing leachate generation and transport from tire shred fills. See NCHRP 435 (Stroup-Gardiner and Wattenberg-Komas 2013d), ASTM (2012), Minnesota DOT (Edstrom et al. 2008), Washington DOT (Baker et al. 2003) for additional information and discussion on environmental considerations.
Combustion Potential	The tire shred gradation and design considerations were developed, in part, to prevent combustion of tire shred fills. These design details prevent or minimize the amount of infiltration of water and air into tire shred fill.
Design Considerations	Limit layers to 10 feet in thickness. Keep the tire shred fill above the water table. Provide good surface drainage of roadway surface to avoid water seepage through the shredded-tire fill. Tire shreds should be separated from the surrounding soil by completely wrapping with a geotextile. Metal fragments must be firmly attached to the chips, with 98 percent embedded in the rubber to prevent exposed wire strands from puncturing tires or construction equipment. Place a minimum 3-foot thick soil cap on the top and side slopes of the tire chip fill to minimize pavement deflections and provide confinement. Place 2-foot soil surcharge for 60 days to minimize post construction settlement due to compressibility of tire shreds. Top of tire shred embankment should be a minimum of 5 feet below the top of subgrade elevation. Multiple 10-foot tire shred layers should be separated by 3 feet of soil fill. Drainage pipes beneath the fill should be located at least 3 feet below the bottom of the of the tire shred layer. Drainage features that could provide free access to air should be avoided at the bottom of the fill.
Construction Considerations	Spread using a track-mounted dozer in a lift thickness of 3 foot or less. Compact using sheep's foot rollers, smooth drum rollers, or by repeated passes with a D-8 dozer. Use multiple passes of compaction equipment, since compressibility decreases after 5 to 8 cycles of loading. Anticipate 35 percent volume reduction during compaction, plus 10 percent shrinkage under loading of soil cover and pavement base course.

PennDOT has developed specifications for the use of TDA in embankment fills, and for the production of TDA aggregates in embankments, as listed below:

- Special Provision (SP), Item 9203-0100 - Select Borrow Excavation, Structure Backfill, Tire Derived Aggregate
- Special Provision (SP), Item 9703-0100 - Production of Tire Derived Aggregate for Embankment and Backfills

As part of these special provisions, requirements have been developed for grain size, exposed metal present in the TDA, amount of metal fragments present with the rubber, and presence of deleterious materials (e.g., oils, gasoline, etc.). Additionally, PennDOT prescribes the encapsulation of the tire shreds with a geotextile, a maximum height of 10 feet for each layer of TDA (multiple layers necessary for higher backfills), and a minimum distance of 3 feet between two adjacent TDA layers.

1.4.2.1.1.3 Example Case Histories

Given the long history of usage, several case histories exist in the literature that summarize the performance of TDA in transportation applications. For example, there was a case history as part of a highway improvement project in Southern Oregon where an existing embankment was widened and raised (Figure 1.23). However, the widened section will apply a load and cause downward movement of the embankment in a landslide. Geotechnical investigations showed that slide movement could be decreased by reducing the embankment load and adding a downslope counterbalance. As a result, they decided to use shredded tire as lightweight fill material. Observation showed that the embankment over shredded tires performed well, and there were no signs of settlement, sloughing, or erosion (Upton and Machan 1993).

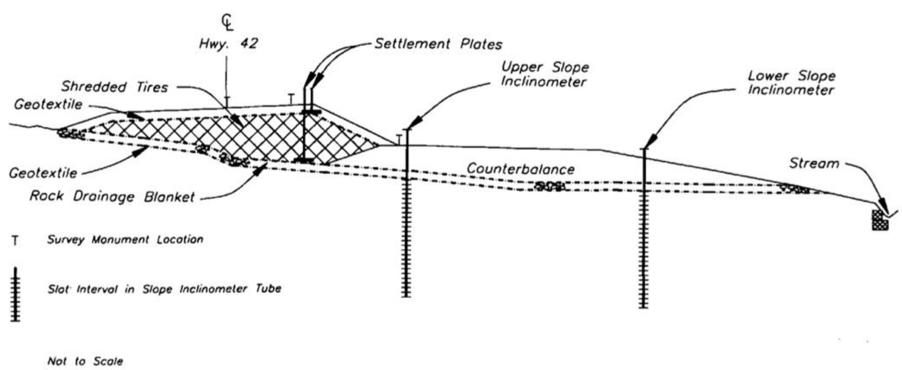


Figure 1.23. Cross section of landslide using TDA (Upton and Machan 1993).

In another TDA project, the California Integrated Waste Management Board (CIWMB) teamed with the Mendocino County Department of Transportation to repair 160 feet of roadway along Marina Drive near the City of Ukiah in California. The project involved removing the original soil fill used to construct the road and replacing it with the lighter tire-derived aggregate (TDA) as a form of slope stabilization for a landslide that had developed. The in-place unit weight of TDA was 8.6 kN/m^3. Figure 1.24 shows the cross-section of the project. Over the deepest fill areas, the output after two years following project completion was only 50 mm of settlement (Kennec, Inc.

2008).

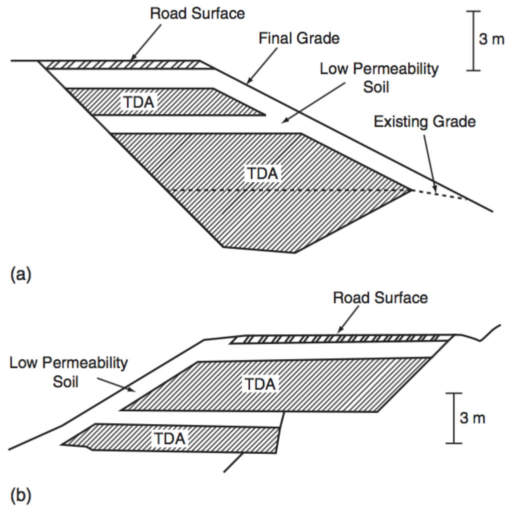


Figure 1.24. Use TDA fill as landslide repair (Ahn et al. 2015).

In 2006 a landslide occurred due to changes in saturation from heavy rain along an 80-m-long section of Geysers Road in Sonoma County, California. It was repaired by replacing some of the slide material with an upper 3-m-thick layer and a lower 1.5-m-thick layer of TDA separated by a 1-m-thick, low-permeability soil. The in-place unit weight was 7.1 kN/m, and finally, over the deepest fill areas, 60 mm of the settlement was measured after two years following project completion (Kennec, Inc. 2008). The use of lightweight TDA resulted in a smaller excavation and a lower-cost repair. This project used approximately 150,000 waste tires and resulted in an overall cost savings to the county of $370,000.

In another project a 200 m long embankment was constructed with TDA for the reconstruction of Route 17 in Windsor and Kirkwood, New York (Dickson et al. 2001). The TDA particle size and thickness was equal to 450 mm and 3 m, respectively. After placement and compaction TDA particles, the layers were covered by 1 to 1.5 m of soil and then were surcharged temporarily with 1.25 to 2.5 m soil layer, which was removed after four months again. The results showed that an initial 27 mm of 3 m thickness compressed between 10-25 mm over 60 days due to the surcharge load.

In 2006, a highway embankment failed in New Brunswick, Canada, because of carelessness in the strength of foundation and project timing. The embankment was reconstructed using two TDA layers (3.0 and 2.1 m thick) with a 1.4-m-thick, low-permeability soil layer between TDA layers (Figure 1.25). At the final step, all the materials were covered by 2.2-m-thick soil cover with the in-place unit weight of 8.1 kN/m^3. As a result, field measurement indicated that the combined time-dependent settlement of both TDA layers was 43 mm over three months (Mills and McGinn 2010).

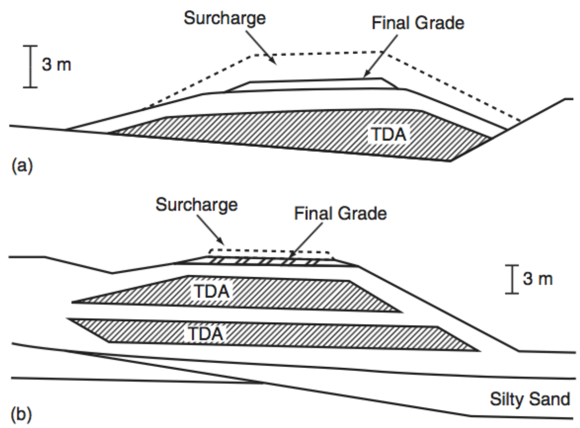


Figure 1.25. Use TDA fill for embankment (Ahn et al. 2015).

Another case history was summarized in Gale et al. (2013) in which new highway construction required an overpass to be built over an existing rail line in Mankato, MN. The 9.1 m embankment showed longitudinal tension cracks due to some distresses and unknown deposits in the foundation soils (Figure 1.26). As a result, the upper 4.6 m of the embankment was reconstructed with TDA to prevent excessive settlement and ensure satisfactory long-term performance. The settlement after one year was approximately 60 cm. Although the TDA fill was thicker than the recommended limit as per ASTM D6270, no sign of self-heating reaction was observed for this project (Gale et al. 2013).

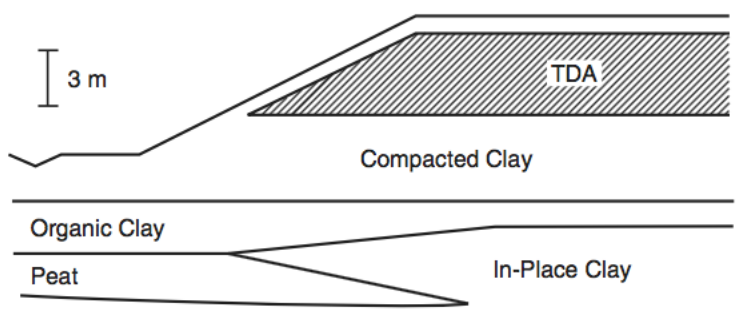


Figure 1.26. Use TDA fill for embankment (Ahn et al. 2015).

1.4.2.1.1.4 Costs

The total cost of TDA depends on many factors such as raw material costs, delivery to the site, placement and compaction costs. For example, in Minnesota the price of delivering tire shreds to a site varies from \$1.25 to \$3.25/yd^3 (\$5 to \$12/ton). In Oregon, from a distance of 150 to 250 miles, the price has been reported as approximately \$30/ton (Upton and Machan 1993). Another significant cost factor is the existence of any environmental program developed by states to encourage the recycling of tire-derived rubber. Such reimbursement programs can decrease the cost of TDA by as much as half (Ahmed and Lovell 1993; Upton and Machan 1993). In addition to direct expenses of the material, the use of TDA as a fill material often requires some monitoring program. The price of installing and applying features, such as settlement plates, temperature sensors, pan lysimeters varies depending on the size and nature of the project and can add several thousands of dollars to the overall project costs (Staseff and Sangiuliano 2011).

1.4.2.1.2. Wood Fibers

The use of wood fibers in geotechnical construction historically arose from the need to stabilize low-volume roadways in geographic areas where sands and gravel fill materials were expensive or difficult to source but a thriving lumber industry existed where wood waste material could be more readily sourced (Russell 2015). Wood waste material can consist of a mixture of bark, wood shavings, wood chips, wood scraps, and mineral grit that is a by-product of the lumber, paper, construction, or other industries. The two most common geotechnical applications for wood waste material are as an erosion control countermeasure on slopes and as a material to stabilize embankments. Composted material, including wood waste, has been one of the recognized techniques in order to reduce or omit erosion effects. WSDOT recommends the classification scheme in Table 1.23 to differentiate between different wood fiber compositions.

1.4.2.1.2.1 Manufacturing Process

In general, the term "wood fiber" as applied to geotechnical engineering encompasses a range of word-based products, including hog fuel, sawdust, planer chips, and wood waste. Hog fuel is a kind of ground wood and bark burned in a boiler to create steam. Sawdust is the residue of cutting logs into lumber. When the logs are cut into the final dimension, the excessive part of those materials can be used as planer chips. Wood chips is another equivalent term for planer chips. The typical operation for fabricating wood chips follows typical logging practices where a feller-buncher grabs a tree before cutting it and skidders transport the tree after felling. The skidders consist of large grapples that help to hold log bunches while skidded to a chipper unit (Bowman et al. 1987). Wood waste is a more generic terms and includes hog fuel, sawdust, planer chips, or a combination of the three (Kilian and Ferry 1993).

1.4.2.1.2.2 Engineering Properties & Design/Construction Considerations

For the purpose of wood fiber as a geotechnical material, much research has taken place to explore the strength, hydraulic conductivity, and compressibility of wood-soil blends (e.g., Baker et al. 2003; Etim et al. 2017; Nath et al. 2018; Oluremi 2019). Nath et al. (2018) showed that as the percentage of wood ash increased when mixed with a low plasticity clay, the UCS increased to as much as twice the UCS of the untreated clay. However, the addition of wood caused failure to occur at lower strain levels and exhibited more post-peak decrease in stress. Similarly, Oluremi (2019) revealed that the maximum UCS of a wood-soil blend occurred at 2% waste wood ash content. Nath et al. (2018) also examined the drained strength of the wood ash-clay mixture and found that wood increased the friction angle but did not have a significant impact on the drained cohesion. Sadasivam and Reddy (2015) found similar results where wood materials have more cohesion and friction angle comparable to soil.

Table 1.23. WSDOT wood fiber classification (Allen and Kilian 1993)

Class	General Appearance	Appearance of Decay[a,b]	Particle Strength (Breaking)[c,d]	Particle Stiffness (Bending Capacity)[d]
1	Woodlike, sharply defined graininess	Fresh: Sharp color, fresh woody smell, no disintegration	Cannot be broken with fingers	Retains its shape with force
2	75% of material is woodlike, well defined graininess	Initial signs of decomposition: Distinct color, definite wood smell, very little disintegration of wood fibers	Very difficult to break with fingers	Easily returns to original shape with release of force
3	50% of material is woodlike, complete but poorly defined graininess	Middle stage of decomposition: Fading color, weak wood smell, some disintegration of wood fibres	Breaks with firm finger force	Shape is permanently, but slightly, distorted with force
4	25% of material is woodlike, only partial graininess remains	Advanced stage of decomposition: Fading color, organic smell, mostly disintegrated	Breaks easily with fingers	Shape is permanently distorted with force
5	No longer woodlike, no graininess	Completely decomposed: Dull color, foul smell, completely disintegrated	Squeezes between fingers	No longer returns to original shape; spongy

[a]Primary emphasis is on disintegration
[b]All discriptors may not apply
[c]Standard testing size is 2" x 1/2" x 3/8"
[d]Moisture content for tested sample is "wet to touch"

Multiple studies have examined the hydraulic conductivity of wood fibers. In one study, different percentages of burnt and unburnt sawdust were added separately to soil. The hydraulic conductivity results showed that soil mixed with unburnt sawdust exhibited higher hydraulic conductivity than soil mixed with burnt sawdust ash (Figure 1.27) (Etim et al. 2017). The probable reason is that unburnt sawdust sample has larger particle sizes. Oluremi (2019) revealed a range of hydraulic conductivity from 1.5×10^{-10} to 4×10^{-9} mm/s depending on waste wood ash content up to 10%.

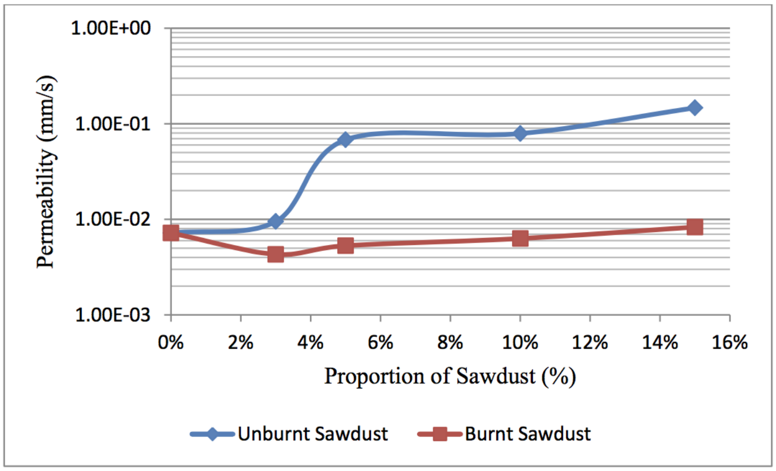


Figure 1.27. Variation of permeability coefficient with sawdust (burnt and unburnt) (Etim et al. 2017).

In addition to strength testing, Nath et al. (2018) also examined the compressibility of wood-clay blends. Using a one-dimensional consolidation apparatus, Nath et al. (2018) found that the compression index (C_C) decreased with an increase in the wood ash content and the initial void ratio increased for the ash-treated soil. Hardcastle and Howard (1991) examined the compressibility of a wood fiber fill used to reestablish the grade of an airport runway after removal of problematic soils and significant settlement had occurred. Using samples of the wood fill, Hardcastle and Howard (1991) determined compression on the order of 4% of the fill thickness under effective stress increases up to 360 psf. Subsequent secondary compression testing yielded values for the secondary compression index larger than fine-grained soils but smaller than organic clays.

In general, the suitability of wood waste material for geotechnical construction is highly dependent on the scale and scope of the project along with geographic constraints. The species of the raw wood product from which the waste material is derived along with the climate/humidity where the raw wood product was used can have a pronounced effect on their durability. For example, softwoods such as pine, cedar or spruce tend to be more durable. Wood fills in the northern Great Lakes region have lasted for decades on forest roads (Bowman et al. 1987) but have been ineffective over long periods of time in the warmer and more humid southeast (Russell 2015). The burial of wood fiber products can improve their long-term durability as the anaerobic environment results in less exposure to oxygen and subsequent rot when the wood is moist. For example, buried wood chips and chunkwood in road applications have been shown to last more than 20 years in some cases (Russell 2015). Table 1.24 summarizes some general design and construction specifications for wood fiber materials when used for geotechnical construction.

Table 1.24. Design and construction considerations for wood fiber (Schaefer et al. 2017a)

Item	Guideline
Design Parameters	**Moist Density:** 45 to 60 pcf **Angle of Shearing Resistance:** Sawdust – 25° to 27° Hogfuel – 31° Wood Chips – 30° to 49° **Permeability:** 1 x 10^{-5}m/s **Compressibility:** Loose volume reduces 40 percent on compaction. **Vertical subgrade reaction coefficient:** 1300 to 1450 psi in top 2 feet, roughly corresponding to a CBR of 1
Environmental Considerations	Potential environmental effects of the leachate include: depletion of available dissolved oxygen in groundwater. lowering of groundwater pH because of acidic nature of leachate, which has pH of 4 to 6. potential contamination of water with toxins. Methods to reduce contamination include: reducing water infiltration into wood fiber by drains and capping. treatment of leachate. barriers between wood fiber fill and adjacent bodies of water.
Design Considerations	Restrict particle size to 6 inches maximum to prevent development of large voids. Less than 30 percent should be finer than 0.5 inches to minimize the use of fine uniform sawdust. Use fresh wood fiber to prolong the life of the fill. Use side slopes of 1.5H:1V or flatter. Employ surface treatment with cover material of thickness 2 feet or more to protect slope from erosion and minimize deterioration of wood fibers. Restrict height of fill to about 16 feet and reduce air penetration into wood to minimize the possibility of spontaneous combustion.
Construction Considerations	Truck-mounted equipment is used to spread fiber in 12 to 20-inch lifts. Two passes with a fully loaded hauling truck weighing 33 kips or more is usually sufficient to properly compact wood fiber.

1.4.2.1.2.3 Example Case Histories

Most case histories in the literature for wood fiber products examine their use as an erosion control countermeasure in slopes or as a lightweight embankment fill material. For example, Demars et al. (2000) showed that wood waste filter has the best erosion control performance after multiple storm events at a field site with a slope of 1 vertical to 2 horizontal. Additionally, wood waste mulch at a thickness of 0.75 inches (19mm) protected the soil from being eroded as effectively as a thicker covering (Demars et al. 2000). Similarly, Buchanan et al. (2002) found that treatment of an embankment with small wood chips reduced erosion by 22%, large wood chips reduced erosion by 78%, and the mixture of chip sizes reduced erosion by 86% (Buchanan et al. 2002).

In another case study, an embankment section was part of a newly constructed two-lane highway. The fill was 180 m long and 13.4 m high (Figure 1.28) and constructed over organic sandy and clayey silt. Monitoring of the embankment showed no stability problems or excessive settlements, even after post-construction (Allen and Kilian 1993). Consequently, WSDOT deemed the performance of wood fiber acceptable and allowed its use for permanent applications with design lives of more than 50 years (Allen and Kilian 1993).

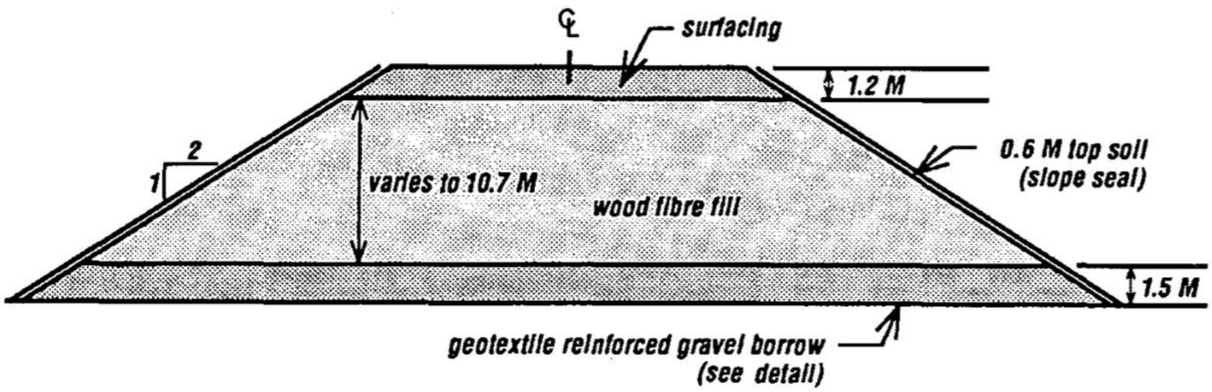


Figure 1.28. Cross section of wood fiber embankment (Allen and Kilian 1993).

1.4.2.1.2.4 Costs

As noted previously, the use of wood fiber products for geotechnical construction is highly dependent on availability from logging and other timber-based industries. This has a major impact on the costs. When readily available, the raw material costs can be as low as a few dollars per cubic yard (e.g., Hardcastle and Howard 1991). This can result in significant cost savings. For example, in the WSDOT case history summarized in Allen and Kilian (1993), if lightweight fills were not used, the construction of the proposed bridge would cost approximately $1.7M. The other alternative proposed was ground improvement using stone columns at an estimated cost of $1.5M. However, the use of wood fiber as a lightweight fill in the embankment resulted in an estimated cost of a little less than $1.0M, which was more than $500,000 cost savings compared with the ground improvement alternative and more than $700,000 less than the original bridge design (Allen and Kilian 1993).

1.4.2.1.3. Coal Combustion By-Products

As the name suggests, coal combustion by-products are the residue waste materials that are produced by coal fired power plants. Generally, coal combustion by-products (CCP) refer to four materials: boiler slag, bottom ash, fly ash, and flue gas desulfurization (FGD). These materials were often landfilled, but within the last few decades recycling applications have been identified for these by-products, including in geotechnical fill applications. Of the four CCP, fly ash is the most commonly used in highway applications (Stroup-Gradiner and Wattenberg-Komas 2013b). The American Coal Ash Association (ACAA) estimated that the volume of coal combustion by-products production in 2005 was approximately 123 million, with the portion of fly ash and bottom ash totaling 71 and 18 million, respectively. The chemical composition of ash directly depends on the properties of the composition of the coal burned in power plants. Typically, the major constituents of these ashes are SiO_2, Al_2O_3, and Fe_2O_3.

Based on chemical composition and type of coal burned, two major classes of fly ashes are identified in ASTM C618-15 (ASTM 2015). These are designated as class F and class C. Incineration of fresher lignite or sub-bituminous coal results in Class-C fly ashes. These are naturally pozzolanic and exhibit self-cementing abilities (i.e., these ashes harden and gain strength over time in the presence of water). They have more than 20% lime (CaO) and are rich in alkali and sulfate (SO_4), and usually, they do not require an activator. For Class-C fly ash, the sum of SiO_2, Al_2O_3, and Fe_2O_3 should be in the 50–70% range per ASTM C618-15 (ASTM 2015). Class-F fly ashes are generated by the incineration of older, harder bituminous and anthracite coals. They possess less than 10% lime (CaO) and are naturally pozzolanic. However, the glassy silica and alumina of Class-F fly ashes require water and an activator (quicklime, hydrated lime, or ordinary Portland cement) to react and produce cementitious compounds. For Class-F fly ash, the sum of SiO_2, Al_2O_3, and Fe_2O_3 should be more considerable than 70% per ASTM C618-15 (ASTM 2015).

1.4.2.1.3.1 Manufacturing Process

Each of the aforementioned CCP is obtained from a different location along the typical steam generating process in a coal fired power plant (Figure 1.29). The process of ash fabrication is started by stocking coal. Then the coal is grinded in grinding plants in a very fine dust. The coal is blown into a furnace and the coal ash is produced. The fine solid particles of coal ash remained at the electrostatic precipitator is fly ash while the solid coarse particles remained at the bottom of the boiler called bottom ash or boiler slag depending on whether the molten ash is water cooled or not (Ballisager and Sorensen 1981). Typically, the ratio of fly ash to bottom ash is 80:20 by weight (Kim et al. 2005; Rai et al. 2010). Fly ash slurry that results from the process can also be pumped into a large pond and allowed to dry, resulting in pond ash (Figure 1.30). FGD is obtained at the tail end of the process when sulfur dioxide (SO_2) is removed from the exhaust resulting in a mixture of gypsum ($CaSO_4$), calcium sulfite ($CaSO_3$), fly ash, and unreacted lime or limestone.

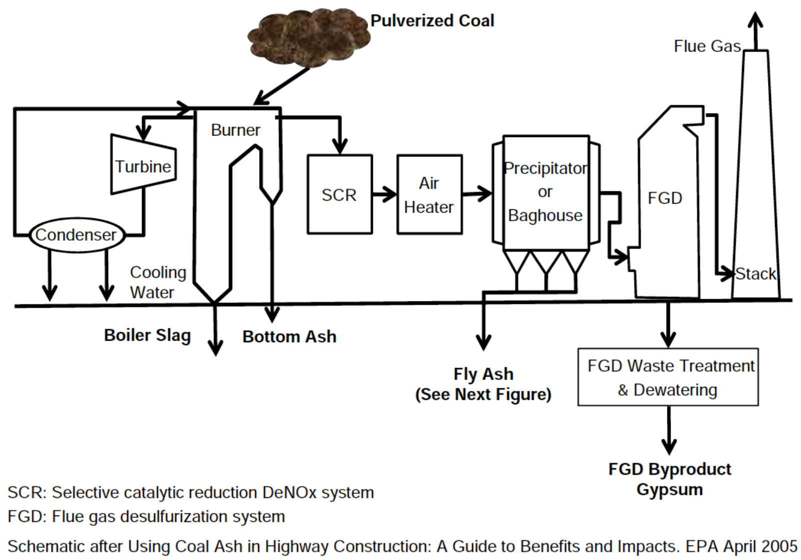


Figure 1.29. Production of coal combustion by-products (Horiuchi et al. 2000).

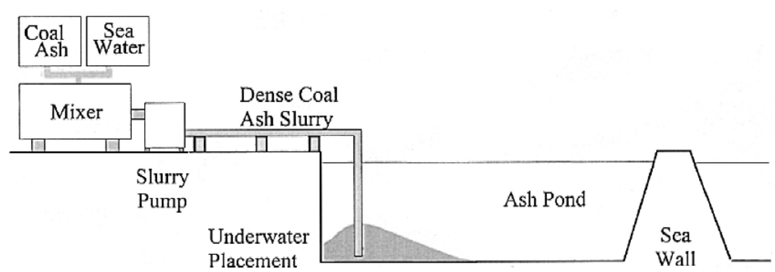


Figure 1.30. Concept of slurry reclamation for pond ash (Horiuchi et al. 2000).

1.4.2.1.3.2 Engineering Properties & Design/Construction Considerations

Since fly ash is the most commonly used CCP for civil engineering, this section will focus on documenting previous efforts to investigate its engineering and use in design. Fly ash has a number of civil engineering applications, including in concrete admixtures to enhance the performance of concrete, in combination with lime and aggregate to produce a quality stabilized pavement base [i.e., pozzolanic-stabilized mixtures (PSMs)], as mineral filler in hot-mix asphalt (HMA) paving applications, in grouts for pavement subsealing, for chemical and/or mechanical stabilization of soils, in flowable fills, and as a borrow material to construct structural fills and embankments (ACAA 2003). Consequently, its engineering properties have been extensively examined. The most commonly investigated engineering properties include California Bearing Ratio (CBR), shear strength, hydraulic conductivity, and compressibility (Digioa and Nuzzo 1972; McLaren and

Digioia 1987; Singh and Panda 1996; Toth et al. 1988; Raymond and Smith 1996; Lee et al. 2001a; Pandian 2004; Kaniraj and Gayathri 2004; Puppala et al. 2006; Sivapullaiah and Moghal 2011a; Sivapullaiah and Moghal 2011b; Lal and Mandal 2012).

Given that fly ash has often been used in pavement applications, it is unsurprising that its performance in the CBR test has been investigated. CBR is a penetration test used to evaluate the subgrade strength of materials to determine the thickness of pavement and its component layers. It is an important parameter in the design of flexible (i.e., asphalt) pavements. CBR values have been found to range from approximately 15% - 27% under unsoaked conditions and 6% - 14% under soaked conditions (Toth et al. 1988). This compares favorably to naturally occurring soils whose CBR values range from 3% - 15% for fine-grained materials, 10% - 40% for sand and sandy soils, and from 20% - 80% for gravels and gravelly soils (Hough 1969).

The studies conducted on the shear strength of fly ash revealed that pozzolanic fly ashes derive the greater part of their shear strength from internal friction, with little contribution from apparent cohesion (Digioa and Nuzzo 1972; Singh and Panda 1996). This friction angle has been observed to range between 26° - 42° range (Puppala et al. 2006) with a mean friction angle value of approximately 34° (McLaren and Digioia 1987). However, in a study by Lal and Mandal (2012), the fly ash collected from a power plant in India did exhibit cohesion (approximately 20 kPa) during direct shear testing at the optimum moisture content. The maximum friction angle of 32° was recorded when the fly ash compacted to dry of optimum at 12% (Lal and Mandal 2012).

Given its usage in embankment fills, some research has been performed to characterize the flow of water through fly ash. The hydraulic conductivity of properly compacted fly ashes can range 10^{-4}–10^{-6} cm/s, matching the range exhibited by silty sand to silty clay soils (Hough 1969). In one study, Pandian (2004) investigated the range of hydraulic conductivity of fly ash, pond ash, and bottom ash and found values of 8×10^{-6} to 1.9×10^{-4} cm/s for fly ashes, 5×10^{-5} to 9.6×10^{-4} cm/s for pond ashes, and 9.9×10^{-5} to 7×10^{-4} cm/s for bottom ashes (Pandian 2004). Lal and Mandal (2012) performed falling head permeability testing on fly ash specimens prepared with different moisture contents 12%, 16%, 20% [dry side of optimum], 24% [OMC], 28%, 32% [wet side of optimum]. The results show that the Koradi fly ash in their study had permeability values in the range of 1.4×10^{-7} to 6.3×10^{-8} m/sec.

Pandian (2004) found that fly ashes tend to have higher void ratios relative to most soils, which affects their compressibility with compression index (C_C) values varying from 0.049 - 0.284 for fly ashes, 0.052 - 0.30 for pond ashes, and from 0.057 - 0.484 for bottom ashes.

In terms of design considerations, fly ash obtained from silos can be delivered with close controls on moisture content and grain size distribution. This is not the case for other forms of CCP, particularly pond ash where the moisture content or grain size distribution can vary tremendously depending on location within the pond. Generally, the mechanical behavior of fly ash shares a lot of similarities to those of silt, including its corresponding difficulties as an engineering material (e.g., dusting, erosion and frost susceptibility). For this reason, it is typically recommended to avoid the use of fly ash as a fill material below the groundwater table or when adequate drainage is not possible in the design. Table 1.25 summarizes of necessary considerations for design and construction with fly ash materials.

Table 1.25. Design and construction considerations for fly ash (Schaefer et al. 2017a).

Item	Guideline
Design Parameters	**Density Range, Compacted:** 70 to 90 pcf **Shear Strength:** 33° to 40°, c = 0, for Type F; Class C is self-hardening, so the shear strength will vary as it cures **Permeability:** Range of 1 x 10^{-6} to 1 x 10^{-9} m/s **Compressibility:** C_c = 0.05 to 0.37, C_{cr} = 0.006 to 0.04 **Grain Size Range:** 0.005 to 0.074 mm **Specific Gravity:** 1.9 to 2.5 **Atterberg Limits:** Non-plastic
Environmental Considerations	The leachate is alkaline, with pH of 6.2 to 11.5. Calcium, sulfate, and boron are soluble constituents, which can leach and migrate. The EPA (Rittenhouse 1993) has stated fly ash as non-hazardous.
Design Considerations	Where the groundwater table is high, a drainage blanket should be provided below the fly ash fill to promote a capillary cutoff and prevent frost heave and resiliency of the subgrade. Runoff from paved surfaces should be discharged into a drainage system. Surface waters from peripheral areas should be diverted away from the embankment to minimize infiltration into the fly ash. The side slope of embankments should be covered with at least 2 feet of soil to prevent erosion. If concrete is to be formed directly on fly ash, a polyethylene barrier should be placed on the fly ash to prevent moisture absorption from the fresh concrete and to serve as a moisture barrier. Use fly ash in the concrete to reduce sulfate attack.
Construction Considerations	Fly ash behaves like silt, thus, dusting will occur when dry, and compaction is difficult when wet. Some means for adding water should be available on site to keep the water content near optimum for compaction. Surface protection to minimize erosion may be required. Compaction is obtained with smooth drum vibratory rollers or self-propelled, pneumatic-tired rollers. Use 10-inch lifts and compact the fly ash immediately after spreading. The use of test strips to develop the most efficient compaction procedures is advisable.

1.4.2.1.3.3 Example Case Histories

Fly ash has a long history as a highway construction material across many applications (e.g., Table 1.26). Consequently, the following presents just a short overview of two notable and well-documented case histories in the literature. Yoon et al. (2009) presents a case history where a test embankment was constructed to aid INDOT in understanding the performance of fly ash and bottom ash mixtures. The height, length, and width of the test embankment were equal to 7.6, 60, and 100 m respectively (Figure 1.31). Each layer of ash was placed across the length of the structure, and then a steel track bulldozer was used to set and adjust the compaction appropriately. After five months, monitoring and inspection of the test embankment found only 80 mm of settlement. Moreover, after four months passing, the embankment's lateral movement was negligible.

Table 1.26. Summary of embankment construction projects using high volumes of fly ash (Yoon et al. 2009).

State	Year	Project description	Estimated tonnage or volume
Arizona	1980	I-40, Joseph City	46,300 m^3
Delaware	1987	I-495, Wilmington	6,110 m^3
Illinois	1972	LR 437, Waukegan	188,700 m^3
Massachusetts	1978	John Scott Blvd., Norton	3,820 m^3
Minnesota	1979	SR 13, Eagan	267,400 m^3
Minnesota	1975	High Bridge Plant, St. Paul	9,932 m^3
Ohio	1981	I-480, Avon	30,000 t
Ohio	1983	US 35, Gallia County	27,000 t
Pennsylvania	1967	Culver Road, Hopewell Township	145,160 m^3
Virginia	1978	SR 665, Carbo	300 t
Wisconsin	1976–1977	Milwaukee	120,000 t
Wyoming	1976	US 60, Morgantown	40,000 t
Wyoming	1971	US 250, Fairmont	5,000 t

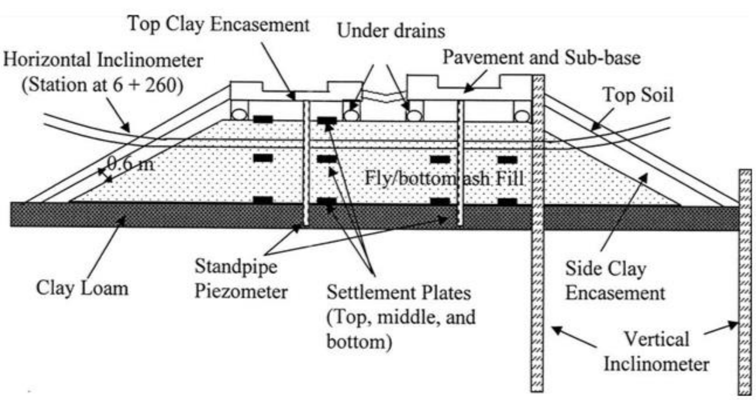


Figure 1.31. Schematic cross section of the test embankment in Yoon et al. (2009).

In another study (Alizadeh 2016), several Controlled Low Strength Materials (CLSM) mixtures made with fly ash were examined to understand their performance in terms of load-carrying capacity and deformations for abutments. A full-scale abutment was constructed in the laboratory (Figure 1.32). The results showed that the CLSM abutment was capable of carrying typical bridge loads (up to 780 kN of vertical loading) with a reasonably significant safety factor and with minimal deformations.

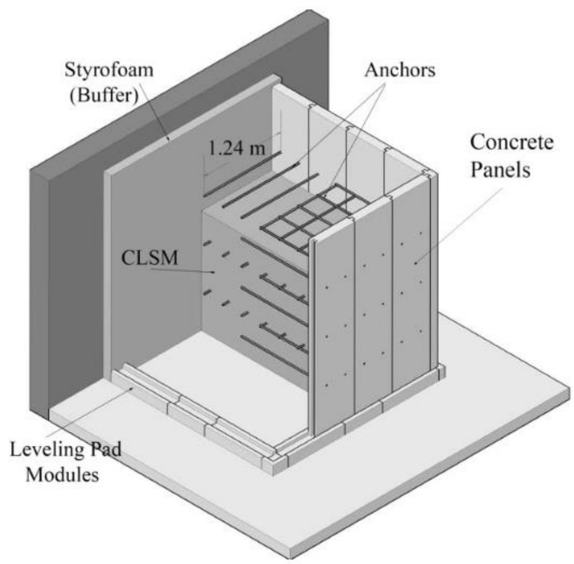


Figure 1.32. CLSM abutment in the Alizadeh (2016) study.

1.4.2.1.3.4 Costs

Only limited cost information was found in the literature despite the use of fly ash across several highway construction applications. Some major factors can directly or indirectly affect the economic aspects of fly ash technology. Since fly ash is a solid waste, most of its price is associated with transport and handling operations. Stroup-Gardiner and Wattenberg-Komas (2013b) noted that there is a significant difference between the final cost of CCP of power plant owners with power plants without their own landfills, so that typical plant landfill costs range from $3 to $15/ton for plants with their own landfills. Landfilling through another company increased costs from $10 to $35/ton. Siddiki et al. (2004) reported that the cost of loading and hauling byproducts plays an important role and can even dominate other costs. Based on their report, the cost of loading and 15-mi (24-km) hauling to a construction site ranges from $3 - $4 per ton. However, this price can be reduced to approximately $1 per ton if backhauling other materials such as coal and limestone for a power plant. In contrast, the cost for landfilling is in a range between $3 to $35 per ton which is much higher. A study on bottom ash showed that adding bottom ash to clay for permeability reduction requires 10 to 15 percent clay to meet the requirement and could range between $1 - $2 which are 60% - 70% less than the cost of natural aggregates (Wasemiller and Hoddinott 1997).

1.4.2.1.4. Slag

Slag material is typically a term used to refer to a byproduct of the production of ferrous materials. This byproduct is a kind of residue of metals production from ore and refinement of impure metals. First of all, ore, coke, and limestone mix are combusted. The residue obtained after cooling is referred to as molten slag. Slag produced from the production of iron is commonly referred to as blast furnace slag, while slag from steelmaking is commonly known as steel slag. Other types of slags exist, including those obtained as byproducts from non-ferrous metals, including copper and nickel slag, lead/zinc slags, and phosphorous slag (Jahangirnejad et al. 2013).

1.4.2.1.4.1 Manufacturing Process

Tare the most common slag byproducts are obtained from the production of ferrous materials and include steel slag, boiler slag, and blast furnace slag. Like other slags, steel slag (SS) is a byproduct made during the separation process of steel from impurities. The portion mass of this product is about 15% of the original steel. The product is finalizing as a molten liquid, which solidified by cooling down. The chemical composition of this product is a complex solution of silicates and oxides. Boiler slag (BS) is coarse, granular, incombustible CCP, which is collected from wet bottom boilers of furnaces that burn coal (ECOBA) (Smolar et al. 2016). It is extracted from molten ash in wet, water cooled bottom boilers. It forms when slag falls from the furnace in a hot molten state and is discharged in cold water where the particles crystallize, solidify, and form angular glassy particles with sizes less than 1⁄8-inch to 3⁄8-inch. Blast furnace slag is the remaining part obtained from pig iron in a blast furnace. It is a kind of molten liquid that is a solidified version of silicates and oxides upon cooling. However, if the slag leaves to become cool slowly, it is named air-cooled blast furnace slag, but if the slag is cooled quickly by water or water and air, vitrified slags are produced called granulated slag and pelletized slag (O'Flaherty 1988). The other name is ground granulated blast furnace slag (GGBS). Blast furnace slag is produced from iron blast furnaces as a byproduct of the iron-making industry. It results from the fusion of a limestone flux with ash from coke and the siliceous and aluminous residue remaining after the reduction and separation of the iron from the ore (Nidzam and Kinuthia 2010).

1.4.2.1.4.2 Engineering Properties & Design/Construction Considerations

Compaction, permeability, shear strength, and dynamic properties have been investigated to determine the behavior of slag byproducts in civil engineering applications (Malasavage et al. 2012; Kumar et al. 2019; Wang et al. 2019). Kumar et al. (2019) found that the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) of blast furnace (BF) slag was similar to granular soils (21kN/m^3 and 10%, respectively). Similarly, both copper and zinc slag were found to exhibit similar compaction characteristics to granular soils (Prasad and Ramana 2016a,b). Kumar et al. (2019) also examined the shear strength of BF slag using a direct shear test according to ASTM D3080-98. The result shows that the BF slag exhibited post-peak strain softening behavior attributed to the presence of bond resistance between slag particles from high CaO content. The peak friction angle and cohesion of the tested BF slag were found to be 36.8° and 16.3 kPa. Prasad and Ramana (2016a,b) found much higher friction angles for copper and zinc slags (49.1° and 51.7°, respectively). Wang et al. (2019) examined the permeability of steel slag and a mixture of steel slag and silt. Tables 1.27 and 1.28 show the summary of the experimental results for permeability. Additionally, Malasavage et al. (2012) examined the permeability of a blended mixture includes dredged material and steel slag furnace. The results from that study showed that the hydraulic conductivity of the dredged high plasticity organic soil increased from 10^{-8} cm/s to 10^{-5} cm/s when the steel slag content increased to 80%.

Table 1.27. Permeability of modified silt with different amounts of steel slag (Wang et al. 2019).

Mixing ratio of steel slag	Average water temperature T (°C)	k_T (cm/s)	η_T/η_{20}	k_{20} (cm/s)
	15.0	1.691×10^{-7}	1.133	
0%	14.5	1.676×10^{-7}	1.148	1.920×10^{-7}
	14.5	1.692×10^{-7}	1.148	
	13.7	3.348×10^{-6}	1.178	
30%	13.7	3.312×10^{-6}	1.178	3.923×10^{-6}
	13.5	3.39×10^{-6}	1.178	
	14.2	5.307×10^{-6}	1.163	
50%	14.1	5.236×10^{-6}	1.163	6.131×10^{-6}
	14.0	5.281×10^{-6}	1.163	
	13.5	3.155×10^{-5}	1.178	
70%	13.5	2.827×10^{-5}	1.178	3.467×10^{-5}
	13.5	2.847×10^{-5}	1.178	
	13.0	8.99×10^{-5}	1.194	
90%	13.0	9.15×10^{-5}	1.194	1.011×10^{-4}
	13.0	7.27×10^{-5}	1.194	
	11.9	1.97×10^{-4}	1.227	
110%	11.8	3.19×10^{-4}	1.227	3.276×10^{-4}
	11.8	2.85×10^{-4}	1.227	

Table 1.28. Permeability of three kinds of steel slag at standard temperature (Wang et al. 2019).

Material	Number	D_r (%)	e	k_T (cm/s)	T (°C)	η_t/η_{20}	k_{20} (cm/s)
Fine steel slag	1	22	0.493	7.839×10^{-3}	28.2	0.833	6.53×10^{-3}
	2	30	0.532	8.182×10^{-3}	30.8	0.781	6.39×10^{-3}
	3	45	0.615	6.376×10^{-3}	29.8	0.798	5.08×10^{-3}
	4	47	0.625	4.610×10^{-3}	28.2	0.833	3.84×10^{-3}
	5	66	0.690	4.034×10^{-3}	28.0	0.833	3.36×10^{-3}
	6	75	0.724	3.821×10^{-3}	29.5	0.806	3.08×10^{-3}
Coarse steel slag	1	9	0.567	8.9×10^{-2}	32.0	0.765	6.81×10^{-2}
	2	33	0.605	7.4×10^{-2}	28.1	0.833	6.16×10^{-2}
	3	47	0.635	7.073×10^{-2}	28.2	0.833	5.89×10^{-2}
	4	53	0.661	4.66×10^{-2}	17.0	1.077	5.02×10^{-2}
	5	60	0.720	3.2×10^{-2}	18.0	1.050	3.36×10^{-2}
	6	69	0.824	3.05×10^{-2}	18.2	1.050	3.20×10^{-2}
Gravel steel slag	1	38	0.422	2.77×10^{-2}	18.0	1.050	2.91×10^{-2}
	2	50	0.482	2.38×10^{-2}	18.8	1.025	2.44×10^{-2}
	3	56	0.510	1.61×10^{-2}	17.0	1.077	1.72×10^{-2}
	4	68	0.592	6.4×10^{-3}	18.0	1.050	6.72×10^{-3}
	5	72	0.620	0.99×10^{-4}	18.8	1.025	1.01×10^{-4}
	6	78	0.715	6.80×10^{-5}	18.0	1.050	7.20×10^{-5}

Generally, slag materials exhibit performance in highway applications that is similar to typical granular soils. Assuming the slag is obtained from one production plant, their characteristics are more consistent relative to other byproducts (e.g., fly ash) since there is tight quality control and quality assurance for the primary iron and steel products from which ferrous slags are derived (Stroup-Gardiner and Wattenberg-Komas 2013c). However, slag properties can vary widely between plants given the differences in technologies employed and source materials. Steel slags tend to be dense and less porous than other slags, which can lead to a high heat capacity. Additionally, steel slags in water can result in high pH levels that can be corrosive to aluminum or galvanized steel products (e.g., pipes, piles, etc.). Tables 1.29 and 1.30 highlight some of the necessary design considerations when using boiler slag and blast furnace slag materials.

Table 1.29. Design and construction guidelines for boiler slag (Schaefer et al. 2017a).

Item	Guideline
Design Parameters	**Dry Density, Loose:** 60 to 78 pcf **Dry Density, Compacted:** 82 to 102 pcf **Optimum Moisture:** 8 to 20% (Stroup-Gardiner and Wattenberg-Komas 2013b) **Angle of Shearing Resistance:** 38° to 42° **Coefficient of Permeability:** 0.3 to 0.9 mm/s **Grain Size Range (Percent Passing):** 90 to 99% on #4, 62 to 89% on #8, 16 to 46% on #16, 4 to 23% on #30, 2 to 12% on #50, 1 to 7% on #100, and 0 to 5% on #200 (Stroup-Gardiner and Wattenberg-Komas 2013b) **Atterberg Limits:** Non-plastic **Compressibility:** Comparable to sand, at same relative density
Environmental Considerations	After 4 days of soaking, the pH of the water solution is generally in the range of 6.7 to 7.0. Barium has been detected by toxicity tests, but at levels well below the EPA specified standard. There are no known environmental concerns with the use of this material.
Design Considerations	The aggregate is durable and satisfies acceptable limits for soundness tests. The aggregate works well as an underdrain filter material, provided the gradation requirements are met. Side slopes should be covered with a minimum of 2 feet of cover material since exposed material has low stability. Specify standard proctor compaction, AASHTO T 99, since some degradation occurs during laboratory compaction in accordance with AASHTO T 180.
Construction Considerations	Compact with several passes of a pneumatic roller or a smooth-drum, vibratory roller. Keep water content at or above optimum water content, as determined by AASHTO T 99. 6 to 10 passes are usually sufficient. Material must be kept wet since there could be a loss in stability when material dries.

Table 1.30. Design and construction guidelines for blast furnace slag (Schaefer et al. 2017a).

Item	Guideline
Design Parameters	**Compacted Moist Density:** 70 to 94 pcf, varies with size and gradation **Gradation:** Can be graded to any specified size from 4 inches down. **Angle of Shearing Resistance:** 35° to 40° **Permeability and Compressibility:** Depends on final specified gradation. Generally similar to gravel and sand.
Environmental Considerations	Slag contains small amounts of sulfur in combined alkaline compounds. The pH of water in contact with slag is generally in the range of 8 to 12, which tends to inhibit corrosion. Some washing of the aggregate may be required to control the pH to 11 or less to meet AASHTO specifications. There are no known environmental concerns.
Design Considerations	The slag behavior is similar to natural angular gravel and sand deposits. The highest internal stability occurs for aggregate that is well graded with a maximum particle size of 16 inches. The amount passing #200 sieve should be limited to 5 to 7 percent. However, the density increases for well-graded materials. If lightweight fill is desirable, uniformly graded materials should be specified. Absorption in slag is usually in the range of 1 to 6 percent by weight. Slag is highly resistant to weathering and abrasion, and can be placed below the water table and next to lakes and rivers.
Construction Considerations	Slag can be placed and compacted in the same manner as natural gravel and sand.

1.4.2.1.4.3 Example Case Histories

Ghionna et al. (1996) the use of steel slag as structural fill material for embankments. They constructed a scaled embankment with mixtures of steel slag and other fill materials. The results from these efforts showed that diluting steel slag as with other fill material (e.g., typical granular aggregates) reduced the risk of swelling potential. According to the plate load test performed, the elastic modulus of the treated embankment was acceptable for their application and equal to 55 MPa.

Malasavage et al. (2012) examined the compaction performance of dredged material (DM) blended with steel slag fines (SSF). Two modified proctor compaction criteria of 92% and 90% were considered for two different DM-SSF blends (20⁄80 and 50⁄50). A trial embankment was modeled constructed on the natural ground that was made of 1 m of compacted clay. The fill materials were blended first, distributed with a bulldozer and then compacted with an 8-ton vibratory roller. The details dimensions of the embankment were rectangular core dimensions of approximately 3.6 m (12 ft) high, 3.6 m (12 ft) wide, 15.2 m (50 ft) long with 3∶1 end slopes and 2∶1 side slopes. Based on CPT testing and triaxial testing of samples of the embankment, the performance of the blended DM-SSF mixture of Dredged Material-Steel Slag Furnace was acceptable with no noticeable swelling or metal leaching issues.

Kataoka et al. (2017) examined the hydraulic and mechanics properties of steel flag combined with fine grained soils. To do so, a full-scale test embankment was constructed from three sections: one control and two sections mixed with slag (Figure 1.33). The mix ratio of steel slag for the two types of mixed materials was 25%. Each embankment dimension was equal to 3.5 m in length, 2 m in height (the thickness of each leveled layer was 25 cm; a total of 8 layers), and 4 m in crown width. The slope gradient was 1:18. The embankment investigation showed that there is some difference between the output from the section made by pure soil with the one mixed with slag. A standard penetration test was used on the sections after the construction finished, and the road opened to large truck traffic. The blow count of the soil section was 10 to 15 six months after construction, while tests on section mixed with slag had blow counts equal to 40. The other factor was evaluating the PH of each section. For this purpose, they constructed ditches at the embankment toe to measure the PH of leakage. The result proved that the leakage of highly alkaline water from inside the embankment could be controlled by sufficient compaction of the slag soil mixture.

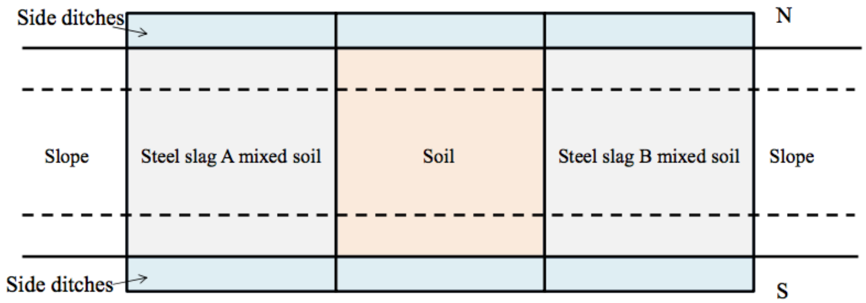


Figure 1.33. Schematic plan view of constructed embankment in Kataoka et al. (2017) case study.

1.4.2.1.4.4 Costs

USGS (2021) provides a good summary of the production and use of iron and steel slag in the United States, which includes some discussion of recent cost information. The following information relies heavily on this reference. Domestic slag sales over the last five years (2016 – 2020) ranged between 14 - 17 million tons, with about equal distribution between blast furnace slag and steel slag in terms of tonnage sold. However, blast furnace slag accounted for close to 90% of the total dollar value of slag (estimated at $380 million in 2020), most of which was granulated. The price per ton for slag byproducts can vary tremendously between a few cents for some steel slags at a few locations to about $120 or more for some ground granulated blast furnace slags (GGBFS). On average, the price of steel slag over the last five years (2016 – 2020) has increased slightly from $22 per ton to $27 per ton FOB. Because of the low unit values, most slag byproducts can be shipped only short distances by truck, but rail and waterborne transportation can increase the shipping distances.

1.4.2.1.5. Foamed Recycled Glass

Foamed recycled glass [or foamed waste glass (FWG)] is an aggregate made from waste glass and has a multi-porous structure with either continuous or discontinuous voids (Figure 1.34). Based on its composition, FWG exhibits a number of favorable material qualities, including light weight, high rigidity, thermal insulation, chemical inertness and nontoxicity, and low water absorption (Bai et al. 2014). The final shape of FWG aggregates is similar to regular gravel or crushed aggregates, so very little has to be changed with respect to handling and compaction in the field. Figure 1.35 presents a summary of FWG usage in a number of different applications (Lu and Onitsuka 2004).

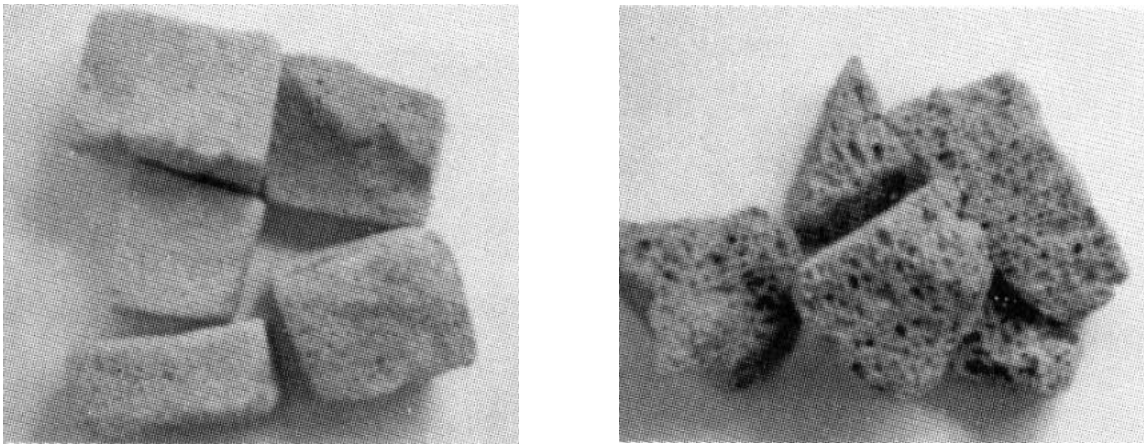


Figure 1.34. Types of foamed waste glass (FWG): (a) discontinuous voids; and (b) continuous voids (Lu and Onitsuka 2004).

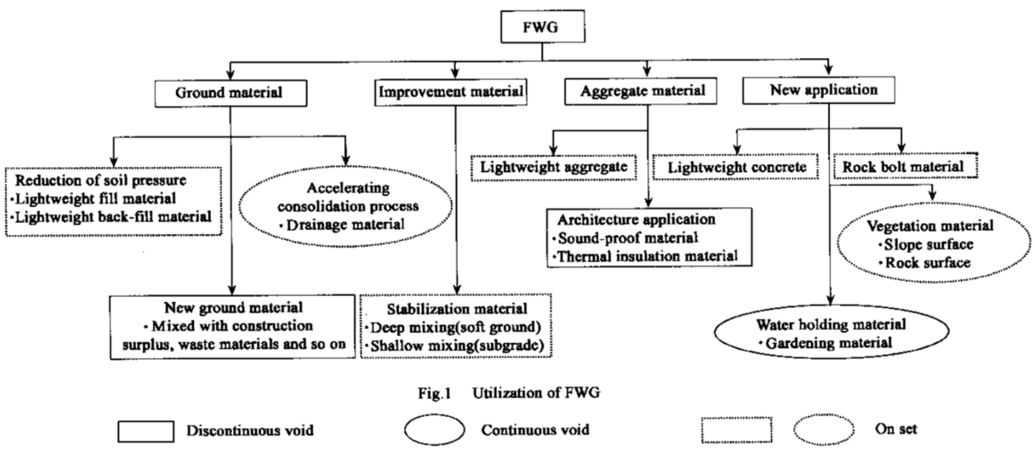


Figure 1.35. Different applications of FWG (Lu and Onitsuka 2004).

1.4.2.1.5.1 Manufacturing Process

The fabrication process begins by crushing the collected glass into powder of approximately 10-100 micrometers size. Afterwards, the glass powder is mixed with a foaming agent in either a wet or dry state and transferred to a furnace to be heated between 700 °C – 1000 °C. During this heating process, the foamed glass powder expands up to five times its initial size and forms a “cake” layer with height of approximately 5 cm – 8 cm. Finally, the foamed glass powder is allowed to cool and harden during which time internal stresses within the mixture result in cracking and separation into aggregate-sized pieces. This process is typically compounded by transport on conveyor belts, loading, and vibration on shipping vehicles (Zegowitz 2010). One of the main differences between the wet-foaming and dry-foaming process is the color of the final product with the wet procedure resulting in black aggregates and the dry procedure in gray aggregates.

1.4.2.1.5.2 Engineering Properties & Design/Construction Considerations

A number of engineering properties have been examined for foamed recycled glass, including particle density, water absorption, CBR, shear strength, and thermal conductivity (e.g., Zegowitz 2010; Arulrajah et al. 2015; Bradette et al. 2019; Lenart and Kaynia 2019; Rieksts et al. 2019).

Arulrajah et al. (2015) performed a series of tests on foamed recycled glass to examine its suitability as a lightweight engineering material. Their results showed that the density of coarse particles of foamed recycle glass was in the range of 4.54-14.79 kN/m^3, which implies that foamed recycled glass can have density lower than water. There was a difference between the performance of the fine particles (2% passing the #200 sieve) relative to the coarse ones. For example, the density of fine particles was three times higher than the coarse ones. The CBR values ranged between 2%-5%. Direct shear testing demonstrated high shear strength relative to typical aggregates with a cohesion of 23.4 kPa and a friction angle of 54.7°. Other studies corroborated these findings with similar values for the engineering properties (Bowles 1988; Sivakugan and Das 2010). Compressive strength testing has also been explored with results showing a range between 300 kPa to 820 kPa at 10 % deformation (Zegowitz 2010).

To examine thermal properties of foamed glass aggregate, Bradette et al. (2019) developed a device with dimension of 100 cm x 100 cm x 100 cm and polyester coverage insulation with a thickness of 15 cm. An innovative approach of downward and upward heat was used to understand the thermal behavior of the material. The heat transfer mechanism included convection, conduction, and radiation. The result shows that equivalent conductivity of 0.18 W/°C.m and an intrinsic permeability of 0.345 x 10^{-6} m^2. In another study, foamed glass was used beneath a section of a road as frost insulation in a location with a mean annual temperature of 0.2 °C. In order to monitor the variation of temperature, ten thermocouples were used at the middle height of each material used to construct the road and at the interface of consecutive layers. Thermal monitoring was performed from 2016 to 2018. The results show that foamed glass aggregate provided excellent insulation from frost (Rieksts et al. 2019).

1.4.2.1.5.3 Example Case Histories

Sato et al. (2002) explored the use of FWG as a lightweight material in Japan in order to stabilize a building foundation and restoration of a failed slope due to heavy rainfall. After construction, slope stability analysis was performed using the Fellenius method and the safety factor was considered as the primary assessment parameter for the FWG. The result shows that FWG was an appropriate choice for both slope and gravity material and could satisfy the demand for stability.

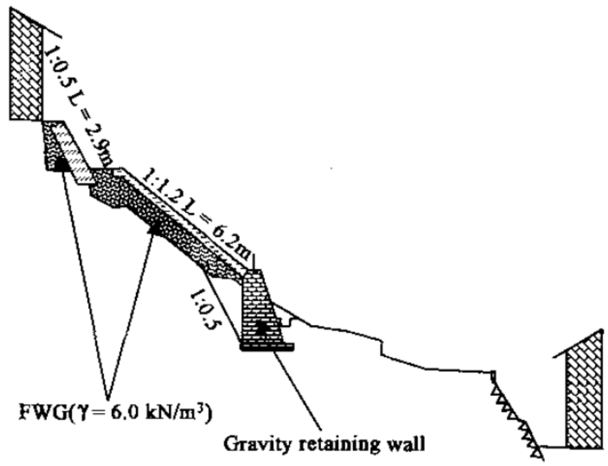


Figure 1.36. Cross section of slope after restoration in Sato et al. (2002) case history.

Hara et al. (1999) explored the use of FWG as a water-holding material to control and avoid slope failures due to heavy rainfall. The results of their testing revealed that the water absorption of FWG with continuous voids is 1.4 times the unit weight so that it would be a good choice as a water-holding material. Figure 1.37 shows the field example from the case history as applied to provide a greening effect for a rock slope in Japan. Hara et al. (1999) found that using 10% FWG is an ideal percentage for greening of the slope.

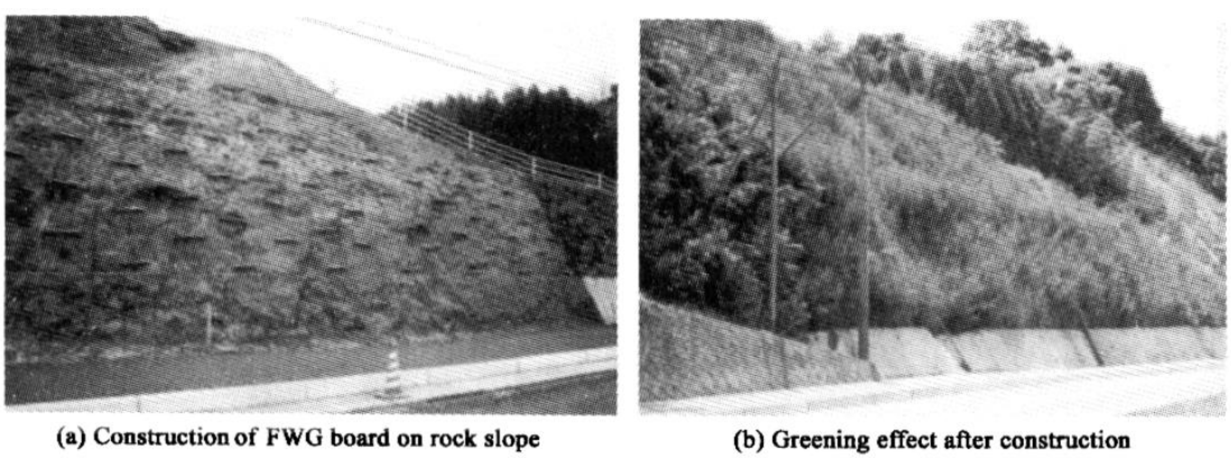


Figure 1.37. Example of FWG greening technique (Hara et al. 1999).

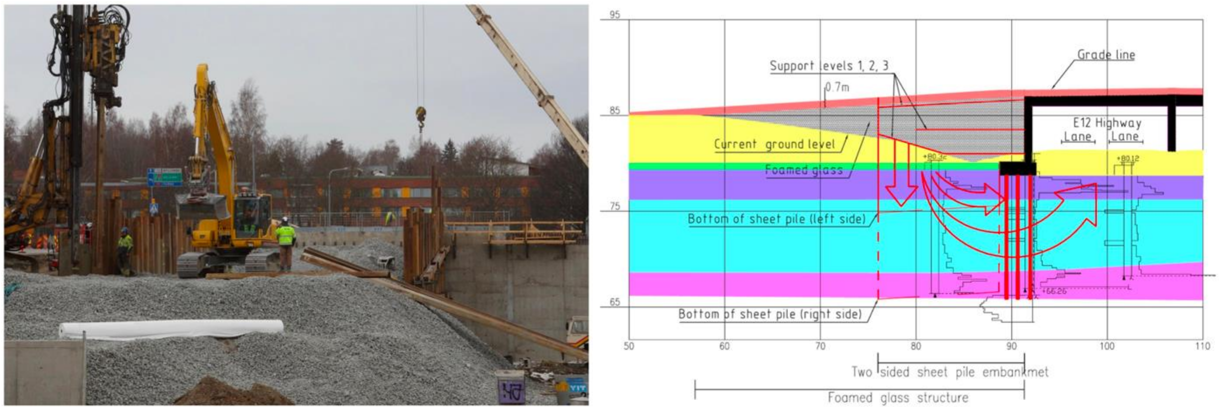


Figure 1.38. Auvinen et al. (2013) FWG case history: (left) longitudinal section of the foamed glass ramp onto concrete shell of tunnel; (Right) foamed glass embankment during construction.

Auvinen et al. (2013) presented information regarding a case history where FWG was used to constructed temporary access ramps for a tunnel project in Hämeenlinna, Finland (Figure 1.38). Foam glass was used in two parts between sheet piles and unsupported sloped embankment with a maximum 3.5 m thick layer. The ramp design was based on a friction angle of 40° and a unit weight of 4 kN/m^2 for the FWG. This resulted in a reduction of embankment loads of 85 kN/m^2 and reduction of earth pressure by approximately 60% compared to typical granular embankment materials.

1.4.2.1.5.4 Costs

Foamyna Canada Incorporation explored the ratio of materials-only cost versus total cost, including the installation to estimate the total cost of foamed glass aggregate. As a result, they published the material-only cost of FWG equals to CAD 50 per cubic meter, while the final cost including installation is typically increased by up to CAD 76.5 per cubic meter. Schneider (2017) performed a comprehensive Life-Cycle Cost Analysis (LCA) comparison between foamed glass aggregate versus EPS geofoams. Table 1.31 presents this comparison based on design equivalent single-axis loads (ESAL).

Table 1.31. LCA comparison: Foam glass LWA versus EPS geofoam (Schneider 2017).

(All values below are in $ CAD.)

Design Artificial Subgrade	LWA	EPS	LWA	EPS	LWA	EPS
Design Lifetime ESALs	1 x 10^6	1 x 10^6	10 x 10^6	10 x 10^6	60 x 10^6	60 x 10^6
Depth Hot Mix Asphalt (mm)	127.0	317.5	190.5	482.6	304.8	711.2
Depth Granular Base (mm)	152.4	152.4	152.4	152.4	152.4	152.4
Depth Granular Subbase (mm)	152.4	152.4	228.6	215.9	393.7	368.3
Depth Lightweight Fill (mm)	5568.2	5377.7	5428.5	5149.1	5149.1	4768.1
Road Length (m)	1000	1000	1000	1000	1000	1000
Road Width (m)	15	15	15	15	15	15
Volume Hot Mix Asphalt (m^3)	1905.0	4762.5	2857.5	7239.0	4572.0	10668.0
Volume Granular Base (m^3)	2286.0	2286.0	2286.0	2286.0	2286.0	2286.0
Volume Granular Subbase (m^3)	2286.0	2286.0	3429.0	3238.5	5905.5	5524.5
Volume Lightweight Fill (m^3)	83523.0	80665.5	81427.5	77236.5	77236.5	71521.5
Cost Hot Mix Asphalt	$492,443	$1,231,106	$738,664	$1,871,282	$1,181,862	$2,757,678
Cost Granular Base	$96,698	$96,698	$96,698	$96,698	$96,698	$96,698
Cost Granular Subbase	$80,582	$80,582	$120,872	$114,157	$208,169	$194,739
Cost Lightweight Fill	$6,387,053	$8,927,109	$6,226,809	$8,547,627	$5,906,321	$7,915,158
TOTAL COST - CONSTRUCTION	**$7,056,775**	**$10,335,494**	**$7,183,043**	**$10,629,764**	**$7,393,049**	**$10,964,273**

1.4.2.1.6. Recycled Plastics

Recent efforts have explored the use of recycled plastics as an engineering fill material. One of the motivations for this development is to prevent landfilling of plastic bottles. The production volume of plastic bottles equal to 1.8 billion kilograms by considering that around 80% of them are shelved and inactive. Plastic material compares favorably to EPS geofoam in terms of weight (e.g., as low as 1% - 2% of soil unit weight) and has some additional advantages, including lower environmental impact, lower cost, and compatibility with petroleum products. However, recycled plastics have a very short history as an engineering fill material in the documented literature.

1.4.2.1.6.1 Manufacturing Process

Waste plastic has many byproducts, but one of the most common is polyethylene terephthalate (PET). However, waste plastic can be produced in other forms: recycled plastic pins (RPP) (Khan et al. 2013; Khan et al. 2014); reused bottles compacted together with adhesive binding agents to form a lightweight material (Graettinger et al. 2005); as an additive in different mixtures such as concrete (Alfahdawi et al. 2016) or soil mixtures (Jin et al. 2019). The process of production begins with collecting and gathering waste plastics materials from different sources. The next step is cutting them into smaller pieces. For this purpose, chipping machines may be used but it may also be possible to manually perform the cutting. Grinding machines can also be employed to then obtain specific ranges in particle sizes after sieving. Chemical methods are also available that use catalysts at a specific temperature and high pressure (Ben Zair et al. 2021).

1.4.2.1.6.2 Engineering Properties & Design/Construction Considerations

Although the use of PET has been limited in civil engineering projects, its compaction, unconfined compressive strength, and hydraulic conductivity properties when combined with soil have been examined by Ojuri and Ozegbe (2016). In that study, PET water bottles were shredded into strip sizes ranging from 0.3 cm × 0.3 cm to 0.7 cm × 0.7 cm. The soil was mixed with PET strips of 0.4%, 0.8%, 1.2%, 1.6%, and 2.0% by weight of soil and 3% cement by same weight of soil. For compaction, modified proctor testing demonstrated that the MDD decreased by increasing PET into the mixture, which pointed to PET acting as a lightweight material in the soil mixtures. For unconfined compressive strength, a Pocket Penetrometer test was used. The results showed that the unconfined compressive strength of the control specimen (i.e., just sand) was 320 kN/m^2, while adding cement increased the strength to 366.7 kN/m^2 (14.6% increase) and adding 2% PET increased it further to 433.3 kN/m^2 (35.4% increase). Finally, Ojuri and Ozegbe (2016) examined the hydraulic conductivity and found that the untreated soil was equal to 7.91×10^{-4} m/s. The samples stabilized by cement and 2% PET exhibited hydraulic conductivity were equal to 7.26×10^{-5} m/s and 1.06×10^{-4} m/s, respectively. The results revealed that the most efficient percentage of PET was 1.6% (Ojuri and Ozegbe 2016).

1.4.2.1.6.3 Example Case Histories

Graettinger et al. (2005) presented a case history in Tuscaloosa, Alabama where compacted plastic bottles were used as structural fill material for a small retaining wall. For this purpose, a polyurethane foam was used as the binding agent to attach recycled plastic bottles to shape blocks. Each block had a volume of 0.227 m^3 (8 ft^3) and was approximately 80% recycled plastic bottles and 20% polyurethane adhesive. The retaining wall design included a flat pad for bikes at the top of the retaining wall and a geotextile layer installed to cover all the surface of the structure and folded back over the material in order to avoid any infiltration of lightweight materials and soil together (Figure 1.39). The results showed that the materials performed well after nine months of observation (Graettinger et al. 2005).

Figure 1.39. Graettinger et al. (2005) case history: (Left) attaching woven geotextile to retaining wall; (Left) laboratory sample of lightweight fill composed of recycled plastics.

Khan et al. (2013) summarized a case study where slop stabilization was performed near highway US 287 in Midlothian, Texas using recycled plastic pins (RPP). In slope stabilization, the RPP are inserted into the slope face to make an additional resistance and increase the factor of safety against sliding failure. The details of the slope and RPP geometry for this case study is presented in Figure 1.40. A total of 429 RPPs were installed in two sections over the course of 4 days. For monitoring the behavior and performance of recycled plastic bottles, Khan et al. (2013) used a high-resolution rain gauge to monitor the daily rainfall and nine RPPs were instrumented with strain gauges. Monitoring after the first year revealed that the control section had undergone larger strain compared to reinforced parts. Additionally, the comparison of the two reinforced sections showed differences in strain due to larger space of pins at the crest in one of the reinforced sections.

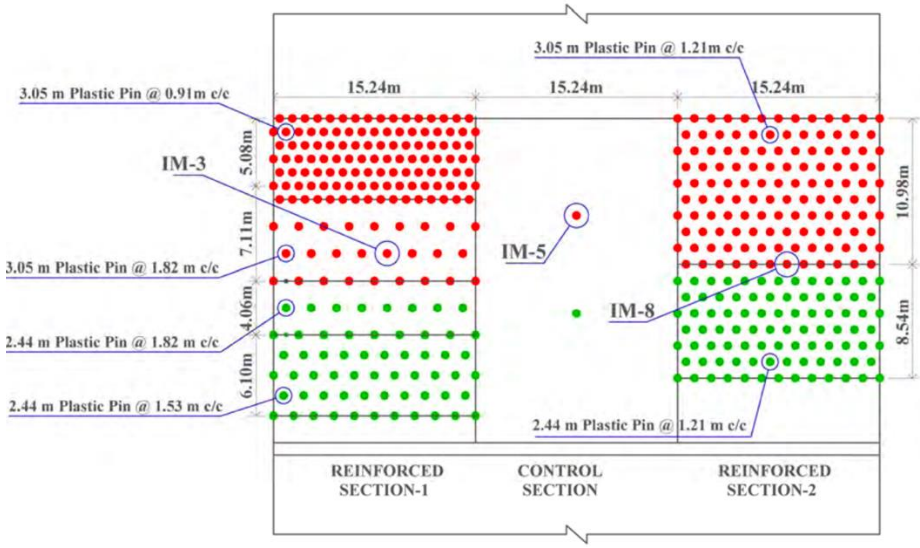


Figure 1.40. Schematic of RPP design at US 287 slope (Khan et al. 2013).

In another case study, Khan et al. (2014) explored the use of RPP on a two-part section of a slope along Highway Loop 12 in Dallas, Texas (Figure 1.41). Due to rainfall, an excessive amount of settlement (30 cm) occurred at the top slope, which resulted in the development of a crack at the footing of the retaining wall. The results from site investigation showed that the soil at the top of the slope was a medium to high plastic clay with high possibility for shrinkage and swelling behavior. Consequently, Khan et al. (2014) developed a finite element model to find the placement of RPP to prevent additional settlement. After the design was selected and construction completed, the sections with smaller spacing between pins had better performance so that the top of the wall displaced more than the bottom of the wall. The results showed that the slope's performance was satisfied with only 0.8 inches at the crest.

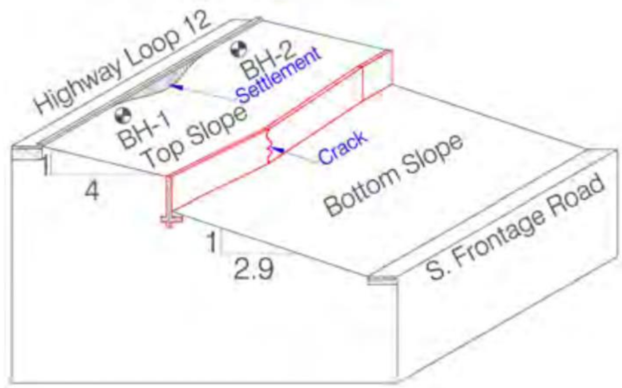


Figure 1.41. Schematic of slope failure in Khan et al. (2014) case history.

1.4.2.1.6.4 Costs

The key aspects related to costs of the PET recycling process include the price of the recycled polymer compared with virgin polymer and the cost of recycling compared with alternative forms of acceptable disposal. As previously noted, the recycling process of PET is made of two main types named mechanical and chemical. Although mechanical process consists of many steps, it is ultimately cheaper, easier, and requires less effort compared to the chemical method. Collection is one of the most significant factors that may affect the recycling price of plastic bottles. Indeed, collection may vary based on locality, type of dwellings (houses or large multi-apartment buildings), and the type of sorting facilities available. The other factor that might affect the price of recycling is the price of petroleum, which is the principle feedstock for plastic production. Its initial price will directly affect the price of bottle production and, subsequently, the recycling process's price. Consider the fact that the price of oil has increased significantly in the last few years, from a range of around USD 25 per barrel to a price band between USD 50–150 since 2005. Also, technological advances can affect the price of recycling directly. This reduction can occur by decreasing the cost of recycling (productivity/efficiency improvements) and/or by closing the gap between the value of recycled resin and virgin resin (Hopewell et al. 2009). Also, mixing PET with concrete can produce cheap polymer concrete resins. Although the price of polymer concrete resin is higher than cement-based materials, because of the high cost of virgin resins, using waste PET bottles recycling combined with production of polyester resin can decreases the cost of resin manufacture when compared to conventional normal resin production (Rebeiz et al. 1991).

1.4.2.2. Non-Lightweight Materials

The following sustainable technologies have similar compacted in-place densities to typical earthen materials used to construct fills. Consequently, they are not considered lightweight and do not exhibit their advantages (e.g., reduced settlements and earth pressure loads). Nevertheless, many of these technologies utilize a stream of waste by-products from the production of other resources and may exhibit cost savings in addition to the reduction in environmental impact.

1.4.2.2.1. Waste Glass

Like many other sustainable building construction materials such as red mud, fly ash, fuel waste, scrap tires, demolition wastes, and blast furnace slag, waste glass was introduced as another sustainable material for use in civil engineering projects. Several studies have investigated the potential applications of waste glass in geotechnical applications, concrete, geopolymers, and other glass-based products.

1.4.2.2.1.1 Manufacturing Process

Waste glass has several common names, such as glass cullet, recycled glass, soda lime glass, crushed glass, or processed glass aggregate. It is recovered from breakages and inferior products made during glass manufacturing (Stroup-Gardiner and Wattenberg-Komas 2013a). For the pre-production process, waste glass collected from the curbside and industrial sectors is transferred to a glass recycling site. The process of waste glass recycling then occurs over four main steps (Figure 1.42).

- At the beginning, every contaminant except glass products such as plastic and paper are removed from the mixture.
- Next step is the crushing process which is manipulated with the aim of crushing equipment.
- Then color sorting is developed by sorting glass pieces in three color categories of white, green, and amber.
- For the last step, another round of debris removal and quality control is done to ensure the clean color sorted glass is ready for use in bottle production industries.

Unfortunately, the color-sorted recovered glass still contains a small percentage of initial waste glass, so a high percentage of this recovered glass cannot be reused to produce bottles due to debris remaining and labels on glass pieces (Disfani et al. 2012). One alternative use for this considerable volume of recovered waste glass has been as a construction material in civil engineering projects.

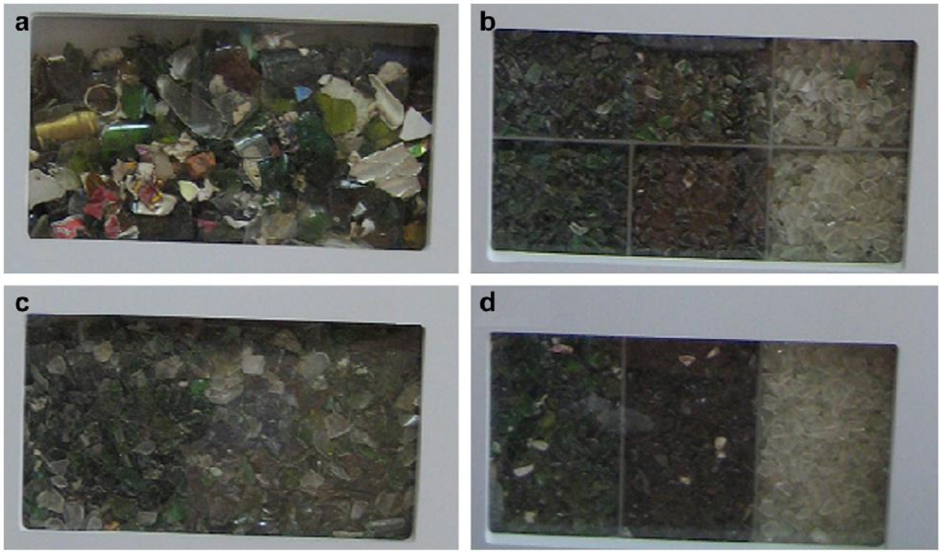


Figure 1.42. Waste glass recovery: (a) collected from curbside; (b) after contaminant removal (c) after color sorting and further crushing; and (d) clean and color-sorted glass (Disfani et al. 2012).

1.4.2.2.1.2 Engineering Properties & Design/Construction Considerations

A number of researchers have explored the engineering properties of waste glass for highway applications (Kenny et al. 1984; Cosentino et al. 1995; Clean Washington Center 1998; FHWA 2008; Wartman et al. 2004; Ooi et al. 2008; Disfani et al. 2011; Disfani et al. 2012). The most comprehensive of the studies available in the literature is the work completed by Wartman et al. (2004). In this study, two different crushed glass aggregate sources were examined. The first crushed glass aggregate was provided by a commercial quarry and aggregate supplier located in southeastern Pennsylvania who used to crush glass to an AASHTO No. 8 gradation or finer. In contrast, the second source was a recycling facility located in eastern Pennsylvania that accepts, processes, and separates commingled waste. The following physical properties were examined for the two sources of waste glass: water and debris content, specific gravity, grain size distribution, minimum and maximum density, USCS and AASHTO classifications, Los Angeles (LA) abrasion, freeze-thaw, sodium sulfate soundness, compaction properties, hydraulic conductivity, consolidation properties, and shear strength. The results for these properties are presented in Table 1.32.

Table 1.32. Summary of Physical and Engineering Properties of Crushed Glass (Wartman et al. 2004)

		Supplier I		Supplier II	
Test/Index	Test standard	As-received	Postcompaction	As-received	Postcompaction
Water content (%)	*ASTM D2216-98*	2.36	—	4.22	—
[range]		[2.03–2.60]		[3.49–5.32]	
Debris content (%)	Gravimetric	0.34	—	1.82	—
[range]		[0.0–0.75]		[0.62–3.41]	
Specific gravity (G_s)	*ASTM C127-88*	2.48	—	2.49	—
Minimum density (g/cm^3)	*ASTM D4254-00*	1.15	—	1.27	—
[range]		[1.14–1.16]		[1.23–1.30]	
Maximum density (g/cm^3)	*ASTM D4253-00*	1.79	—	1.74	—
[range]		[1.77–1.80]		[1.72–1.75]	
Median grain size D_{50} (mm)	—	2.24	1.6	3	2.5
[range]		[1.85–2.62]		[2.70–3.30]	
Coefficient of uniformity, C_u	—	6.2	6.5	7.2	7.8
[range]		[4.3–10.0]		[5.4–7.0]	
Sand content (.075–4.75 mm) (%)	—	91.3	87	70	76
[range]		[89.5–93.0]		[66.5–74.0]	
Fines content (<0.075 mm) (%)	—	3.2	6.2	1.2	2.2
[range]		[0.5–5.0]		[0.2–2.0]	
USCS classification	*ASTM D2487-98*	SW	SW	SW	SW-SM
AASHTO classification	*AASHTO M43-88*	No. 10	No. 10	No. 10	No. 10
LA abrasion (wt. %)	*ASTM C131-96*	24%	—	25%	—
Sodium sulfate soundness (wt. %)	*ASTM C88-99*	6.38%	—	7.1%	—
Hydraulic conductivity[a] (cm/s)	*ASTM D2434-68*	1.61×10^{-4}	—	6.45×10^{-4}	—
[range]		[1.36×10^{-4}–1.85×10^{-4}]		[6.42×10^{-4}–6.64×10^{-4}]	
Modified Proctor	*ASTM D1557-00*				
$\gamma_{d,max}$ (kN/m^3)		18.3	—	17.5	—
w_{opt} (%)		9.7	—	11.2	—
Standard Proctor	*ASTM D698-00*				
$\gamma_{d,max}$ (kN/m^3)		16.8	—	16.6	—
w_{opt} (%)		12.8	—	13.6	—
Direct shear	*ASTM D3080-98*				
Internal friction (deg)					
σ_n (kPa)					
0–60		61–63°	—	59–62°	—
60–120		58–61°	—	55–59°	—
120–200		63–68°	—	47–55°	—
Consolidated drained triaxial[a] internal friction (deg)	USACOE	48°	—	47°	—

The other studies showed similar results for other waste glass sources. For example, specific gravity was typically reported between 2.45–2.52 (Cosentino et al. 1995; Clean Washington Center 1998; FHWA 2008; Wartman et al. 2004; Ooi et al. 2008). Maximum dry density for fine recycled glass (FRG) and medium recycled glass (MRG) were 16.7 and 18.0 kN/m^3, respectively (Disfani et al. 2012) while these values were 10–15% lower compared to natural aggregate within the same soil classification while 16.6–16.8 kN/m^3 and 17.5–18.3 kN/m^3 reported by Wartman et al. (2004) for standard and modified compaction energy respectively. For LA abrasion values, values found in previous research include 24–25% obtained by Wartman et al. (2004) and 27–33% obtained by Ooi et al. (2008). FHWA (2008) determined higher LA values of 30–42% for the recycled glass samples they studied, and also Clean Washington Center (1998) obtained LA abrasion values of

29.9–41.7%. Last but not least, Disfani et al. (2011) reported 24.8% and 25.4% for the LA abrasion values for FRG and MRG, respectively. Both Kenny et al. (1984) and Disfani et al. (2011) reported hydraulic conductivity values for both FRG and MRG. Kenny et al. (1984) reported 2.5×10^{-5} m/s (MRG) and 2×10^{-5} m/s (FRG), while Disfani et al. (2011) reported 2.8×10^{-5} m/s (MRG) and 1.7×10^{-5} m/s (FRG). As shown in Table 1.33, Disfani et al. (2011) reported CBR and direct shear test output values as shear strength properties for both FRG and MRG. CBR and internal friction angle values were approximately the same as previous studies (i.e., CBR = 47-80% and internal friction angle between 47–63°).

Table 1.33. Shear strength parameters of FRG and MRG (Disfani et al. 2011).

Shear strength parameters of FRG and MRG.

Test	FRG	MRG
CBR (%)		
Using standard compaction effort	18–21	31–32
Using modified compaction effort	42–46	73–76
Direct shear test		
ϕ_d (°)		
σ_n (30–120 kPa)	45–47°	52–53°
σ_n (60–240 kPa)	42–43°	50–51°
σ_n (120–480 kPa)	40–41°	–
Triaxial shear test (CD)		
ϕ_{cd} (°)		
σ'_c (30–120 kPa)	40°	42°
σ'_c (60–240 kPa)	38°	41°
σ'_c (120–480 kPa)	35°	41°

1.4.2.2.1.3 Example Case Histories

Grubb et al. (2006) presented a case history to examine the feasibility of using crushed glass (CG) and dredged materials (DM) blends as a structural fill for embankments. Approximately 2,750 m^3 (3,600 yd^3) of crushed glass from the City of Philadelphia curbside-collection program and dredged materials (DM) from Fort Mifflin near the Philadelphia International Airport were blended in order to construct three embankments. The target ratios of dry % by weight for the three embankments were 20/80, 50/50, and 80/20 CG–DM with a blend tolerance of ±5%. The rectangular core dimensions for each embankment was approximately 3.6 m (12 ft) high by 3.6 m (12 ft) wide by 15.2 m (50 ft) long with 3:1 ramps and 2:1 side slopes. The results from the field efforts showed that 20% - 80% CG addition to DM resulted in 1.5 – 5.5 kN/m^3 increases in field dry densities relative to the 100% DM, which were densities not achievable with other DM stabilization techniques such as Portland cement, fly ash, and/or lime (PC/FA/lime) addition. Also, the 20/80, 50/50, and 80/20 CG–DM embankments were characterized by average cone tip resistance on the order of 1.0, 1.5, and 2.0 MPa, respectively (Grubb et al. 2006).

In another field study located on Dalhart Road in Palm Bay, Florida, a 300 ft road section was developed using WG (15% by volume) in addition to reclaimed subgrade as subgrade materials (Cosentino et al. 1995). For the construction procedure, approximately 26 yd^3 of WG were transported to the site. Some field experiments, such as CPT, pressuremeter, in situ densities, and field CBR tests were performed to evaluate the WG performance. The results showed that using WG as a part of fill material provided technical, environmental, and economical benefits when compared to using 100% limerock or cemented base. The technical benefits included:

- A mix of WG with limerock material significantly increased the coefficient of permeability compared to 100% limerock base.
- The elastic modulus of the WG and limerock mix increased very little compared to the values obtained for 100% limerock or cemented base.
- The resilient modulus remained constant despite an increase in the percent of WG content.

1.4.2.2.1.4 Costs

Dames and Moore, Inc. (1993) reported different cost aspects such as WG price in the market, processing glass as an aggregate, and sorting glass for the market, separately. Based on their report, the WG price is approximately \$5 - \$10 per ton, processing glass as an aggregate result in \$7 - \$12 per ton, and sorting glass for the market is \$20 - \$50 per ton (Dames and Moore, Inc. 1993). In terms of transportation, costs for recycled glass range from \$1.00 to \$1.50 per mile, and the glass is worth \$15 to \$50 per ton upon delivery to a recycling center (Heck et al. 1989). Grubb et al. (2006) provided a discussion of costs associated with the case history in Philadelphia. The cost comparison for the three different blends is provided in Table 1.34. The first column describes the costs associated with the pugmilling of CG–DM blends based on the Fort Mifflin demonstration project quantity of approximately 3,058 m^3 (4,000 yd^3). The key difference between columns 2 and 3 relates to how the DM is ultimately acquired [i.e., mining from a containment and disposal facilities (CDF) or unloading from a barge] and processed. Each method varies in terms of initial expenses, but the key is that the total price of CG-DM using a fixed plant is the most economical approach among all the approaches. Although column 2 needs an excessive \$5.23/m^3 (\$4.00/yd^3) for crust management and re-excavation operations, column 3 needs \$12.43/m^3 (\$9.50/yd^3) for using PC/FA/lime to stabilize slurried and/or wet DM directly off-loaded from barges fresh from the actual dredging site instead. The fixed plant would have significantly increased efficiencies and economies of scale over the mobile operation, corresponding to considerable reductions in labor and equipment costs. In contrast, the indirect and miscellaneous expenses would be roughly cut by 50% based on the industry experience (Grubb et al. 2006).

Table 1.34. Cost comparisons for CG-DM blending in the Grubb et al. (2006) case history.

	Field scale CG-DM[a]		Full scale CG-DM[b]		Full scale PC/FA/lime[b]	
	($/m^3)	($/yd^3)	($/m^3)	($/yd^3)	($/m^3)	($/yd^3)
CDF operations	—	—	5.23	4.00	—	—
Mobilization	9.81	7.50	0.65	0.50	1.31	1.00
Equipment	10.46	8.00	3.92	3.00	5.89	4.50
Labor	10.46	8.00	6.54	5.00	7.85	6.00
Indirect/Misc.	2.61	2.00	1.31	1.00	1.31	1.00
Crushed glass	—	—	⟨3.92–6.54⟩	⟨3.00–5.00⟩	—	—
10% PC/FA/lime	—	—	—	—	12.43	9.50
Total	33.24	25.5	11.11–13.73	8.5–10.50	28.79	22.00

[a]Portable pugmill rated at ~1,500 m^3/day (200 t/h) throughput (not operated continuously). Basis: total blending production of ~3,050 m^3 (4,000 yd^3).
[b]Fixed pugmill rated at ~6,000 m^3/day (800 t/h) throughput. Basis: minimum throughput of 382,222 m^3/year (500,000 yd^3/year) in New York City metro area.

1.4.2.2.2. Reclaimed/Recycled Asphalt

Asphalt can be reclaimed and/or recycled in two primary ways: Reclaimed Asphalt Shingles (RAS) and Recycled or Reclaimed Asphalt Pavement (RAP). One of the main applications of RAP/RAS is for reuse in hot-mix asphalt (HMA) applications. However, RAP/RAS is also used directly as a backfill material or as an additive mixture with soils. Additives such as self-cementing fly ash are common for stabilizing RAS for use in structural fill applications (Soleimanbeigi 2012).

1.4.2.2.2.1 Manufacturing Process

RAP is degraded asphalt that is obtained from either milling the surface of old asphalt concrete or removing the full depth of asphalt concrete. Milling just removes the top layer of asphalt, which is around 2 inches in thickness, and the layer beneath layer is used as the base for the next new asphalt layer. Full depth removal, however, uses heavy equipment (e.g., bulldozer) to remove both the asphalt surface and everything on top of the subgrade. Then the removed asphalt is transferred to a facility to be processed with crushing, screening, conveying, and stacking (FHWA 2008). Alternatively, the RAP can be pulverized in place, which is useful when the goals is to reuse these materials directly into the replacement asphalt pavement. In this application, a train operation is used consisting of removing the pavement surface and mixing the reclaimed material by adding some additives (such as virgin aggregate, binder, and/or softening or rejuvenating agents to retrieve the binder quality), and placing and compacting the final mixture in a single pass.

RAS products are produced from two primary sources: (1) extraction of asphalt shingles from roofs during home renovations (called post-consumer asphalt shingle or tear-off shingle); (2) rejected asphalt shingles discarded during the manufacturing process (called manufactured shingle scrap). Asphalt shingle typically consists of 20-35% asphalt cement, 2-15% cellulose felt, 20-38% mineral granule/aggregates, and 8-40% mineral filler/stabilizer. To prepare RAS products, the first step of the process is using a rotary shredder, including two slow-speed blades turning at approximately 50 revolutions per minute. The shingle is subsequently reduced into smaller particles, that are then reduced even further to a nominal size of about 9.5 mm (3/8 in) or finer using a high-speed hammer mill operating at about 800 to 900 revolutions per minute (FHWA 2008).

1.4.2.2.2.2 Engineering Properties & Design/Construction Considerations

Since the origins and materials comprising RAP and RAS are similar, the engineering properties that have been studied for them are also similar. Initially, this section will focus on RAP and then discuss engineering properties of RAS. Much effort exists in the literature to characterize RAP compaction behavior, shear strength, corrosivity, hydraulic conductivity, and CBR (Petrarca and Galdiero 1984; Senior et al. 1994; Bennert and Maher 2005; Cleary 2005; Dikova 2006; Rathje et al. 2006; Guthrie et al. 2007; Stolle et al. 2014). To clarify RAP's compaction behavior, Texas DOT recommended that because the RAP maximum particles are larger than 7/8 inch it is better to use a larger mold, larger hammer, and drop weight (Rathje et al. 2006). Stolle et al. (2014) showed that by adding even up to 50% RAP content, there is negligible difference in the compacted dry unit weight, and it is around 18 kN/m^3 - 21 kN/m^3. To investigate shear behavior, past studies on the shear properties of RAP reveal that these materials have high friction angles with little to no cohesion (Petrarca and Galdiero 1984). Bennert and Maher (2005) showed that the CBR value of asphalt base mixtures including RAP will decrease up to 50 % by adding RAP content up to 25% compared with the control specimen. Guthrie et al. (2007) revealed that the CBR values are directly related to compaction effort and the constituent of the mixture. They claimed that CBR decreased between 13 and 29% by increasing the RAP content to 25%. Furthermore, Stolle et al. (2014) acclaimed previous results by claiming the CBR will decrease from around 100 at control to around 20 by adding RAP content to around 50. Corrosivity and hydraulic conductivity are important parameter for characterizing the suitability of backfill materials behind MSE walls. Corrosivity is evaluated base on the pH and resistivity of the material. Based on the results of Rathje et al. (2006), they figured out the pH of RAP as a backfill material was around 7.8 to 9. Moreover, the amount of resistivity was around 5000 Ω-cm. The hydraulic conductivity was investigated using ASTM D5084, and the results showed that RAP performed similarly to conventional fill materials with k ranging between 38.4×10^{-4} cm/s to 5.5×10^{-4} cm/s.

The general design considerations of RAP as fill material in geotechnical construction are generally the same as conventional fill materials. One of the important things is that all the particles should be crushed. Additionally, it is recommended to avoid submersion of RAP in water.

As noted previously, RAS materials are similar in composition to RAP, and compaction behavior, hydraulic conductivity, compressibility, and shear strength have been explored for RAS in a manner similar to RAP (Holtz et al. 1981; Soleimanbeigi et al. 2011; Soleimanbeigi et al. 2014). A study by Soleimanbeigi (2012) on the compaction behavior of RAS, and stabilized RAS with fly ash, showed that the maximum dry unit weight was equal to 11.3 and 15.9 kN/m^3 respectively compared to typical compacted sandy soils, which typically ranges between 17 and 20 kN/m^3. This comparison showed that the addition of RAS resulted in lower compacted dry unit weights than conventional fill materials. Soleimanbeigi (2012) also found that the hydraulic conductivities of RAS and stabilized RAS varied between 2×10^{-4} cm/s and 9×10^{-6} cm/s depending on confining pressure, which compares favorably to very fine sand, silty sand, and silty clay soil. Soleimanbeigi et al. (2014) compared the hydraulic conductivity of RAP and RAS under similar conditions and found that RAS has lower permeability (i.e., 8.6×10^{-3} cm/s for RAP versus 2.3×10^{-4} cm/s for RAS). Also, Soleimanbeigi et al. (2011) showed that adding 10% - 20% fly ash to RAS will decrease the hydraulic conductivity significantly. Soleimanbeigi et al. (2014) explored the compressibility of RAS and found that the axial strain on RAS at a typical embankment load of 200 kPa was approximately 17.5%, while for sand this axial strain would be much lower at approximately 2%. However, Soleimanbeigi et al. (2014) found that by adding bottom ash as a portion of the mixture with RAS the vertical strain was reduced to approximately 5%. In terms of shear strength behavior, Soleimanbeigi et al. (2014) found that the friction angle of RAS was approximately 33°. Alternatively, Soleimanbeigi (2012) found it to be around 36°, but adding 20% of fly ash content reduced it down to 30°. However, the reduction in friction angle was counteracted by an increase in the cohesion term from 24 kPa to 105 kPa.

As a design consideration, the Missouri Department of Transportation (MoDOT) claimed that one of the most important considerations for RAS is mixture uniformity. One of the solutions was specifying the gradation as per Table 1.35, which is finer. Another issue is the presence of deleterious materials in RAS. The limit range of deleterious material varies according to agency, with 3% cited by Schroer (2008), but as low as 1.5% for Texas Department of Transportation (TxDOT) requirements. Another consideration is the heat caused by exposure to sun and the RAS particles sticking to one another. It has been recommended to cover the stockpile before transporting or alternatively RAS can be mixed with a certain percentage of RAP to preserve its performance (West and Willis 2014).

Table 1.35. MoDOT RAS gradation requirements (West and Willis 2014).

Sieve	⅜"	#4	#8	#16	#30	#50	#100	#200
Percent Passing (%)	100	95	85	70	50	45	35	25

1.4.2.2.2.3 Example Case Histories

The overwhelming number of documented case studies with RAP/RAS for highway applications have focused on their use for HMA pavements (e.g., Li et al. 2008). In fact, no literature existed that documented the construction of a full-scale retaining wall or embankment with RAP/RAS and only references to feasibility studies were available. For example, to show a practical application of RAP in civil engineering projects, Basha et al. (2016) explored the possibility of three different failure mechanisms to obtain the optimum dimensions of a narrow backfill width gravity retaining walls by considering the effect of backfill material. They investigated this subject based on the ratio of gravity wall width (B_w) to height (h) as in Figure 1.43. The results showed that if RAP was used for the backfill, a typical friction angle of 44° would result in a B_w / h ratio in the range of 0.42 to 0.44 when the ratio of backfill width to height (b / h) increases from 0.1 to 0.8.

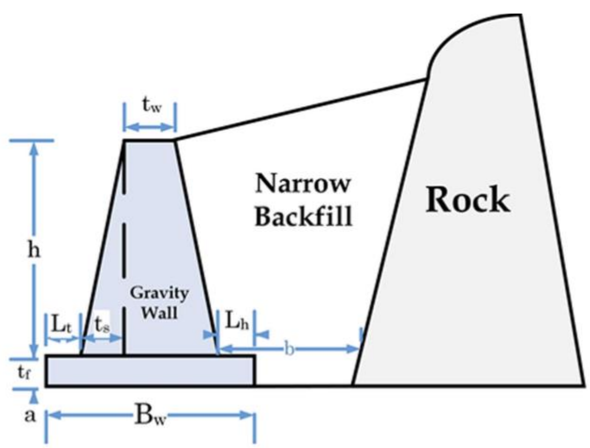


Figure 1.43. Schematic of narrow backfill gravity wall with various parameters for the Basha et al. (2016) case history.

In terms of examining the compressibility of RAS in embankment applications, Soleimanbeigi (2012) performed settlement calculations using typical RAS properties for embankment dimensions of 12 m wide at the top and 2, 5, 10, and 15 m high constructed on a 10 m thick sand deposit. The results show that the settlement caused by RAS structural fill was approximately 1025 mm, which is abnormally high and above tolerable limits due to the high compressibility of RAS. However, stabilizing RAS with 10% fly ash reduced the total embankment settlement to 300 mm.

and increasing the fly ash content even further to 20% reduced the total settlement to negligible levels after a 40-year lifetime.

1.4.2.2.2.4 Costs

Golestani (2015) performed a life cycle cost analysis (LCCA) for incorporating RAS in to HMA in terms of both initial construction cost and 20 years' maintenance period (Table 1.36). Although this analysis was developed for pavement applications, many parts of the analysis are similar for other potential RAS applications, including for use in structural fills. The analysis was based on 2014 prices and used averages between the different states.

Table 1.36. LCCA for RAS in HMA applications based on Golestani (2015).

LCC Components	Unit	Value
Virgin Aggregate	$ / ton	$50
Asphalt Binder	$/ ton	$505 - $697
Trucking	$/ ton / mile	$0.13
Tipping Fee	$ /ton	$24.3 - $91
Shingle Grinding	$ /ton	$14.80
Asphalt Inflation Rate	% / year	%1.1
Trucking Distance [Mine to Plant]	Miles	30
Trucking Distance [Refinery to Plant]	Miles	50
Trucking Distance [Plant to Site]	Miles	10

1.4.2.2.3. Recycled Concrete Aggregates

Recycled concrete aggregates (RCA) are obtained from demolished concrete sources such as rigid pavements (e.g., airport runways) and bridge structures (Rathje et al. 2006). They are produced in a similar manner to natural aggregates with one major difference being the necessary removal of any attached steel or reinforcement. The main application for RCA is reuse in concrete production or as a backfill in earthwork construction such as retaining walls (Gonzalez and Moo-Young 2004; Rathje et al. 2006).

1.4.2.2.3.1 Manufacturing Process

RCA can also be referred to as crushed concrete based on how it is obtained from previous concrete sources. The primary source of crushed concrete is demolished Portland cement concrete used for roadway, airport runway, or concrete structures. The processing of RCA includes its transport to a central processing plant where different steps such as crushing, screening, and ferrous metal recovery are developed. Typically, primary and secondary crushers are used to crush the RCA. In the primary crushers, the particles will be broken down into approximately 3 to 4 in pieces. Before transferring the particles from primary to secondary crushers, any reinforcing steel material will be removed via an electromagnetic separator. Then the secondary crusher further breaks down the particles to the desired gradation. Finally, the gradation will be screened into two coarse and fine gradations. Also, the stockpiling of crushed concrete is done through the screening of the gradations to prevent inadvertent commingling of materials (Rathje et al. 2006).

1.4.2.2.3.2 Engineering Properties & Design/Construction Considerations

A number of studies have explored the engineering properties of RCA, including dry unit weight, shear strength, compaction, drainage capacity, and resilient modulus (Park 2003; Rathje et al. 2006; Bozyurt et al. 2012; Nokkaew et al. 2012; Arulrajah et al. 2013; Edil et al. 2012; Soleimanbeigi et al. 2016). The dry unit weight and gradation of RCA materials can vary tremendously due to their source and the attached particles of mortar to them. The specific gravity of mortar is naturally quite low naturally and as the amount of mortar in the RCA increases, the dry unit weight tends to decrease (De Juan and Gutiérrez 2009; Edil et al. 2012).

Soleimanbeigi et al. (2016) examined the shear behavior based on triaxial testing, which involved the development of a large scale triaxial chamber capable of consolidated drained (CD) testing. During the test, the failure was specified as either 15% axial strain or the maximum deviator stress (σ_{max}), which ever came first. The results of the CD triaxial testing yielded a cohesion of 78 kPa and friction angle of 48°. These results compared favorably with published values in the literature (i.e., 41°-65° for friction angle and up to 55 kPa for cohesion). Additionally, Soleimanbeigi et al. (2016) studied the shear behavior of RCA using large scale direct shear and gyratory shear tests. In this manner, the interface friction angle (δ) between RCA and geosynthetic reinforcement and the efficiency factor ($E\varphi$) could also be evaluated (Table 1.37). Similarly, Arulrajah et al. (2013) also performed large scale direct shear testing to determine the interface shear strength properties of unreinforced and reinforced (Geogrid) construction and demolished (C&D) waste materials. Based on their results in Table 1.38, the best performance was related to geogrid-reinforced RCA. At the same time, the unreinforced RCA had better shear strength properties compared to crushed brick (CB) and RAP.

Table 1.37. Interface friction angles and efficiency factors for different interface materials (Soleimanbeigi et al 2016).

Interface type	Source	Interface friction angle (δ)	Efficiency factor (E_φ)
RCA-Woven geotextile	**This study**	**26.0**	**0.41**
Gravel	Hsieh et al. (2011)	26.9	0.64
Crushed Stone	Hsieh et al. (2011)	30.6	0.41
Sand	Hsieh et al. (2011)	35.3	0.92
FDS	Goodhue et al. (2001)	29.0	0.26-0.32
RCA-Nonwoven geotextile	**This study**	**18.4**	**0.30**
RCA-Uniaxial geogrid	**This study**	**35.8**	**0.62**
Gravel	Nejad et al. (2012)	44.4	0.95
Sand	Nejad et al. (2012)	33.8	0.97
FDS	Goodhue et al. (2001)	26.0-31.0	0.80-0.60
RCA-Biaxial geogrid	**This study**	**31.5**	**0.55**
RCA	Arulrajah et al. (2013)	50.0	0.55
Gravel	Hsieh et al. (2011)	38.7	1.01
Crushed Stone	Hsieh et al. (2011)	43.4	0.66
Sand	Hsieh et al. (2011)	37.0	0.98

Table 1.38. Shear strength properties of unreinforced and geogrid-reinforced C&D materials obtained by Arulrajah et al. (2013).

Material	Cohesion (kPa)	Friction angle (°)
RCA	95	65
RCA + biaxial geogrid	75	50
RCA + triaxial geogrid	83	52
CB	87	57
CB + biaxial geogrid	67	45
CB + triaxial geogrid	80	49
RAP	15	45
RAP + biaxial geogrid	6.5	40
RAP + triaxial geogrid	13	42
Typical construction materials—dense sands and gravels	–	40–48

Edil et al. (2012) examined the compaction behavior of RCA and found that the range of maximum dry unit weight varies narrowly between 19.4 to 20.9 kN/m^3 while the optimum moisture content range was 8.7 to 11.8%, which compares favorably to other recycled alternatives such as RAP. The primary reason mentioned for this performance was the higher absorption capacity because of the porous mortars in RCA (Edil et al. 2012). Relatedly, RCA naturally has high drainage capacity but retains more moisture content due to its hydrophilic cement mortar (Nokkaew et al. 2012). Finally, resilient modulus was studied by Bozyurt et al. (2012) and Nokkaew et al. (2012).

Experiments on seven samples of RCA at optimum moisture content and 95% relative compaction based on modified proctor testing revealed that RCA has a higher range of resilient modulus (163 to 208 MPa) than crushed natural aggregate (152 MPa).

With respect to specific design and construction consideration of this material, some points should be mentioned. One of the main components of RCA is cement paste, which bonds to aggregates. Since cement has high alkalinity properties, the high alkalinity of calcium hydroxide of cement may increase the pH of RCA further than 11, which is above the limit range typically prescribed for backfills (Bruinsma et al. 1997; Kuo et al. 2002; Rathje et al. 2006). Another issue that comes into existence is Tufa ($CaCO_3$). Because of the presence of lime and portlandite [$Ca(OH_2)$] in RCA, Tufa might cause detrimental damages to the drainage system by holding and accumulating water behind the face of the wall (Rathje et al. 2006).

1.4.2.2.3.3 Example Case Histories

Although several experimental studies have conducted to evaluate RCA's properties and their applicability in the field, only a few studies have been documented in the literature that used RCA in real projects. Kawalec et al. (2017) summarizes two case histories where RCA was used on full scale civil engineering construction projects. For the first, a bridge located at Lake Borgne east of New Orleans was demolished after incurring significant damage during Hurricane Katrina. The concrete from this bridge was repurposed for RCA that was used to construct geo-grid marine mattresses along the nearby coast for erosion control. Over 84,000 yd^3 of concrete debris was recycled from the demolition of the bridge for use as fill material in approximately 7,500 marine mattresses. The materials were crushed and screened on site, but the mattresses themselves were filled offsite and barged to the shoreline due to the restriction against construction equipment on the shoreline. Long term observations reported no problems with the marine mattresses nor erosion in the area.

Kawalec et al. (2017) also summarized the successful implementation of RCA as a full material for a new railway line in western Poland. A large component of that project included the construction of an embankment over weak soils. Consequently, the Dynamic Replacement method was proposed whereby granular fill material would be introduced into the subsurface with a mechanical impact caused by dropping a heavy barrel-shaped rammer from as high as 25 m (Figure 1.44). Initially, the rammer is dropped to form a crater that is then backfilled with the granular material, Subsequent blows with the rammer drive the fill material to the desired depth and the process is repeated to form a compact column to the final desired depth. For the case history in Kawalec et al. (2017), the material used for the Dynamic Replacement operation was a mixture of medium and large grain crushed concrete with natural sand. Quality control efforts during construction by means of excavation confirmed the ability to construct the columns with a consistent average diameter of 2.4 m near the top and 1.8 m near the bottom of the column. Only 1 cm of settlement was observed at throughout the site after monitoring for three months post-construction.

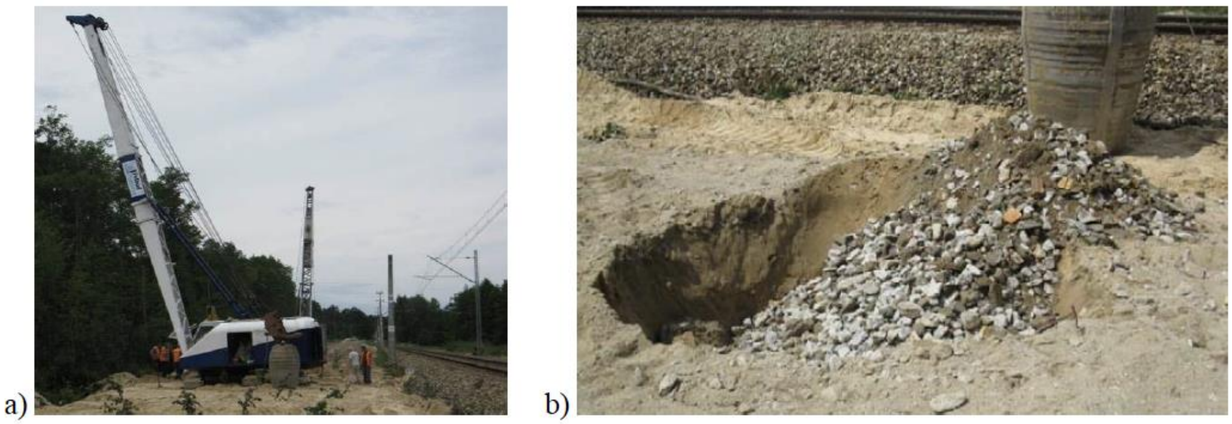


Figure 1.44. The dynamic replacement method: (a) barrel-like rammer attached to crane; and (b) column material (crushed concrete with sand) (Kawalec et al. 2017).

1.4.2.2.3.4 Costs

Donalson et al. (2011) explored the sustainability of RCA for highway applications, which included an investigation of costs. Donalson et al. (2011) divided the economic impact of RCA into three main categories: (1) cost of material; (2) cost of installation; and (3) life-cycle cost. Table 1.39 presents the findings and compares the costs associated with RCA with virgin limestone aggregate as might be employed more typically in highway fill applications. Overall, RCA was found to compare favorably to virgin limestone aggregate in terms of costs.

Table 1.39. Cost comparison between virgin limerock aggregate and RCA in highway construction (Donalson et al. 2011).

	Recycled Concreted Aggregate ($USD per ton of aggregate)	Virgin Limerock Aggregate ($USD per ton of aggregate)
COST OF MATERIAL		
Delivered Price (DP)	12.75	13.20
Price of Raw Material, F.O.B. (RM)	0.00	2.00
Cost of Processing the Material (PR)	3.00	3.00
Cost of Stockpiling the Material (ST)	0.50	0.50
Cost of Loading the Material (LD)	0.50	0.50
Cost of Transporting the Material (TR)	2.75	6.00
Profit (P)	6.00	1.20
COST OF INSTALLATION		
Cost for Design of Application with Material (DR)	0.00	0.00
Cost for Construction with Material (CC)	3.75	4.50
Cost of Testing and Inspection for Proposed Application (RP)	0.70	1.00
Sub-Total Cost of Installation (CI)	4.45	5.50
LIFE-CYCLE COST		
Annual Effective Cost (EC)	1.66	1.82
Cost of Installation (CI)	4.45	5.50
Capital Revovery Factor (CRF)	0.15	0.15
Annual Maintenance Cost (AM)	1.00	1.00

1.4.2.2.4. Waste Foundry Sands

Waste foundry sand (WFS) is a uniformly graded sand byproduct that is produced in the metal casting industry. It is typically used as a molding material to either form the external shape of a cast metal part or as a core material to fill the void space in similar applications. There are approximately 2,300 foundries across the United States that produce WFS, with approximately 28% of their spent foundry sand sent to beneficial reuse programs (Stroup-Gardiner and Wattenberg-Komas 2013d). Some of this spent foundry sand is used in highway applications, including as a structural fill or granular base for roadways.

1.4.2.2.4.1 Manufacturing Process

The WFS manufacturing process broadly consists of three stages (Figure 1.45). First, a casting is produced by pouring molten metal into molds usually made of molding sand and core sand. After the cast is hardened, it is separated from the molding and core materials through the shakeout process. Finally, the castings are cleaned, inspected, and then shipped for delivery, leaving behind any molding and core sand (Javed 1994). In general, there are three types of WFS based on molding processes: (1) green sand molding; (2) chemically bonded process; and (3) shell molding process. Green sand molding is commonly made of four major components: sand (85 - 95%), some form of clay in order to act as a binder for the green sand and provides strength and plasticity (4 -

10%), combustible additives like seacoal, cereal, fuel oil, and wood flour (2- 10%), and water (2-5%). Chemically bonded sands are those that use furan, phenolic urethane, and acid-cured nobake systems, as well as alkyd and phenolic urethane cold box processes. Shell molding is a mixture of sand and thermosetting resin (usually phenolformaldehyde). When these materials are exposed to a heated pattern, a thin shell is made due to the resin's polymerization, which binds the sand particles.

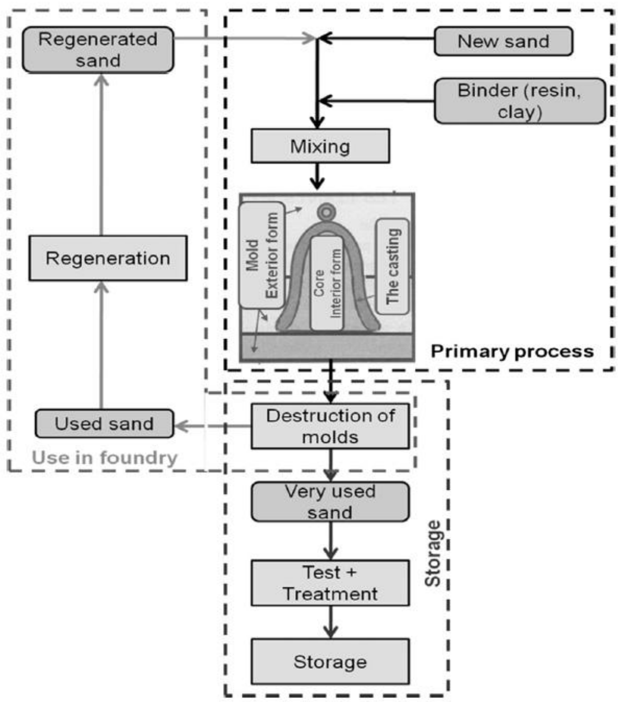


Figure 1.45. Production process of foundry sand: primary use, reuse in foundry process, and storage (Yazoghli-Marzouk et al. 2014).

1.4.2.2.4.2 Engineering Properties & Design/Construction Considerations

Since WFS is a type of granular fill material, a number of research studies have explored the engineering properties useful for characterizing a granular fill material, including compaction, shear strength, and CBR (e.g., Javed and Lovell 1995; Naik and Singh 1997; Partridge and Alleman 1998; Lee et al. 2001b; Dingrando et al. 2004; Deng and Tikalsky 2008). Partridge and Alleman (1998) reported some engineering properties for weathered and fresh WFS based on different ASTM standards (Table 1.40). Overall, the performance of WFS was similar to other sands in terms of CBR, hydraulic conductivity, and compaction characteristics. However, the WFS exhibited cohesion when tested in direct shear and its specific gravity was generally lower than typical sands.

Table 1.40. Engineering properties of WFS based on Partridge and Alleman (1998).

Test		Weathered WFS	Fresh WFS
Direct shear (AASHTO)	Cohesive intercept	83.4 pcf (13.1 kN/m^2)	96.8 pcf (15.2 kN/m^2)
	Friction angle	38°	39°
CBR (AASHTO-193)	CBR	16.8	6.2
Hydraulic conductivity (ASTM D1883, D5084)	Falling head, fixed wall	4.6 x 10^{-6} ft/s (1.4 x 10^{-6} m/s)	5.6 x 10^{-6} ft/s (1.8 x 10^{-7} m/s)
Liquid limit (ASTM D4318)		30.7%	--
Plastic limit (ASTM D4318)		24.7%	NP
Specific gravity (ASTM D854)		2.53	2.46
Percentage of coarse particles (ASTM D422)		78–90%	60%
Percentage of fines (Passing No. 200 sieve)		10–22%	40%
Percentage of clay size particles (<0.005 mm) (ASTM D422)		9%	--
Standard Proctor compaction Method B (ASTM D698)	Optimum moisture content	12.8	27.1
	Maximum dry unit weight (kN/m^3)	18.2	--

Javed and Lovell (1995) specifically examined the differences between waste green sands and raw sands, which includes casting sands. Figure 1.46 presents their results for the compaction characteristics and the California Bearing Ratio (CBR) as a function of moisture content. The results show that the raw sand density varies less with moisture content when compared to the waste greensand. Javed and Lovell (1995) also explored the shear strength performance for five different combinations of WFS with other materials (Table 1.41) at two different relative density ranges (i.e., loose versus dense).

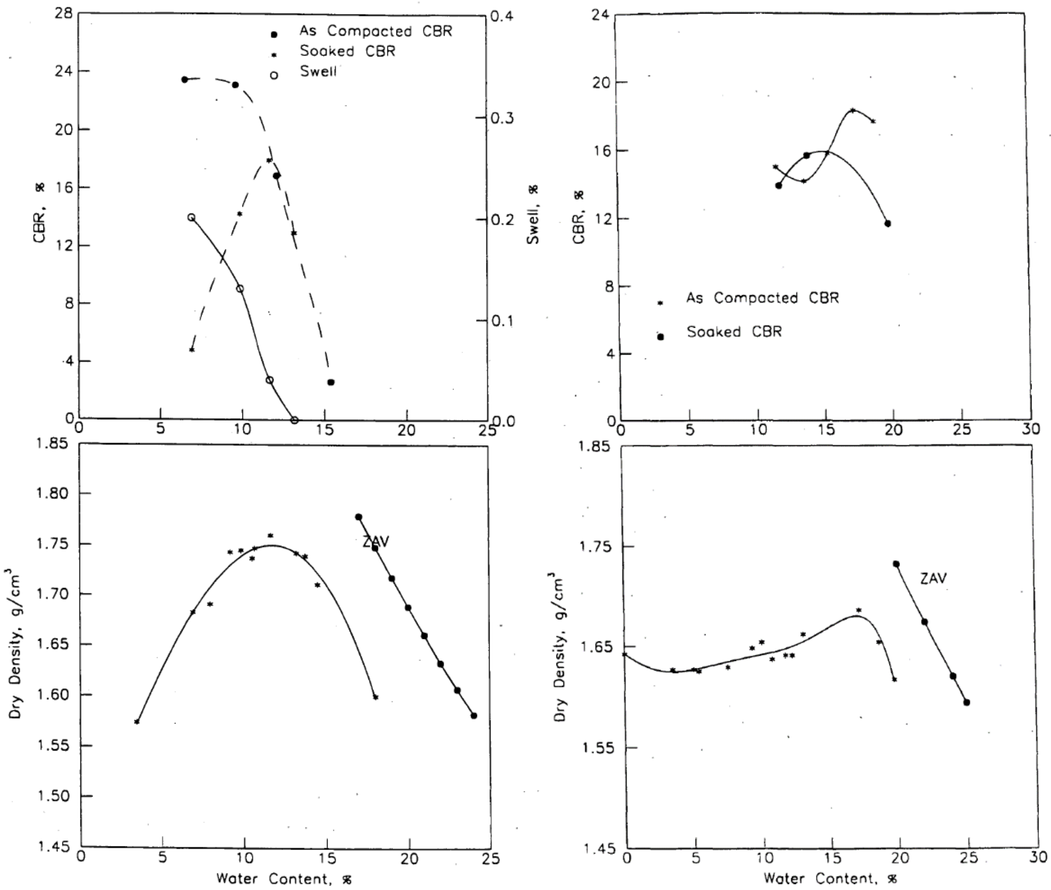


Figure 1.46. Variation of CBR and dry density with moisture content for greensand (Left) and raw sand (Right) (Javed and Lovell 1995).

Table 1.41. Results of direct shear testing of WFS (Javed and Lovell 1995).

Sample #	D_r	Loose		D_r	Dense	
		c (kPa)	ø (deg)		c (kPa)	ø (deg)
G1	29	4.13	32.4	90	9.92	36.6
G3	34	5.17	34.2	98	12.54	40.9
C2	22	0.41	30.4	94	7.17	34.9
S1	31	0.41	30.8	94	4.75	36.5
R1	32	0.21	30.4	88	1.17	33.8
Uniform medium sand[a]	Moderately dense		32-34	Very dense		35-38
Sand[b]	Loose		29-30	Dense		36-41

1.0 psi = 6.89 kPa
[a] (*4*)
[b] (*5*)

WFS has some unique design considerations relative to other granular fill materials. For example, since WFS can exhibit lower density than other sands, the fine particles may be moved easily by the wind, so a lower percentage of fine particles may be observed on top of any constructed embankment. Dusting and foreign sources of sand particles could also cause some changes in the characteristics of the mixture. Finally, the engineering performance of WFS is variable with respect to the stockpile from which it is obtained (Siddiki et al. 2004).

1.4.2.2.4.3 Example Case Histories

Several US states have reported using WFS in construction, including Michigan, Wisconsin, and Ohio. Lovejoy et al. (1996) summarized efforts by Wisconsin to conduct a study to evaluate WFS as a highway embankment material at two test sites (Waupaca and Appleton). Two embankments were constructed on each site, the first with a typical construction soil and the other with WFS. Lysimeters were installed to evaluate the collection of seepage water and handle groundwater monitoring. Results showed that the lysimeters did not collect nearly as much effluent from within the WFS embankments. It means that WFS has low hydraulic conductivity and may cause some excess pore pressure in a WFS embankment and subsequently cause lateral earth pressure behind retaining walls.

Partridge et al. (1999) summarized a case history where an embankment with a slope of 3H:1V was constructed with WFS at County Road 206, just south of US-6, near Butler, Indiana. The embankment was comprised of three different sections. The southernmost subsection was built with clayey soil from a local borrow pit. The center subsection [approximately 374 ft (114 m) long, 280 ft (85.3 m) wide, and 30 ft (9.1 m) high] was built with WFS from the Auburn Foundry. The northernmost subsection was built with natural sand. For this project, a volume of 56,000 yd^3 (42,815 kN/m^3) of WFS was used. During construction, problems related to dusting and foreign objects became a challenge, which was addressed by a magnetic scraper and spraying water. The performance of the WFS was comparable to the natural sand section with very little internal deformations observed and high SPT blow counts recorded after compaction.

1.4.2.2.4.4 Costs

Patridge et al. (1999) provided detailed information about the costs associated with WFS for the County Route 206 demonstration project in Indiana. Auburn Foundry did not charge any costs associated with the WFS material, but the contractor did pay $0.61/$m^3$ for compaction of the WFS. The costs of material and compaction was $8.27/$m^3$ and $3.92/$m^3$ for the B-borrow and clay borrow sections, respectively. For this project, 42,800 m^3 of WFS cost $26,117 to compact, while the same volume of clay borrow on this project would have cost $167,835, including the cost of material, trucking, and compaction. Up to here the difference expenses between WFS embankment and clay borrow shows an 84 % reduction price. However, it should be considered that Auburn foundry spend approximately $450,000 ($10.52/m^3) for trucking the WFS to the job site. The total costs including original cost of siting, constructing and current maintenance costs would be $639,656 ($14.94/m3) to reuse the gained monofill space. If the $450,000 trucking costs is subtracted, then the foundry's net savings (based on monofill disposal costs at the time of the project) was $189,656. The cost of a new monofill can be as much as $1,407,000. Again, by subtracting $450,000 in trucking costs, the net savings would be $957,000. By comparing $189,656 of reused monofill with $957,000 of new monofill, the potential savings available to foundries from the beneficial reuse of WFS was quite clear (Partridge et al. 1999).

1.4.3. Summary

The sustainable fill technologies presented in this chapter represent a broad range of materials. Some compare quite favorable in terms of engineering properties to natural granular materials typically used in structural fill applications. Others require either special considerations or construction practices to maximize performance in highway applications. Regardless, all of these technologies represent an opportunity to increase the sustainability of highway construction and recycle or repurpose materials that would otherwise be sent to landfills. Additionally, as noted in the discussions of costs, some of these materials presented a distinct advantage in terms of project costs in addition to improved sustainability. Any costs savings is often dependent on the specifics of a particular project, particularly the ease with which the material can be sourced and the transportation costs associated with shipping to the project site. The case histories further reinforced the degree with which sustainable fill materials can prove to be cost effective depending on the specifics of a project and the availability of typical structural fill materials.

2. PENNDOT CASE HISTORIES WITH LIGHTWEIGHT/SUSTAINABLE FILLS

This project started with a literature review to gather useful information regarding the types of lightweight and sustainable fill materials, their manufacturing process, engineering properties, and design/construction considerations. In order to develop useful flowcharts and guidelines for these materials as alternatives of engineered fills, the scope of TEM WO 013 proceeded to a review of case histories where alternative fill materials were used. These case histories have primarily included PennDOT projects from across the Commonwealth of Pennsylvania (though most of them are from District 6-0), but also include some from other states. The focus of this chapter is to summarize the PennDOT case histories. Table 2.1 below summarizes the projects that were available to the Temple Research Team and reviewed as part of the case history review efforts. An important aspect of any alternative engineering design is the implications on project costs. Therefore, the focus of TEM WO 013 is on those case histories that include detailed information regarding costs (i.e., construction, operations, materials) when compared with conventional fill materials.

Table 2.1. List of reviewed projects.

Project	**Location**
0095, Section BR0	*Philadelphia*
0095, Section GR2	*Philadelphia*
0202, Section 300	Chester County
0202, Section 311	Chester County
0202, Section 330 - Chester Valley Trail	Chester County
0202, Section 330 - NW4	Chester County
1012, Section C01 - Gulph Rd ov Trout Run	King of Prussia, Montgomery County
SR 0119 - Indiana Hill Bridge	*Punxsutawney, Jefferson County*
SR 217, Section E10	Derry, Westmoreland County
SR 3422, Section 03B - Penn Street Bridge over Schuylkill	Reading, Berks County
SR 4011, Section CSB	Philadelphia
SR 4063, Section A05 - Pearce Mill Road Landslide Remediation	Pine Township, Allegheny County
SR 9015, Section NAV - TN 501 Langley Ave	Philadelphia

Note: Only the italicized projects contained sufficient cost information to be included in this report.

2.1. Selected Projects

Of all the lightweight fills case histories reviewed, only three projects contained sufficient detail to be included in this chapter (Table 2.1). The first two projects are from the ongoing I-95 improvement construction efforts taking place near the Betsy Ross Bridge interchange (0095, Section BRI) and near the intersection with Girard Avenue (0095, Section GR2).

Project 0095-GR2 has several different structures where lightweight fill materials were proposed, including bridges at Shackamaxon Street (S-26064) and Marlborough Street (S-26901), as well as various retaining walls throughout the extent of the projects. The structure located at I-95 over Shackamaxon Street (S-26064) is a simple span bridge, including six different modified components (Abutment 1, Abutment 2, Wingwall A, Wingwall B, Wingwall C, and Wingwall D). The same type of structure exists at I-95 over Marlborough Street (S-26901), and again multiple components (Abutment 1, Abutment 2, Wingwall A, and Wingwall D) were modified with lightweight fills. Moreover, Section GR2 includes four different wall sections that used lightweight materials as engineered fills: (1) Wall 9, Segment 0225 from station 305+34.22 to 308+44.22; (2) Wall 10, Segment 0224 from station 303+1.48 to 308+52.87; (3) Wall 11, Segment 0224 from station 309+68.22 to 314+33.22; and (4) Wall 12A, Segment 0224 from station 315+55.04 to 317+05.00.

The 0095-BRI project files described six different areas where lightweight fill materials were proposed: (1) I-95 NB and I-95 SB from station 492 + 00 to 500 + 00; (2) I-95 NB and I-95 SB from station 500 + 00 to 510 + 00; (3) Ramp EE and Ramp F; (4) Ramp YY; (5) Retaining Wall A (between Mainline and Ramp YY); and (6) Retaining Wall C (along with Ramp F). Also, there is a cost analysis comparison among compensating fill and column supported embankment as two improvement approaches.

The SR 0119 - Indiana Hill Bridge project is located in Jefferson County along State Route 119 in the borough of Punxsutawney, PA. This project is a roadway improvement project that includes multiples phases, including:

- Removal of an existing three-span structure over the Mahoning Shadow Trail
- Replacement of an existing three-span structure with embankment fill
- Realignment of the Mahoning Shadow Trail and construction of an emergency access road
- Placement of a pedestrian box culvert
- Addition of a climbing lane on S.R. 119 South

The remaining projects in Table 2.1 are not summarized in this report. Either there was no specific discussion of any lightweight fill technologies, or there was insufficient discussion of costs associated with lightweight engineered fills used in the projects. In some cases, the project files included plans, field logs, photos, contractual documents, and commentary regarding the project's process, but no useful engineering reports by which to fully understand the full extent of the work proposed/constructed in the project.

2.1.1. Project 0095, Section GR2

The I-95 improvements in section GR2 contained multiple structures for which lightweight fill materials were implemented during construction after a proposed re-design highlighted potential costs savings. The following sections summarize each of these structures, the proposed designs, and a comparison of the costs associated with lightweight materials approach relative to the original proposed design.

2.1.1.1. 0095 Over Shackamaxon Street (S-26064)

This part of the project concerns the modification of an existing simple span bridge (S-26064) located on I-95 over Shackamaxon Street in Philadelphia County, Pennsylvania. The existing bridge comprises of a composite pre-stressed concrete spread box beam supported by two Abutments (1 and 2), including Wingwalls (A, B, C, D) on spread footings. Abutment 1 includes Wingwall A in the northbound and Wingwall B in the southbound. Abutment 2 includes Wingwall C in southbound and Wingwall D in northbound.

As a result of the planned widening of I-95, the abutments and wing walls will be extended significantly in the northbound lane while the extension of widening in southbound is minimal and includes 4.4 ft beyond the existing foundation at the location of Wingwall B and 4.9 ft beyond the existing foundation at the location of Wingwall C. The initial design includes the construction of spread footings on soils improved by jet grouting and 2-3 ft of geogrid mat. However, the project was re-evaluated as a Value Engineering (VE) project with lightweight concrete as part of the backfill and an undercut fill to reduce vertical stress imposed to the structure and subsequently reduce the vertical deformation.

2.1.1.1.1. Geotechnical Considerations

The subsurface investigation revealed that the embankment (fill) soils lie on alluvial soils belonging to the Trenton Gravel Formation, while the underlying alluvial and residual soils are weathered from the Wissahickon Formation. Due to the variability of the site stratigraphy, each layer was assigned separate design parameters based on the previously-approved Geotechnical

Report completed by URS Corporation (URS, 2011). To reduce and limit the structure's vertical deformation, the lightweight concrete used in this project had the following properties:

Table 2.2. Lightweight Concrete Material Properties Prescribed for 0095, Section GR2.

Material Property	**Value**
Compressive Strength	40 lbs/in^2
Unit Weight	30 lbs/ft^3
Elastic Modulus	270 kips/in^2
Poisson's Ratio	0.2
Coefficient of Lateral Earth Pressure (at rest)	0.25

The software program FoSSA (ADAMA Engineering, 2007) was used to calculate the settlements of the abutments and wing walls. The concept of net pressure of the settlement was used to load balance and prevent excessive settlements. The results of the settlement analysis demonstrated that lightweight concrete was a better choice than jet grouting. Use of lightweight concrete as a replacement for conventional structural backfill material decreases the weight of the abutments and wing walls and subsequently reduces the driving forces on the wall and anticipated settlements. However, all of the conventional backfill material cannot be replaced with lightweight concrete because the structure will face problems in terms of sliding and overturning. For this reason, both materials were incorporated into the design to optimize with consideration of external stability and settlements. The height of structural fill material varied for the different structures (Figs. 2.1 – 2.6):

- Abutment 1 and Wingwall A = Elevation 16.5 ft to elevation 21 ft
- Abutment 2 and Wingwall D = Elevation 15.5 ft to elevation 21 ft.
- Wingwall B = Elevation 18.75 ft to 8 ft below the roadway pavement
- Wingwall C = Elevation. 17.75 ft to 10 ft below the roadway pavement

Subsequent analysis with the PennDOT ABLRFD software showed that partial use of lightweight concrete as the partially backfilling for Abutment 1 (Northbound), Abutment 2 (Northbound), Wingwall A, and Wingwall D was not in the tolerable range of 0.5 inches. As a result, a 5 ft undercut filled with lightweight concrete was incorporated beneath Abutment 1 (NB), Abutment 2 (NB), and Wingwalls A and D in order to reduce the vertical settlement further while maintaining external stability of the wall. The settlement results of Wingwall B and C were found to be within an acceptable range so the partial use of lightweight concrete as the partially backfilling was sufficient for the mentioned sections. Consequently, a 3ft thick layer of 2A coarse aggregate was incorporated below Wingwall B and a 2ft thick layer of 2A coarse aggregate was incorporated below Wingwall C.

2.1.1.1.2. Additional Design Recommendations

Figures 2.1 to 2.6 present the typical cross section for design of the proposed abutment and wingwall structures. Additionally, the following design recommendations were highlighted in the geotechnical reports:

- Use of a geocomposite layer behind the lightweight concrete is necessary to collect any subsurface drainage and should be extended to the back of the structure.
- Dewatering should be considered as a conservative alternative to prevent the entrance of any possible groundwater.
- Slope cut inclination should not be more than 1.5H:1V.
- Proposed abutments and wingwalls should bear a minimum of 4.5 ft below the proposed surface.

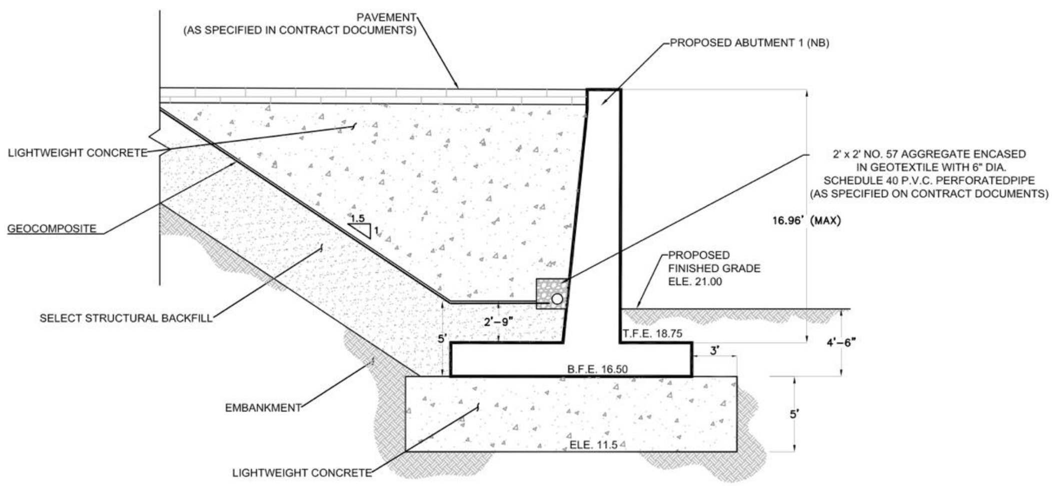


Figure 2.1. Typical cross section for Abutment 1(NB) (0095 Over Shackamaxon Street, S-26064).

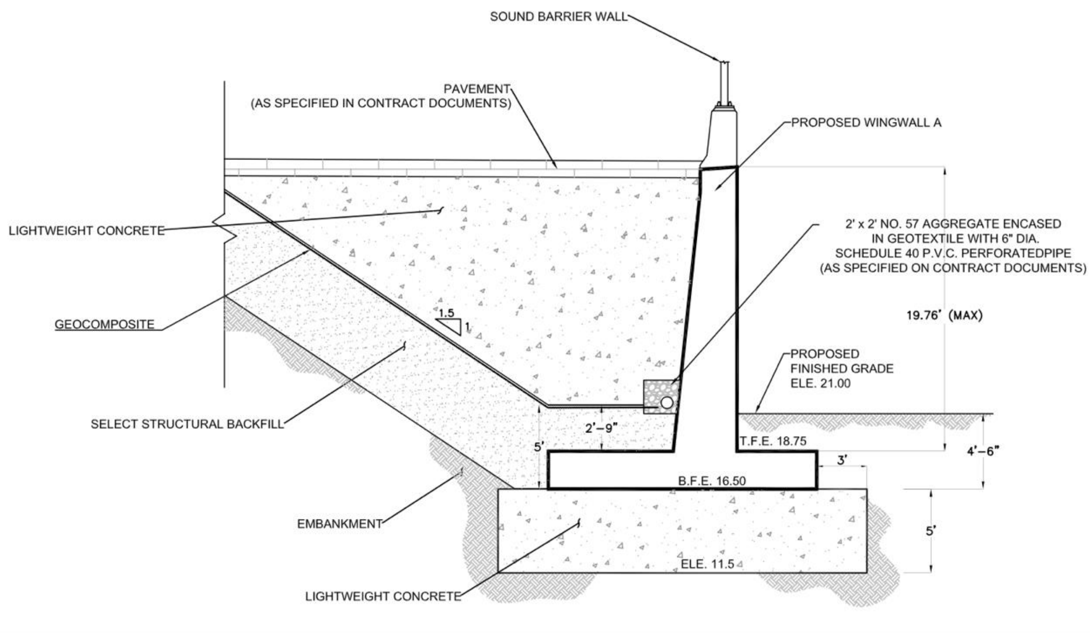


Figure 2.2. Typical cross section for Wingwall A (0095 Over Shackamaxon Street, S-26064).

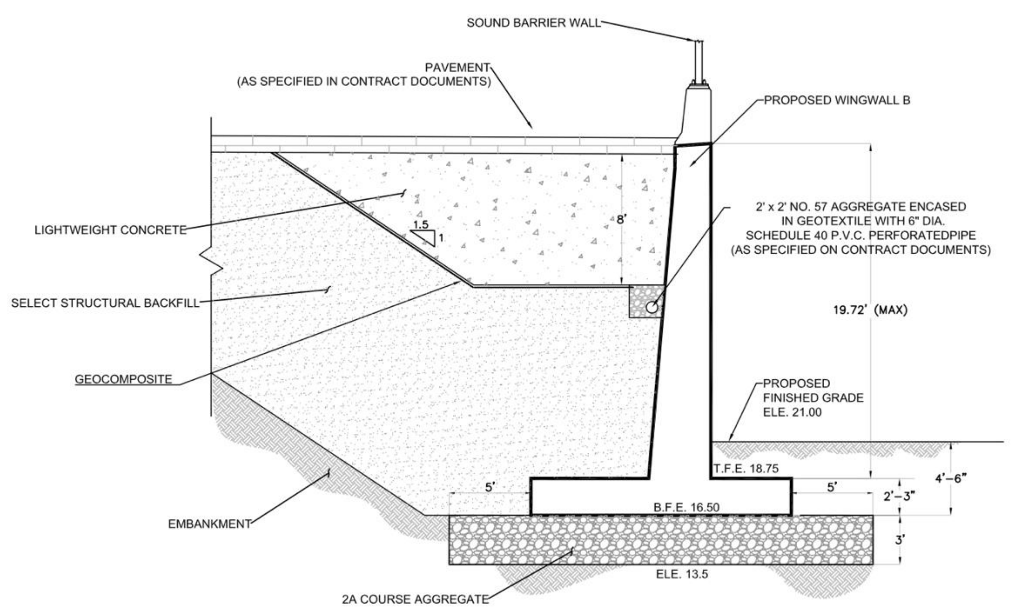


Figure 2.3. Typical cross section for Wingwall B (0095 Over Shackamaxon Street, S-26064).

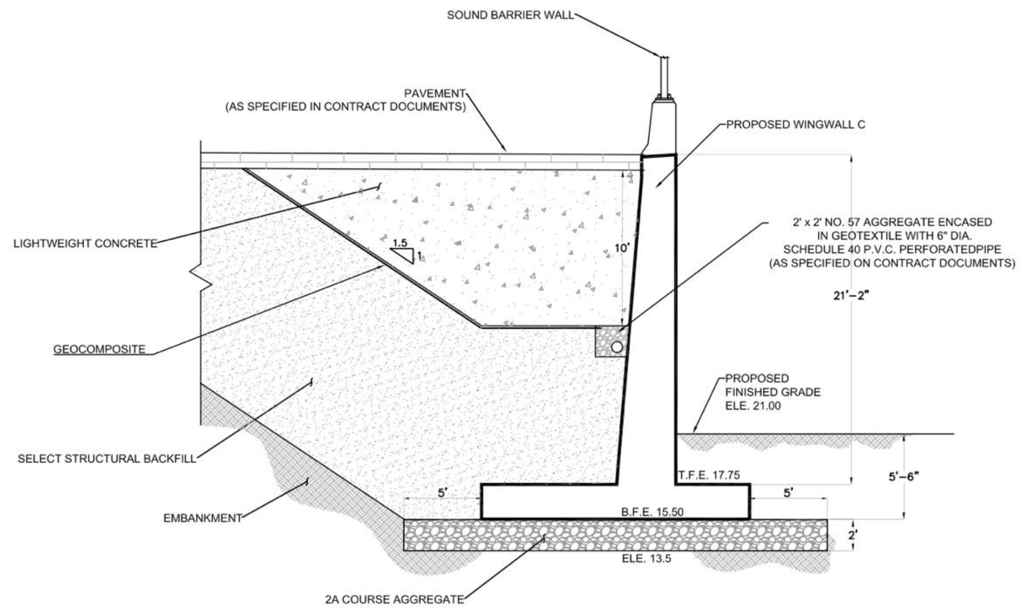


Figure 2.4. Typical cross section for Wingwall C (0095 Over Shackamaxon Street, S-26064).

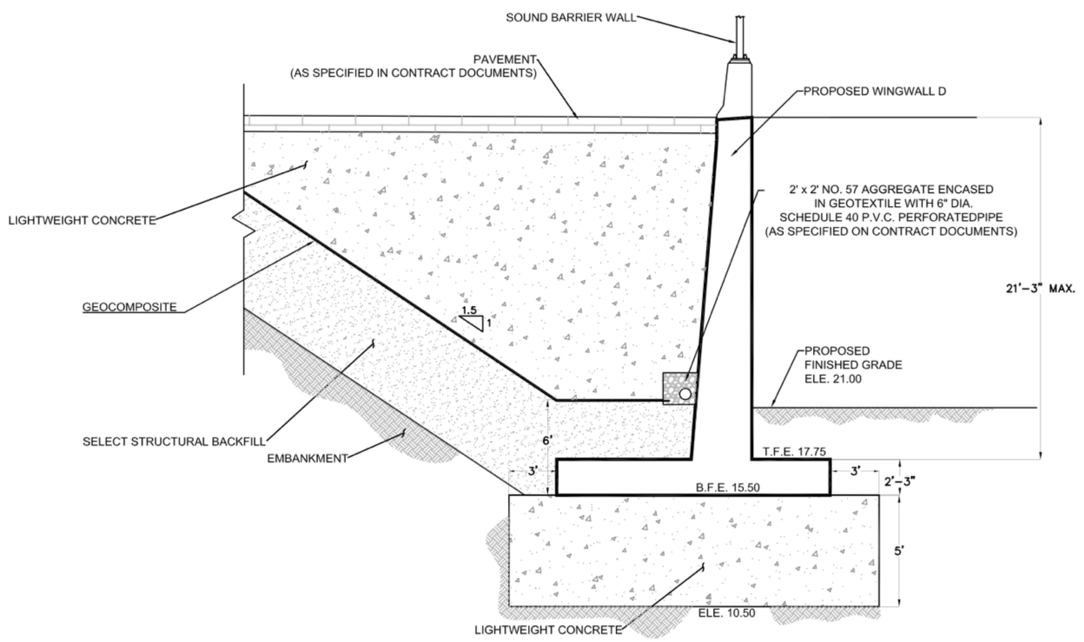


Figure 2.5. Typical cross section for Wingwall D (0095 Over Shackamaxon Street, S-26064).

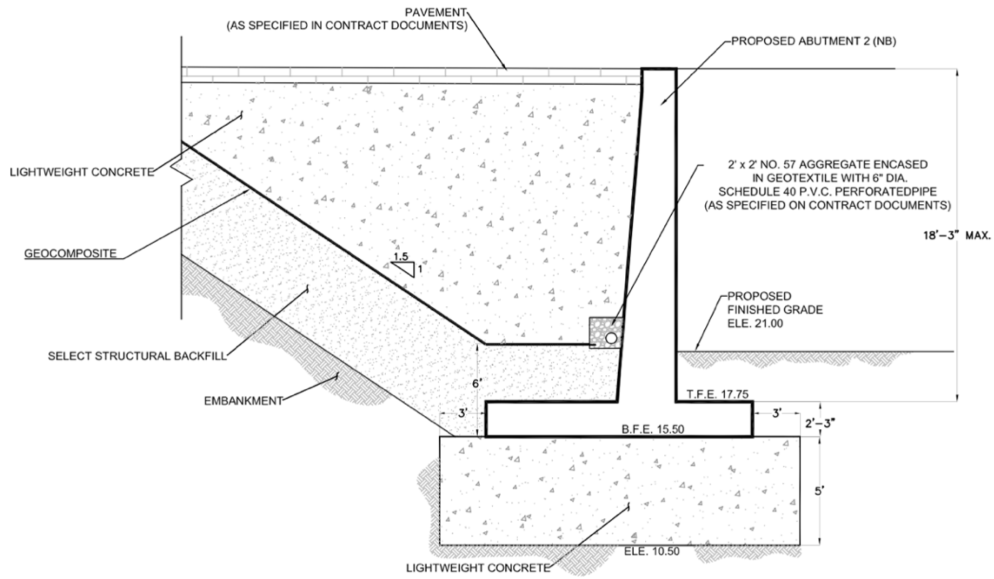


Figure 2.6. Typical cross section for Abutment 2 (NB) (0095 Over Shackamaxon Street, S-26064).

2.1.1.1.3. Cost Comparison

As noted previously, the original project design called for the use of jet grouting as a ground improvement technique in combination with spread footings. Prior to construction, a VE report was generated by Earth Engineering Incorporated on behalf of the contractor (James J. Anderson Construction Company, Inc.) that discussed the use of lightweight concrete fill as an alternative to jet grouting. As part of the VE efforts, anticipated costs were computed for the lightweight fill alternative approach and compared to jet grouting. Tables 2.3 – 2.6 present the associated costs with the alternative design for each of the structural components in the Shackamaxon project.

Table 2.3. Abutment 1 and Wingwall A Cost Analysis (0095 Over Shackamaxon Street, S-26064).

	Volume (Unit)	**Unit Price ($)**	**Deduct ($)**	**Add ($)**
Original Cost Deduct				
Structural Backfill	145 CY	55	7,975	
Additional Costs to Avoid Jet Grouting				
Undercut	320 CY			
Cost Unsuitable	480 Tons	57.37		27,537
Shoring Area	70(L)×9.5(D)=665 SF	30		19,950
Lightweight Concrete in Undercut	320 CY	85		27,200

Lightweight Concrete Backfill	145 CY	85		12,325
Geocomposite	130 SY	9.90		1,287

Table 2.4. Abutment 2 and Wingwall D Cost Analysis (0095 Over Shackamaxon Street, S-26064).

	Volume (Unit)	**Unit Price ($)**	**Deduct ($)**	**Add ($)**
Original Cost Deduct				
Structural Backfill	145 CY	55	7,975	
Additional Costs to Avoid Jet Grouting				
Undercut	245 CY			
Excavation/Disposal	367.5 Tons	57.37		21,083.48
Shoring Area	65(L)×11(D)=715 SF	30		21,450
Lightweight Concrete in Undercut	245 CY	85		20,825
Lightweight Concrete Backfill	145 CY	85		12,325
Geocomposite	120 SY	9.90		1,188

Table 2.5. Wingwall B Cost Analysis (0095 Over Shackamaxon Street, S-26064).

	Volume (Unit)	**Unit Price ($)**	**Deduct ($)**	**Add ($)**
Original Cost Deduct				
Structural Backfill	76 CY	55	4,180	
Undercut	75 CY			
Additional Costs to Avoid Jet Grouting				
Excavation/Disposal	112.5 Tons	57.37		6,454.13
Shoring Area	0 SF			
2A in Undercut	75 CY	50		3,750
Lightweight concrete Backfill	76 CY	85		6,460
Geocomposite	55 SY	9.90		544.50

Table 2.6. Wingwall C Cost Analysis (0095 Over Shackamaxon Street, S-26064).

	Volume (Unit)	**Unit Price ($)**	**Deduct ($)**	**Add ($)**
Original Cost Deduct				
Structural Backfill	214 CY	55	11,770	
Additional Costs to Avoid Jet Grouting				

Undercut	76 CY		
Excavation/Disposal	114 Tons	57	6,498
Shoring area	0		
2A in Undercut	76 CY	50	3,800
Lightweight concrete Backfill	214 CY	85	18,190
Geocomposite	70 SY	9.90	693

2.1.1.2. 0095 Over Marlborough Street (S-26901)

This part of the 0095, Section GR2 project modifies an existing simple span bridge (S-26901) that carries traffic from I-95 over Marlborough Street in Philadelphia County, Pennsylvania. It is located approximately 570 ft away along I-95 NB from the Shackamaxon Street bridge section of the project described in section 1.2.1.1. Similar to the Shackamaxon Street bridge, the existing bridge at Marlborough Street is made of pre-stressed concrete box beam supported by two Abutments (1 and 2), including Wingwalls (A, B, C, D) on spread footings. As before, the abutments and wing walls will be extended significantly in the northbound lane and minimally for the southbound lanes. The initial design again includes the construction of spread footings on soils improved by jet grouting and 2-3 ft of geogrid mat, though lightweight fills were recommended as a VE project to save costs by reducing the vertical stresses and subsequent settlements caused by the structure.

2.1.1.2.1. Geotechnical Considerations

Given the extensive similarities between the two structures, the geotechnical design considerations, analytical approach, and final recommendations were nearly identical for the Marlborough Street bridge section when compared to the Shackamaxon Street bridge section. Again, load balancing was attempted whereby some of the structural backfill was partially replaced by lightweight concrete to reduced settlements but not negatively impact the external stability of the wall due to overturning and sliding. To satisfy these requirements, again it proved necessary to design an undercut section of lightweight concrete fill beneath the spread footing of each wall. Figures 2.7 – 2.10 present typical cross sections for each of the components of the structure.

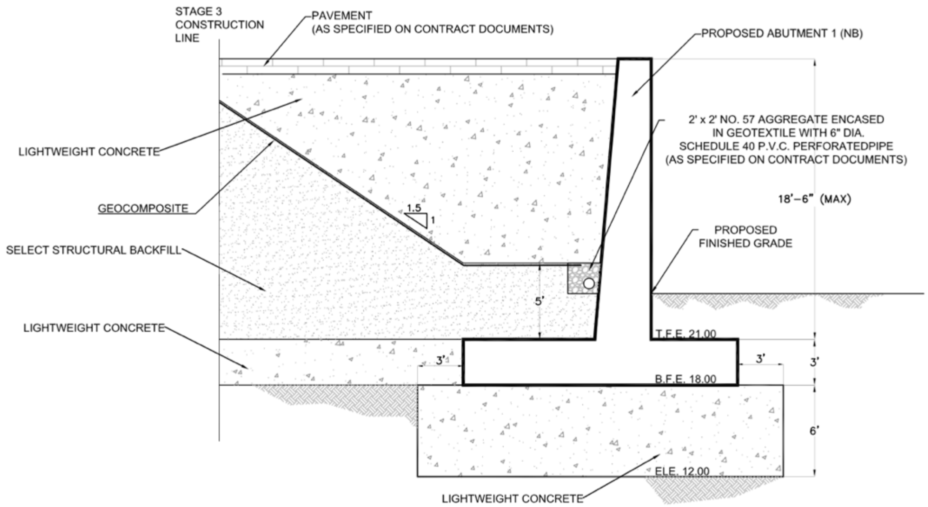


Figure 2.7. Typical cross section for Abutment 1 (NB) (0095 Over Marlborough Street, S-26901).

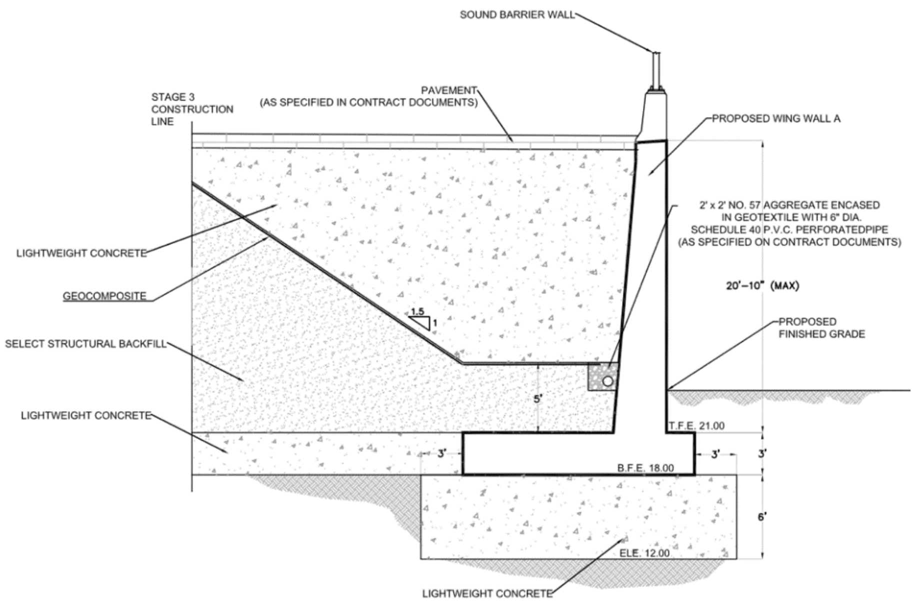


Figure 2.8. Typical cross section for Wingwall A (0095 Over Marlborough Street, S-26901).

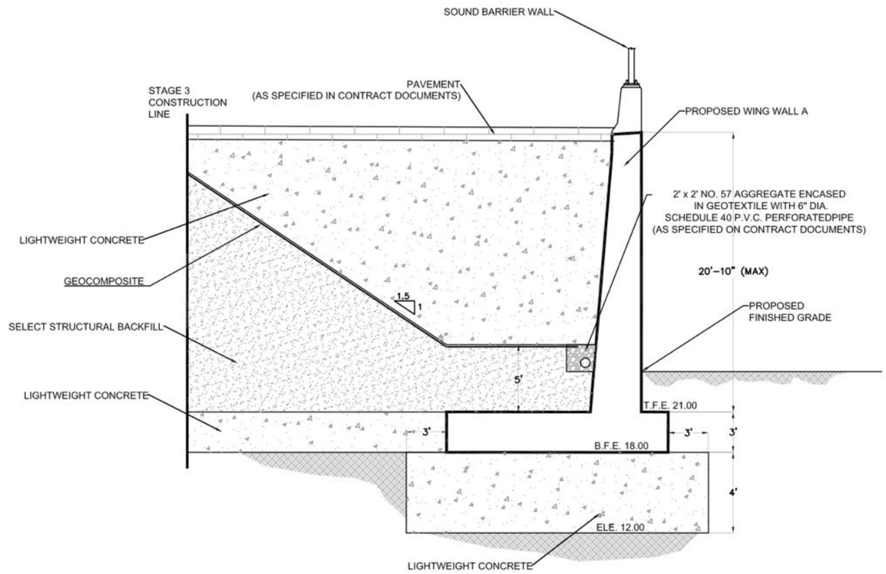


Figure 2.9. Typical cross section for Wingwall D (0095 Over Marlborough Street, S-26901).

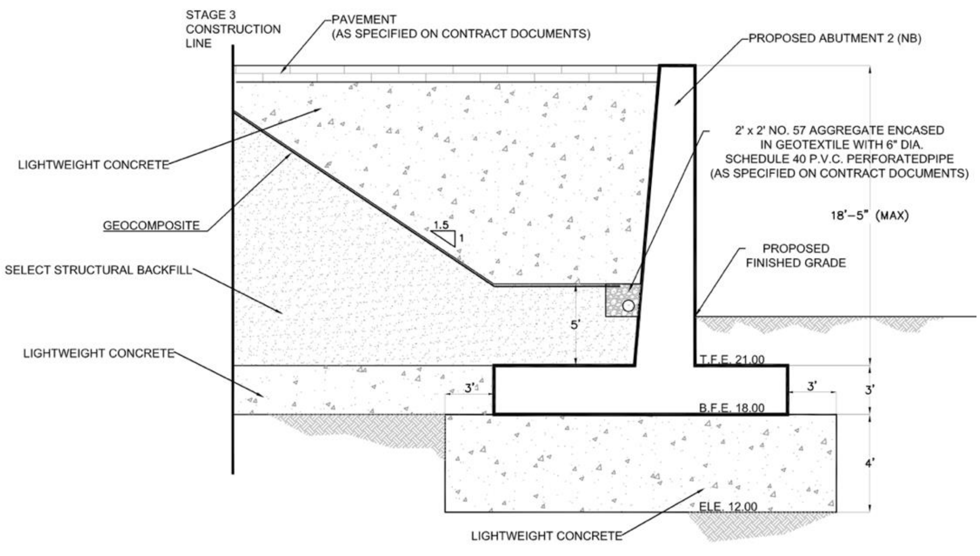


Figure 2.10. Typical cross section for Abutment 2 (NB) (0095 Over Marlborough Street, S-26901).

2.1.1.2.2. Cost Comparison

Tables 2.7 and 2.8 present the associated costs with the alternative design for each of the structural components in the Marlborough project. As with the Shackamaxon section, use of jet grouting was avoided and instead lightweight concrete was incorporated into the backfill and undercut fill after an Earth Engineering Incorporated VE report generated on behalf of the contractor (James J. Anderson Construction Company, Inc.) demonstrated the potential for significant costs savings.

Table 2.7. Abutment 1 and Wingwall A Cost Analysis (0095 Over Marlborough Street, S-26901).

	Volume (Unit)	Unit Price ($)	Deduct ($)	Add ($)
Original Cost Deduct				
Structural Backfill	440 CY	55	24,200	
Additional Costs to Avoid Jet Grouting				
Undercut	320 CY			
Cost Unsuitable	480 Tons	38		18,240
Shoring Area	480 SF	30		14,400
Lightweight Concrete in Undercut	320 CY	85		27,200
Lightweight Concrete Backfill	440 CY	85		37,400
Geocomposite	110 SY	9.90		1,089

Table 2.8. Abutment 2 and Wingwall D Cost Analysis (0095 Over Marlborough Street, S-26901).

	Volume (Unit)	Unit Price ($)	Deduct ($)	Add ($)
Original Cost Deduct				
Structural Backfill	440 CY	55	24,200	
Additional Costs to Avoid Jet Grouting				
Undercut	215 CY			
Cost Unsuitable	322.5 Tons	57.37		18,501.83
Shoring Area	400 SF	30		12,000
Lightweight Concrete in Undercut	215 CY	85		18,275
Lightweight Concrete Backfill	440 CY	85		37,400
Geocomposite	135 SY	9.90		1,336.50

2.1.1.3. 0095 Section GR2 Retaining Walls

In addition to the Shackamaxon and Marlborough Street bridge sections, 0095-GR2 also contained four different segments of precast modular concrete T-Walls® retaining wall systems [Wall 9 (S-32707), Wall 10 (S-32599), Wall 11 (S-32669), and Wall 12A (S-32472)]. The T-walls® will support the widening of both southbound and northbound lanes for I-95 near the interchange with Girard Avenue. Structure-mounted sound barrier walls and vehicular barriers will also be located at the top of the T-walls®. As with the bridge structures at Shackamaxon and Marlborough Street, the original design for the T-walls® required the use of jet grouting to improve the subgrade soils along with a 2 ft geogrid-reinforced aggregate mat beneath the walls. Lightweight concrete backfill was proposed as a Value Engineering design to decrease costs by limiting the vertical stresses imposed by the walls and reducing settlements within tolerable limits. The following sections provide information about each of the walls as well as the cost comparison relative to jet grouting.

2.1.1.3.1. Summary of Wall Geometries

The Wall 9 structure (S-32707) (Figure 2.11) is located at Southbound Station 305+34.22, where it abuts a temporary sign structure S-32718 (Tower A). The end of the wall is located at Station 308+44.22, where it abuts the proposed bridge wingwall for the Shackamaxon Street bridge structure (S-26064) as described in section 1.2.1.1 of this report. Segment 1 of the wall extends from the beginning Southbound Station 305+34.22 to Station 307+59.22, with a top of leveling pad elevation of 14.0 ft and a bottom stem width of 16 ft. Segment 2 thereafter extends to the end of wall Station 308+44.22, with the top of the leveling pad extending to elevation 16.54 ft and a bottom stem width of 15 ft. Based on this information, the retaining wall will be approximately 310 ft long and have a maximum height of 24.91 ft. The structure-mounted sound barrier wall located on top of Wall 9 will have a maximum height of 11.8 ft and the vehicular barrier will be 3.5 ft high.

Wall 10 (S-32599) (Figure 2.12) is located near Wall 9, but will support widening of northbound lanes of I-95. The wall begins at Northbound Station 303+11.48 and ends at Station 308+52.87, where it abuts the proposed bridge wingwall for the Shackamaxon Street bridge structure (S-26064). A sign structure with an independent foundation system will be located between Station 305+05.78 and 305+27.87. Wall 10 will be approximately 400 ft long and have a maximum height of 25.5 ft. As with the other walls in the section of GR2, a structure-mounted sound barrier wall and vehicular barrier will be located at the top of the wall, with maximum heights of 14 ft and 3.5ft, respectively.

Wall 11 (S-32669) (Figure 2.13) begins at Northbound Station 309+68.65 and ends at Station 314+33.65. The beginning of the wall abuts the proposed bridge wingwall for the Shackamaxon Street bridge structure (S-26064) and the end of the wall abuts the proposed bridge wingwall for the Marlborough Street bridge structure (S-26901). Wall 11 will be approximately 465 ft long and contains two segments. Segment 1 is 225 ft long and will have a maximum height of 25.35 ft and Segment 2 will be 240 ft long and have a maximum height of 24.05 ft. Similar to the other walls in this section of GR2, Wall 11 will have a structure-mounted sound barrier wall (14 ft maximum height) and vehicular barrier (3.5 ft height) located at the top of the retaining wall.

Wall 12A (S-32472) (Figure 2.14) begins at Northbound Station 315+55.04 where it abuts a sign structure and ends at Station 317+05.00. The existing wall along this section of I-95 pinches together toward the end of Wall 12A, which resulting in the design of Wall 12A into two segments. Segment 1 was originally designed on spread footing on soil improved by jet grouting. This segment begins at Station 315+55.04 and ends at Station 316+50.00. Segment 2 was designed on driven piles and begins at Station 316+50.00 and ends at Station 317+05.00. Only Segment 1 was re-designed to use lightweight fill material as part of the VE efforts. It is approximately 95 ft long and will have a maximum height of 24.84 ft and a traffic barrier and sound barrier wall as with the other walls.

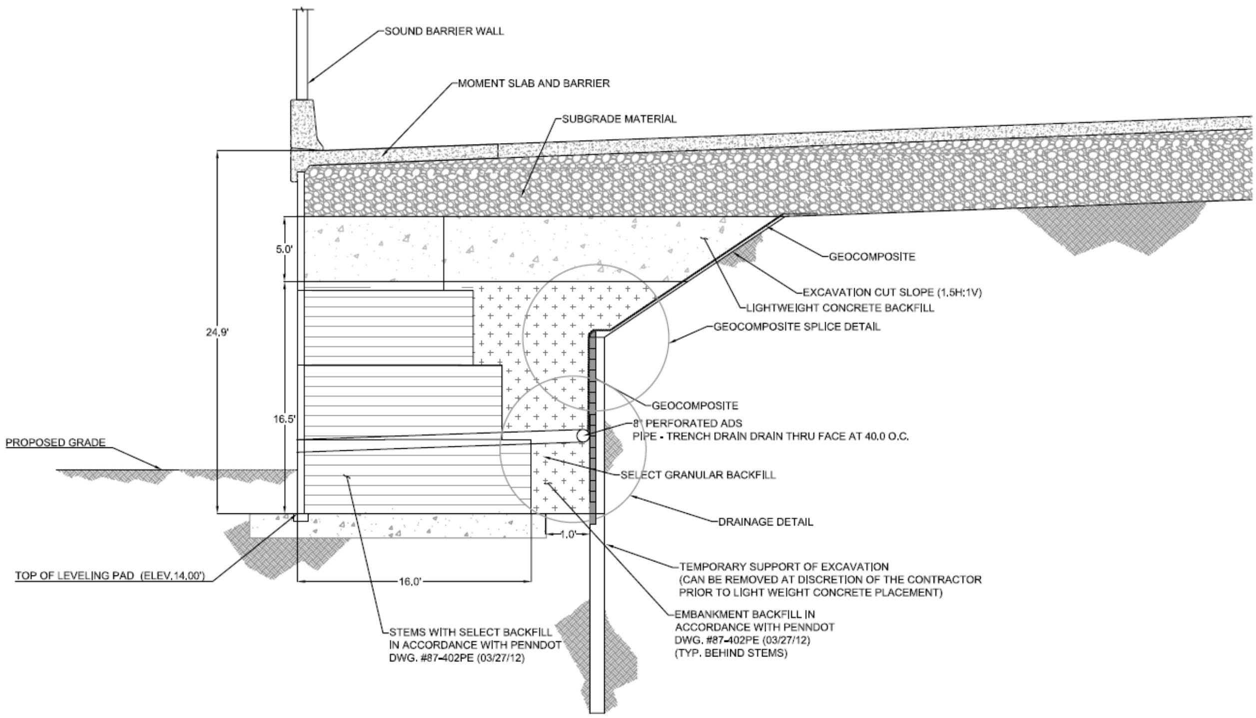


Figure 2.11. Typical cross section for Wall 9 (S-32707).

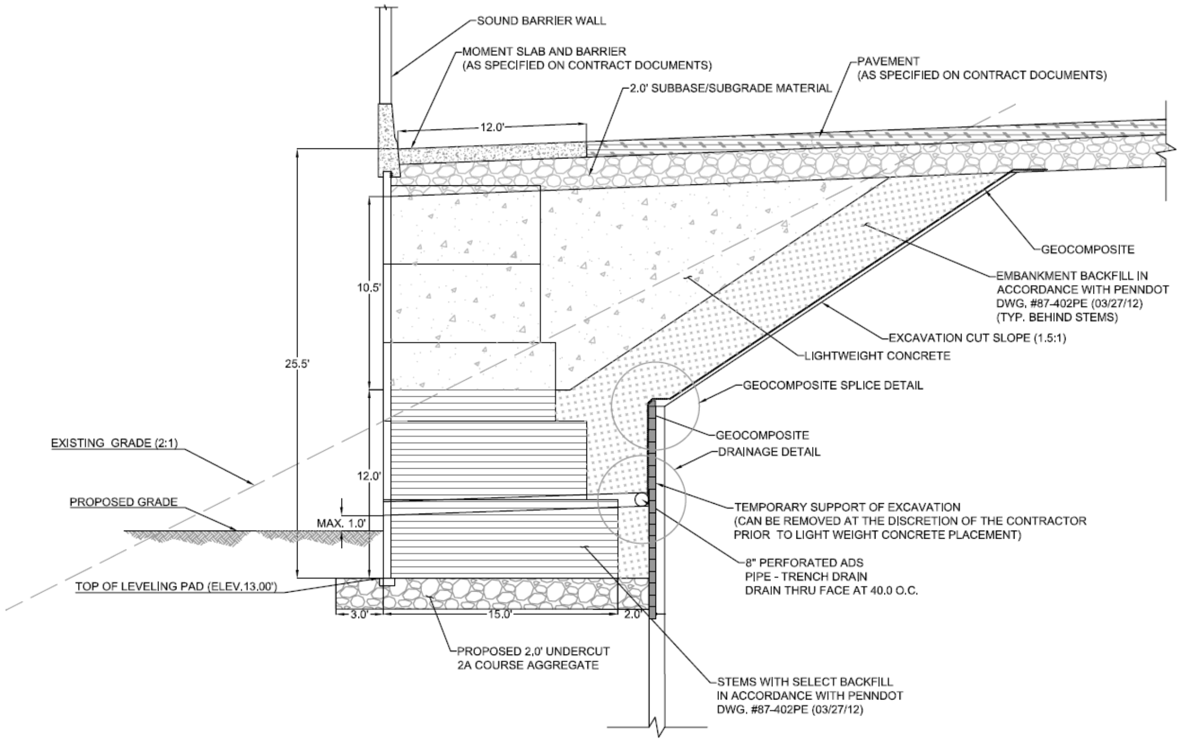


Figure 2.12. Typical cross section for Wall 10 (S-32599).

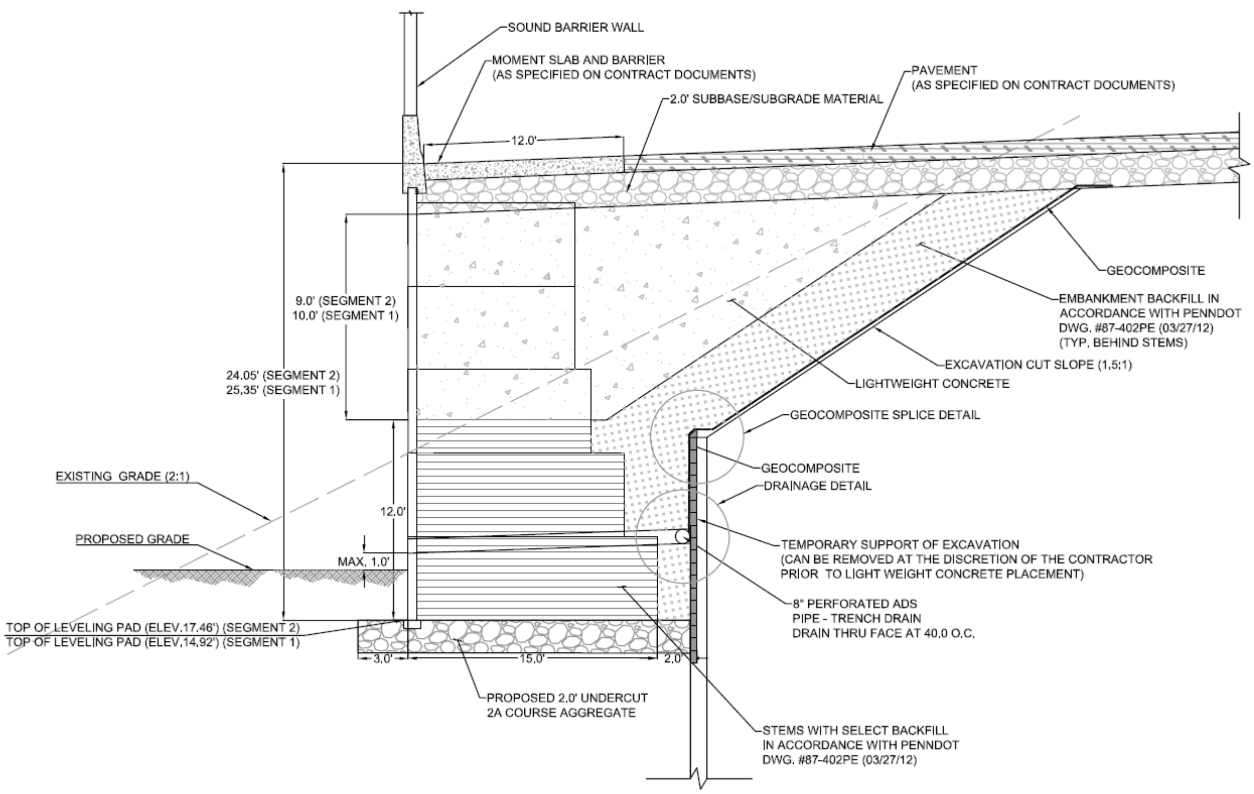


Figure 2.13. Typical cross section for Wall 11 (S-32669).

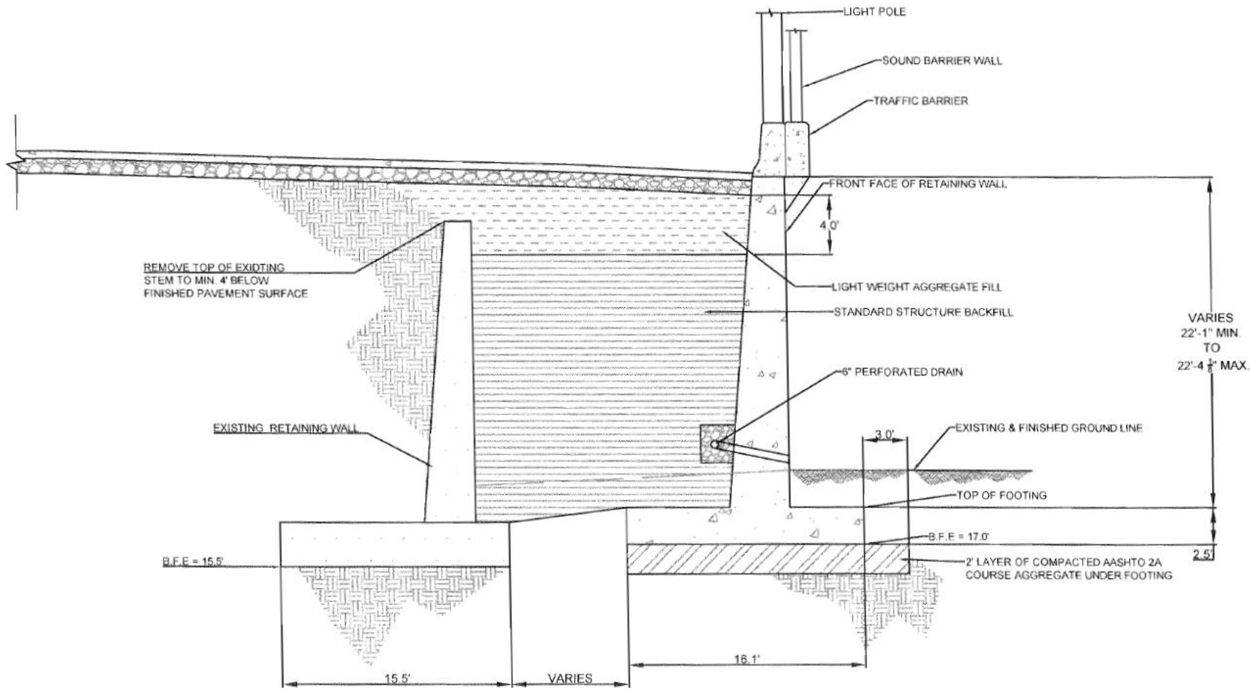


Figure 2.14. Typical cross section for Wall 12A (S-32472).

2.1.1.3.2. Cost Comparison

As noted previously, a VE report was prepared by Earth Engineering Incorporated (EEI) on behalf of the contractor for 0095-GR2 (James J. Anderson Construction Company, Inc.) for Wall 9 (S-32707), Wall 10 (S-32599), Wall 11 (S-32669), and Wall 12A (S-32472). The focus of the VE report was to propose an alternate design for the proposed walls that uses lightweight fills to reduce vertical loads and anticipated settlements to tolerable limits. This eliminate the need for ground improvement of the subsurface soils using jet grouting as originally proposed for the design of these walls. As part of the VE package efforts, the contractor estimated the potential cost savings associated with this alternate design. Tables 2.9 – 2.12 present the cost analysis performed by the contractor for each of the proposed walls.

Table 2.9. 0095, Section GR2 Wall 9 (S-32707) Cost Analysis.

	Volume (Unit)	**Unit Price ($)**	**Deduct ($)**	**Add ($)**
Original Cost Deduct				
Select Backfill	2498 CY	55	137,390	
Common Backfill	1443 CY	25	36,075	
Additional Costs to Avoid Jet Grouting				

Select Backfill	4500 CY	55		247,500
Lightweight Concrete Backfill	1920 CY	85		163,200
Geocomposite	1050 SY	9.90		10,395
Common Borrow Cap	566 CY	25		14,150
8" ADS Subdrain	310 LF	10		3,100
8" Outlet	180 LF	15		2,700

Table 2.10. 0095, Section GR2 Wall 10 (S-32599) Cost Analysis.

	Volume (Unit)	Unit Price ($)	Deduct ($)	Add ($)
Original Cost Deduct				
Select Backfill	3481 CY	55	191,455	
Common Backfill	1443 CY	25	36,075	
Additional Costs to Avoid Jet Grouting				
Select Backfill	3970 CY	55		218,350
Lightweight Concrete Backfill	4000 CY	85		340,000
Geocomposite	1400 SY	9.90		13,860
Common Borrow Cap	987 CY	25		24,675
8" ADS Subdrain	422 LF	10		4,220
8" Outlet	187 LF	15		2,850
Additional Trucking and Disposal Fee for Unused Backfill Materials	1030CY	32.78		33,763.40

Table 2.11. 0095, Section GR2 Wall 11 (S-32669) Cost Analysis.

	Volume (Unit)	Unit Price ($)	Deduct ($)	Add ($)
Original Cost Deduct				
Select Backfill	4146 CY	55	228,030	
Common Backfill	2362 CY	25	59,050	
Additional Costs to Avoid Jet Grouting				
Select Backfill	4185 CY	55		230,175
Lightweight Concrete Backfill	4460 CY	85		379,100
Geocomposite	1280 SY	9.90		12,672

Common Borrow Cap	896 CY	27		24,192
8" ADS Subdrain	465 LF	10		4,650
8" Outlet	204 LF	15		3,060
Additional Trucking and Disposal Fee for Unused Backfill Materials	1515 CY	32.78		49,661.70

Table 2.12. 0095, Section GR2 Wall 12A (S-32472) Segment 1 Cost Analysis.

	Volume (Unit)	**Unit Price ($)**	**Deduct ($)**	**Add ($)**
Replace AASHTO 57 with Lightweight Aggregate				
Length of Segment 1	95.92 LF			
Width of Lightweight Aggregate	20 FT			
Depth of Lightweight Aggregate	4 FT			
Volume of Lightweight Aggregate	284.21 CY			
Original Cost Deduct				
Weight of #57	130 lbs/cf			
Tons	498.79 Tons			
Cost FOB site		18.40		
Net Cost inc. 8% tax			9,911.95	
Cost Add to Place Lightweight Aggregate				
Weight of Lightweight Aggregate	50 lbs/cf			
Tons	191.8 Tons			
Cost FOB site		110		
Net Cost inc. 8% tax				22,785.84

2.1.1.4. Final Cost Savings

As part of the VE submission package, the overall costs for 0095, Section GR2 were estimated using the original jet grouting design and the alternate design utilizing lightweight fill materials for the retaining walls. It was estimated that jet grouting added $1.806M to the overall costs of the project (Table 2.13). Table 2.14 presents the final cost analysis for both designs when considering all of the aforementioned structures proposed for 0095, Section GR2. The analysis shows that the initial cost of using lightweight fill materials is substantially higher relative to conventional fill materials. However, the additional $1.806M associated with jet grouting increases the overall costs such that the design with lightweight fills is less expensive to construct overall. As Table 2.14 estimates, the use of lightweight fills in the retaining structures and elimination of jet grouting results in a total cost savings of approximately $300,000 for Project 0095, Section GR2, when engineering fees are included.

Table 2.13. Details of Costs Associated with Jet Grouting for 0095, Section GR2.

Item	**Cost**
Jet Grouting Mobilization	+ $190,000
8000 CY @ $200 per CY	+$1,600,000
Utility Monitoring during Jet Grouting	+ $16,000
Total Cost	+$1,806,000

Table 2.14. Final Project Cost Analysis for 0095, Section GR2.

	Conventional Fill	**Lightweight Concrete**
Design Alternatives	Final Cost of Conventional Fill (Not Including Jet Grouting) +$778,286.95	Final Cost of Lightweight Concrete Fill +$2,202,417.98
	Final Cost of Conventional Fill (Including Jet Grouting) $778,286.95 + $1,806,000 = +$2,584,286.95	+$2,202,417.98
Net Saving		+$381,868.97
Engineering Fees (@ 50%)		-$175,200 / 2 = $87,600
Total Value Engineering Cost Reduction		+$300,000

2.1.2. Project 0095, Section BRI

The 0095, Section BRI project encompasses an earth fill section that supports the existing continuous, multi-span concrete bridge carrying northbound and southbound traffic on I-95 near the Betsy Ross Interchange. This section starts approximately 500 ft north of Frankford Creek at Station 492+50 and ends about 1,800 feet away at Station 515+00 where I-95 continues as a steel bridge. This section was originally constructed as part of the original I-95 project in the 1960's and is very complex in terms of the number and geometry of the bridges carrying traffic to and from the Betsy Ross Bridge, I-95, and nearby Aramingo Avenue. The goals of the 0095, Section BRI project are to increase the number of travel lanes on I-95 and improve traffic flow, increase service life of the structures and place portions of I-95 on grade. The following sections describe the proposed work at 0095, Section BRI, the proposed geotechnical solutions for the I-95 improvements, and a comparison of the costs associated with each design alternative.

2.1.2.1. Scope of Project

Figure 2.15 presents a plan view of the limits of the 0095, Section BRI project based on the preliminary geotechnical report generated by STV Incorporated in 2011. In addition to the mainline roadway, there are multiple associated on- and off-ramps (Ramp YY, combined Ramp E-F, an extension of Ramp EE and Ramp F) that will be modified as part of the Section BRI construction efforts. Due to the complexity of the site and the necessary staging of construction efforts, the BRI section has three subset construction sections (BR0, BR2, and BR3.). Section BR0 comprises Ramp EE and Ramp F, which carry traffic from I-95 to the Betsy Ross Bridge. The proposed ramp structures will replace the existing Ramp E. Ramp EE will be constructed from station 92+42.49 to 105+85, while Ramp F will be parallel to Ramp EE and start at Station 808+63.86 and end at Station 818+00. Sections BR2 and BR3 will comprise the mainline roadway from Station 492+50 to Station 515+00 and Ramp YY from Station 19+50 to Station 29+50.

Figure 2.15. Scope of Project 0095, Section BRI.

2.1.2.2. Proposed Geotechnical Solutions

When the portion of I-95 associated with Section BRI was originally constructed, pile foundations were the selected design alternative to carry the structural loads due to the soft soils in the area. However, due to the economics of new bridge construction and concern over deterioration of existing piles because of salt laden roadway runoff, other alternative foundation designs were considered for the new roadway. This included removal of the compressible soils in the section, various in-situ ground improvement techniques (vibro-compaction, rammed aggregate piers, jet grouting, deep soil mixing), and preloading with or without prefabricated vertical drains (PVD). However, two primary design alternatives were ultimately recommended for further consideration: (1) Compensating Fill; and (2) Column Supported Embankment. The concept behind a compensating fill is to increase the roadway grade without applying any excessive overburden pressure on the existing underlying layers. To accomplish this a portion of roadway fill material will be replaced by lightweight engineered fill. Column supported embankment consists of a geosynthetic-reinforced granular soil supported on a pattern of vertical columns. The load on top of the embankment is transferred to the geosynthetic-reinforced granular soil and subsequently transferred to a stratum with acceptable bearing capacity. The following sections provide additional details about the proposed alternative designs for section BRI.

2.1.2.2.1. Compensating Fill

Before selecting an appropriate lightweight fill material several key factors must be evaluated. One of the most critical parameters is the range of unit weight of the materials because it directly affects the amount of material needed (i.e., costs) and the weight of the proposed structure. Environmental concerns, logistics of placement, and durability are other parameters that may affect the final selection of material. For the BRI section, multiple commonly-used lightweight materials were evaluated, including fly ash and air-cooled slag (unit weight ranges between 70 to 95 pcf), expanded shale (40 to 65 pcf), lightweight foamed concrete (20 to 50 pcf), and expanded polystyrene (1 to 2 pcf). Lightweight foamed concrete can easily adjust to different unit weights based on the requirement of each project. Moreover, the material has no associated environmental issues and only requires a modest excavation depth due to its low unit weight. In this way, there is no necessity to excavate below the groundwater table and risk net uplift due to buoyancy. The foaming agent used to create lightweight foamed concrete is readily available from a number of manufacturers nationwide (e.g., Elastizell Corporation, Cellular Concrete, Inc., Cematrix, Inc., Geofill Cellular Concrete, etc.). Consequently, lightweight foamed concrete was chosen as the lightweight alternative for the proposed compensating fill design for BRI.

Based on the condition of the project, two different subcategories of lightweight foamed concrete were prescribed:

- Class IV has a higher density of 42 pcf and, subsequently, higher strength material and used for better traffic load distribution. This class's minimum compressive strength is 120 psi, and the thickness is considered a fixed depth equal to 2 feet.
- Class II has a lower density of 30 pcf and subsequently lower strength material and used as fill material between the bottom of Class IV and the bottom of the excavation. The minimum compressive strength of this class is 40 psi.

Figure 2.16 presents a schematic of the compensating fill concept using the proposed lightweight foamed concrete so that the reconstructed I-95 mainline roadway does not impose additional weight on the underlying compressible soils and cause additional settlement. The existing fill will be excavated to a depth H_{exc} and the existing ground surface will be raised by ΔH to the proposed roadway profile grade using the two classes of lightweight foamed concrete. The proposed pavement section for the I-95 mainline roadway consists of a 17-in Plain Concrete Pavement layer, a 4-in Asphalt Treated Permeable Base, and a 10-in No. 2A Subbase aggregate layer. Tables 2.15 – 2.17 present the estimated H_{exc} and ΔH as a function of Station along the mainline I-95 roadway and the corresponding ramps assuming the existing fill material has a unit weight of 95 pcf. The shaded rows represent section of the proposed roadways where a compensating fill design alternative is not feasible because the excavation would need to be extended below the groundwater table. Figure 2.17 presents an example cross section for the proposed compensating fill design alternative based on the proposed dimensions defined in Tables 2.15 – 2.17. It was noted that the application of the compensating fill section would necessitate the construction of a temporary Ramp EE overlap with Ramp X, which supports low-level structure and transitions to a separate embankment located on the virgin ground between Ramp X and the proposed Ramp EE/F.

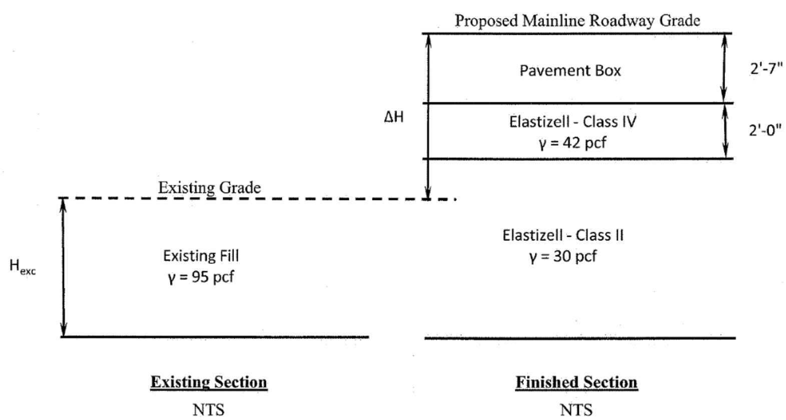


Figure 2.16. Example schematic of the compensating fill design alternative using lightweight foamed concrete.

Table 2.15. Compensating fill estimates for the I-95 mainline roadway for Section BRI.

I95 Mainline - BRI Earth Fill Section						
		Nominal				Class II
Sta	PGL	OGS	ΔH	H_{exc}	B.O.E	Thick.
	Elev.	Elev.	(feet)	(feet)	Elev.	(feet)
492	18.08	12.3	5.8	7.3	5.0	8.5
493	18.48	12.3	6.2	7.5	4.8	9.1
494	18.88	12.4	6.5	7.7	4.7	9.6
495	19.28	12.5	6.8	7.8	4.7	10.0
496	19.68	12.6	7.1	7.9	4.7	10.4
497	20.08	12.7	7.4	8.1	4.6	10.9
498	20.48	12.9	7.6	8.2	4.7	11.2
499	20.88	13.2	7.7	8.2	5.0	11.3
500	21.28	13.5	7.8	8.3	5.2	11.5
501	21.68	13.7	8.0	8.3	5.4	11.7
502	22.08	14.8	7.3	8.0	6.8	10.7
503	22.48	14.3	8.2	8.4	5.9	12.0
504	23.03	14.6	8.4	8.6	6.0	12.5
505	23.88	14.9	9.0	8.8	6.1	13.2
506	24.88	15.2	9.7	9.1	6.1	14.2
507	25.88	15.5	10.4	9.5	6.0	15.3
508	26.88	16.8	10.1	9.3	7.5	14.8
509	27.88	18.2	9.7	9.1	9.1	14.2
510	28.88	19.5	9.4	9.0	10.5	13.8
511	29.88	11.0	18.9	13.4	-2.4	27.7
512	30.88	10.8	20.1	13.9	-3.1	29.4

Table 2.16. Compensating fill estimates for the I-95 Ramp E-F and Ramp EE for Section BRI.

I95 BRI Earth Fill Section Ramps EF/EE						
		Nominal				Class II
Sta	PGL	OGS	ΔH	H_{exc}	B.O.E	Thick.
	Elev.	Elev.	(feet)	(feet)	Elev.	(feet)
EE93	23.10	17.9	5.2	6.7	11.2	7.6
EE94	21.67	16.8	4.9	6.5	10.3	7.0
EE95	20.54	19.1	1.4	4.9	14.2	2.0
EE96	20.36	16.7	3.7	6.0	10.7	5.3
EE97	20.91	17.2	3.7	6.0	11.2	5.4
EE98	21.55	18.0	3.6	5.9	12.1	5.1
EE99	22.34	18.9	3.4	5.8	13.1	4.9
EE100	23.22	20.1	3.1	5.7	14.4	4.5
EE101	23.25	21.3	2.0	5.2	16.1	2.8
EE102	21.80	21.2	0.6	4.5	16.7	0.8
EE103	20.17	11.6	8.6	8.2	3.4	12.4
EE104	18.54	10.9	7.6	7.8	3.1	11.1
EE105	16.91	9.0	7.9	7.9	1.1	11.5

Table 2.17. Compensating fill estimates for the I-95 Ramp E-F and Ramp F for Section BRI.

I95 BRI Earth Fill Section Ramps EF/F						
Sta	PGL Elev.	Nominal OGS Elev.	ΔH (feet)	H_{exc} (feet)	B.O.E Elev.	Class II Thick. (feet)
F808	24.37	16.8	7.6	7.8	9.0	11.0
F809	22.91	18.7	4.2	6.2	12.5	6.1
F810	21.45	17.5	4.0	6.1	11.4	5.7
F811	20.11	15.7	4.4	6.3	9.4	6.4
F812	19.67	14.8	4.9	6.5	8.3	7.0
F813	20.12	14.4	5.7	6.9	7.5	8.3
F814	20.70	13.4	7.3	7.6	5.8	10.6
F815	21.28	13.6	7.7	7.8	5.8	11.2
F816	21.86	14.6	7.3	7.6	7.0	10.5
F817	22.66	14.0	8.7	8.3	5.7	12.6
F818	24.41	16.1	8.3	8.1	8.0	12.1

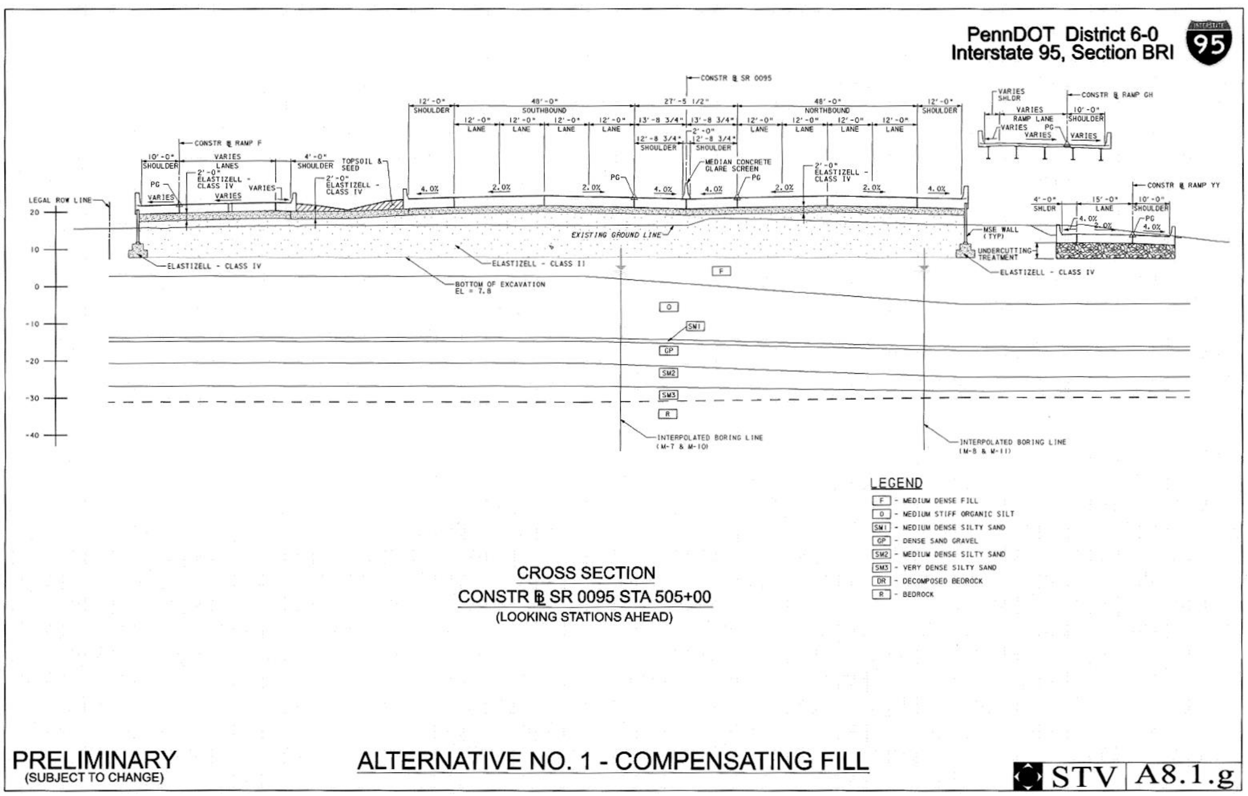


Figure 2.17. Example cross section at Station 505+00 for the Section BRI compensating fill design alternative.

2.1.2.2.2. Column Supported Embankment

For this proposed design alternative, the roadway embankment is supported on a geosynthetic-reinforced granular soil load transfer platform that rests on a series of vertical columns extending down to a more competent bearing layer below compressible soil. In this manner no new loads are imposed on the existing compressible soils at the site and no long-term primary consolidation settlements or secondary compression occur. Also, since the embankment and pavement structures are constructed to final grade, immediate elastic settlements are also avoided. The columns are typically constructed as either vibro-concrete columns (VCC) or controlled-modulus columns (CMC), though the CMC option can present potential proprietary issues since the requisite design and installation technology is patented by Menard USA. Therefore, for Section BRI, the VCC technology was recommended in addition to the use of steel HP12x53 piles or 12-inch square precast prestressed concrete piles.

Based on the proposed grade line (PGL) elevations, column supported embankments were determined to be feasible between approximately Station 492+00 and Station 510+00. Between these stations, excavation to a nominal 3 ft depth below the existing ground line will allow construction of a 3 ft load transfer platform while maintaining adequate fill height to develop full soil arching to transfer the entire embankment weight to the underlying columns (Figure 18). This design alternative results in less excavation than the compensating fill alternative and therefore prevents the bottom of the excavation from encountering the ground water table. Additionally, buoyancy is not an issue since the materials used for construction are not lightweight. The load transfer platform would include a minimum of three layers of internal geosynthetic reinforcement to stiffen the soil and develop beam-type response during load transfer. Based on preliminary analysis for the purpose of exploring the design alternatives for Section BRI, the load transfer platform was specified to be supported by 2 ft width/diameter columns and 8 ft center-to-center spacing in a square pattern. Figure 2.19 presents a typical proposed cross section for the same mainline roadway Station presented in Figure 2.17 for the compensating fill solution.

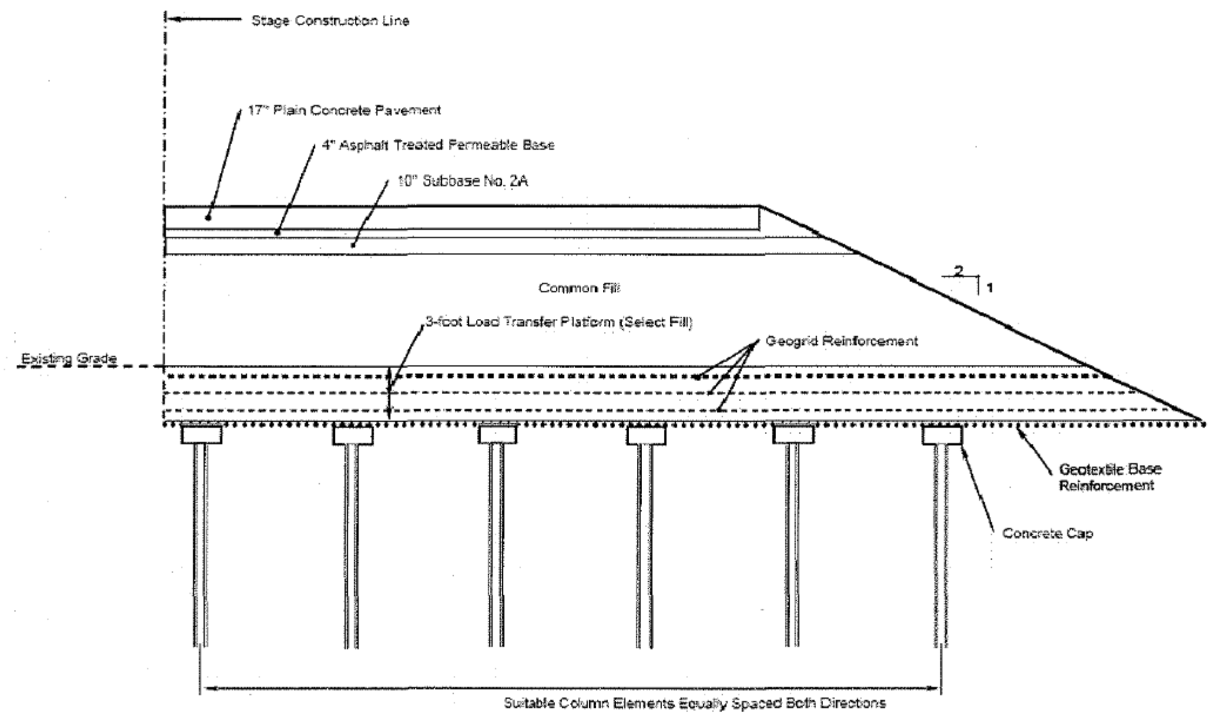


Figure 2.18. Example of a typical column supported roadway section as proposed for 0095, Section BRI.

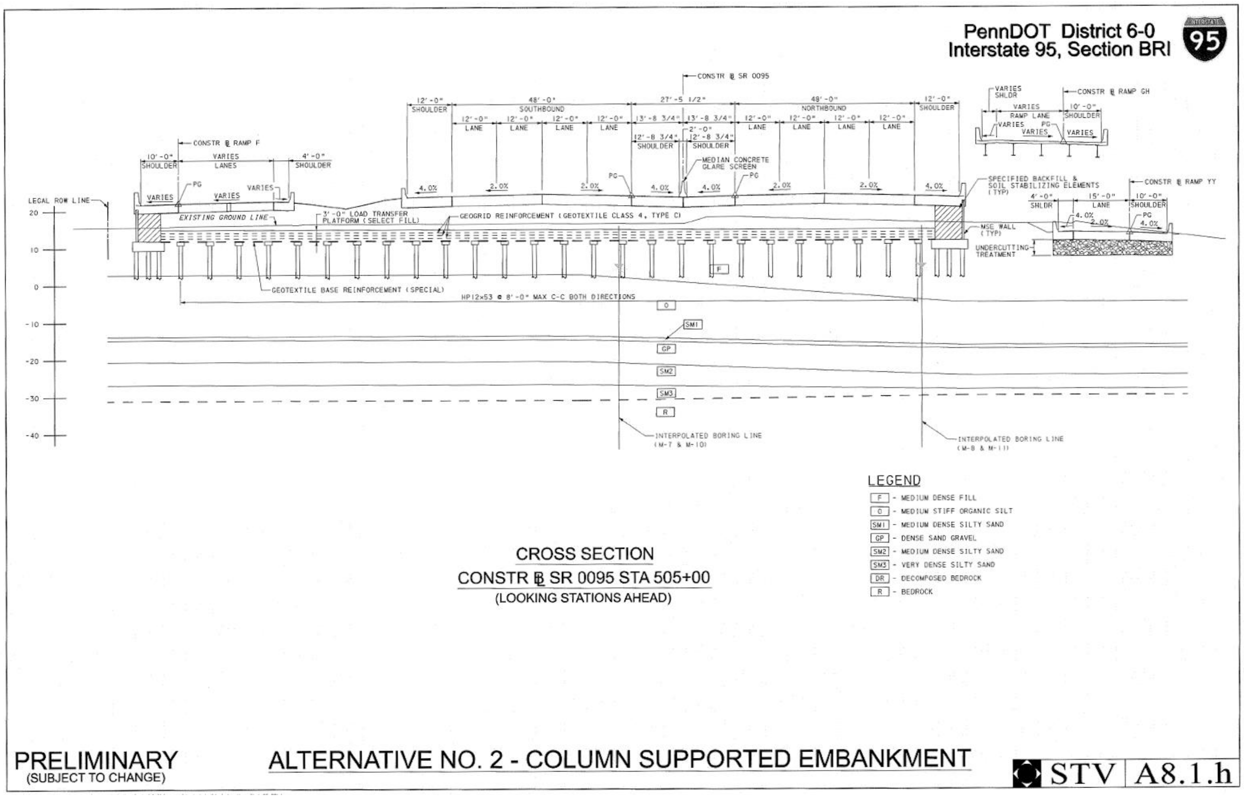


Figure 2.19. Example cross section at Station 505+00 for the Section BRI column supported embankment design alternative.

<u>2.1.2.3. Comparison of Design Alternative Costs</u>

As noted previously, a compensating fill approach and column supported embankments were determined to be the most feasible design alternatives given the constraints of Section BRI. To understand and evaluate the merits of each method, both alternatives were compared relative to technical and financial concerns as highlighted in Table 2.18. Both alternatives meet the preliminary objectives of the project to eliminate around 340,000 square feet of existing bridge deck while making a stable base for the widened and reconstructed roadway. Additionally, both design alternatives are estimated to cause no additional loading on the underlying compressible soil. As a result, they do not result in any significant settlement to the mainline roadway and associated ramps, though the column supported embankment alternative does carry with it a smaller overall risk for adverse performance with respect to settlements over the design life. However, the compensating fill design alternative results in simpler and faster construction with less potential for encountering unexpected field conditions. Ultimately, the compensating fill approach results in lower overall cost for the project as highlighted in Table 2.18 and further detailed in Table 2.19. The final cost evaluation estimates the overall costs of using compensating fill as $46.9 million. In contrast, the overall costs are higher for column supported embankments with either the 12-inch square precast prestressed concrete piles ($48.9 million) or the HP 12×53 steel piles ($55.6 million). These costs represent as much as half the overall costs of the Section BRI proposed efforts ($91.1 million). This highlights the importance of selecting a proper design alternative and the role of lightweight fill materials in potentially lowering construction costs.

Table 2.18. Comparison of the two design alternatives for 0095, Section BRI.

Alternative No. 1 Compensating Fill	**Alternative No. 2 Column Supported Embankment**
Sta. 492+59 to Sta. 510+70 & Ramps EE, F, E-F, & YY	Sta. 492+59 to Sta. 510+70 & Ramps EE, F, E-F, & YY
Design Life: 100 years	Design Life: 100 years
Construction Cost: $ 46.9 Million	Construction Cost: $ 48.9 Million w/12" Prestressed Piles [$55.6 Million w/HP12x53]
Advantages / Disadvantages:	**Advantages / Disadvantages:**
Eliminates 343,500 square feet of new bridge deck and associated life cycle maintenance costs from PennDOT Asset Management Program.	Eliminates 343,500 square feet of new bridge deck and associated life cycle maintenance costs from PennDOT Asset Management Program.
No additional load imposed on foundation soils.	No additional load imposed on foundation soils.
Immediate settlement compensated during construction. No additional primary consolidation settlement induced. Completed roadway is susceptible to minor ongoing secondary consolidation of underlying normally consolidated compressible soils due to existing fill estimated at less than one inch over 40 to 50 year pavement life cycle. Impact on rideability is minimal.	Immediate settlement compensated during construction. No additional primary consolidation settlement induced. Completed roadway is not susceptible to ongoing secondary consolidation of underlying normally consolidated compressible soils. No impact on rideability.
Requires sampling & testing of excavation spoils for proper disposal.	Requires sampling & testing of excavation spoils for proper disposal.
Excavate existing fill to 8.5-foot depth (typical).	Excavate existing fill to 3-foot depth (typical).
Does not require excavation dewatering.	Does not require excavation dewatering.
Proper excavation, characterization, transportation and disposal means and methods will need to be employed by the contractor to manage this material in compliance to PADEP regulations.	Proper excavation, characterization, transportation and disposal means and methods will need to be employed by the contractor to manage this material in compliance to PADEP regulations.
(continued on next page)	(continued on next page)

Table 2.18 (cont.). Comparison of the two design alternatives for 0095, Section BRI.

Alternative No. 1 Compensating Fill	**Alternative No. 2 Column Supported Embankment**
No additional ground improvement required.	Requires driving of 391,000 LF of steel H or precast prestressed concrete piles to end bearing on bedrock, provided with precast caps for better load bearing. Potential for shallow obstructions.
Fill consists of 215,000 CY of lightweight foamed concrete placed in two-foot lifts using a foaming agent.	Fill consists of common and select earth fill with geosynthetic reinforcement of base platform.
Fill placement with familiar concrete placement means & methods, with expert guidance of foaming agent manufacturer.	Fill placement with conventional earthwork means and methods, with expert guidance of geosynthetic manufacturer.
Vertical faces self supporting and provided with precast MSEW facing panels and steel strip reinforcement that also serve as forms.	Vertical face along outside edges of roadway requires MSEW. Temporary vertical face for first construction stage requires geosynthetic wrapped face construction.
Roadway utility trenches can be excavated using conventional equipment and backfilled with lightweight foamed concrete.	Roadway utilities constructed conventionally within common fill volume.
Completed construction susceptible to localized seismically induced settlement during a seismic event. The use of lightweight foamed concrete fill results in better performance than comparable roadway earth embankments in immediate project vicinity.	Piles provide ductile support in transferring roadway embankment loads to bedrock during a seismic event, and must be designed for potential additional downdrag loads.

Table 2.19. Detailed estimate of costs associated with the 0095, Section BRI Compensating Fill design alternative.

PRELIMINARY GEOTECHNICAL ENGINEERING REPORT COST ESTIMATE
ALTERNATIVE 1
COMPENSATING FILL

AREA 1: I-95 NB & I-95 SB STA 492+59 TO STA 500+00 ON STABILIZED EARTH					
CLASS 1 EXCAVATION, SPECIAL	57,435	CY @	$75.00 /CY	=	$4,307,605
FOREIGN BORROW EXCAVATION	2,937	CY @	$15.00 /CY	=	$44,055
ELASTIZELL CLASS II	59,179	CY @	$65.00 /CY	=	$3,846,617
ELASTIZELL CLASS IV	9,995	CY @	$65.00 /CY	=	$649,675
CONCRETE GLARE SCREEN	741	LF @	$60.00 /LF	=	$44,460
CONCRETE SINGLE FACE BARRIER	1,440	LF @	$55.00 /LF	=	$79,200
17" CONCRETE PAVEMENT W 4" STAB BASE & 10" SUBBASE	13,525	SY @	$130.00 /SY	=	$1,758,250
TEMPORARY EXCAVATION AND SUPPORT SYSTEM	1	LS @	$270,000.00 /LS	=	$270,000
			SUB-TOTAL	=	*$10,999,862*

AREA 2: I-95 NB & I-95 SB STA 500+00 TO STA 510+00 ON STABILIZED EARTH					
CLASS 1 EXCAVATION, SPECIAL	100,582	CY @	$75.00 /CY	=	$7,543,679
FOREIGN BORROW EXCAVATION	8,590	CY @	$15.00 /CY	=	$128,850
ELASTIZELL CLASS II	98,840	CY @	$65.00 /CY	=	$6,424,625
ELASTIZELL CLASS IV	14,866	CY @	$65.00 /CY	=	$966,290
CONCRETE GLARE SCREEN	1,000	LF @	$60.00 /LF	=	$60,000
CONCRETE SINGLE FACE BARRIER	1,986	LF @	$55.00 /LF	=	$109,230
17" CONCRETE PAVEMENT W 4" STAB BASE & 10" SUBBASE	17,596	SY @	$130.00 /SY	=	$2,287,480
TEMPORARY EXCAVATION AND SUPPORT SYSTEM	1	LS @	$360,000.00 /LS	=	$360,000
			SUB-TOTAL	=	*$17,880,154*

AREA 3: RAMP EE & RAMP F ON STABILIZED EARTH					
CLASS 1 EXCAVATION, SPECIAL	38,030	CY @	$75.00 /CY	=	$2,852,223
FOREIGN BORROW EXCAVATION	3,016	CY @	$15.00 /CY	=	$45,240
ELASTIZELL CLASS II	42,807	CY @	$65.00 /CY	=	$2,782,431
ELASTIZELL CLASS IV	6,675	CY @	$65.00 /CY	=	$433,875
CONCRETE SINGLE FACE BARRIER	2,819	LF @	$55.00 /LF	=	$155,045
16" CONCRETE PAVEMENT W 4" STAB BASE & 8" SUBBASE	7,045	SY @	$120.00 /SY	=	$845,400
			SUB-TOTAL	=	*$7,114,214*

AREA 4: RAMP YY ON STABILIZED EARTH/UNDERCUTTING TREATMENT					
CLASS 1 EXCAVATION, SPECIAL	25,297	CY @	$75.00 /CY	=	$1,897,239
FOREIGN BORROW EXCAVATION	10,580	CY @	$15.00 /CY	=	$158,700
ELASTIZELL CLASS II	6,289	CY @	$65.00 /CY	=	$408,754
ELASTIZELL CLASS IV	921	CY @	$65.00 /CY	=	$59,865
CONCRETE SINGLE FACE BARRIER	3,878	LF @	$55.00 /LF	=	$213,290
16" CONCRETE PAVEMENT W 4" STAB BASE & 8" SUBBASE	7,346	SY @	$120.00 /SY	=	$881,520
			SUB-TOTAL	=	*$3,619,369*

RETAINING WALL A (BETWEEN MAINLINE AND RAMP YY)					
MSE/T-WALL RETAINING WALL	18,510	SF @	$30.00 /SF	=	$555,300
CLASS A CEMENT CONCRETE	12	CY @	$800.00 /CY	=	$9,600
			SUB-TOTAL	=	*$564,900*

RETAINING WALL C (ALONG RAMP F)					
MSE/T-WALL RETAINING WALL	19,830	SF @	$30.00 /SF	=	$594,900
CLASS A CEMENT CONCRETE	23	CY @	$800.00 /CY	=	$18,400
			SUB-TOTAL	=	*$613,300*

TOTAL	=	$40,791,798
15% CONTINGENCY	=	$6,118,770
GRAND TOTAL	=	**$46,920,000**

Table 2.20. Detailed estimate of costs associated with the 0095, Section BRI Column Supported Embankment design alternative.

PRELIMINARY GEOTECHNICAL ENGINEERING REPORT COST ESTIMATE
ALTERNATIVE 2
COLUMN SUPPORTED EMBANKMENT (STEEL H-PILES)

AREA 1: I-95 NB & I-95 SB STA 492+59 TO STA 500+00 ON STABILIZED EARTH

CLASS 1 EXCAVATION, SPECIAL	28,994	CY	@	$75.00 /CY	=	$2,174,550.00
FOREIGN BORROW EXCAVATION	27,283	CY	@	$15.00 /CY	=	$409,245.00
SELECTED MATERIAL BACKFILL	17,159	CY	@	$40.00 /CY	=	$686,360.00
CONCRETE GLARE SCREEN	741	LF	@	$60.00 /LF	=	$44,460.00
CONCRETE SINGLE FACE BARRIER	1,440	LF	@	$55.00 /LF	=	$79,200.00
17" CONCRETE PAVEMENT W 4" STAB BASE & 10" SUBBASE	13,525	SY	@	$130.00 /SY	=	$1,758,250.00
GEOTEXTILE BASE REINFORCEMENT (SPECIAL)	18,414	SY	@	$8.00 /SY	=	$147,312
GEOGRID REINFORCEMENT (CLASS 4, TYPE C GEOTEXTILE)	55242	SY	@	$4.00 /SY	=	$220,968
STEEL BEAM TEST PILES, HP 12x53	39	EA	@	$5,000.00 /EA	=	$195,000
STEEL BEAM BEARING PILES	119,539	LF	@	$54.00 /LF	=	$6,455,106
STEEL BEAM PILE TIP REINFORCEMENT, HP 12 x 53	2,267	EA	@	$150.00 /EA	=	$340,050
PRECAST PILE CAPS	2,267	EA	@	$300.00 /EA	=	$680,100
TEMPORARY EXCAVATION AND SUPPORT SYSTEM	1	LS	@	$112,500.00 /LS	=	$112,500
				SUB-TOTAL	=	*$13,303,101*

AREA 2: I-95 NB & I-95 SB STA 500+00 TO STA 510+00 ON STABILIZED EARTH

CLASS 1 EXCAVATION, SPECIAL	46,461	CY	@	$75.00 /CY	=	$3,484,575
FOREIGN BORROW EXCAVATION	45,729	CY	@	$15.00 /CY	=	$685,935
SELECTED MATERIAL BACKFILL	22,655	CY	@	$40.00 /CY	=	$906,200
CONCRETE GLARE SCREEN	1,000	LF	@	$60.00 /LF	=	$60,000
CONCRETE SINGLE FACE BARRIER	1,986	LF	@	$55.00 /LF	=	$109,230
17" CONCRETE PAVEMENT W 4" STAB BASE & 10" SUBBASE	17,596	SY	@	$130.00 /SY	=	$2,287,480
GEOTEXTILE BASE REINFORCEMENT (SPECIAL)	24,794	SY	@	$8.00 /SY	=	$198,352
GEOGRID REINFORCEMENT (CLASS 4, TYPE C GEOTEXTILE)	74382	SY	@	$4.00 /SY	=	$297,528
STEEL BEAM TEST PILES, HP 12x53	52	EA	@	$5,000.00 /EA	=	$260,000
STEEL BEAM BEARING PILES	138,707	LF	@	$54.00 /LF	=	$7,490,178
STEEL BEAM PILE TIP REINFORCEMENT, HP 12 x 53	2,894	EA	@	$150.00 /EA	=	$434,100
PRECAST PILE CAPS	2,894	EA	@	$300.00 /EA	=	$868,200
TEMPORARY EXCAVATION AND SUPPORT SYSTEM	1	LS	@	$150,000.00 /LS	=	$150,000
				SUB-TOTAL	=	*$17,231,778*

AREA 3: RAMP EE & RAMP F ON STABILIZED EARTH

CLASS 1 EXCAVATION, SPECIAL	19,513	CY	@	$75.00 /CY	=	$1,463,475
FOREIGN BORROW EXCAVATION	21,824	CY	@	$15.00 /CY	=	$327,360
SELECTED MATERIAL BACKFILL	10,766	CY	@	$40.00 /CY	=	$430,640
CONCRETE SINGLE FACE BARRIER	2,819	LF	@	$55.00 /LF	=	$155,045
16" CONCRETE PAVEMENT W 4" STAB BASE & 8" SUBBASE	7,045	SY	@	$120.00 /SY	=	$845,400
GEOTEXTILE BASE REINFORCEMENT (SPECIAL)	11,859	SY	@	$8.00 /SY	=	$94,872
GEOGRID REINFORCEMENT (CLASS 4, TYPE C GEOTEXTILE)	35577	SY	@	$4.00 /SY	=	$142,308
STEEL BEAM TEST PILES, HP 12x53	27	EA	@	$5,000.00 /EA	=	$135,000
STEEL BEAM BEARING PILES	81,272	LF	@	$54.00 /LF	=	$4,388,688
STEEL BEAM PILE TIP REINFORCEMENT, HP 12 x 53	1,456	EA	@	$150.00 /EA	=	$218,400
PRECAST PILE CAPS	1,456	EA	@	$300.00 /EA	=	$436,800
				SUB-TOTAL	=	*$8,637,988*

Table 2.20 (cont.). Detailed estimate of costs associated with the 0095, Section BRI Column Supported Embankment design alternative.

AREA 4: RAMP YY ON STABILIZED EARTH/UNDERCUTTING TREATMENT							
CLASS 1 EXCAVATION, SPECIAL	22,525	CY	@	$75.00	/CY	=	$1,689,375
FOREIGN BORROW EXCAVATION	13,519	CY	@	$15.00	/CY	=	$202,785
SELECTED MATERIAL BACKFILL	1,317	CY	@	$40.00	/CY	=	$52,680
CONCRETE SINGLE FACE BARRIER	3,878	LF	@	$55.00	/LF	=	$213,290
16" CONCRETE PAVEMENT W 4" STAB BASE & 8" SUBBASE	7,346	SY	@	$120.00	/SY	=	$881,520
GEOTEXTILE BASE REINFORCEMENT (SPECIAL)	1,603	SY	@	$8.00	/SY	=	$12,824
GEOGRID REINFORCEMENT (CLASS 4, TYPE C GEOTEXTILE)	4809	SY	@	$4.00	/SY	=	$19,236
STEEL BEAM TEST PILES, HP 12x53	9	EA	@	$5,000.00	/EA	=	$45,000
STEEL BEAM BEARING PILES	20,328	LF	@	$54.00	/LF	=	$1,097,712
STEEL BEAM PILE TIP REINFORCEMENT, HP 12 x 53	484	EA	@	$150.00	/EA	=	$72,600
PRECAST PILE CAPS	484	EA	@	$300.00	/EA	=	$145,200
				SUB-TOTAL		=	*$4,432,222*

RETAINING WALL A (BETWEEN MAINLINE AND RAMP YY)							
MSE/T-WALL RETAINING WALL	18,510	SF	@	$30.00	/SF	=	$555,300
STEEL BEAM TEST PILES, HP 12x53	6	EA	@	$5,000.00	/EA	=	$30,000
STEEL BEAM BEARING PILES	9,793	LF	@	$54.00	/LF	=	$528,822
STEEL BEAM PILE TIP REINFORCEMENT, HP 12 x 53	228	EA	@	$150.00	/EA	=	$34,200
CLASS A CEMENT CONCRETE	483	CY	@	$800.00	/CY	=	$386,400
REINFORCEMENT BARS	72,334	LB	@	$1.35	/LB	=	$97,651
SPECIFIED BACKFILL	2,397	CY	@	$40.00	/CY	=	$95,880
				SUB-TOTAL		=	*$1,728,253*

RETAINING WALL C (ALONG RAMP F)							
MSE/T-WALL RETAINING WALL	19,830	SF	@	$30.00	/SF	=	$594,900
STEEL BEAM TEST PILES, HP 12x53	12	EA	@	$5,000.00	/EA	=	$60,000
STEEL BEAM BEARING PILES	21,284	LF	@	$54.00	/LF	=	$1,149,336
STEEL BEAM PILE TIP REINFORCEMENT, HP 12 x 53	456	EA	@	$150.00	/EA	=	$68,400
CLASS A CEMENT CONCRETE	959	CY	@	$800.00	/CY	=	$767,200
REINFORCEMENT BARS	143,792	LB	@	$1.35	/LB	=	$194,119
SPECIFIED BACKFILL	4,941	CY	@	$40.00	/CY	=	$197,640
				SUB-TOTAL		=	*$3,031,595*
				TOTAL		=	$48,364,937
				15% CONTINGENCY		=	$7,254,741
				GRAND TOTAL		=	**$55,620,000**

2.1.3. Project S.R. 0119, Section 559 - Indiana Hill Bridge Borough of Punxsutawney

S.R. 0119, Section 559 is a roadway improvement project located in Jefferson County along S.R. 0119 in the borough of Punxsutawney, PA. The efforts associated with this project include:

- Removal of an existing three-span steel beam bridge over the Mahoning Shadow Trail
- Widening of an existing embankment to accommodate extension of an existing truck-climbing lane
- Full-height embankment placement to replace the existing three-span steel beam bridge
- Realignment of the Mahoning Shadow Trail and construction of an emergency access road
- Placement of a pedestrian box culvert (10'-6" in height and 12'-0" in width)
- Addition of a climbing lane on S.R. 0119 South

The embankment widening would extend the existing lane for truck-climbing and become part of the new roadway after removing the existing structure. Since the soil beneath the existing structure is soft compressible soil, embankment placement will be divided into two phases (Phase 1 and 2). The pedestrian box culvert under S.R. 0119 will maintain pedestrian traffic through the new location of the Mahoning Shadow Trail on the western side of the project perpendicular to S.R. 0119. The proposed access road will serve as an emergency road along the western side of the newly widened section of the embankment to carry the traffic load from the western side of S.R. 0119 to the relocated pedestrian trail culvert under S.R. 0119. Table 2.21 summarizes the proposed construction efforts for S.R. 0119, Section 559.

Table 2.21. Proposed construction efforts for S.R. 0119, Section 559 project.

Roadway Baseline	Stationing (Work Limits ±)	Proposed Construction	Length (feet)
S.R. 0119	133+00 to 135+50 141+40 to 146+40	Milling and Overlay	800
	135+50 to 141+40	Full-depth Pavement Reconstruction	590
	135+10 to 136+70 138+90 to 145+00	Embankment Widening	777
	136+70 to 138+90	Full Embankment Placement	220
Mahoning Shadow Trail	0+00 to 10+94	Trail Relocation Full Embankment Placement	1,094
Access Road	10+05 to 14+50	Full Embankment Placement Pavement Construction	445
	10+05 to 15+49	Cut/Fill Grading	99

2.1.3.1. Geotechnical Considerations

As noted in Table 2.21, much of the efforts for the project includes widening of the existing embankment and construction of a proposed full-width embankment to replace an existing bridge structure. There are some factors that may cause some geotechnical issues in this project. Based on subsurface investigation, the presence of alluvial clay with low blow counts may cause some serious stability issues. Moreover, lab results indicated very high silt/clay content potentially indicating wetland deposits below the proposed embankment construction. All of these factors led to concerns with a solution that would utilize conventional fill material. Consequently, various design alternatives were considered, including various rock toe benches, use of ground improvement with jet grout columns, construction of a retaining wall, construction of drilled shafts to pin the slopes, and lightweight fill materials (EPS Geofoam blocks) to decrease embankments loads on the soft soils at the base of the fill.

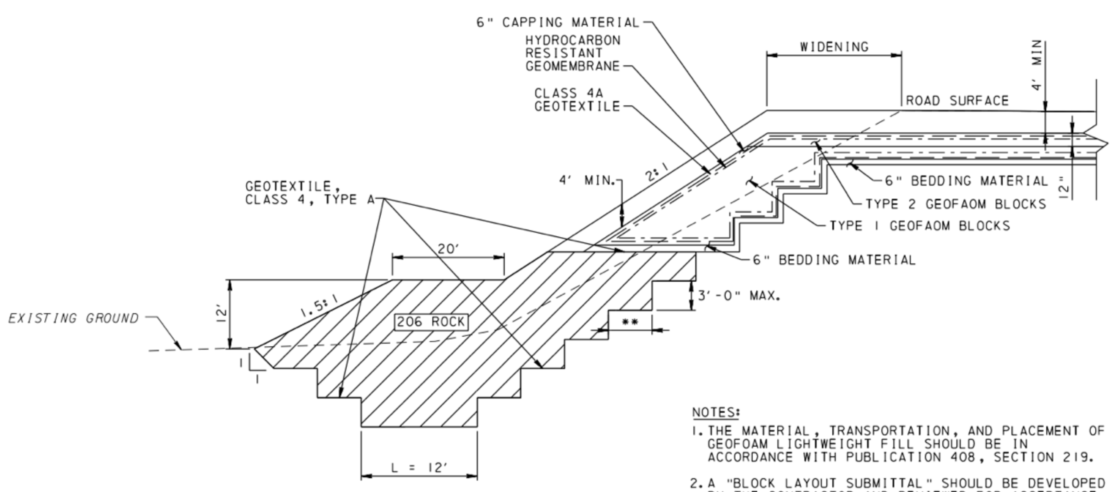
After careful consideration of these alternatives, it was recommended that embankment construction proceed to use random embankment fill material, 206 Rock, and EPS Geofoam blocks. Benching will be required to construct all widened embankments with the locations prescribed in Table 2.22. The slopes of the random embankment material are 2H:1V, while in the buttress area where Geofoam is used, the slopes will be constructed at a maximum 1½H:1V up to a maximum height of 12 feet. Geofoam was specifically prescribed for construction of the widened embankment from station 135+20 to 136+90 (Left Offset) due to the presence of soft clay as determined from the subsurface investigations. Additionally, analysis using the GSTABL7 software led to concerns regarding long-term slope stability if conventional fill materials were used. Figure 2.20 shows the details of a typical embankment cross section using Geofoam.

Table 2.22. Summary of recommended fill bench locations for S.R. 0119, Section 559 project.

Roadway Baseline	Approximate Station	Offset	Proposed Construction[1]	Detail Type
S.R. 0119	135+20 to 136+90	Left	1½:1 Rock Buttress 2:1 Embankment Widening	Geofoam
	136+10 to 136+65	Right	2:1 Embankment Widening	Sliver Fills < 12' High
	136+65 to 137+80	Right	2:1 Embankment Fill	Side Hill Toe Bench
	136+90 to 137+50	Left	2:1 Embankment Fill	Sliver Fills > 12' High, Modified
	138+20 to 140+15	Left/Right	2:1 Embankment Fill/ Widening	Sliver Fills > 12' High, Modified
	140+15 to 144+30	Left	2:1 Embankment Widening	Sliver Fills > 12' High, Modified
	144+55 to 145+10	Left	2:1 Embankment Widening	Sliver Fills < 12' High
Access Road	14+25 to 14+75	Right	2:1 Embankment Widening	Sliver Fills < 12' High

As previously highlighted, to control and stabilize the consolidation and settlement, the embankment will be constructed in two phases between approximate Stations 137+00 to 138+50.

Phase 1 was chosen for the area with the maximum height of embankment and under the existing structure. Phase 1 involves placing embankment fill under the existing structure and around the bridge piers. The height of the embankment is estimated as 22 ft based on the overhead clearance so that construction equipment will be operated and moved quickly. This would place the top of the embankment fill approximately 15 feet below the bridge beams. To complete Phase 1, the embankment should be allowed to settle until the cohesive soils below the embankment reach 90% consolidation (approximately 2½ to 4½ months). After completing the Phase 1 consolidation, the road can be closed and a detour put in place, the bridge removed, and the embankment completed to the proposed grade (Phase 2). After the embankment is then constructed to grade in Phase 2, the embankment and underlying soils should be allowed to settle until 1 inch of additional consolidation settlement occurs (approximately 1 to 2½ months). To reduce these times, it is recommended that the Contractor surcharge the embankment by overbuilding the embankment height by 5 to 10 feet above the proposed grade. Settlement monitoring plates are to be installed to monitor the deformation and consolidation during embankment placement and after the embankment has been constructed to grade and surcharged.



GEOFOAM DETAIL

NOT TO SCALE
AS SHOWN ON CROSS SECTIONS

ROADWAY BASELINE	STATION	OFFSET	PROPOSED SLOPE (H:V)	MINIMUM TOE WIDTH (FEET)
SR 0119	135+20 TO 136+90	LEFT	1½:1 2:1	12

NOTES:

1. THE MATERIAL, TRANSPORTATION, AND PLACEMENT OF GEOFOAM LIGHTWEIGHT FILL SHOULD BE IN ACCORDANCE WITH PUBLICATION 408, SECTION 219.
2. A "BLOCK LAYOUT SUBMITTAL" SHOULD BE DEVELOPED BY THE CONTRACTOR AND REVIEWED FOR ACCEPTANCE BY THE REPRESENTATIVE PRIOR TO CONSTRUCTION.
3. BEFORE PLACEMENT OF FIRST LAYER OF BLOCKS, PLACE A LEVELING COURSE CONSISTING OF 6-INCH LOOSE LAYER OF BEDDING MATERIAL.
4. FULLY ENCAPSULATE ALL BLOCKS IN ONE CONTINUOUS LAYER OF HYDROCARBON RESISTANT GEOMEMBRANE.
5. FULLY COVER THE GEOMEMBRANE WITH CLASS 4, TYPE A GEOTEXTILE.
6. A 6-INCH MINIMUM LAYER (HORIZONTAL AND VERTICAL) OF CAPPING MATERIAL SHOULD BE PLACED ON TOP OF THE GEOTEXTILE.
7. COVER BLOCKS WITH A MINIMUM OF 4 FEET OF MATERIAL BUT NOT EXCEED 6 FEET VERTICALLY AND HORIZONTALLY. TOTAL COVER THICKNESS INCLUDES EMBANKMENT AND CAPPING MATERIAL.
8. ROCK USED TO CONSTRUCT EMBANKMENT MUST MEET SECTION 206.2(a) 1.d. IN ADDITION, IT MUST BE BLASTED SANDSTONE. IT MUST NOT CONTAIN SILTSTONE, CLAYSTONE, OR SHALE.

Figure 2.20. Typical embankment cross section with Geofoam details for S.R. 0119, Section 559.

2.1.3.2. Costs of Lightweight Fill Materials

The EPS Geofoam blocks that were used for the embankments in the S.R. 0119, Section 559 project were of two types. Type 1 was used as the main embankment fill blocks and were $100 per cubic yard. The Type 2 blocks were denser and used directly under the roadway to ensure durability from the traffic live loads. The Type 2 blocks were more expensive at $160 per cubic yard. Based on the total cubic yardage of EPS geofoam blocks, the approximate total material costs were $463,000. However, since EPS Geofoam is damaged when exposed to petroleum solvents (gasoline, diesel, etc.) use of a geomembrane was necessary to protect the blocks. The unit price of the geomembrane used on S.R. 0119, Section 559 was $65 per square foot. Based on the total square yardage needed to cover the EPS Geofoam blocks, there was an additional $275,000 in costs associated with this lightweight fill technology for this project. Consequently, the overall lightweight fill material costs amounted to approximately $738,000, which was about 18% of the overall project costs of $4M.

2.2. Summary

Included in this chapter are three projects that serve as example case histories and highlight the use of lightweight fill technologies in PennDOT projects. One of the case histories (0095, Section GR2) was a VE project where the lightweight fill alternate design was shown to reduce overall costs when compared to the originally proposed ground improvement design. The proposed lightweight concrete fill was used as backfill behind multiple retaining walls and as an undercut fill below some of the walls to reduce the structural loads imposed on the underlying compressible soils. The second case history (0095, Section BRI) considered the use lightweight foamed concrete (e.g., Elastizell) as a potential design alternative to support the widening of I-95 and associated ramps and compared it to a column supported embankment design. Both designs were deemed suitable for meeting the desired technical constraints imposed by the site conditions and proposed structures, but the lightweight foamed concrete design was estimated to reduce the overall project costs. The final case history (S.R. 0119, Section 559) primarily consisted of embankment widening and placement efforts at a site with poor subsurface soils and nearby wetlands conditions near the toe of the embankment slopes. The use of lightweight EPS Geofoam blocks was incorporated into the design after consideration of multiple other design alternatives. The reduced loads from the blocks improved the stability of the embankment slopes and resulted in predicted settlements within tolerable limits. The overall costs of materials associated with the EPS Geofoam design was increased by the need for a geomembrane cover for the blocks to prevent damage from petroleum solvents. Additionally, the geomembrane was heavy, labor intensive, and difficult to bend around the EPS Geofoam blocks.

Though there were not many projects for which information was sufficient to document as a case history in this chapter, the diverse nature of these projects allows for a few conclusions to be drawn. First, though the project applications were varied (retaining wall backfill, undercut fill, embankment fill), the common theme that led to the consideration of lightweight fill materials was the presence of problematic soils prone to excessive settlement and/or loss of stability. Second, the overall costs for lightweight fill materials can indeed be quite high relative to conventional fill materials. However, the first two case histories demonstrated that changes to the overall design afforded by the use of lightweight fill materials can recuperate costs in other areas (e.g., eliminating the need for expensive ground improvement or pile foundations, reductions in overall construction timeline, etc.). Consequently, the alternate lightweight fill designs proved cost-effective. Finally, the use of lightweight fill materials can potentially introduce additional concerns that may increase the material costs because other materials become necessary in the design. For example, the S.R. 0119, Section 559 project necessitated additional geosynthetics to protect the lightweight fill technology employed in that particular application. Therefore, consideration of designs that incorporate lightweight fill materials should anticipate the needs for either additional material costs or construction efforts that can potentially negate their cost effectiveness.

3. NON-PENNDOT CASE HISTORIES WITH LIGHTWEIGHT/SUSTAINABLE FILLS

The focus of this chapter is to summarize case histories from projects associated with agencies outside of PennDOT. Wherever available, the summaries of these non-PennDOT case histories include a discussion on project costs in addition to the geotechnical and design considerations.

3.1. Selected Projects

These case histories were obtained during literature review and by communicating with various lightweight/sustainable fill material stakeholders, including manufacturers, installation contractors, and governing agencies (e.g., other state DOTs). Additionally, these selected projects include multiple lightweight and sustainable fill material technologies.

3.1.1. I-15, Salt Lake City

Highway 15 (I-15) in the Salt Lake Valley was initially constructed from 1998 to 2001. Its preliminary capacity has since become insufficient for the growing population and traffic flow. Hence, the Utah Department of Transportation (UDOT) started the process of reconstruction of Interstate I-15 by widening a 26 km section of this highway from station 600 North to 10600 South, including the reconstruction of 144 bridges and 160 mechanically stabilized earth (MSE) retaining walls. The primary purpose of widening the current highway was adding a general purpose lane, a high occupancy vehicle (HOV) lane, and an auxiliary lane between ramps on both the north and southbound sides of the interstate during 3.5 years.

Figure 3.1. I-15 Alignment and Geofoam Placement Areas in Salt Lake City.

<u>*3.1.1.1. Geotechnical Considerations*</u>

Widening the I-15 alignment required the construction of large embankments with a height of 8 m to 10 m. However, there were some factors that may cause some geotechnical issues for constructing this volume of embankment. Based on the subsurface investigation, the foundation soils consisted of soft clay that has the potential to produce preliminary consolidation settlement exceeding 1 m at many locations. Moreover, the utilities that existed beneath the freeway must be relocated to avoid any damage due to settlement caused by the new embankment construction. All of these factors resulted in many geotechnical and economic concerns with potentially using conventional fill material. As a result, expanded polystyrene (EPS) geofoam was chosen to construct the embankments. This extremely lightweight material allowed the existing utilities to remain in service without the cost of relocation and avoided the excessive settlement due to the high unit weight of conventional fill materials.

The typical cross-section of geofoam embankments was made of multiple layers, as shown in Figure 3.2. The first layer from the bottom was a minimum of 0.3 m of base sand that was graded and leveled for the placement of geofoam blocks. The next layer was made of the preexisting granular embankment, graded at a 1.5H:1V (33.7 degrees) backslope adjacent to geofoam blocks with 0.82-m high, 1.2-m wide, and 4.9-m long. The third layer consisted of a reinforced concrete load distribution slab with 0.150-m thickness, used to protect the geofoam from local overstressing. The fourth and last layers were 0.61 m untreated pavement base course and a 0.35 m unreinforced Portland cement concrete pavement.

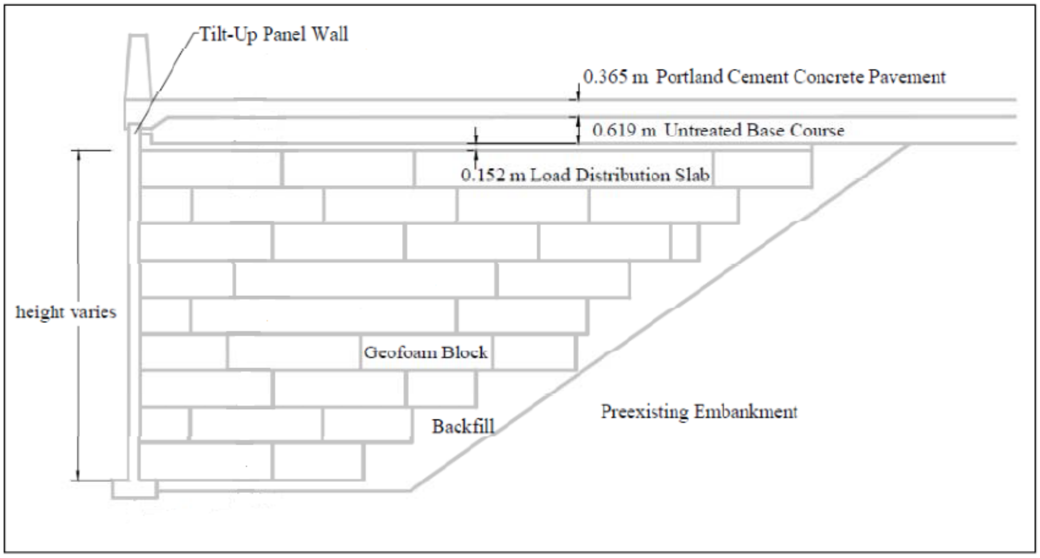


Figure 3.2. Typical Cross-Sectional View of EPS blocks for I-15 Alignment.

As the major fill material of the embankments, it was necessary for the geofoam blocks to have specific physical properties to resist vertical stresses from the live load and dead load of traffic applied on Portland cement concrete pavement in addition to lateral deformation caused by soil weight and seismic loads. As a result, a design method was required to find the best EPS type for the project. For this purpose, both Type VIII and Type II geofoam were chosen based on ASTM C-578 described in Bartlett et al. (2000). Although both EPS types have been accepted for this project, Type VIII was selected as the approved type due to lower density than other types. Table 3.1 describes the typical EPS properties extracted from ASTM-C-578-95 mentioned in Bartlett et al. (2000). Table 3.1 includes the value of density, compressive resistance (at 10% compressive strain), flexural strength, and water absorption of both Type VIII and Type II geofoam in addition to their ASTM test procedure.

Table 3.1. Typical EPS Properties from ASTM-C-578-95 (from Bartlett et al. 2000).

Physical Property	**ASTM Test Procedure**	**Type VIII Accepted Value**	**Type II Accepted Value**

Density	D1622	18 kg/m^3	22 kg/m^3
Compressive Resistance	D1621	90 kN/m^3	104 kN/m^2
Flexural Strength	C203	208 kN/m^2	276 kN/m^2
Water Absorption	C272	3	3

As part of the design process, different researchers presented limitation for stress and strain criteria applied to geofoam embankments. Bartlett et al. (2000) talked about stress limitation caused by traffic applied to EPS embankment. They claimed that, in order to limit long-term creep deformation, the total of dead load and live load applied to the geofoam blocks must be restricted. For this purpose, the stress value due to the self-weight of overlying material as dead load must be limited to 30 percent of compressive resistance of Type VIII geofoam, while the allowable live traffic load must not be greater than 10 percent of compressive resistance. Hence, the total stress applied on geofoam blocks due to the load combination of dead and live load could not exceed 40 percent of the compressive resistance.

On the other hand, Farnsworth et al. (2008) considered strain limitation so that the long-term settlement of up to 75-mm or less was the expected and permitted value in a 10-year post-construction period for the foundation settlement. Additionally, in terms of construction and post-construction global strain, two limit criteria were set for two periods of time; the end of construction and over 50 years. In this way, a limit of 1 percent strain was designed for the end of the construction, while 2 percent strain was considered for over a 50 year period.

After choosing the appropriate EPS for the embankments and determining the design criterion for them in terms of maximum allowable stress and creep strain, it was recommended to monitor the construction and post-construction performance of this technology and compare them with the design criterion. EPS blocks were located as chosen fill material for several embankments through I-15; however, two EPS embankment locales: (1) 100 South Street, (2) 3300 South Street, were focused as typical cross section examples of this study. For the 100 South Street site, EPS blocks were used to minimize the settlement applied to utilities and soft clay foundation beneath the embankments, while the purpose of using geofoam for the site location at 3300 South Street was improving the global stability of high approach fill at a rail road crossing. As a result, some instruments were required to be installed to monitor the behavior of the sections, including geofoam blocks. For this purpose, basal vibrating wire (VW) total earth pressure cells placed in the sand underneath the EPS, horizontal inclinometers placed on the top and bottom of the EPS section, and magnet extensometer placed within the geofoam fill. Magnet extensometer was used to measure the vertical compression of the geofoam embankment during the placement of the overlying materials and pavement section. VW total pressure cells were used to measure the vertical and horizontal stresses and horizontal inclinometers strains were used to develop in the geofoam embankment. Moreover, horizontal inclinometers were installed to measure the differential settlement along a cross section of an embankment width.

To record the data from these instruments, several locations were considered for installation. However, this report presents the two EPS embankment locations previously mentioned: (1) 100 South Street, (2) 3300 South Street, as typical cross section examples. For the 100 South Street location, two instrumentation arrays were placed (a north and south array). These arrays were located in the southbound geofoam embankment on the western side of I-15, where it intersects with 100 South Street. A typical cross section of the southbound portion EPS embankment at 100 South Street is presented in Figure 3.3. According to Figure 3.3, by considering each row of EPS blocks as a layer, magnet extensometers were located at layers 0, 1.5, 3.5, 5.5, 7.5, 8.5, and 9. On the other hand, two horizontal inclinometers placed near the base and near the top of the embankment were responsible for recording horizontal displacement, while basal VW were placed in the sand underneath the EPS. The monitored instrumentation has shown 1% total compression strain at the end of construction, and it is expected it will increase to 1.7% compression strain after 50 years.

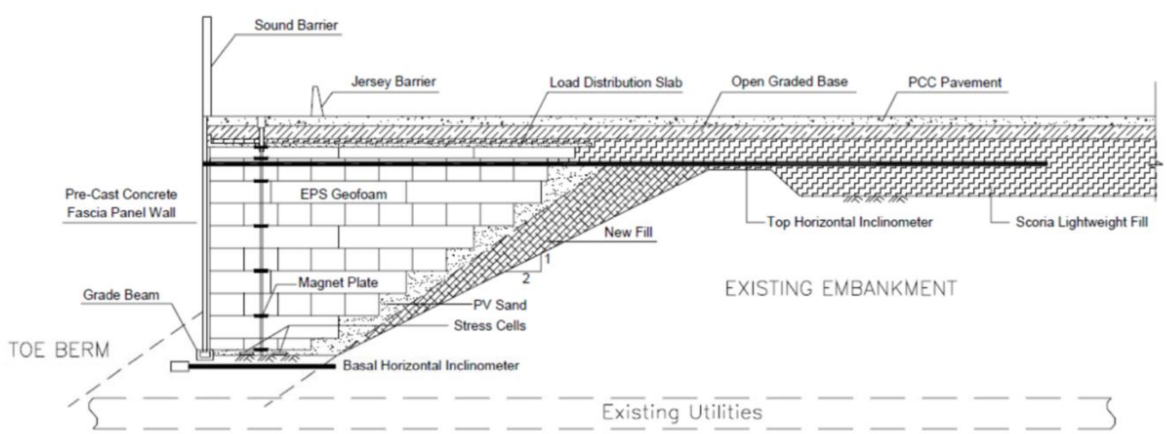


Figure 3.3. Cross Sectional View of the EPS Embankment and Instrumentation at the 100 South Street.

For other locations at 3300 South Street, two instrument arrays were installed at mainline stationing 25+347 m, and stationing 25+315 m, respectively. Each array consisted of magnet extensometers, VW total pressure cells and survey points. Magnetic extensometers were installed at various height intervals according to Figure 3.4, while VW 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. Based on instrument measurements, the data has shown 40 mm of post-construction foundation settlement in a 5.5-year post construction period. This settlement was summed up of 15 mm settlement due to placement of the fill and pavement materials on top of the EPS and 25 mm due to the placement of the toe berm, which was constructed at the base of the wall. The settlement reached to 50 mm in a 10 year post-construction period.

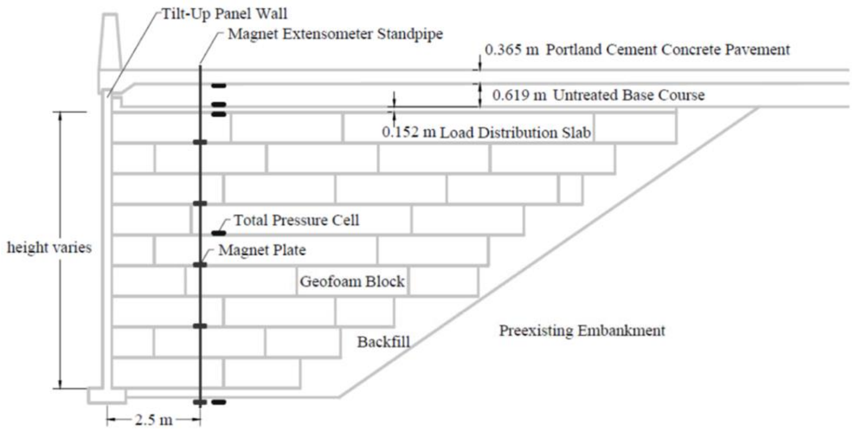


Figure 3.4. Cross Sectional View of the EPS Embankment and Instrumentation at the 3300 South Street.

3.1.1.2. Conclusion

The I-15 reconstruction project provides a successful case history regarding the usage of EPS geofoam blocks. These blocks not only helped to progress the rapid construction of large embankments over soft soils but also prevented settlement damage to buried utilities that crossed or ran parallel to the new alignment and limited foundation settlement in an acceptable range. Comparing the results obtained for foundation settlement at the 3300 South and compression strain at 100 South Street sites with the aforementioned design criterion in previous sections revealed that all the results were in an acceptable range. So both 3300 South and 100 South street have met the design criteria. As a result, it shows that Geofoam embankments had the best overall settlement performance of the results monitored.

3.1.2. I-64, Intersection of Route 199 and the Route 646 Connector

This project is located at the intersection of Route 199 and the Route 646 Connector in York County, Virginia, approximately 3 km (2 mi) north of Williamsburg. The objective of this study was to evaluate the behavior of shredded tire mixed with soil as an embankment material. To see the behavior of the material mentioned above, a scheduled experiment was considered by constructing two highway embankments next to the Route 646 Connector in the summer of 1993. The embankments were divided into two north and south sides of the Route 646 Connector through Route 199. Moreover, the end of the south side and the beginning of the north side of the embankments were terminated by soil sections for future linking of both embankments over the Route 646 Connector. In the following section, all of the efforts, including the detail of the proposed embankment and the parameters evaluated, will be described in detail.

3.1.2.1. Geotechnical Considerations

As mentioned previously, two embankments were constructed to compare the behavior of shredded tire and conventional fill materials in terms of stress and strain behavior at the top and base of the embankment. Each embankment consisted of both shredded tire and soil sections. The south embankment included a 160 m long shredded tire section in addition to a 30 m soil section at the end, while the north embankment was 80 m long of a shredded tire section adjoining 30 m conventional soil at the beginning, as shown in Figure 3.5. To accomplish the embankment construction and accelerate long-term settlement, 1.5 m of uncompacted soil surcharge was placed on top of the soil and tire sections. Each shredded tire section had a maximum height of 6 m and consisted of an approximate 50/50 volumetric ratio (visual determination) of shredded tires to the soil since shredded tire cannot be used alone due to the high compression rate. The soil mixed with the shredded tire at the south embankment was red clayey silt, while the soil mixed with the shredded tire at the north embankment was yellow silty sand. Since a compacted unit weight of tire shreds typically ranges from 3.1 to 7.1 kN/m^3 with the average value of 5.1 kN/m^3 and a compacted unit weight of a soil fill used on the project is approximately 17.3 kN/m^3, the compacted unit weight of 50/50 soil/tire mix was estimated as 11.2 kN/m^3. Besides unit weight values, tire shreds pieces should have had a maximum dimension of 25 cm (10 in), a maximum surface area of 260 cm^2, at least one sidewall severed, and no loose metal strands in order to meet the requirements of VDOT's Special Provision for Shredded-Tire.

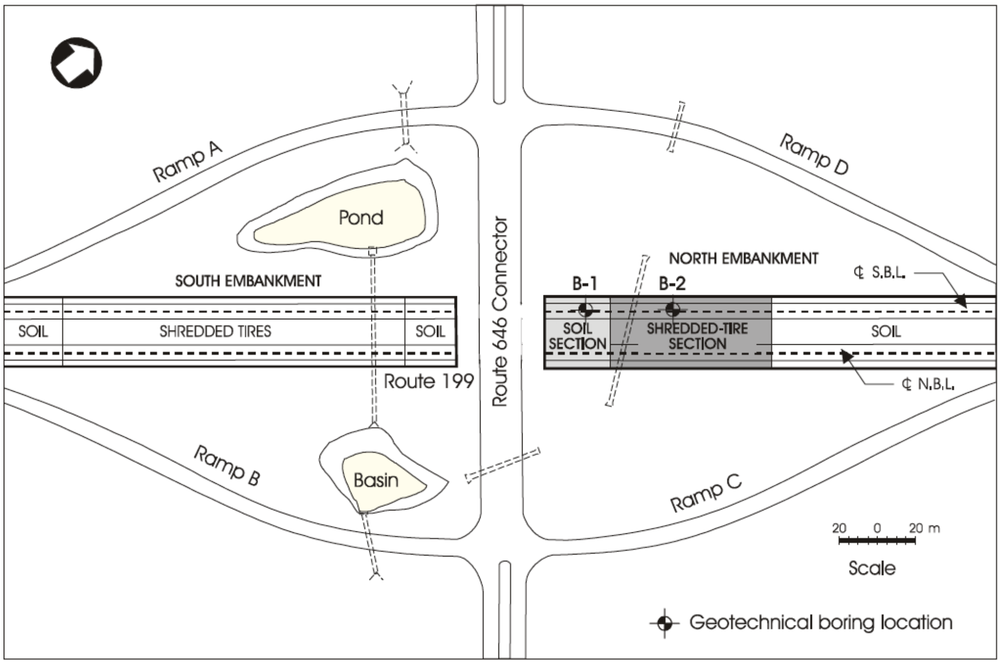


Figure 3.5. Site Location at the Intersection of Route 199 and the Route 646 Connector.

After determining the composition structure of the embankment section, their field performance was evaluated in terms of stress and strain exerted by soil self-weight of overlying material or traffic load. Consequently, instrumentation was required to be installed in the shredded-tire and conventional embankment sections north of the Route 646 Connector. For this purpose, pressure cells and settlement sensors were chosen to measure stress and strain applied to the sections mentioned earlier. As shown in Figure 3.6, two pressure cells were located at the base of both tire and soil sections, while two settlement sensors, one at the base and one at the top of the embankment, were installed for each tire and soil section. Pressure cells were installed to measure the soil pressure, while settlement sensors were responsible for measuring the embankment settlement at the mentioned locations.

Soil pressure applied to the embankment sections was measured in three different phases: (1) at the end of construction without a surcharge, (2) after a surcharge placement, and (3) nine months after construction. The stress reported at the base of the conventional fill material and shredded tire section at the end of construction was 83 kPa and 20 kPa, respectively. After placement of the surcharge, the values increased to 90 kPa and 30 kPa. Finally, by June 1994, approximately nine months after construction, vertical stresses stabilized, and the values reached 63 kPa and 28 kPa below the soil and tire/soil sections. The common point concluded among all the phases was that by using a mix of shredded tire and soil, the vertical stress would be roughly 0.6 of the stress exerted by a conventional soil embankment of the same geometry. So the vertical stress exerted by a shredded tire and soil mixture was significantly lower than the conventional fill section.

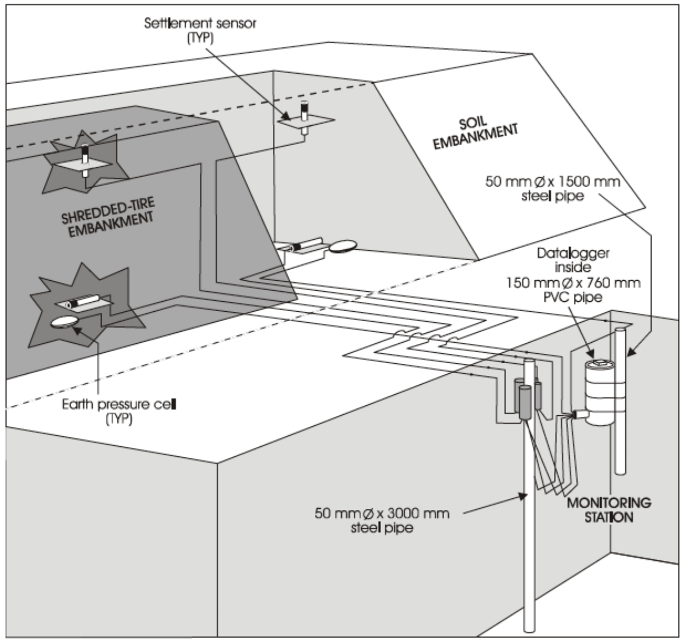


Figure 3.6. Instrumentation Setup at the Embankment Located in Site Location.

Embankment settlements were also measured in addition to the induced stresses. The embankment settlements measured by settlement sensors were reported in two different periods because the rest of the settlement readings became erratic at the end of July 1994, which rendered subsequent data unreliable. In December 1993, approximately four months after construction, the settlement measured on top of the embankment sections were 52 mm and 30 mm at the shredded-tire and soil sections, respectively, indicating a 1.7 ratio between the two measurements. The settlement ratio between the two sections was somewhat steady so that in July 1994, measured values for the shredded-tire and soil sections were 105 mm and 55 mm. Unlike what happened to the soil pressure, shredded-tire embankment settlements were roughly twice (1.9) the magnitude of the conventional embankment settlement through several months after construction. This ratio was not all too surprising since the tire shreds had a higher compressibility rate than the conventional fill layer, though the presence of 1.5 m of uncompacted soil surcharge was not ineffective and played an important role in the value of the embankment settlement.

3.1.2.2. Project Cost

After describing geotechnical considerations regarding the soil pressure and embankment settlement, a cost comparison between the shredded tire section and conventional soil section was reported, as shown in Table 3.2. The cost details were divided into two embankment sections located at the north and south of the Route 646 Connector. Although the unit price of the shredded tire was the same for both embankment sections, the final cost of shredded tire for the south embankment was higher than the north section due to the higher volume embankment section at the south section. It should be noted that the price of the shredded tire was calculated in a different approach compared to conventional soil. Indeed, it was paid based on loose volume delivered to the site instead of a common compacted in-place volume for the regular fill material. As a result, due to the 30 percent compression of tire shreds, its unit cost was at least 37% higher than the conventional fill. This issue may cause a cost overrun of $425.509 as a difference between the total estimated cost and the total final cost. Apart from the cost overrun of the tire shreds, by considering the total estimated cost and shredded tire, it is estimated that the overall shredded tire material costs used for both embankment sections amounted to approximately $437,078, which was about 62% of the overall estimated costs of $704,179.

Table 3.2. Cost in detail for both North and South Embankments

Item	Estimated Quantity	Final Quantity	Unit	Unit Price	Cost
North Embankment					
Construction Surveying			L.S.	2,625	2,625
Surplus Regular Excavation	18,809	21,318	m^3	1.6021	34,153
Borrow Excavation	10,972	16,143	m^3	9.8640	159,234
Settlement Plates	4	4	EA.	1,000.00	4,000
Shredded Tires	9,091	18,029	m^3	10.3701	186,966
Surcharge	4,261	8,632	m^3	9.8640	85,150
South Embankment					
Construction Surveying			L.S.	2,625.00	2,625
Surplus Regular Excavation	27,067	30,677	m^3	1.6021	49,147
Borrow Excavation	15,789	26,083	m^3	9.8640	257,280
Settlement Plates	4	4	EA.	1,000.00	4,000
Shredded Tires	13,082	24,119	m^3	10.3701	250,112
Surcharge	8,555	9,570	m^3	9.8640	94,396
Total Final Cost					$1,129,688
Total Estimated Cost					$704,179
Cost Overrun					$425,509

3.1.2.3. Conclusion

A comparison of the two shredded tire and conventional soil sections in terms of soil pressure, embankment settlement, and cost of the project revealed that shredded tire mixed with soil was not a good alternative in accordance with the settlement behavior and cost of the project. However, it exerted lower vertical stress at the embankment due to lower unit weight of shredded tire rather than conventional soil material. Indeed, a high compressibility rate of shredded tire not only might be expected to settle twice the magnitude of conventional embankments but also can cause a significant difference between the estimated cost and final cost of the project as a cost overrun since the contractors should be paid based on the loose volume of this material delivered to the site so their compacted in-place volume would be higher at the site. However, the use of shredded tires as lightweight, sustainable material can be prudent in other aspects not mentioned in this study, such as waste tire disposal, since the current study used an estimated 1.7 million discarded tires of shredded tire sections of embankments.

3.1.3. I-64/I-264, Witchduck

This case history was the I-64/I-264 interchange improvement project located in the cities of Virginia Beach and Norfolk, Virginia (Figure 3.7). The project was started at the intersection of I-264 and I-64 and extended 0.3 miles towards the east of Witchduck Road on I-264. The project includes several changes such as the construction of a new I-264 Off-ramp A (Ramp A) and demolition of the existing off-loop, realignment of On-loop B, and improvements to Witchduck and Grayson Roads. Several objectives supported the proposed changes, such as evaluating and characterizing the subsurface condition in addition to developing a geotechnical recommendation for the design and construction of roadway improvement and major structures such as the proposed bridge and retaining walls. However, the main focus of this case history was the geotechnical analyses of proposed embankments and retaining walls supporting Ramp A and Loop B.

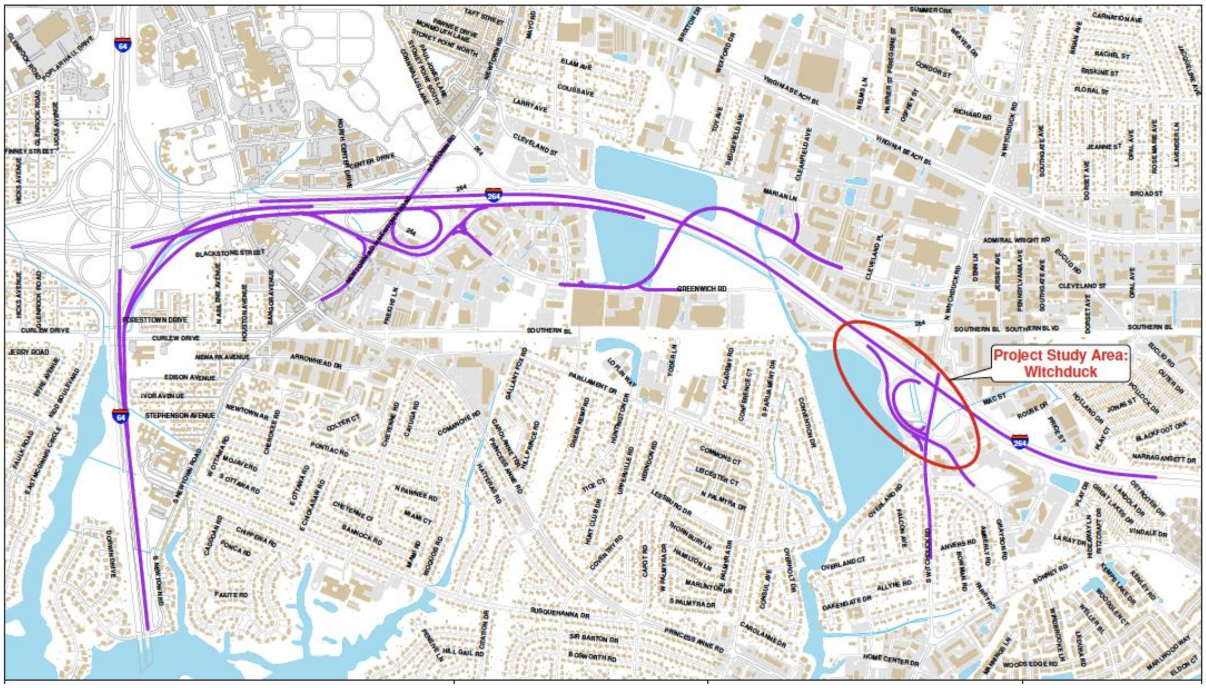


Figure 3.7. Approximate Site Location Area.

<u>3.1.3.1. Geotechnical Considerations</u>

As mentioned in the previous section, Ramp A and Loop B were proposed to be supported by retaining walls and embankments. Ramp A was carried by two MSE walls with heights of approximately 28 ft and 8 ft and lengths of 400 ft and 375 ft, while loop B will be laid on an embankment with a 2H:1V slope and height up to approximately 16 ft. As an essential design process of these structures, global stability and settlement analyses were performed to ensure that the minimum required factor of safety and the acceptable magnitude of settlement within a specific timeframe were met. To develop the analyses, stations 14+00 and 21+00 for Ramp A and station 19+00 for Loop B were selected.

The proposed fill geometry for each station is described in Table 3.3. In addition to the proposed fill geometry of each station, some other assumptions such as MSE fill unit weight, retained fill unit weight, and subsurface layers' type was assumed. Based on the subsurface investigation, six distinct subsurface strata were distinguished from the top towards the bottom: (1) Fill or reworked in-situ soil; (2) Loose to medium dense sands (upper sand); (3) Very soft to soft gray clays and clayey sands (upper clay); (4) Medium dense to dense sands (lower sand); (5) Medium dense to dense sands (lower sand); and (6) Medium dense to dense and stiff to hard, green-gray sands and clays. Each layer had some properties such as elevation, internal friction angle, and unit weight that were beyond this summary and obtained from subsurface investigation and laboratory experiments. Moreover, three unit weights corresponding to the conventional fill layer (130 pcf), No.57 open-graded (110), and lightweight expanded shale (70 pcf) were used in different combinations for both MSE fill and retained fill unit weight to evaluate the stability of proposed models.

Table 3.3. Summary of Selected Stations

Critical Section Station	Associated Structure	Proposed Fill Geometry
Ramp A 14+00	Retaining Wall	Maximum Fill Height = 28ft Fill Width = 85ft
Ramp A 21+00	Retaining Wall	Maximum Fill Height = 8ft Fill Width = 60ft
Loop B 19+00	Embankment	Maximum Fill Height = 16ft Fill Width = 80ft

After considering all the assumptions mentioned above, proposed models were developed with the aid of computer programs. Slope/W (2012, Version 8, Geo-Slope International) and CONSOL (version 3.0, CGPR, Virginia Tech) were used to complete the global stability analyses, and magnitude and time rate of settlement analyses, respectively. For global stability purposes, the main output of the program, as mentioned above, is a factor of safety (FS). So to evaluate the output obtained from the program, an initial assumption and design consideration were required. Consequently, two values of 1.3 and 1.5 were assumed as FS against global stability failure for both non-critical and critical applications, respectively. VDOT defined critical application for embankment and wall when walls of 15 feet or greater and slopes of 25 feet or greater; otherwise, the application will be counted as non-critical.

Additionally, the magnitude and time rate of settlement was important since it was helpful to estimate approximate waiting periods following fill placement to aid in the construction planning process. For this reason, the time necessary to reach less than 1 inch of settlement was assumed as the criteria. So, if the anticipated phasing and wait times necessary to meet required settlement cannot be reached, ground improvement techniques or lightweight fill can be used as alternatives, and prefabricated vertical drains (PVD) can be used to accelerate the consolidation settlement.

The results of the analyses showed that, in terms of FS, the worst case scenario and best case scenario occurred when using conventional fill layer (130 pcf) and lightweight expanded shale (70 pcf) for both retained and MSE fill. By considering both stations 14+00 and 21+00, the FS obtained for using conventional fill was in the range of 1.1-1.3, while FS for using lightweight expanded shale fill was in the range of 1.5-1.6. In the case of using a conventional fill layer in Loop B, the FS met the requirement and was equal to 1.5; thereby, no other alternatives were required to be analyzed. On the other hand, in the case of settlement, by assuming anticipated time for settlement less than 1 inch for conventional fill (130 pcf) at location 14+00 (Ramp A) as output, the results showed that the time for not using PVD was one year, while the time with PVD with 5 ft spacing was reduced to four months.

3.1.3.2. Conclusion

The study showed that using lightweight expanded shale can significantly increase FS and meet the minimum of 1.3 and 1.5 as safety factors of both non-critical and critical applicants. It means that by using expanded shale as the lightweight materials alternative, there is no potential failure due to low stability for the embankments and MSE walls. Moreover, the results showed that using PVD can significantly decrease the estimated consolidation time rates to less than half. Although the use of lightweight materials was not investigated for settlement analyses, PVD, as an auxiliary acceleration alternative, can significantly reduce the consolidation settlement, which is an essential factor for foundation design.

3.1.4. I-95 Southbound, Cowan Boulevard

The approximate project location of this case history was along I-95 between mile markers 129.3 and 135.3 in the city of Fredericksburg, Stafford County, and Spotsylvania County, Virginia, as shown in Figure 3.8. The project includes converting the existing southbound (SB) general purpose (GP) lanes to a new collector distributor (CD) and constructing new southbound GP lanes within the existing median. Also, some widening was required to the outside of the existing southbound general purpose lane. One of the areas that must support the new GP lane was the existing Cowan Boulevard Bridge. So the objective of this study was to evaluate the subsurface condition of the existing Cowan Boulevard Bridge to support the new GP lanes.

Figure 3.8. Approximate Site Location of the Project

3.1.4.1. Geotechnical Considerations

As mentioned previously, the proposed I-95 SB-GP lanes below the existing Cowan Boulevard bridge were selected to investigate the subsurface characteristics. The final site grade along the alignment centerline was EL 231 and will slope down to east and west of the I-95 SB-GP lanes with the slopes of 2H:1V and 4H:1V, respectively, to reach the existing site grade. Moreover, in the I-95 NB and SB direction, site grades will vary from about EL 203 at the northern limit (Station 3473+50) to EL 235 at the southern limit (Station 3470+70). Figure 3.9 shows a typical cross section of Cowan Boulevard bridge at the station between 3471+50 to 3473+50. Based on the elevation mentioned for the different directions, the final site grades will require the placement of up to 27 ft of new fill.

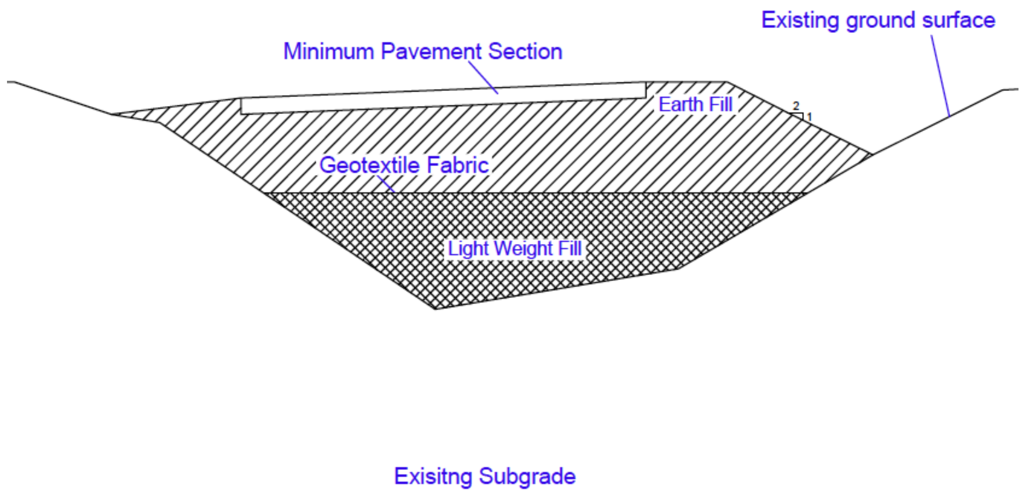


Figure 3.9. Typical Cross Section of Cowan Boulevard.

The new fill placement at the selected location can lead to soil settlement, and subsequently, this settlement could cause downdrag on the existing pier foundations. As a result, the total settlement of the existing ground was limited to 1/2 inch over 20 years within 100 feet of the bridge (i.e., from Station 3470+70 to Station 3473+50) below the 3 ft of regular weight earth fill. To accomplish this, a variety of fill materials, including regular earth fill, VDOT No. 57 stone, and light-weight fill materials (foamed glass from Aero Aggregates or expanded slate from Stalite) with different unit weights were used at three different stations 3471+50, 3472+50, and 3473+50 (Table 3.4).

Table 3.4. Fill Materials Selected for the Project

Fill Material	Moist Unit Weight, (pcf)
Normal Earth Fill	125
VDOT No. 57 Stone	110
Expanded Slate (Stalite)	60
Foamed Glass (Aero Aggregates)	20

The three stations selected for the fill materials corresponded to the deepest fill placement anticipated for the project. As a result, at least three settlement plates were installed at the base of the new fill at Stations 3471+50, 3472+50, and 3473+50 at points to monitor the settlement behavior. The results of the monitored settlements of the different fill alternatives exhibited a range of 0.4 in to 0.7 in. Use of VDOT No. 57 Stone at both 3472+50 and 3473+50 stations corresponded with 0.7 in and 0.6 in of settlement, respectively, where both values were higher than the 0.5 in maximum allowable settlement. But, using foamed glass aggregates at the same stations reduced the settlements to 0.4 inches and meets the maximum permitted settlement, and also, the section did not face any problem. Finally, using normal earth fill at 3471+50 section resulted in a settlement less than 0.5 inches.

3.1.4.2. Conclusion

Comparing the estimated settlement obtained at different stations mentioned in the previous section with the maximum allowable settlement showed that for South of Station 3471+50, normal weight fill could be used since the value was less than maximum allowable. But from Station 3471+50 to 3473+50, foamed glass aggregate must be used, since using VDOT No. 57 Stone did not meet the requirement. This comparison revealed that foamed glass aggregate is a good alternative for the locations where settlement potential exists.

3.1.5. Route 7, Section 2 - Wittpenn Bridge over Hackensack River

This case history was the memorandum of a project located at Route 7 Hackensack River_Wittpenn Bridge, New Jersey. The memorandum presented foamed glass aggregate (FGA) as an alternative material to replace other proposed lightweight materials [expanded shale lightweight aggregate (LWA) and geofoam]. As a result, the objective of the study was to evaluate the replacement of FGA for the aforementioned lightweight materials at two locations where the grade will be raised up on a soil layer consisting of organic or soft soil, and in roadway embankment supported by vibro-concrete technology. The purpose of using lightweight materials in both mentioned locations was to minimize the additional pressure resulting from raising the grade and any subsequent settlements. For using material over soft soil layer, lightweight material can be used by over-excavating existing soil and backfilling with lightweight material to the bottom of the proposed pavement or the bottom of the topsoil or non-vegetated layer within the shoulders. Also, vibro-concrete column technology was used under the flat area of the embankment to avoid settlements. However, it is not possible to use this technology at some limited areas such as sloping sides of embankment or to be installed over utilities. Hence, lightweight materials can be used in mentioned limited spaces to minimize the settlement. As an alternative lightweight material, FGA has some advantages over the other lightweight materials.

For example, it does not need additional considerations such as gasoline-resistant geomembrane or concrete topping slab compared to geofoam blocks. Moreover, FGA has less unit weight than expanded shale, which might be helpful in applied pressure to underlying layers. All of these factors led to evaluate all three lightweight materials to choose the most viable choice for the project.

3.1.5.1. Geotechnical Considerations

The geotechnical considerations of this project were the additional pressure from the toping pavement, substitution of lightweight materials with FGA for pavement, and substitution of lightweight materials with FGA for inaccessible spaces under the embankments supported by Vibro-concrete columns. When evaluating the additional pressure exerted from the toping pavement, the applied stress transmitted from traffic loading to underlying layers should be limited to avoid breaking the material such as FGA and control the additional pressure on lightweight materials beneath the pavement. The calculation showed that to restrict the stresses to 1000 and 1500 psf, 32 in and 24 in were acceptable depths for permanent and temporary pavement, respectively. As a result, two types of pavement were described for the case of LWA material as regular section and FGA as modified section, as shown in Table 3.5. Table 3.5 shows that to reach the equivalent depth of pavement on FGA, the 6 and 10 inches dense graded aggregate layer of LWA section selected for temporary and permanent pavement should be increased to 14 inches for the modified section using for FGA. In the geofoam section, the regular section was used in addition to a 4-inch thick concrete slab; however, this slab was not required for FGA and was removed from the design.

Table 3.5. Selected Sections for different Pavements

	Regular section (LWA)	**Modified section (FGA)**
Permanent Pavement	3" HMA 12.5 ME Surface Course 4" HMA 19 ME Intermediate Course 7" HMA 25 M64 Base Course 4" Asphalt Stabilized Drainage Course 10" Dense Graded Aggregate Base Course Geotextile Roadway Stabilization	3" HMA 12.5 ME Surface Course 4" HMA 19 ME Intermediate Course 7" HMA 25 M64 Base Course 4" Asphalt Stabilized Drainage Course 14" Dense Graded Aggregate Base Course Geotextile, Roadway Stabilization
Temporary Pavement	2" HMA 12.5 ME Surface Course 3" HMA 19 ME Intermediate Course 5" HMA 25 M64 Base Course 6" Dense Graded Aggregate Base Course	2" HMA 12.5 ME Surface Course 3" HMA 19 ME Intermediate Course 5" HMA 25 M64 Base Course 14" Dense Graded Aggregate Base Course Geotextile, Roadway Stabilization

For substituting lightweight materials with FGA for pavement, all three lightweight materials were compared in four typical cross sections, as shown in Figures 3.10 and 3.11. The Figures showed that by assuming a constant value for the properties of all other layers, the less unit weight of lightweight material, the less over-excavation will be needed to reach the same additional pressure on the subsoil. The unit weight of materials were as the following; geofoam (2.4 pcf), FGA (22 pcf), and LWA (65 pcf). So, according to Figures 3.10 and 3.11, FGA required less over excavation than LWA; however, it required more over excavation compared to geofoam. So it showed that, in terms of over excavation depth of soil layer, using FGA has been demonstrated as better than LWA, but the geofoam block case required less over excavation compare to FGA due to lower unit weight.

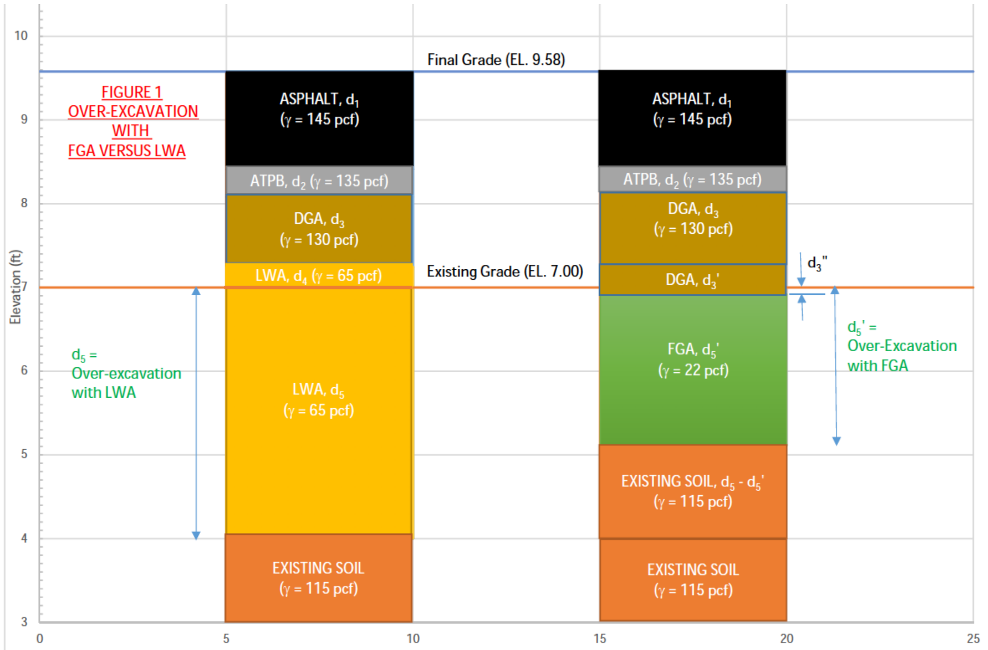


Figure 3.10. Cross Section including LWA (Left) and FGA (Right)

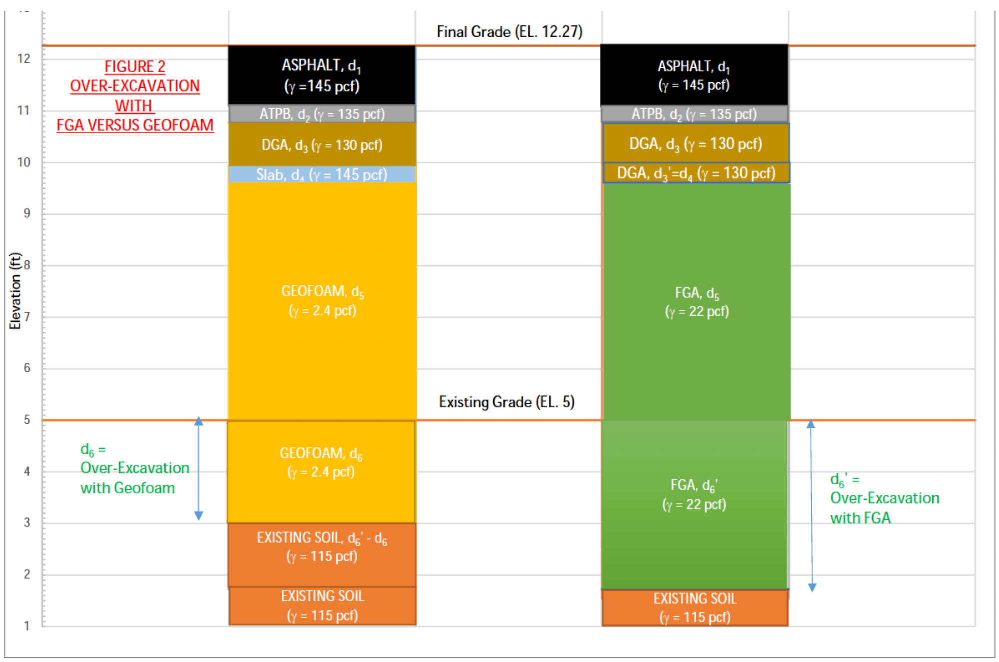


Figure 3.11. Cross Section including Geofoam (Left) and FGA (Right)

Finally, substituting lightweight materials with FGA for inaccessible spaces under the embankments supported by Vibro-concrete columns showed that LWA used in limited areas could be replaced partially by FGA and partially by select fill. Indeed, 60% of the bottom was LWA, and the remaining 40% was by common fill.

3.1.5.2. Conclusion

According to all three cases evaluation, it was revealed that FGA is a viable substitution to LWA and geofoam. On the one hand, FGA eliminates the need for gasoline-resistant geomembrane and concrete topping slab associated with the use of the geofoam blocks in addition to reducing the unknown potential damage exerted by geofoam blocks to utilities, though its unit weight was higher than geofoam blocks. On the other hand, the unit weight of FGA was lower than LWA, so it required less over excavation soil depth compared to LWA and subsequently resulted in less cost for the project. Moreover, in the case of equivalent depth for pavement, the FGA section did not cause any excessive cost to the project compared to the geofoam section, though the thickness of dense graded aggregate base should be increased, which subsequently affected the budget of the project and could increase the final cost, albeit not by a significant amount.

4. Guidelines for Appropriate Selection of Lightweight/Sustainable Fill Alternatives

The focus of this chapter is to provide guidelines for how lightweight materials can be considered in the geotechnical engineering design process for highway applications. Specifically, various lightweight materials are discussed with respect to how they compare to typical design alternatives for retaining walls, embankment fills, and similar highway-related geotechnical structures. This chapter discusses lightweight fill design alternatives based on the previous literature review and case history review efforts.

4.1. Comprehensive Guidelines

In any engineering design process, the goal is to design the most efficient and cost-effective system to meet the needs of the particular project. In the case of highway-related geotechnical projects (e.g., embankments, retaining walls, slopes, roadways), this often entails the use of shallow foundations on stiff strata (e.g., compacted fills, bedrock if located near the surface etc.), simple compaction efforts with typical construction equipment, and/or removal and recompaction of problematic soils. However, different issues might arise in highway-related geotechnical projects prior to or during their construction phases. These include problems such as drainage, frost-heave, insulation thermal behavior, or technical issues like load bearing capacity, low stability, high lateral force, and settlement issues. These issues may impart risks to not only the geotechnical structure itself, but also utilities such as high pressure gas pipeline or electrical cables that exist onsite. These factors can lead to limitations in the use of conventional geotechnical designs and consideration of alternative approaches, some of which may be more expensive or have limitations relative to site conditions. For example, various deep foundation systems can be proposed to bypass problematic soil conditions and distribute the applied loads to stiffer strata. Ground improvement is a strategy approach which can increase the performance of various soils and prevent potential damage to overlying structures. Table 4.1 combines data acquired and summarized in Munfakh (1997a), Elias et al. (2006), and Schaefer et al. (2017a,b) to present several solution alternatives based on the desired function for a particular project. However, some design alternatives may be inappropriate for certain project types and/or site conditions.

Lightweight fill materials are useful and applicable in various applications as a design alternative (Table 4.1) for embankment construction, earth retention backfills, culverts, and slope stabilization fills. Their major attributes are decreased applied loads, reductions in settlements, elimination or reduction of the need for surcharge loading, reductions in lateral loads, and reductions in construction time. For this purpose, to get a better sense of each lightweight material attributes and their use in geotechnical infrastructure, this study focuses on the important factors related to pre-construction phase activities, construction phase activities, and economic considerations regarding each technology. Included at the end of the chapter is a set of guidelines for how to approach the engineering design process for typical highway infrastructure projects and consider lightweight and/or sustainable fills materials.

Table 4.1. Technologies classified by function (Munfakh 1997a; Elias et al. 2006; and Schaefer et al. 2017a,b).

Function	Technologies	
Increase Shear Strength and Bearing Resistance	• Deep Foundations • Vibro-Compaction • Dynamic Compaction • Compaction Grouting • Mixing Methods • Prefabricated Vertical Drains (PVD)	• Stone Columns • Rammed Aggregate Piers • Chemical Stabilization • Mechanical Stabilization • Shear Key • Resisting Berm
Increase Density	• Vibro-Compaction • Dynamic Compaction • Blasting Compaction	• Compaction Grouting • Mixing Methods • PVD
Decrease Permeability	• Bulk-infill Grouting • Chemical Grouting • Jet Grouting • Deep Mixing Methods	
Control Settlement	• Deep Foundations • Columns Supported Embankment • Reinforced Load Transfer Platform • Non-Compressible Columns • Mixing Methods • Vibro-Compaction • Dynamic Compaction • Stone Columns	• Rammed Aggregate Piers • Chemical Stabilization • Mechanical Stabilization • Encapsulation • Expanded Polystyrene (EPS) • Foamed Glass Aggregate (FGA) • Lightweight Cellular Concrete (LCC)
Increase Elevation	• Expanded Polystyrene (EPS) • Foamed Glass Aggregate (FGA) • Lightweight Cellular Concrete (LCC) • Column Supported Embankment	
Increase Drainage	• PVD • Aggregate Columns • Geotextile Encased Columns • Electro-osmosis	• Geosynthetics in Pavement Drainage • Stone Columns • Foamed Glass Aggregate (FGA)
Accelerate Consolidation	• PVD • Aggregate Columns • Geotextile Encased Columns	
Decrease Imposed Loads	• Granular Fills (Wood Fiber; Blast Furnace Slag; Fly Ash; Boiler Slag; Expanded Shale, Clay, and Slate; Tire Shreds; Foamed Glass Aggregate) • Compressive Strength Fills (EPS, LCC)	• Rock Toe Bench • Jet Grouting Columns • Deep Foundations
Provide Lateral Stability	• Mechanically Stabilized Earth (MSE) Walls • Reinforced Soil Slopes • Soil Nailing	• Expanded Polystyrene (EPS) • Foamed Glass Aggregate (FGA) • Lightweight Cellular Concrete (LCC)
Increase Resistance to Liquefaction	• Aggregate Columns • Deep Dynamic Compaction • Deep Mixing	• Jet Grouting • Vibro-Compaction

4.1.1. Applications and Design Alternatives

Choosing the right technology for a project can be challenging and should be accomplished after a comprehensive evaluation of the conditions of a project. In fact, choosing a proper technology is related to many factors (Mitchell 1981; Holtz 1989; Munfakh 1997b):

- Depth, area, and volume of weak or damaged soil
- Soil type and its properties
- Availability of materials (both conventional and lightweight materials)
- Availability or requirement of skilled labors and specialty equipment
- Construction and environmental conditions
- Site accessibility and constraints, such as waste disposal, erosion, water pollution
- Effects on adjacent facilities and structures
- Experience and preference of contractors and designer
- Scheduled time
- Costs

In order to explore these factors with respect to various lightweight fill materials in highway construction, common geotechnical structures in transportation system (embankments, culverts, earth retaining systems) are discussed in the following sections. Design alternatives are described for each project type so that they can be contrasted against lightweight fills in later sections of the chapter. In order to streamline the discussion, it is assumed that typical alternatives (i.e., shallow foundations, removal and recompaction of problematic soils, deep foundations) have already been considered for each of the applications during the design process. Consequently, the discussion in the following sections focuses on alternative designs to those "default" approaches. This often entails the use of ground improvement and similar techniques. Subsequent sections will then highlight the role of lightweight fills as another alternative design approach for typical highway construction projects.

4.1.1.1. Highway Embankments

Stability and excessive settlement are important factors in the design of embankment systems. Excavation/replacement and ground improvement methods are two comprehensive solutions often used to address any issues related to stability and excessive settlement. Ground improvement methods used for embankment differ according to the type of soil being treated. If the subsurface soils are granular, the ground improvement treatment typically relies more on densification. While, if the soil at site location is cohesive, consolidations, weight reduction, or reinforcement using vertical reinforcing elements are used as remediation alternatives. On the other hand, if the soil close to the site location is weak, the treatments would be different as densification with heavy vibratory rollers, chemical stabilization with admixtures, or reinforcement with geosynthetics

(Munfakh 2003). Some ground improvement methods are common for use in embankments without specific consideration for soil condition. In the following sections, each method is briefly described.

4.1.1.1.1. Excavation and Replacement

One of the conventional ways to improve the subsurface beneath an embankment is excavating organic material or very soft clay and replace them with select granular material (Figure 4.1). This process may either partially or completely replace the unsuitable soil layer. Complete excavation may be appropriate when the bottom of the soil is as shallow as 20 ft below the surface while partial excavation is a better choice when the very soft surface deposit is either quite deep or is underlain by a significantly stronger material (Ariema and Butler 1990). Using a heavy load device on soft and compressible soil to replace and refill that soil with proper material is one challenging issue associated with this approach. However, the biggest challenge with excavation and replacement is typically related to regulated/hazardous soils and groundwater. Removal of these soils incurs additional costs associated with disposal and the need for onsite groundwater treatment and removal.

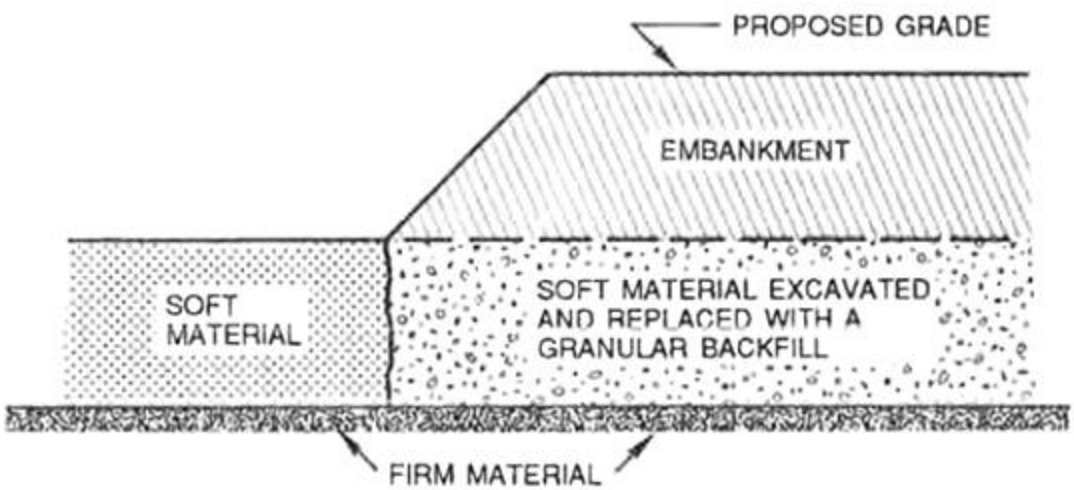


Figure 4.1.* Complete excavation and replacement *of soft material below a proposed embankment (Ariema and Butler 1990.)

4.1.1.1.2. Prefabricated Vertical Drains (PVDs)

Prefabricated Vertical Drains (PVD) are composed of a plastic core with longitudinal channels covered by a fabric filter of high clogging resistance. The can be installed by penetrating in poor soils to dissipate the pore water pressure in the layer, typically in combination with surcharging

(Figure 4.2). One of the most common applications of PVDs is accelerating the consolidation for constructing embankments over soft soil where the total settlement is not acceptable (Li and Rowe 2001; Rowe and Taechakumthorn 2008). Despite many advantages for PVD technology, there are some potential disadvantages regarding installing and storing these materials (Schaefer et al. 2017a). The equipment required for PVD installation must have 5 to 10 feet more space than the depth of installation, which can limit its consideration on some site locations. Also, the material used for this technology must be covered during storage to avoid degradation due to exposure to sunlight. Finally, installation of PVDs requires the need for specialty contractors since typical heavy highway and structure contractors do not perform that kind of construction.

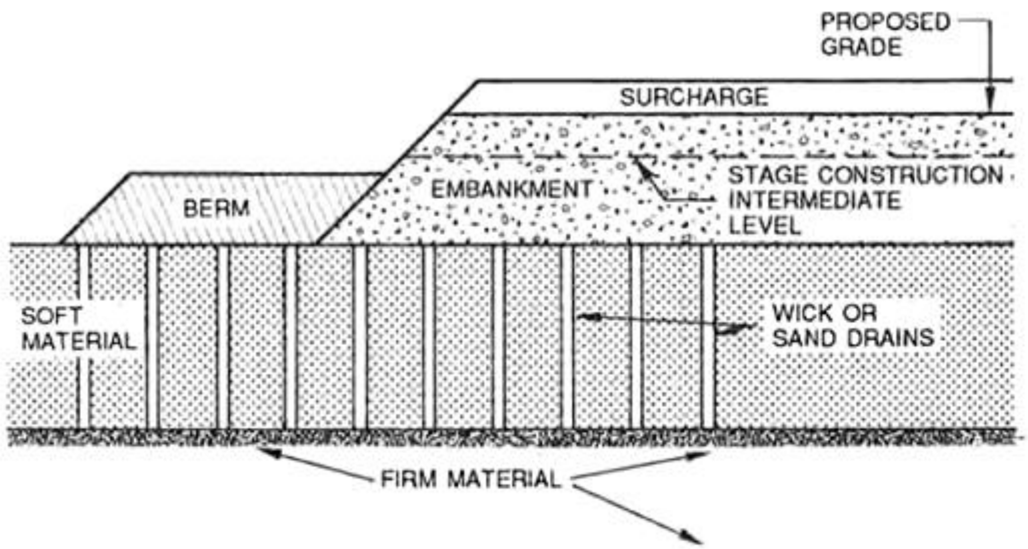


Figure 4.2. Stabilization by consolidation with a surcharge fill and wick or sand drains (Ariema and Butler 1990).

4.1.1.1.3. Column Supported Embankment

Column Supported Embankments (CSE) use several stiff vertical inclusions to transfer the applied load below compressible soil layers (Figure 4.3). In this way, the system prevents differential settlement which might occur at the surface of an embankment. CSE can be used in various applications such as bridge approach embankments (Hoppe and Hite 2006), roads and railways traversing soft ground (Collin 2004; Collin et al. 2005b), and embankment widening projects (Han and Akins 2002). CSEs can be supported with or without a geosynthetic-reinforced load transfer platform (LTP) (Figure 4.3). The LTP transfers the embankment load to the columns without any excessive deformations between columns. The columns themselves can be constructed using a number of different approaches. For example, deep foundations systems such as steel H-piles can be driven as support columns. Ground improvement techniques (e.g., stone columns, controlled modulus columns, vibro-concrete columns, etc.) can also be used to construct the rigid inclusion necessary to support the embankment. Although this method has many advantages, its high initial construction cost compared to other alternatives can be counted as the big disadvantage of this technology. Additionally, as similarly commented for other design alternatives, CSE require the use of specialty contractors for installation.

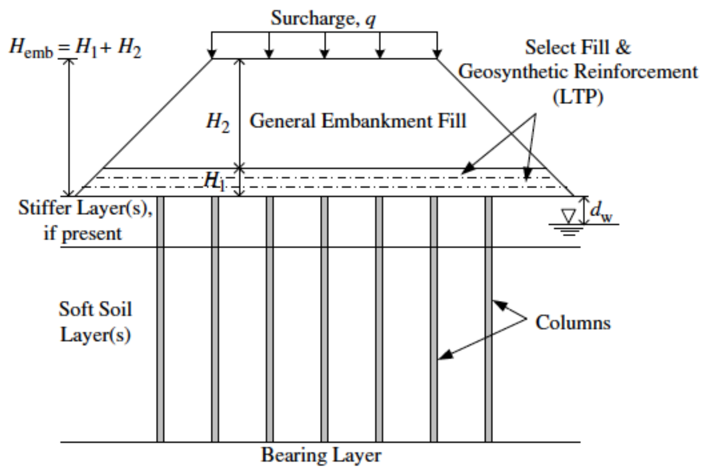


Figure 4.3. Column-supported embankment with geosynthetic reinforcement (Filz et al. 2019).

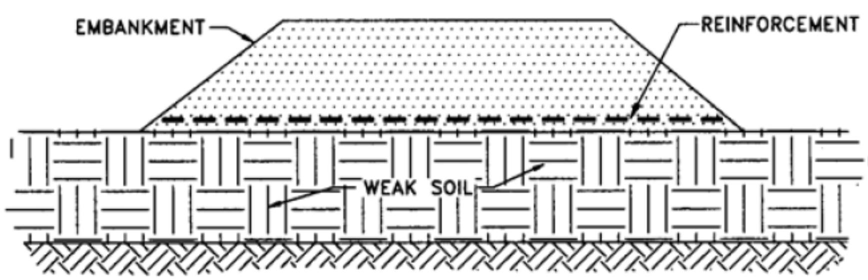


Figure 4.4. Example of reinforced embankment application (Schaefer et al. 2017b).

4.1.1.1.4. Reinforced Embankments

Construction of conventional embankments over soft subgrade soils can lead to several potential failure modes. One of these failure modes is the tendency for lateral spread of the embankment due to horizontal earth pressures. Generally, horizontal shear stresses are counteracted by the foundation soil. However, if the foundation soil provides insufficient lateral support, other alternatives can compensate for this deficiency by using high-strength geotextile or geogrid reinforcements (Jia et al. 2021) as well as berms (Ariema and Butler, 1990). The purpose of a geosynthetic layer is to increase lateral sliding stability and prevent such failures by spreading the load over the full width of the embankment. The geosynthetic is usually installed at the base of the embankment with its strong direction perpendicular to the centerline of the embankment (Figure 4.4). The reinforced layer provides stability until the underlying soft soils consolidate, gain strength, and are capable of supporting the embankment. A berm is a kind of physical stabilization that places additional material near the sides of the embankment (Figure 4.5). The berm is usually installed during the phase of construction of a project. This technology improves stability by adding a counterweight to the sides of the embankment where lateral displacement may otherwise take place. The disadvantages of this method are not reducing the total settlement, not reducing the time to achieve primary consolidation, and the necessity of detailed field observation during the construction phase, including monitoring of pore pressures, total settlement, and rate of settlement (Schaefer et al. 2017b).

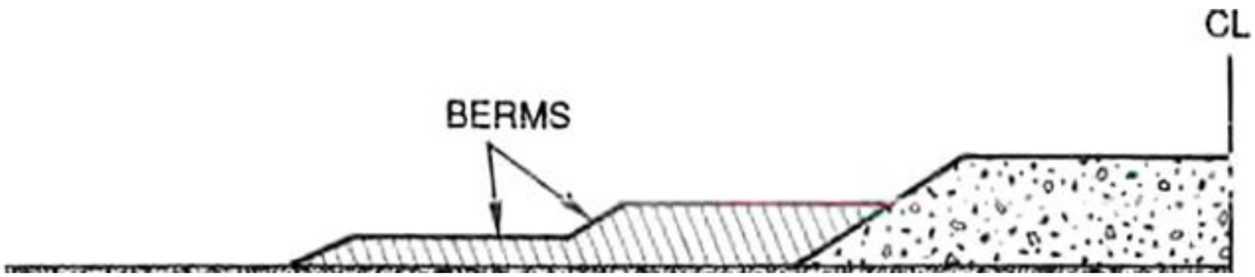


Figure 4.5. ***Embankments stabilized with berms (Ariema and Butler 1990).***

4.1.1.1.5. Dynamic Compaction and Vibro-Compaction

Specialty compaction methods (i.e., dynamic compaction and vibro-compaction) are a common ground improvement technique employed during embankment construction on problematic soils (Figure 4.6). During dynamic compaction, high energy impacts are caused by dropping a heavy weight from a significant height to compact any loose soils with low bearing capacity and high compressibility (Hamidi et al. 2009). Vibro-compaction is a method that improves the weak soil by partial displacement with selected granular material compacted with slender, cylindrical vibrators (Baumann and Bauer, 1974). The primary purpose of compaction is to densify natural soil to increase bearing resistance, reduce settlement, and minimize the collapse of large voids or susceptible soil. Despite the improvements offered by these technologies, some disadvantages exist for both types of this method. Dynamic compaction induces ground vibrations that can travel significant distances as well as lateral ground displacements. These could prove detrimental in urban areas with many nearby structures and embedded utilities. To control vibrations, lightweight tampers and low drop heights can be combined with limiting deep dynamic compaction to only the area within property line. Dynamic compaction may also be limited in terms of the depth of improvement. Also, for site consisting of very loose deposits, a layer of granular fill is required for the operation to provide a working platform for equipment and limiting tamper penetration at impact. Vibro-compaction is only effective for granular soils and may be limited in depth similar to deep dynamic compaction. Both of these specialty compaction methods are not typically performed by heavy highway and structure contractors, so a specialty ground improvement contractor must be involved in the project.

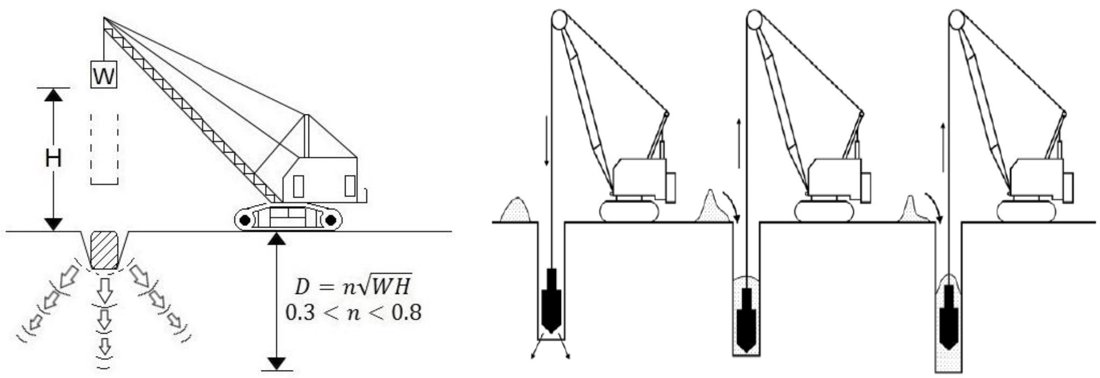


Figure 4.6. Schematic illustration of deep dynamic compaction (Left) and vibro-compaction processes (Right) (Schaefer et al. 2017a).

4.1.1.1.6. Deep Mixing Method (DMM)

The deep mixing method (DMM) is a ground improvement technique where in situ soils are blended with cementitious materials. It is also referred to as deep soil mixing or cement deep soil mixing (DSM/CDSM). Other materials such as binders may also be combined with the cement and soil. The result of the mixing process is a blended material of higher strength and lower compressibility than the untreated soil. Deep mixing can be operated either in a dry or wet approach (Bruce et al. 2013). Wet mixing introduces the cementitious materials and binders to the soil in a slurry form. The dry approach uses binders in powder form that react with the water already present in the soil. The wet method is more useful for a large area like embankment projects due to low mobilization costs and high design strength mixture, while the dry method can be used for a variety of project and have lower mobilization costs than wet method, though the design strength is lower as well. Figure 4.7 shows how an embankment can be reinforced with DMM. In terms of issues with this design approach, DMM has higher mobilization and unit costs than other design and ground improvement alternatives. It also necessitates use of specialized design in terms of construction, specifications, and QC/QA practices. Additionally, any kind of obstructions like cobbles, boulders, dense sand deposits, and buried logs can interfere the operation of equipment. Likewise, DMM can interfere with and potentially damage any buried utilities and structures.

However, compared to other ground improvement techniques, the geometry of the treated area is better known. As with other specialty ground improvement techniques, typical heavy highway contractors do not perform such work and a specialty contractor must be identified for the project.

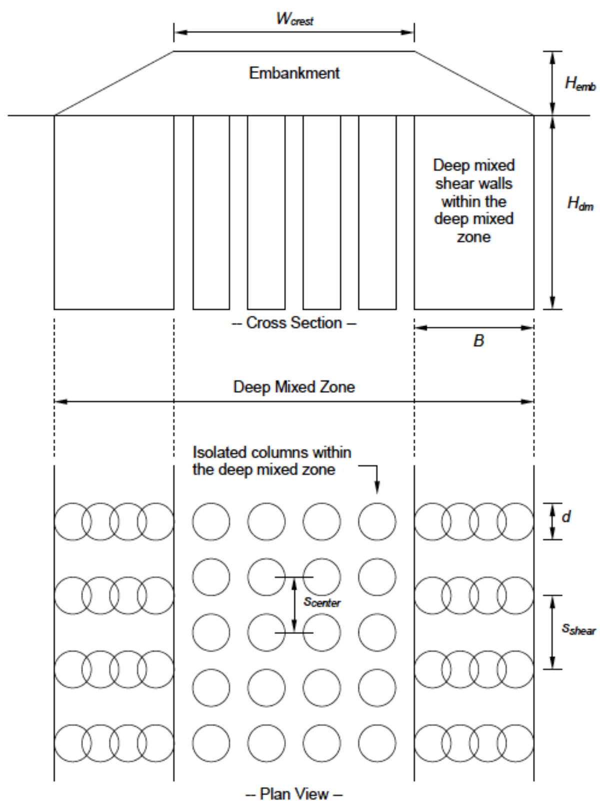


Figure 4.7. Typical arrangement for deep-mixed zone beneath an embankment (Bruce et al. 2013)

4.1.1.2. Culverts and Buried Structures/Utilities

Culverts are a kind of buried structure that allows flow of material (typically water) beneath a road, railway, trail, or similar system. Tunnels, pipelines, and other utilities are other forms of underground structures. Safety and performance of culverts are a chief concern among highway engineers. In fact, in urban environments, construction of embankments, retaining walls, and bridge abutments over or adjacent to buried structures is often unavoidable. However, the addition of more material over these buried structures can damage them due to the additional stresses and/or imposed settlements on the surrounding soils. Buried structures can be quite challenging to repair given their location below the ground surface and they are often a part of important infrastructure.

The pressure imparted on a buried structure can be influenced by many factors, including the characteristics of the surrounding soil, structure geometry, stiffness of the buried structure, and installation conditions in addition to the height of the fill over the structure and surface surcharges (Marston and Anderson 1913; Marston 1930; and Spangler 1950a). Buried structures are not typically designed to bear high pressure of overlying materials as such a design would be cost prohibitive and inefficient. Consequently, two approaches are typically recommended to address potential issues with excessive pressures on buried structures. First, it may be possible to replace and/or relocate any utilities or buried structures affected by the excessive pressures. However, this is typically not the case due to site constraints, logistical issues, and/or concerns regarding costs and effects on project schedule and staging. The other approach is to reduce earth pressure applied over the buried structure/utilities to prevent probable damages. Lightweight fills can play a role in this approach, though specific discussion of these materials is reserved for later sections of this chapter. Other methods to reduce pressures on buried structures include induced trench construction (imperfect ditch method) (Spangler 1950a) and geogrid reinforced soil platform bridge (El Naggar et al. 2015; Meguid et al. 2017), which are described in more detail below.

4.1.1.2.1. Induced Trench Method

When the vertical displacement over buried structures is greater than the adjacent soil, the earth pressure may be reduced due to positive arching theory. This concept has led to the development of the induced trench method (also known as the imperfect ditch or trench method). In this method, vertical displacement is induced intentionally by replacing part of the fill material over a culvert with compressible material. The column of soil above the culvert settles downward relative to the adjacent compacted soil, which mobilizes shear stresses that act upward on the interior prism of soil as shown in Figure 4.8. These shear stresses support part of the weight of the column of soil above the conduit and reduce the load on the culvert. In order for no localized differential settlements to occur in the overlying roadway, the fill above the culvert must be sufficiently high so that the induced shear stresses terminate at a horizontal plane within the embankment itself. In that scenario, no relative settlements and transfer of load takes place above that horizontal plane (i.e., plane of equal settlement). The design process therefore is concerned with computing the settlement ratio, which compares the magnitude of the relative movement between the compressible soil above the conduit and the adjacent soil. The theoretical loads on the conduit can then be estimated from the settlement ratio using chart-based solutions.

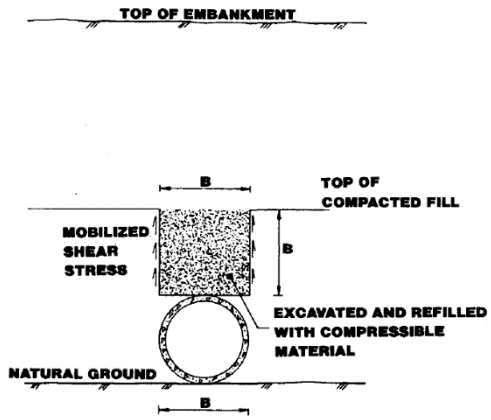


Figure 4.8. Schematic of the imperfect ditch method (Vaslestad et al. 1993).

4.1.1.2.2. Geogrid Reinforced Platform Bridge

A geogrid reinforced platform bridge is another method for reducing buried structure loads. This method works by bridging layers consisting of granular fill and one or multiple layers of geogrid reinforcement (Figure 4.9). This system transfers and distributes the stresses applied to the soil on top of the buried pipelines through geogrid layers towards piles that are driven into the weak subgrade soil. This concept is very similar to the load transfer platform described for column supported embankments.

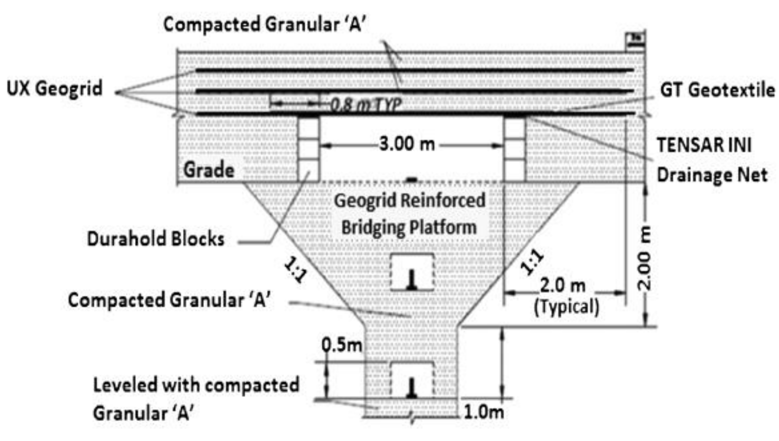


Figure 4.9. Geogrid reinforced platform bridge section (El Naggar et al. 2015).

<u>4.1.1.3. Earth Retaining Structures</u>

In many highway applications it becomes necessary to support fills or cuts with retaining walls. Design of these walls entails analysis of lateral earth pressures to ensure stability. Additionally, these earth retaining structures also must have adequate vertical support from their foundations to avoid bearing capacity failures and/or excessive settlement. Concrete walls, including those with pre-cast elements, are a common design alternative in many highway retaining wall applications. Alternative strategies include reinforced walls (i.e., MSE walls), anchor walls, grouting, soil nailing, and deep soil mixing. Some of these strategies refer to entirely new wall designs, while some describe methods by which to ensure adequate stability of any kind of retaining structure (e.g., by stabilizing foundation soils), including pre-cast concrete walls. The following sections briefly discuss these potential design alternatives.

4.1.1.3.1. Reinforced Soil Wall

Mechanically stabilized earth (MSE) walls contain soil reinforcement, backfill material, facing elements, and a foundation (Elias et al. 2001). The reinforcement length ensures the backfill material is stable enough to resist lateral earth pressures. Figure 4.10 shows different applications of MSE walls, including for highway infrastructure. MSE walls are often more cost-effective than typical reinforced concrete walls. However, they require more quality control during construction, larger space, use of select granular fills, and availability of more specialty materials such as suitable geosynthetics (or metallic strips).

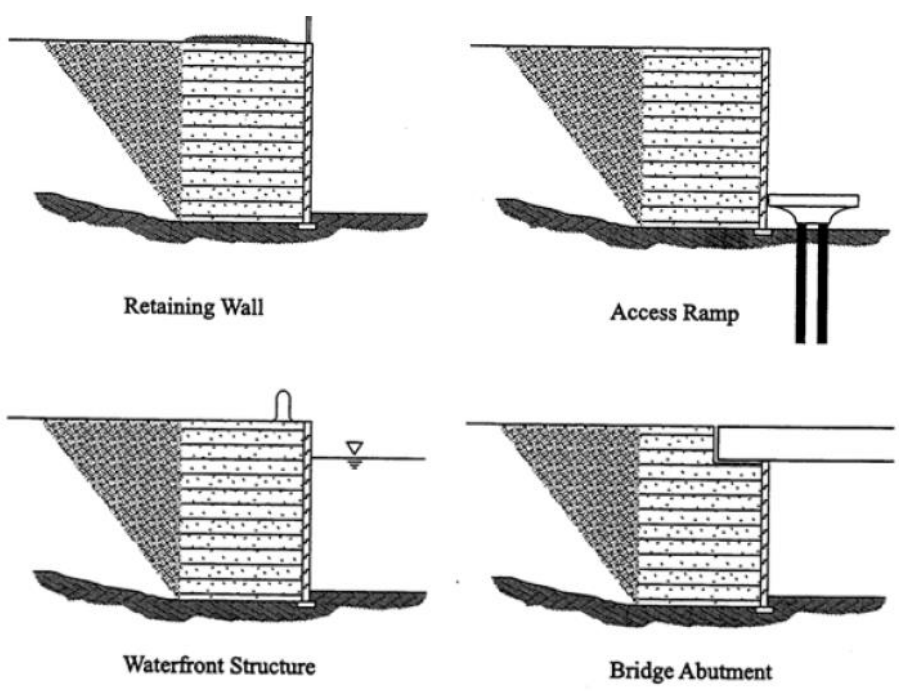


Figure 4.10. Representative MSE wall applications: Retaining Wall (Top-Left), Access Ramp (Top-Right), Waterfront Structure (Bottom-Left), and Bridge abutment (Bottom-Right) (Schaefer et al. 2017b).

4.1.1.3.2. Anchoring

A ground anchor is a structural system that includes anchorage, free stressing (unbonded) steel length, and bond length (Figure 4.11). Anchorage is a combination of anchor head, bearing plate, and trumpet (Sabatini et al. 1999). The system is installed by filling grout into drilled holes created by steel bars and transmit the prestressing force from the prestressing steel (bar or strand) to the ground surface or the supporting structure (oftentimes a retaining wall). The important benefits offered by this system are the ability to withstand large horizontal wall pressure, eliminate the need for select backfill, eliminate the need for deep foundation support, and unobstructed workspace for excavation. The anchoring system is a common useful method for earth retention systems (Long 2001) and soil stabilization (Koerner 2015). As an application, an anchored wall can be useful in grade separation to construct depressed roadways, roadway widenings, and roadway realignments in highways or any other locations where different elevations exist. Also, this system can be combined with walls, horizontal beams, or concrete blocks to stabilize slopes. The horizontal beams or concrete blocks help to transfer the ground anchor loads to stable soil. Disadvantages of this method include costs and difficulty in accommodating anchors in locations with limited clearance (e.g., urban environments, buried utilities, etc.).

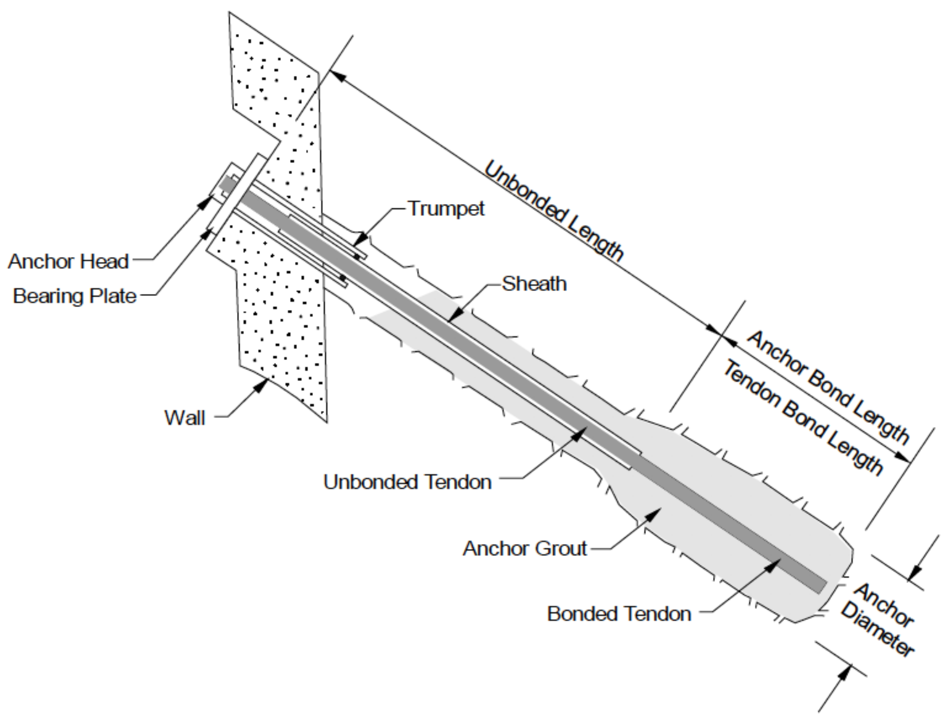


Figure 4.11. Components of a ground anchor (Sabatini et al. 1999).

4.1.1.3.3. Grouting

Grouting injects controlled quantities of cementitious material into the soil through small diameter boreholes. There are a number of ways to perform grouting with differences in its composition/mix and how the cementitious material is introduced into the soil: (1) slabjacking; (2) permeation grouting; (3) compaction grouting; (4) jet grouting; (5) soil fracture grouting; (6) fissure grouting (Lunardi 1997). Grouting can be used as a ground improvement technique to increase retaining wall foundation stability or as a way to generate an alternate wall design (Shen et al. 2012a). In particular, jet grouting can be used to construct grout columns and create earth retaining systems in the form of secant or diaphragm walls. This can be accomplished using either overlapping columns, staggered columns, or a truss design (Figure 4.12). An overlapping column is a row of columns spaced at varying distances and useful for short term protection. Staggered columns have an ease and speed of construction and are useful for long-term protection. Finally, the truss design method can confine the ground between the row of jet-grouted columns to help the entire system's stability. For slope stabilization, grouting can be used in the form of fan shaped, buttress, and caissons (Figure 4.12). For foundation soils, jet grouting can be used to improve the soils under spread footing and large diameter caissons. Spread footing on the improved ground is more useful for seismic areas, while large diameter caissons are suitable for foundations on slopes needing stabilization or in river channels (Figure 4.13). However, grouting in all the aforementioned applications suffer from some disadvantages, including: (1) difficulty in controlling heave, especially in cohesive soil; (2) unsuitability for underpinning; (3) need for specialty operators in some types; (4) need for specialty equipment; and (5) inability for some grouting types to work at the near surface without additional support.

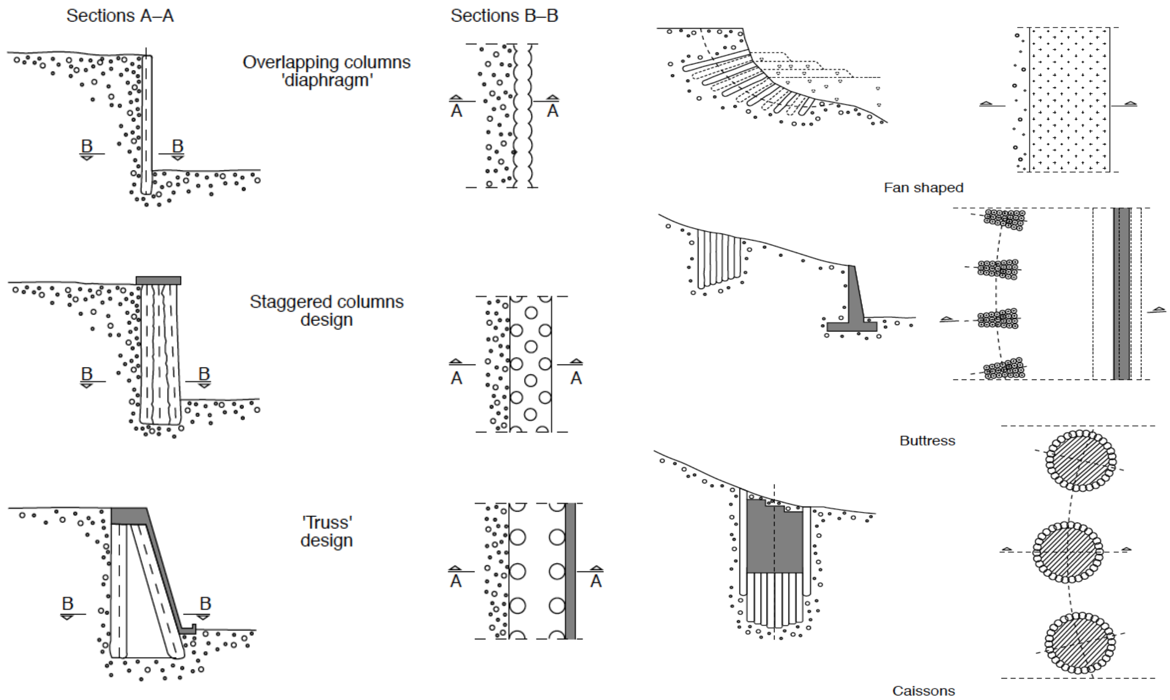


Figure 4.12. Typical applications of grouting for earth retaining structures (Left) and slope stabilization (Right) (Lunardi 1997).

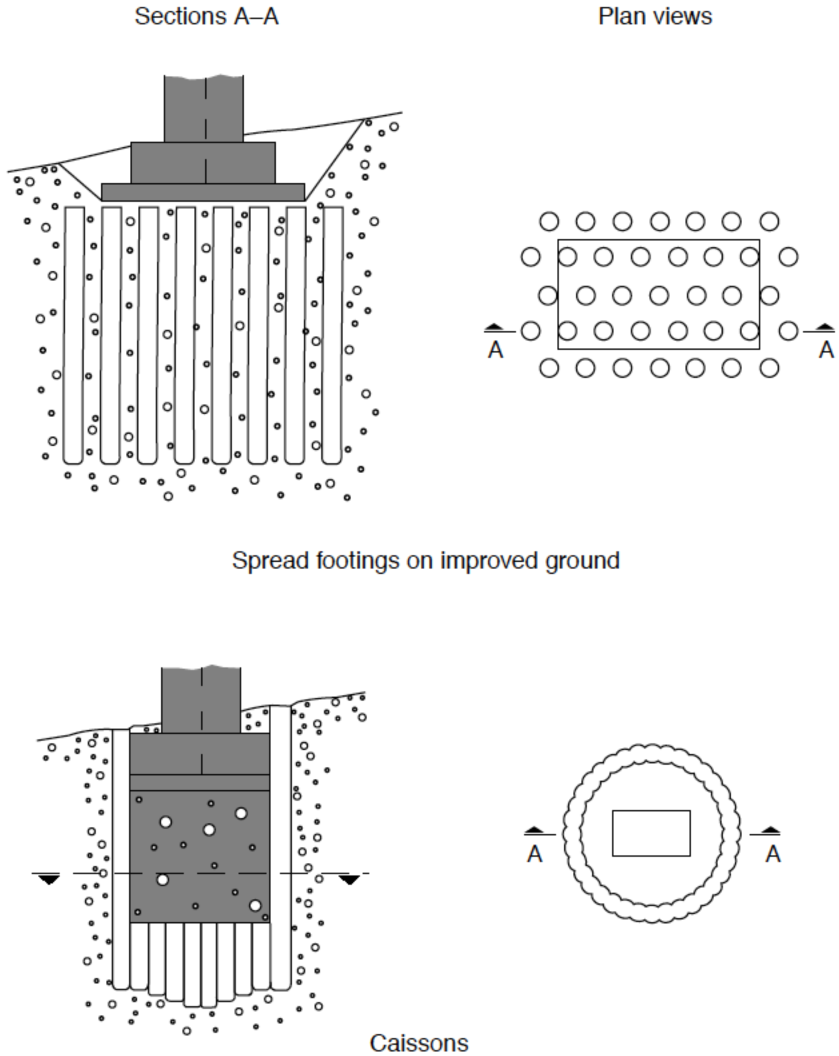


Figure 4.13. Typical designs for foundation work including grouting (Lunardi 1997).

4.1.1.3.4. Soil Nailing

Soil nailing involves the insertion of relatively slender reinforcing elements into pre-drilled holes in the soil followed by grouting. These reinforcing elements are typically inserted at a regularly spaced interval with a downward angle into a slope or behind a retaining wall. This process can involve either standard rebar elements as used in typical reinforced concrete construction, or proprietary solid or hollow-stem bars. For hollow-stem bars, the drilling is typically performed with a sacrificial drill bit so that insertion can occur simultaneously with grouting. Soil nailing occurs through a top-down sequence and is followed for each lift separately. The nails and initial shotcrete facing are installed for each lift, and then a final shotcrete or cast-in-place concrete (CIP) facing is installed (Figure 4.14). Roadway cuts, repair and reconstruction of the existing retaining structures, road widening under existing bridge abutments, tunnel portals, and shored mechanically stabilized earth (SMSE) walls are example applications for soil nailing (Figure 4.14). Soil nailing suffers from similar limitations to anchored systems, including the need for adequate space/clearance for construction, monitoring of deflections in case of critical wall or slope movement, and concerns with buried structures and utilities.

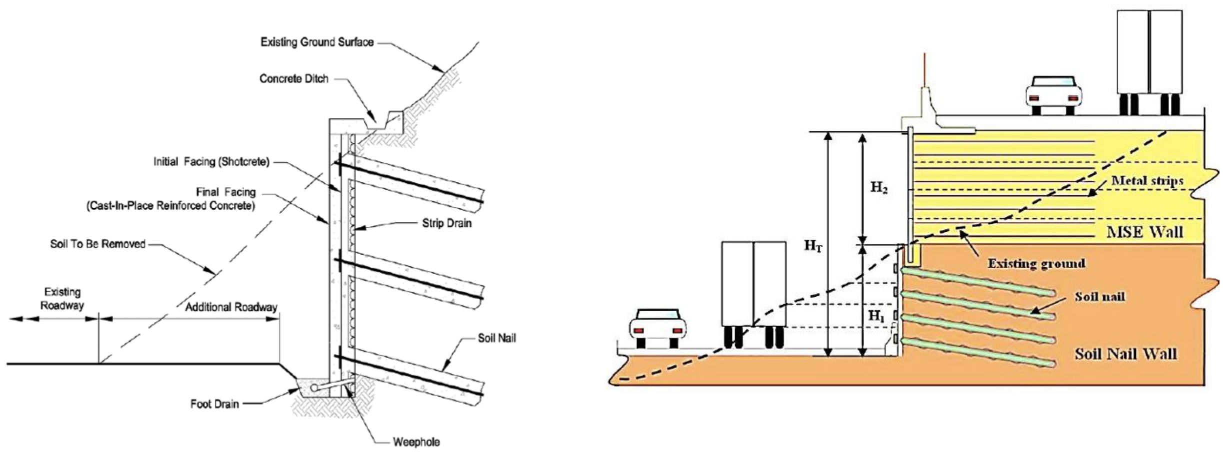


Figure 4.14. Example applications of soil nailing: (Left) roadway cut (modified after Porterfield et al. 1994); (Right) retaining wall (Wood et al. 2009).

4.1.1.3.5. Deep Mixing Method (DMM)

As mentioned previously, DMM is a useful way to stabilize the foundation layer of a vast area like an embankment. However, it can also be utilized for the foundation layer of a more limited area like such as an earth retaining system. This technique can increase the strength, decrease permeability, and reduce the compressibility of the material that will support a retaining wall. Figure 4.15 shows example applications where DMM has been used to stabilize foundation soils beneath retaining systems and other similar highway infrastructure.

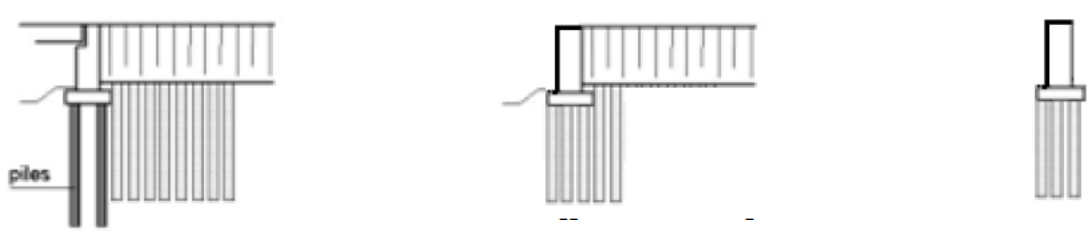


Figure 4.15. Examples of DMM for foundation soils: (Left) abutment; (Middle) retaining wall; (Right) bridge pier (Bruce et al. 2013).

4.1.1.4. Pavement Subgrades

The subgrade provides a suitable support layer for effective performance of a pavement system. The role of the subgrades for roadways is thus the same as a foundation for a structure. As with structures founded on weak layers, pavement constructed over weak layers and soft soils as the subgrade will encounter issues with settlement, bearing capacity failures, and similar geotechnical concerns. Soft near-surface strata are a common geological occurrence in many areas, especially near river floodplain where roadways and highways are often constructed. Moreover, other issues such as drainage problems, elevation changes, and similar concerns can come up for roadway projects. As a result, the first step of designing a roadway is ensuring a suitably stable subgrade. Stabilization of subgrade for pavement soil usually has three goals in mind: (1) creating a construction platform over problematic soil to facilitate placement and compaction of the remaining pavement layers, including subbase, base, and surface; (2) to increase the strength of weak subgrade and reduced compressibility issues; and (3) reduce moisture susceptibility of fine-grained soil. However, finding a proper way to stabilize the subgrade layer may vary according to the specifics of the site conditions and subsurface soils. Munfakh (2003) classified problematic subgrade strata as expansive soils, collapsible soils, karstic ground, liquefiable soils, and tropical soils and reported that each type has a unique group of remedial strategies for stabilization. Nevertheless, there are several methods that are often used for many problematic subgrade conditions, including chemical stabilization, mechanical stabilization (i.e., thick granular layers and blending, geosynthetic layer, and recycled material), and moisture control (Schaefer et al. 2017b). These methods are discussed further in the following sections.

4.1.1.4.1. Mechanical Stabilization

Mechanical stabilization refers to any design alternatives that improve the stability of the subgrade by introducing another material. This includes the use of granular layers and blending, geosynthetics, and/or recycled material. A thick granular layer and blending is mainly used to provide a construction platform for other pavement layers. This layer should be thick enough to spread any traffic loads properly and prevent any excessive stress level more than the bearing capacity of the subgrade; otherwise, it may result in rutting. Use of a geosynthetic layer has both features of separation and stabilization. Use of a geosynthetic in combination with a granular layer (base/subbase) helps prevent intermixing of the subgrade layer and increases the drainage properties of the pavement. Also, use of geosynthetics can reinforce the soil to reduce the thickness of gravel required for the working platform and improve the short term and long term bearing capacity of the roadbed support. Finally, recycled materials can be incorporated into the subgrade to improve performance. This can have the added bonus of reducing material costs and increasing the sustainability of the project. One common recycled material used in pavement support is recycled asphalt pavement (RAP), where old pavement that has been previously ripped up as part of a mill and overlay project is reused.

Each of the aforementioned mechanical stabilization methods has some potential disadvantages. For example, use of a thick granular layer and blending can be expensive in terms of material costs for high-quality aggregate. However, if the quality of aggregate is low, then problems can arise with respect to freeze/thaw performance and drainage. Also, blending of a granular material with an extremely soft natural subgrade may result in excessive intermixing within the soft soil layer and poor performance. Geosynthetics can increase material costs and selection of the most appropriate geosynthetic at a given site may necessitate additional laboratory testing and/or construction of test sections. Moreover, storage, handling, and placement of the geosynthetic material as well as compaction of the remaining pavement layers can introduce difficulties. Finally, recycled materials such as RAP are more variable in their composition and may not allow much control over material properties. For example, RAP may have lower CBR values than typical aggregate materials used for subgrades. Some recycled materials may introduce environmental concerns, may be susceptible to freeze/thaw degradation and corrosion when in contact with some materials like aluminum or galvanized steel pipes.

4.1.1.4.2. Chemical Stabilization

Chemical stabilization refers to the use of chemical admixtures (e.g., Lime, Cement, Fly-ash, Bitumen, etc.) with the existing subgrade soil to improve its performance. The purpose of chemical stabilization is to control and/or mitigate soil permeability and add cohesive shear strength of the resulting mixed material. Reducing permeability occurs due to filling or partial filling of the voids between soil particles. An increasing in shear strength also can occur as the result of binding the particles together and adding cohesion. The main disadvantages of this approach are the need for adequate curing time to allow the chemical changes to occur, dust control issues, and the need for lab testing as QC/QA method for some mixtures to ensure their long term performance with the native subgrade.

4.1.1.4.3. Moisture Control

Adequate control of the moisture content can ensure adequate compaction for the subgrade beneath a pavement system. To that effect, trench drains or horizontal blanket drains can be used as conventional methods to dewater and control the moisture. Additionally, use of geosynthetics and soil encapsulation are other methods to support drainage systems and protect moisture-sensitive soils from large variations in moisture content, respectively. However, there are some difficulties associated with attempts to control moisture of the subgrade. For example, the efficiency of any drainage systems may be difficult to predict, especially when one goal is to adequately resolve the required time for any subgrade improvements from moisture control.

4.1.2. Selected Lightweight/Sustainable Fill Materials

Lightweight and/or sustainable materials can serve as a suitable design alternative for the highway infrastructure applications described in the previous section. They can address some of the issues that prevent the effectiveness of typical designs (e.g., shallow foundations, soil removal and recompaction, deep foundations). Additionally, in some cases, lightweight/sustainable fill materials can outperform the alternative designs presented in previous sections, many of which were ground improvement techniques. For example, lightweight/sustainable fills may reduce costs due to simplified construction methods and lack of specialty equipment for installation. However, special considerations may be required for each material to ensure adequate performance when considering it as a design alternative. The following sections present these considerations for the most common lightweight/sustainable fill materials in highway construction, including Expanded Polystyrene (EPS) Geofoam blocks, Lightweight Cellular Concrete (LCC), Expanded Shale, Clay, and Slate (ESCS), and Foamed Glass Aggregate (FGA). Included is discussion of cost information as well as issues relevant to site activities prior to and during construction.

4.1.2.1. Expanded Polystyrene (EPS) Geofoam Blocks

Expanded polystyrene (EPS) is a low density product that has many practical applications across a range of industries. In the context of highway construction, EPS is usually made in the shape of rectangular blocks for use as a fill material. Its history can be traced to Oslo, Norway in the early 1970s after EPS Geofoam blocks were used for an embankment project. The following sections discuss some of the specific issues regarding EPS Geofoam blocks when considered as a design alternative for highway-related projects.

4.1.2.1.1. Typical Applications and Geometry

Use of EPS Geofoam as a lightweight fill material can be accommodated in most geotechnical construction. However, in a few cases like MSE walls, reinforcement layers like metallic strips anchored to the wall render EPS Geofoam blocks impractical. The following sections discuss geometry considerations for EPS Geofoam blocks in different highway infrastructure applications.

4.1.2.1.1.1 Embankments

Typical EPS Geofoam embankment geometry is trapezoidal, where with slopes that depend on the final elevation of the embankment. The construction configuration from the bottom to top is as follows:

- Placement of a leveling course (usually sand fill) at the base of the excavated area to create a leveled and free draining surface prior to EPS placement.
- Placement of the stepped-side EPS blocks.
- Placement of a protective cover (geomembrane) between EPS and the material on top of the blocks as well as the side.
- Placement of a granular fill layer to fill the void between EPS blocks and the slope sides.
- Placement of a pre-cast concrete (PCC) slab consisting of No. 19 steel reinforcing bars at a spacing of 254 mm (10 in.) at the interface of EPS block and the overlying pavement to reduce stresses and distribute applied from traffic loads toward the blocks. The presence of a concrete slab also helps to anchor various highway hardware such as safety barriers, signage, and lighting if the overburden applied over EPS do not prevent potential uplift of the blocks [Figure 4.16(a)].

4.1.2.1.1.2 Retaining Walls

The case of retaining walls shares similar geometry considerations to embankments, except that the blocks must be covered by a facing layer at the wall side to protect the EPS from damage caused by environmental factors. Typically, a sand layer is placed to allow a level base for the lowest layer of blocks near the abutment. Additional fill material is used to fill gaps between blocks and the sloping soil where the blocks are stepped. To fill this area, granular backfill material is the most suitable choice because it fills the narrow area properly and can play as a drainage system to remove any standing water behind the retention system and then prevent buoyancy effects within the EPS blocks. The selection of the type of facing material is typically based on the following items: (a) material must be self-supporting and physically attach to the EPS blocks; (b) architectural/aesthetic requirements; and (c) cost. Common facing materials include:

- Precast PCC panels, either full height or segmental (such as used in MSE walls)
- Segmental Retaining Wall (SRW) blocks, which are typically PCC
- Shotcrete
- Geosynthetic vegetative mats
- Exterior insulation finish systems (EIFS)
- Wood panels or planks
- EPS-compatible paint for temporary fills

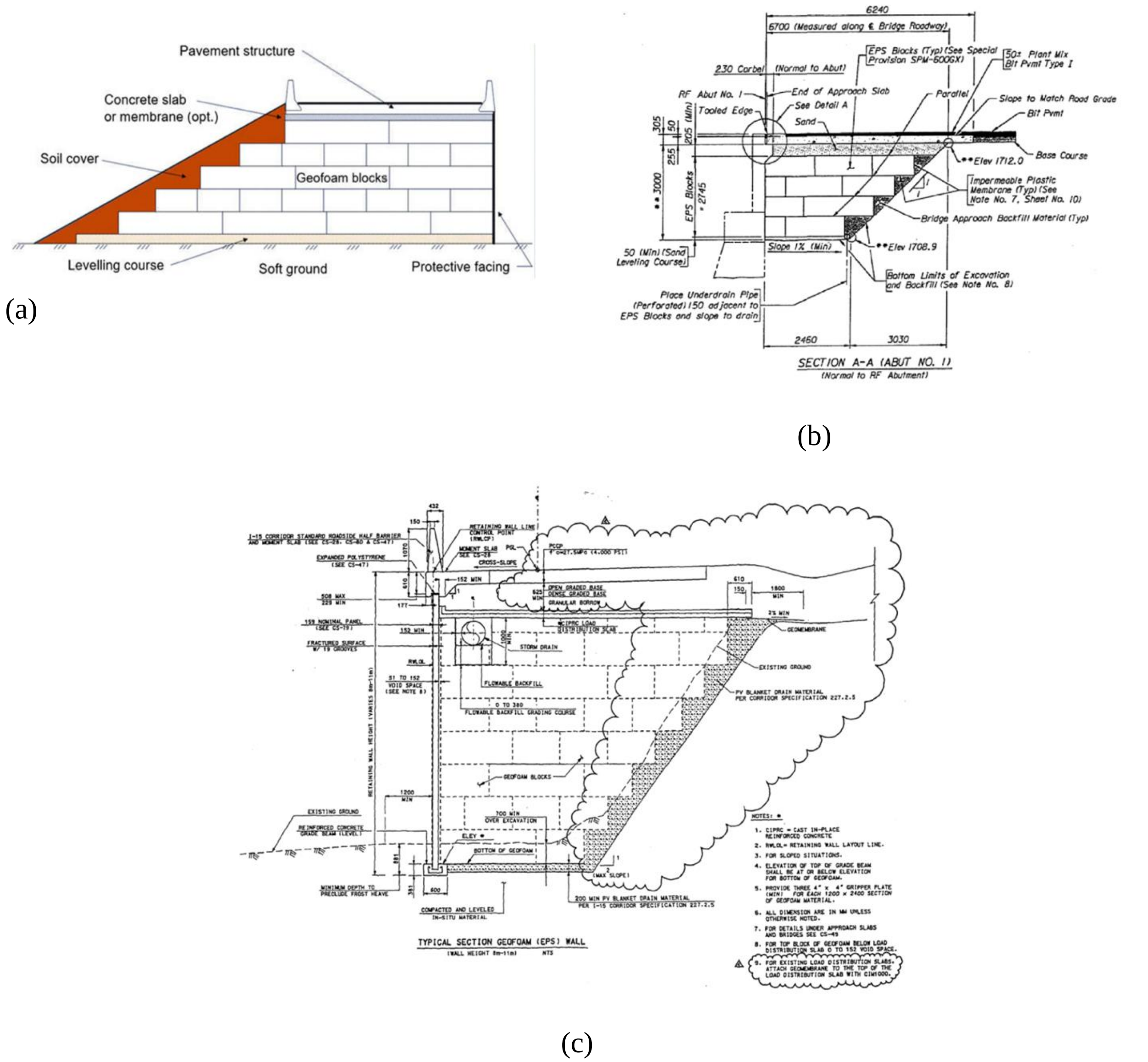


Figure 4.16. (a) Typical cross section of Embankment using EPS blocks (Aabøe et al. 2019), (b) Typical cross section of Abutment using EPS blocks (Right) (Stark et al. 2004), (c)Typical cross section of Retaining Wall using EPS blocks (Stark et al. 2004).

Of these materials, EFIS has been claimed to be the most compatible with EPS blocks due to the following advantages (Riad, 2005):

- Compatible with EPS blocks in terms of stiffness, deformations and other mechanical and material properties

- Significantly lighter weight can reduce applied loads on the existing subgrade
- Simplified design, construction, and maintenance based on the elimination of the pinned connections tying the exterior panels and the load-distribution slab located at the top of the EPS blocks
- It can be applied in a rapid manner at any time after the placement of EPS

After choosing a proper type of facing, a protective cover (i.e., geomembrane) is typically placed between the EPS and the material on top of the blocks as well as the sides. Finally, the blocks are often capped with a PCC slab for load distribution purposes as in embankment applications (Beinbrech and Hillmann 1997) [Figure 4.16(b)].

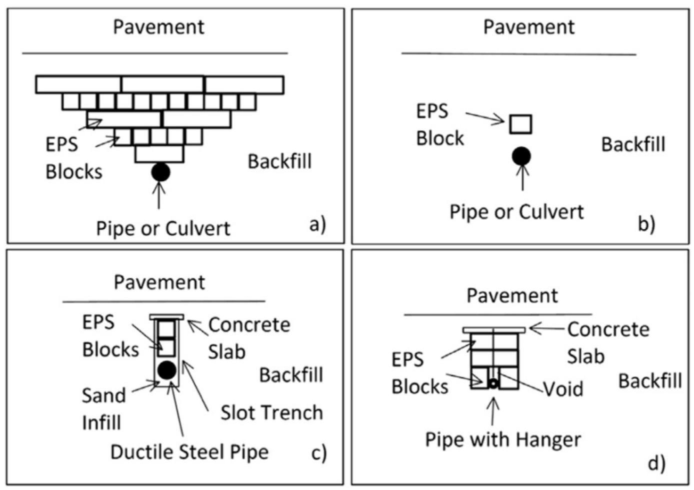


Figure 4.17. General methods of protecting pipelines and culverts from vertical ground displacement: (a) light-weight cover or embankment constructed over pipe or culvert; (b) "imperfect trench method" with compressible inclusion EPS block placed above pipe or culvert; (c) slot-trench light-weight cover system with EPS block placed in slot; and (d) EPS post and beam system with head space void.

4.1.2.1.1.3 Culverts and Buried Structures

For this purpose, EPS Geofoam can be used in different strategies such as lightweight embankments, imperfect trenches, slot-trench cover systems, and post and beam cover systems to mitigate the applied earth pressure over buried pipelines and culverts as shown in Figure 4.17 (Bartlett et al. 2015). The construction of a lightweight embankment over buried structures and/or the imperfect trench method has similar geometry considerations as regular embankments (section 4.2.2.1.1.1). Slot trench lightweight cover system is a design alternative created by placing EPS blocks on top of the pipe in the trench excavation where a bedding sand layer surrounds the pipe. The post and beam cover system is a method consisting of 0.6 m headspace void above the pipeline to allow for settlement of the foundation soil without imposing any forces on the soil. The system is completed by using two EPS blocks on the side as posts and placing EPS block capping on top as a beam.

4.1.2.1.2. Considerations Prior to Placement

Site preparation is a prerequisite step before placing EPS for geotechnical structures. Its correct execution is vital for internal stability and overall constructability otherwise. Site preparation may vary according to different geotechnical projects and need more or less evaluation process. For example, in the case of slope repair including EPS, the stability of the excavated area and the adjacent slope should be evaluated because the EPS blocks cannot resist against external applied earth forces from the adjacent slope material due to their low density. The development of cracks can indicate the need for temporary stabilization to prevent sloughing, otherwise an earth retaining system may be necessary.

After evaluating the excavated area, it is time to prepare the subgrade for EPS blocks placement. The prerequisite condition to construct a stable structure including EPS block is having a smooth and stable subgrade for the first EPS blocks layer. For subgrade preparation, the following considerations are necessary:

- The subgrade should be free of debris or large pieces of vegetation. Moreover, the soil particles at the subgrade should not be larger than sand to gravel with the particle size of 2 - 19 mm (0.08 - 0.8 in.) because the greater size can cause physical damage such as puncturing, gouging, or broken corners to the first layer of EPS blocks overlying on the subgrade.
- No standing water, accumulated ice, or snow is allowed since the presence of water can cause a problem to level and damage blocks.
- A drainage system must be placed below EPS to prevent water accumulation and/or between adjacent slope material and the EPS blocks to collect and divert seeping water and mitigate any potential seepage pressures. Permanent drainage system is typically overlaid on geotextile as a separate layer and is covered by a granular layer (Figure 4.18). In addition to a permanent drainage system, a temporary drainage system is required during the construction phase to dewatering any probable natural seepage; otherwise, during heavy rainfall, an uplift phenomenon may occur due to the low density of the blocks and subsequent hydrostatic uplift forces.

Figure 4.18. Placement of pipe drains to divert water away from the area where EPS-block Geofoam is to be placed (Alabama Department of Transportation).

- EPS blocks should not be placed on frozen subgrade unless in a case where the designer has considered the ground thawing process beneath the fill.
- Vertical deviation of the coarse sand/fine gravel bedding surface may be no more than +/- ½ in. over 10 feet distance. The role of this sand bedding layer is to cover the coarser in-situ material as well as leveling the first layer of blocks. In some cases, when removing or replacing the large aggregate size of the subgrade is impossible, a bed of sand with 12 to 25 mm thick can be placed over the existing foundation soil surface and separated with a geotextile layer to prevent intermixing of the sand bedding and subgrade soil.
- Using a granular drainage layer overlying a geotextile layer can be helpful by providing a stable working platform besides its main role, which is working as a drainage system.

There are also additional considerations regarding block damage since damaged blocks cannot be repaired or reused in a project. Consequently, precautions must be taken during shipment, handling, and storage at the site to reduce the risk of damage. For shipment issues, the most prominent problem is choosing the right method to transport the blocks to the site. Shipment can be accomplished with a tractor-pulled trailer either with a flat-bed or a closed box with each having advantages and disadvantages (Figure 4.19). For example, using enclosed trailers ensures less damage compared with an open flat-bed. In open flat-beds, EPS blocks should be securely strapped to prevent any movement during shipment. This secure strap can cause numerous indentations and breakage. To address this problem, structural angles can be used at the edges of exterior blocks where the straps have contact with the surface of blocks (Figure 4.20a). In contrast to the additional considerations of EPS transport in an open flat-bed, unloading the blocks is much easier due to access from all sides of the trailer. Consequently, unloading blocks from enclosed trailers is more time and labor intensive. In terms of unloading, various methods can be utilized, including specialized gripper lifting devices, forklifts, or straps connected to trackhoes. It is recommended to use a steel angle at every edge of EPS blocks to minimize the probable damage from straps and damage due to swinging the EPS blocks during lifting with a trackhoe (Figure 4.20).

Figure 4.19. (a) Unloading EPS blocks from open flat-bed trailer upon arrival at job site (Virginia Department of Transportation) (b) Unloading EPS blocks from closed upon arrival on site (Alabama Department of Transportation).

Figure 4.20. (a) Using a trackhoe to move EPS blocks and putting structural steel angles to protect bottom edges of blocks from damage due to straps. The top edges of the blocks are unprotected (Virginia Department of Transportation) (b) Moving EPS blocks to storage area using a forklift (Virginia Department of Transportation).

The final pre-construction consideration is stockpiling the EPS blocks at the site before placement. A secure storage area should be assigned for blocks based on the following attributes:

- It should not allow any vehicles or equipment to pass over the blocks.
- The storage area should be located away from any heat sources, tobacco smoking, or any construction activity that can cause ignitions due to the high flammability of the blocks.

- Blocks should be secured to prevent their movement by wind and subsequent potential injury of workers. The EPS block should be ballasted with counterweight from soil, sandbags, or similar material. A daily hazard assessment should be performed to ensure the proper condition of the environment for block placement, including insufficient wind speeds.
- In terms of UV damage protection, temporary storage is not required for short-term exposure to sunlight. However, for long-term protection, any dark colored geomembrane cover should be avoided since it may cause the blocks to melt, crumble, and otherwise distort. So, the best choice to cover the blocks from sunlight is using plastic sheets and secure the sheets with sandbags (Figure 4.21).
- When the air temperature goes below freezing, a thin layer of ice can develop on the exposed surface of blocks and cause them to be slippery and hard to handle. Consequently, the blocks should not be placed over other blocks because there is the potential of sliding due to water, wind, or other horizontal loads.
- Adequate temporary drainage is required to prevent flotation of the blocks.
- Damaged blocks can degrade the performance of the EPS fill. However, if the blocks are damaged and their use is unavoidable, the degraded surface can be potentially removed with a pressure washer (Bartlett et al. 2000).

Figure 4.21. Use of plastic sheeting to protect EPS blocks from UV exposure and sand bags to secure the plastic sheeting until embankment construction is completed (Virginia Department of Transportation).

4.1.2.1.3. Considerations During Placement

The construction phase of a project with EPS Geofoam blocks focuses on cutting blocks to appropriate sizes (if necessary) and block placement using equipment such as mechanical connectors or shear keys to keep the blocks matched together. Smaller blocks such as those that are 2.4 m in length (instead of 4.8 m) can be unloaded and placed more rapidly because they are easier to move. In some cases, it may be necessary to trim the blocks into a smaller shape prior to placement. Geofoam can be trimmed and placed around an irregular space to ensure continuous coverage. Figure 4.22(b) shows an example in which trimmed EPS covers the surface around a pipe to preserve the continuity of the row of EPS blocks. The amount of cutting that is performed in the field should be minimized if possible since it is a more laborious and expensive process compared to when performed during manufacturing. However, smooth and precise surfaces can be cut in the field with a hot wire cutting device. A chain saw or wire saw can also be used (Figure 4.22a). Although the use of a hot wire has not been documented to cause ignition of an EPS block, the availability of a fire extinguisher during the cutting process is recommended. Various methods exist to help to move the blocks, including by hand with an installation crew, use of straps to move each block, and use of a gripper carrying device or a scissor clamp (preferred to avoid damage) that allows two people to carry each block (Figure 4.23). However, the use of a gripper is not recommended since it may cause damage to blocks.

Despite being relatively simple to move around at a site, the placement of EPS Geofoam blocks does necessitate a number of important considerations in order to ensure adequate long-term performance. Much of the effort during construction centers around ensuring the blocks are stable and do not shift after they are placed. The following highlights some key considerations related to block placement:

- Blocks should be tightly matched on all sides.
- A minimum of two layers of Geofoam blocks are required beneath the road since one layer has the potential to shift under traffic loads and leads to premature pavement failure (Horvath 1999c).
- The placement of blocks should be in a way to minimize the vertical joints between them.
- The orientation of each layer alongside the longitudinal axes should be perpendicular to both below and above layer. In better words, it is necessary to place the long dimension of the blocks in each layer perpendicular to the long dimension of blocks at the subsequent layer. This job helps to minimize vertical shifting and sliding between blocks.
- The top surface of EPS blocks should be parallel with the final pavement surface, so any desired change in elevation grade should be assigned and adjusted by sloping the foundation soil surface prior to placement of blocks (PIARC 1997).
- The upper surface of EPS blocks should be horizontal.

- The calculated settlement of EPS configuration pattern should not exceed 1:200 (vertical: horizontal) (Briaud et al. 1997).

Phased construction is a common strategy in highway construction so that one part of the project can be completed before beginning the next phase. This typically ensures part of a roadway can remain open and usable while another part of the site is under construction. However, special care must be exercised in phased construction with EPS Geofoam blocks considering the preceding discussion regarding stability of the blocks (e.g., Figure 4.24).

(a) (b)

Figure 4.22. EPS block installation: (a) Using a hot wire cutter to cut EPS blocks, Woodrow Wilson Bridge (Virginia Department of Transportation); (b) trimming EPS and placement around a manhole.

Figure 4.23. EPS block installation: (a) Placement crews lifting and moving EPS blocks using gripper (Alabama Department of Transportation); (b) use of a scissor clamp to place blocks as part of the new Topaz Bridge project in Idaho (Horvath).

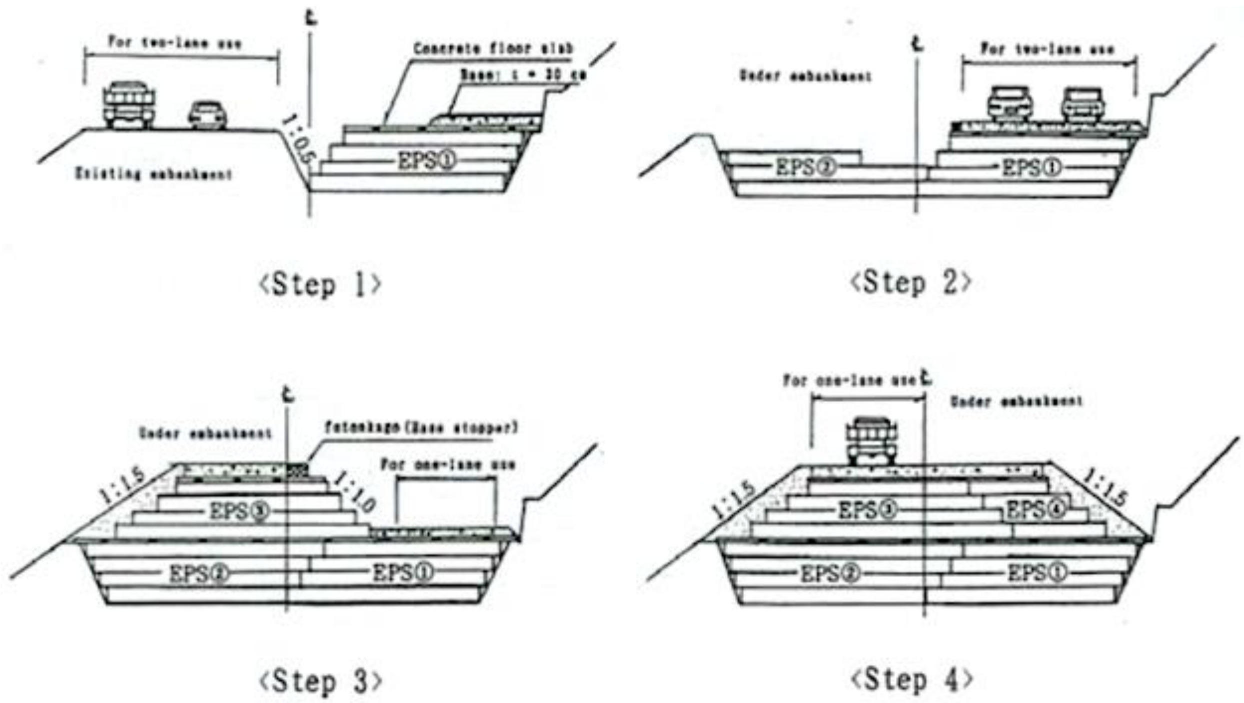


Figure 4.24. Cross-section view of potential method for phased construction of EPS-block Geofoam roadway embankment fills (Tsukamoto 1996).

Blocks should be tightly matched together on all sides to eliminate gaps at the vertical joints between the blocks. In placing EPS blocks, no gaps greater than 0.07 feet (20 mm) on vertical joints between adjacent blocks are allowed. Adhesive chemicals and/or metal connectors can be used to ensure blocks stay together. Urethane adhesive is a kind of glue that is compatible with EPS blocks and helps them prevent any horizontal movement or sliding. The metal connecters are typically prefabricated barbed metal plates and can be installed on horizontal surface blocks by piercing into them (Figure 4.25). These connectors can be used when the calculated resistance forces along the horizontal planes of EPS blocks cannot bear the horizontal driving forces. Mechanical connectors can help EPS blocks remain fixed in place when they are subjected to wet, icy, or windy conditions, and to prevent shifting the blocks under traffic where relatively few layers of blocks are used (Duskov 1994). It is recommended to use two connectors for each 4 ft x 8 ft section of EPS as a minimum requirement to prevent the block movement. However, it should be mentioned that the installation of these steel pieces can add a significant cost to a project, so they should be avoided in projects when their presence is not necessary. Additionally, it is not clear whether they are effective in resisting seismic loads based on conflicting studies into that issue (Sheeley 2000; Negussey et al. 2001). The other method to prevent EPS blocks movement is using shear keys. Shear keys can be used as a block or portion of a block of EPS and placed at various locations within the fill mass to interrupt the horizontal failure between layers of EPS blocks.

Figure 4.25. Photo showing arrangement of EPS blocks and placement of mechanical connectors (Virginia Department of Transportation).

The use of EPS block in situations with a shallow ground water table is not recommended. However, if the EPS Geofoam will be submerged, it is recommended to consider some additional overburden to counteract the buoyancy effect. As a rule of thumb, each meter (3.28 ft.) of submergence of a block of EPS below water requires 500 mm (20 in.) of normal-weight surcharge fill material on top of EPS. Or for every 100 millimeters (4 in.) of submergence of an EPS block, there must be 50 millimeters (2 in.) of soil or pavement on top of the EPS blocks to waive the buoyancy effect on EPS blocks. Obviously, using an excessive amount of overburden fill material or pavement cannot be helpful in conditions where the groundwater level may rise due to extreme floods. Because this excessive volume of material over EPS blocks may cause some detrimental effects such as excessive settlement and/or instability in crisis phenomena like a flood or heavy rain. As an alternative, using vertical ground anchors can be helpful for resisting uplift forces of EPS blocks and prevent the blocks from movement along all sides. This system can work by penetrating the EPS blocks and be embedded in the underlying foundation soil. The head of anchors can be covered by a reinforced concrete slab cast over the surface of the EPS blocks.

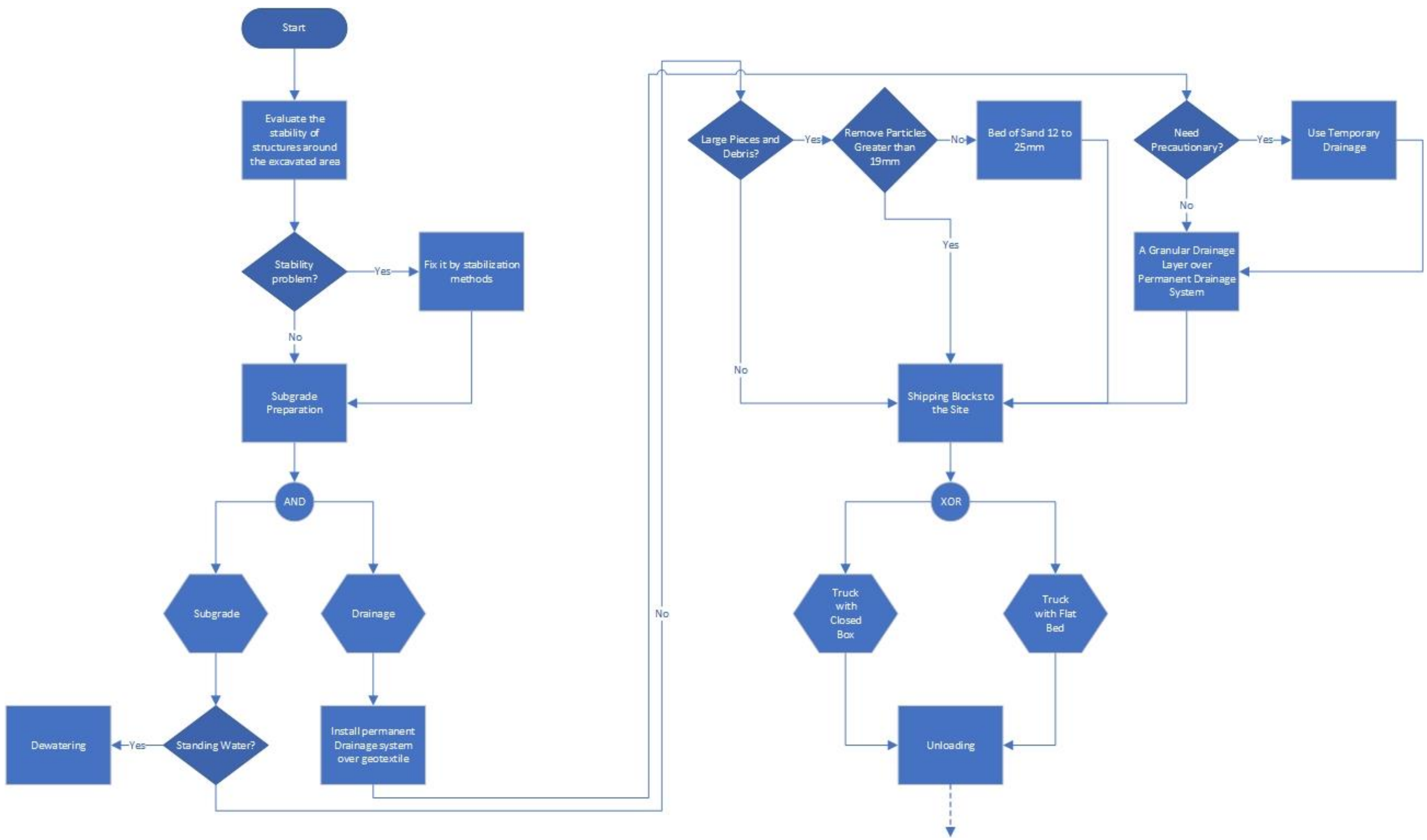
As with pre-construction considerations, it is recommended to avoid placing the EPS Geofoam blocks in contact with any chemical or heat sources due to the high combustion potential and sensitivity to chemical substances. In fact, EPS blocks should be protected using a geomembrane to prevent contact with all organic solvents such as acetone, benzene, and paint thinner; hydrocarbons; chlorinated hydrocarbons; ketones; ethers; esters; petroleum based solvents such as gasoline and diesel fuel; and open flames. EPS blocks should be considered a combustible product, so they should be avoided from open flame or any ignition source. It should be noted that if any blocks are chemically or physically damaged, they should be removed and replaced with new blocks.

After placement of the EPS blocks is complete, they should be separated from additional support layers (e.g., pavement layers) with either a geogrid, a geomembrane that resists hydro-carbon spills, geocell with soil, geotextile with soil, or a soil-cement mixture. This offers two advantages. First, it can enhance the overall performance and life of the pavement system by providing reinforcement, separation, and/or filtration. Second, it can enhance the durability of the EPS blocks so that they are not damaged, unleveled, or moved. A PCC is another separation layer that is useful when traffic flow will take place over EPS blocks. The PCC can provide sufficient lateral confinement of unbound pavement layers as well as load distributing of upper load traffic and mitigate its effect towards the EPS blocks. The thickness of this layer is typically 100 to 150 mm (4 to 6 in.). However, use of a PCC slab represents a significant cost, so it must be used if only determined necessary during design.

Vehicles and construction equipment such as earthmoving equipment should be avoided on the EPS block or separation layer. For example, guidelines from the United Kingdom recommend the maximum allowable weight of compaction equipment to be limited to 4 kips/ft of roll and a maximum applied pressure of 400 lb/ft^2 (Sanders and Seedhouse 1994). Another approach considered to minimize the damage to EPS blocks is to use lightweight equipment to push around 300 mm (12 in) of soil or aggregate on the EPS or separation layer prior to compaction of the material for pavement construction. This activity would be done by using small bulldozer or front-end loader. Any additional unbound and bound pavement layers can be placed normally; however, traffic flow of the surface by trucks or heavy equipment should be minimized until the pavement is completed. To compact the material of a pavement structure located over EPS, a plate vibrator has been found to be the most suitable (Duskov 1997). Use of the nuclear density gauge is not recommended for verifying compaction of the pavement sublayers because EPS contains hydrogen so the device may show spurious water content. In this case, it is better to use traditional procedures such as a sand cone penetrometer to obtain the total unit weight or density of the unbound material followed by oven or other traditional methods in order to dry of soil samples and determine the water content.

4.1.2.1.4. Summary of Considerations

The preceding discussion highlights several of the key considerations related to use of EPS Geofoam blocks as a design alternative in highway construction. It is clear from this discussion that special care must be exercised when using this technology to ensure adequate long-term performance. Figure 4.26 presents a useful flowchart that highlights the details of EPS-specific considerations at a site.


Start
Evaluate the stability of structures around the excavated area
Stability problem?
Yes
Fix it by stabilization methods
No
Subgrade Preparation
AND
Subgrade
Drainage
Standing Water?
Yes
Dewatering
Install permanent Drainage system over geotextile
No
Large Pieces and Debris?
Yes
Remove Particles Greater than 19mm
No
Bed of Sand 12 to 25mm
Yes
No
Need Precautionary?
Yes
Use Temporary Drainage
No
A Granular Drainage Layer over Permanent Drainage System
Shipping Blocks to the Site
XOR
Truck with Closed Box
Truck with Flat Bed
Unloading

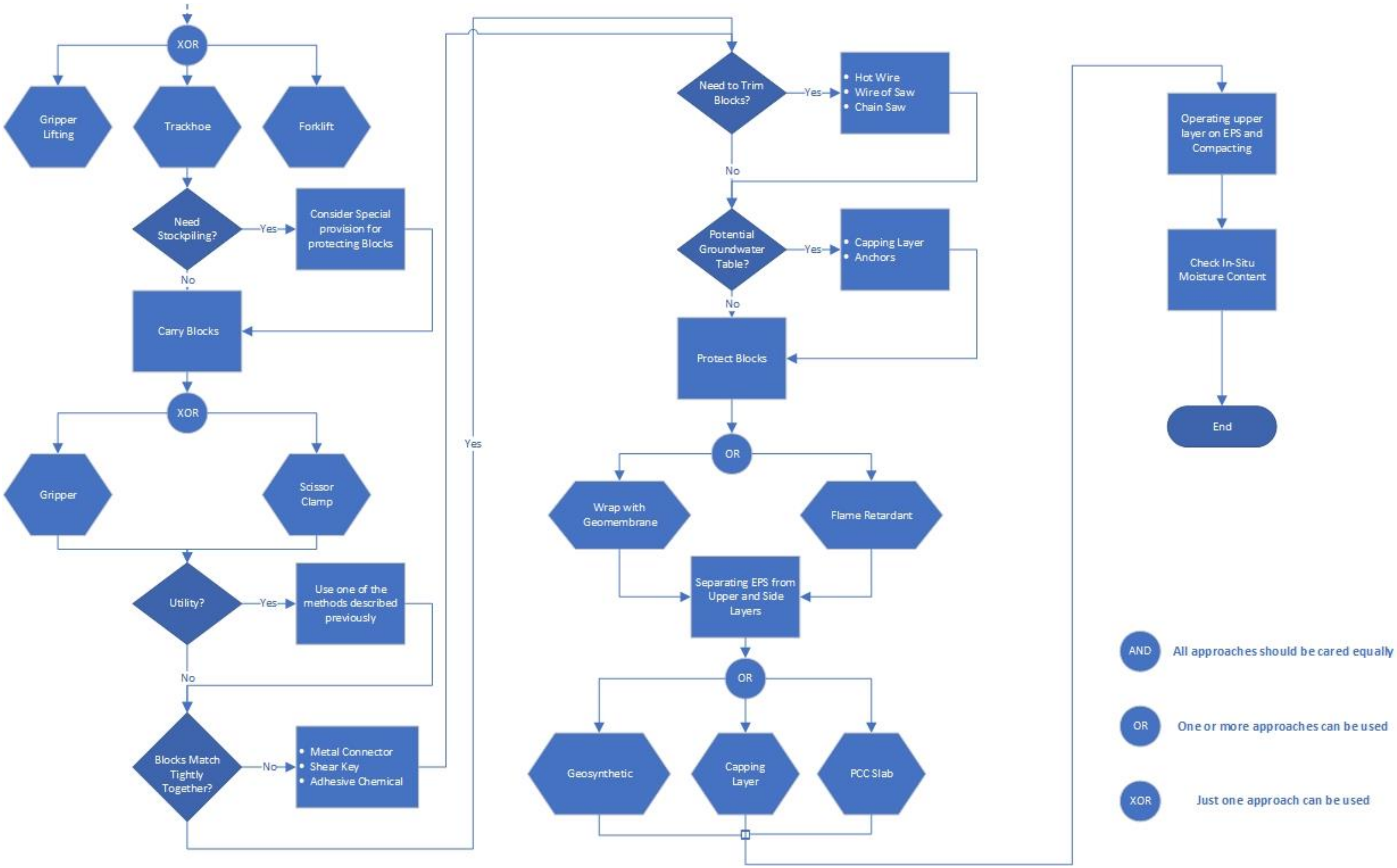


Figure 4.26. Flowchart Guideline for EPS Operation.

4.1.2.1.5. Cost Information

Table 4.2 provides cost information for EPS Geofoam separated by whether the costs are estimated at source, delivered, and/or in-place. The wide range in cost estimates is a result of the many factors that affect costs, such as material costs, transportation costs, quantity of material, availability of the material in addition to placement and/or compaction costs. The basic of ingredients for producing EPS blocks is by-products of fossil fuels, so the cost of material production at the source is directly dependent on fluctuations in the price of oil. Though Table 4.2 provides an approximate range in EPS costs, there are several factors that ultimately affect the overall costs associated with a project. This makes it difficult to perform a comprehensive costs analysis for EPS Geofoam (or any other lightweight fill technology) across different projects and establish a precise range in costs because no two projects are ever the same. Nevertheless, review of several case histories can provide some context for how EPS Geofoam compares to other design alternatives when applied to specific highway projects.

Table 4.2. Typical range in costs for EPS-block Geofoam at Source, Delivered, and In-Place (Schaefer et al. 2017a).

Material	**Material Cost/yd^3 FOB at source**	**Delivered Material Cost/yd^3 FOB at Project**	**In-Place Cost/yd^3**
EPS-block geofoam	\$40 to \$80	\$40 to \$80	\$40 to \$100

The first case history compares the costs of two embankments in terms of using EPS as fill material and the total removal and replacement (TRR) of the peat with borrow fill. As shown in Table 4.3, the estimated cost of the removal and replacement option was \$339,617 (about 28 percent) more than the EPS embankment option. Despite the need for additional materials and construction (i.e., concrete slab, EPS blocks, and compacted crushed aggregate), the use of EPS Geofoam reduces the costs associated with the peat excavation relative to the TRR case. This is because, no excessive stresses are applied by the EPS blocks and so full replacement of the peat is not required. This case history highlights how EPS Geofoam can be an economic alternative to traditional removal and replacement of problematic soils in highway embankment applications. In addition to the cost savings, EPS blocks provide other benefits relative to the TRR design alternative, including: shorter construction time, no dewatering, no need for sheet piling, and no additional right of way.

In another case history, two MSE walls with EPS fills and conventional fill were compared by the Wyoming DOT. Based on Table 4.4, the total estimated cost of the design based on EPS blocks is \$21,036 (28 percent) more than the original MSE wall option. The significant portion of this difference is \$37,000, which is attributed to additional materials and labor for the EPS blocks, geomembrane, sand base, as well as labor to drill holes in the abutment for monitoring instrumentation. It is reported that since it was the first EPS-block bridge approach project in

Wyoming, the higher price was likely related to a lack of experience by local contractors. Nevertheless, the case history noted the ease and speed of EPS-block placement and the possible placement of EPS blocks in adverse weather conditions as benefits that made EPS a better design alternative for this project.

Table 4.3. Cost Comparison Between EPS and Removal/ Replacement Alternatives for State Route 109, Noble County, Indiana (Zaheer 1999).

Items	**EPS Embankment (Plan)**				**Total Removal & Replacement (Option)**			
	Quantity	Unit	Unit Cost in U.S. Dollars	Cost in U.S. Dollars	Quantity	Unit	Unit Cost in U.S. Dollars	Cost in U.S. Dollars
Common Excavation	2,318.9 (3,033)	m^3 (yd^3)	\$5.23 (\$4.00)	\$12,132	2,318.9 (3,033)	m^3 (yd^3)	\$5.23 (\$4.00)	\$12,132
Peat Excavation	7,703.7 (10,076)	m^3 (yd^3)	\$5.23 (\$4.00)	\$40,304	77,256.7 (101,048)	m^3 (yd^3)	\$5.23 (\$4.00)	\$404,192
Borrow Fill	0	0	\$0	\$0	81,119.3 (106,100*)	m^3 (yd^3)	\$6.54 (\$5.00)	\$530,500
Concrete Slab	3,066.1 (3,667)	m^2 (yd^2)	\$40.66 (\$34.00)	\$124,678	0	0	\$0	\$0
EPS Block	4,708 (6,157)	m^3 (yd^3)	\$86.58 (\$66.20)	\$407,593	0	0	\$0	\$0
Compacted Crushed Aggregate No. 8 Stone	1,360.8 (1,500)	Mg (ton)	\$16.53 (\$15.00)	\$22,500	0	0	\$0	\$0
Total Cost				\$607,207				\$946,824

Table 4.4. Cost Comparison Between EPS and MSE Approaches for the Moorcraft Bridge Structure (Stark et al. 2004)

ITEM	**UNIT**	**UNIT COST**	**QUANTITIES for MSE Wall**	**QUANTITIES for EPS Wall**	**MSE COST**	**EPS COST**
Misc.Force Account Work	-	-	-	-	\$15,000	\$37,000
Dry Excavation	m^3 (yd^3)	15.70 (12.00)	1,192.7 (1,560)	1,529.1 (2,000)	\$18,720	\$24,000
Bridge Approach Backfill	m^3 (yd^3)	20.93 (16.00)	856.3 (1,120)	642.2 (840)	\$17,920	\$13,440
Bridge Approach Fill Reinforcing Fabric	m^2 (yd^2)	1.67 (1.40)	4,214.1 (5,040)	3,160.6 (3,780)	\$7,056	\$5,292
TOTAL					\$58,696	\$79,732

A third case history examined two bridge approaches and compares EPS against MSE walls (Stark et al. 2004). The estimated volumetric costs for the EPS and MSE Walls were \$202 and \$100 per

m^3 (\$154 and \$76 per yd^3) of wall. Although the cost of using an EPS approach fill was \$9,330 (32 percent) higher than the MSE Wall, the EPS block approach fill system was used on one side because of its speed of construction and because of its lightweight properties (Table 4.5). The biggest cost difference was the material costs of the 146 m^3 EPS blocks, which highlights that using EPS in smaller applications such as retaining walls may not be as cost effective as larger applications such as embankment fills.

Table 4.5. cost Comparison Between EPS and MSE Approaches for the N.F. Shoshone Bridge Structure (Stark et al. 2004).

	EPS Block Approach Costs			MSE Approach Costs		
Item	Quantity	Unit Cost	Cost	Quantity	Unit Cost	Cost
Dry Excavation	210 m^3 (275 yd^3)	\$18.00 (\$13.76)	\$3,780	230 m^3 (300 yd^3)	\$18.00 (\$13.76)	\$4,140
Bridge Approach Backfill Material	20 m^3 (26 yd^3)	\$30.00 (\$22.94)	\$600	210 m^3 (275 yd^3)	\$30.00 (\$22.94)	\$6,300
Geotextile for Foundation Separation	110 m^2 (132 yd^2)	\$6.75 (\$5.64)	\$742	0	0	\$0
Geotextile for MSE	0	0	\$0	850 m^3 (1112 yd^3)	\$1.75 (\$1.34)	\$1,487
Expanded Polystyrene Blocks	146 m^3 (191 yd^3)	\$104.00 (\$79.52)	\$15,184	0	0	\$0
HDPE Geomembrane	200 m^2 (239 yd^2)	\$4.50 (\$3.76)	\$900	0	0	\$0
Reinforced Concrete Approach Slab	88 m^2 (105 yd^2)	\$96.00 (\$80.27)	\$8,448	88 m^2 (105 yd^2)	\$96.00 (\$80.27)	\$8,448
Underdrain Pipe (Perforated) 150 mm	6 m (20 ft)	\$34.50 (\$10.52)	\$207	4.5 m (15 ft)	\$34.50 (\$10.52)	\$155
Underdrain Pipe (Non-Perforated) 150 mm	15 m (49 ft)	\$31.00 (\$9.45)	\$465	15 m (49 ft)	\$31.00 (\$9.45)	\$465
		Total Cost =	\$30,326		Total Cost =	\$20,995

<u>*4.1.2.2. Lightweight Cellular Concrete (LCC)*</u>

Lightweight Cellular Concrete (LCC) is another low density product made of a foamed agent, water, and cement. Unlike EPS Geofoam, the LCC material itself is essentially cast in place whereby bulk cement is trucked to the site and batched on a truck-mounted plant to produce a slurry that is combined with preformed foam. The following sections discuss some of the specific issues regarding LCC when considered as a design alternative for highway-related projects.

4.1.2.2.1. Typical Applications and Geometry

For the case of LCC, it is more common as a fill layer in smaller scale structures compared with other alternatives (e.g., EPS Geofoam), though this product is capable of offering benefits for other applications mentioned in previous sections. For example, PennDOT constructed a ramp embankment with LCC to eliminate a low-lying bridge structure. Given its primary usage for small scale projects, the focus of this section will primarily be on retaining walls and bridge abutments where LCC is more commonly considered.

In case of backfilling an existing abutment of a bridge: (1) Removing a portion of existing backfill material. (2) A geocomposite layer is required between the LCC and select structural backfill to collect any subsurface drainage. The layer will be extended towards the outlet drainage system located parallel to the abutment. (3) The inclined section of LCC must be at a maximum Slope of 1.5H: 1V. (4) Aggregate mat consists of coarse aggregate may necessary to distribute the structure pressure more uniformly to the foundation soil and is located from the bottom of the proposed footing elevation to top of the existing footing (Figure 4.27).

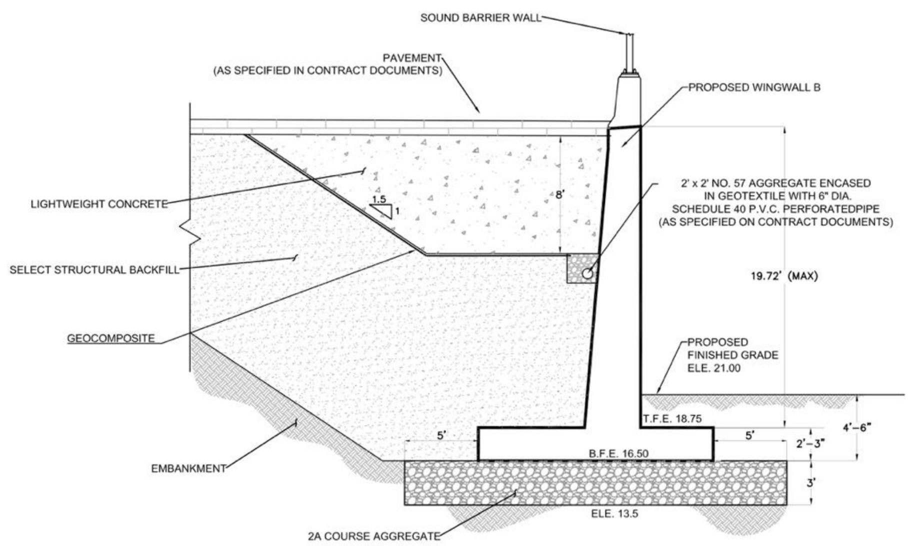


Figure 4.27. Typical cross section of abutment using LCC.

In the construction of retaining walls: (1) the retaining wall should be supported on 6 inches leveling pad, (2) the coarse aggregate mat should be placed underneath the wall. (3) The fill material usually includes LCC, select granular material, and select backfill material with different portions based on the designed process. The slope of the inclined section must be the same as the case of abutment with a maximum of 1.5H:1 V. (4) A geocomposite layer will be installed behind the LCC at the slope of 1.5H:1 V and will be continued towards the end of the vertical portion to collect any subsurface drainage. (5) The panels backfilled with LCC must be anchored to the ground and each other, or to the cured LCC to resist the fluid pressure until the LCC has set sufficiently. (6) Panel joints will be covered with geotextile with a small apparent opening size to prevent any material leakage. (7) It is required to finish the top of each lift with a slight slope away from the wall face to prevent ponding and the possibility of absorption into LCC (Figure 4.28).

For MSE Walls constructed with LCC, (1) each lift must cure before starting the subsequent lift. Each lift must be followed the typical thickness of vertical spacing (30 in.) for the reinforced MSE Wall. (2) The metal strips used in MSE Walls as reinforcement layer should have at least 6 inches of LCC fill material as their coverage layer. (3) As with the case of retaining walls, the panels backfilled with LCC must be anchored to the ground and each other, or to the cured LCC to resist the fluid pressure until the LCC has set sufficiently. (4) Panel joints should be covered with geotextile with a small apparent opening size to prevent any material leakage (Figure 4.29). After considering the geometry considerations, the process is followed by required activities and operations prior to and during the construction phases (Harbuck 1993; Taylor and Halsted 2021).

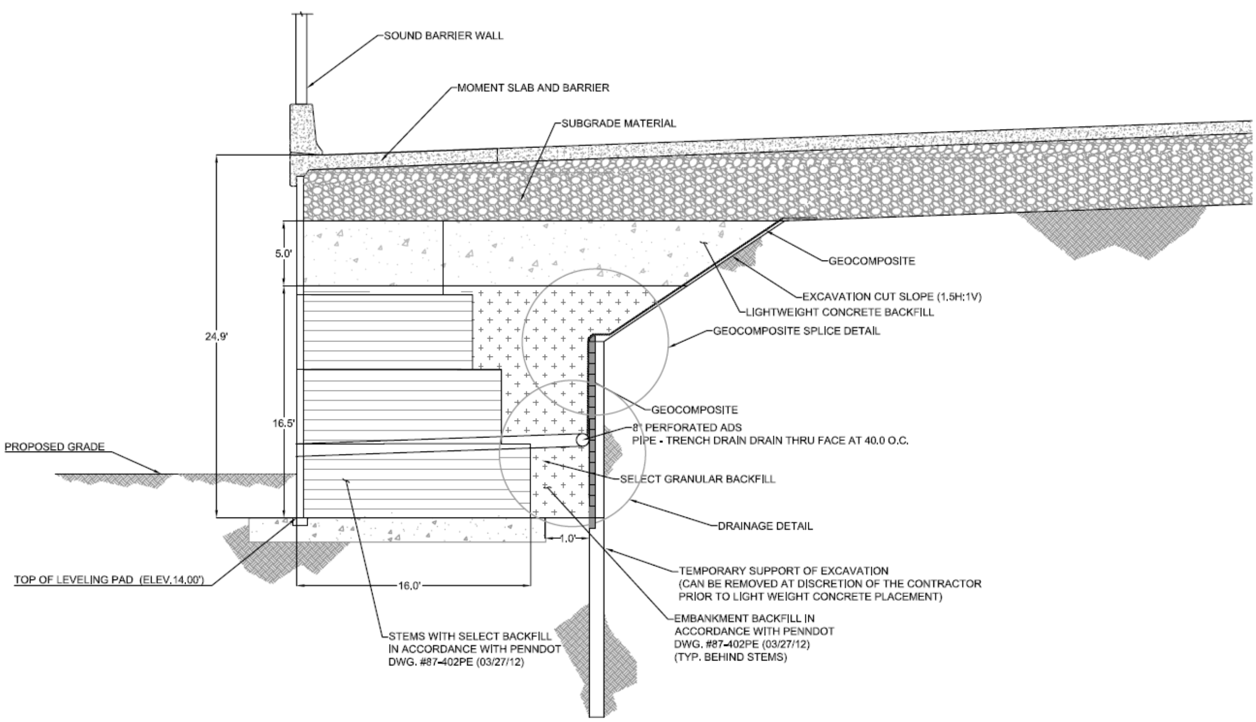


Figure 4.28. Typical cross section of retaining wall using LCC.

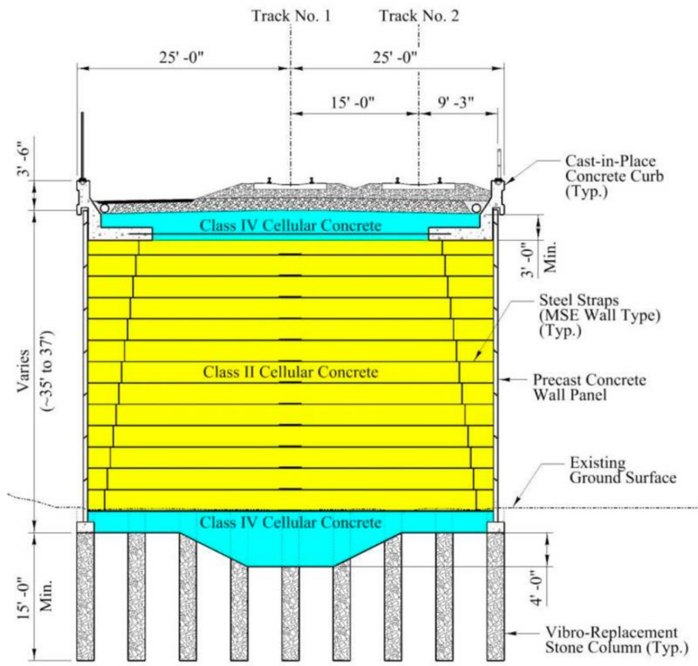


Figure 4.29. Typical cross section of MSE wall using LCC.

4.1.2.2.2. Considerations Prior to Placement

The keys aspects of LCC prior to placement include transportation considerations, on-site mixing of the LCC, and selection of a proper pumping device. It is recommended that the LCC should not be allowed to set and then be remixed. Instead, it should be kept plastic until allowed to set in its final location. Transport of a ready mixture of LCC to a job site is therefore avoided because the excessive vibrations and/or prolonged periods of transport may change the properties of the primary design of the mixture.

With respect to mixing, the cement, water slurry, and foaming agent are transferred separately to the job site and then the slurry is produced based on experiment design for the specific needs of the project. To mix the initial ingredients of LCC in site location, there are two types of productions system: batch mixing and auger mixing. Batch mixing is used when high accuracy is required, and its production rate is 30 to 50 yd^3 (22.9 to 38.2 m^3) per hour. On the other hand, auger mixing is typically produced through a mobile volumetric concrete truck and involves a rotating shaft. Its production rate can vary from a standard 30 yd^3 (22.9 m^3) per hour to up to 500 yd^3 (382.3 m^3) per hour for the largest equipment. The required equipment for the preparation of LCC mixture with the batching method is a unit to dilute and mix the foaming agent, a mixing/calibrating unit, a cement truck with a hopper to measure the cement, and a water tanker. The measured foam agent is placed in a dilution chamber and is mixed with water. This mixed product is routed to a mixing/calibrating unit where measured cement is already added. Afterward, the mix is ready to be pumped into place.

In general, there are three types of pumping devices available for LCC: progressive cavity, peristaltic, and piston pumps. A progressive cavity is considered as the most reliable method for pumping LCC. This pump is extremely steady and does not have any pulsing during the operation. However, it should be avoided from dropping any objectives like rocks or other solids in it. Peristaltic (squeeze pump) can easily transport the LCC. Its significant benefit is separating cementitious material from the pumping mechanism, which is very helpful when the mixture is sticky. Finally, piston pumps utilize a check valve and a piston retracting system. This pump is not required for cases where many surfaces need to be cleaned and high pressure pumping potential exists.

Figure 4.30. LCC placement at the LCG Columbia Storage project (Source: Cell-Crete Corporation).

Figure 4.31. Placement of separated LCC lifts.

4.1.2.2.3. Considerations During Placement

Placement of the LCC is the first phase of construction and be operated by pumping through a hose as shown in Figure 4.30. For placement, the issues relevant to the condition of the area, time of placement, and the thickness of lift are important. First, the condition of area should be prepared by scarring the surface. If the placement area is large, it should be divided into smaller sections to ensure ease of pumping into lifts (Figure 4.31). After preparing the area, the time of placement and thickness of each lift can play an important role. It is recommended that each lift be placed between two to four hours before moving on to another location and not to vibrate during or after placement, or remix the product after it has set. The maximum height for each lift of cellular concrete should be 2 ft (0.6 m). This ensures that excessive heat of hydration does not develop, which can negatively affect the air void content of LCC. Additionally, this height allows workers to place the lifts by laying the hose on the ground to minimize the voids next to formwork. Subsequent lifts can be placed once the previous lift can be walked on without excessive surface penetration (up to 1.0 in. [25 mm] is acceptable) or after a minimum 12-hr up to three to seven days as a waiting period regardless of temperature. However, the time for curing can depend on the ambient temperature at the site, with colder weather conditions (i.e., less than 60°F) requiring up to 20 hours of curing time. It is recommended that LCC only be placed in the allowable range of 32°F (0°C) to 100°F (38°C). Otherwise, special provisions detailed in ACI 523 2006 are necessary to ensure shrinkage is not an issue or excessive curing time.

Water absorption and freeze-thaw cycles are two weather condition which indirectly relevant to temperature. These two items are considered as major construction concerns for LCC. As with other lightweight fill materials, placement below the water table should be avoided due to the buoyancy effect. Water absorption can change the properties of the LCC. This can be prevented by placing a sheet of polyethylene at the bottom of the lift area, constructing the concrete curtain wall on the side of placement, and sealing the top lift with an asphalt emulsion. Also, placing underdrains at the base of curtain walls, wingwalls, abutments, and at the pavement edge can increase the drainage and subsequently reduce the potential for LCC water absorption. For freeze-thaw concerns, a subbase or a lift of LCC with a denser unit weight can be placed over the final lift as an insulation layer to prevent movement.

When LCC is used to support utilities, the utility can be supported on a temporary blocking or bracing during placement, then the fill can be placed easily to surround and submerge the utility. For post-construction installation, the fill can be excavated with the aim of a backhoe, jackhammer, pipe jacking, boring operations, or even hand tools. Moreover, LCC can be used for grading and profiling or providing a side and slope. For grading and profiling, two methods are common; (1) placing the fill in stepped shape with the thickness of 15 cm or 30 cm lifts and then trimmed and overlain with an asphalt truing-and-leveling course, (2) overpouring the top lift and then removing the excessive parts with hand. Moreover, to provide a side or slope, the fill can be operated in stepped shapes and then covered by conventional fill, topsoil, or slope protection.

Additional effort is recommended in terms of screeding and using more labors if a smooth surface is required. In general, to ensure good performance of the final product, no backfill, surcharge, traffic loading, or any other load types should be permitted until the LCC achieves a compressive strength of at least 20 psi. Also, a grade of up to 3% is possible for the final lift and previous lifts do not need to be graded this way for successful placement. Upon completion, the final finished surface of LCC shall be within ± 0.1 ft of the plan elevation. The best way to protect LCC upon completion is to place the surface finish as soon as possible. For this purpose, the surface of LCC can be protected by using durable polyethylene plastic sheet to hold in the preliminary moisture for curing and reducing the potential of shrinkage.

4.1.2.2.4. Summary of Considerations

The preceding discussion highlights several of the key considerations related to use of LCC as a design alternative in highway construction. As with EPS Geofoam blocks and other lightweight fill technologies, LCC is a viable option for various highway-related geotechnical projects assuming that the preceding considerations are kept in mind to ensure adequate fill performance. Figure 4.32 presents a useful flowchart that highlights the considerations specific to LCC when implemented at a site.

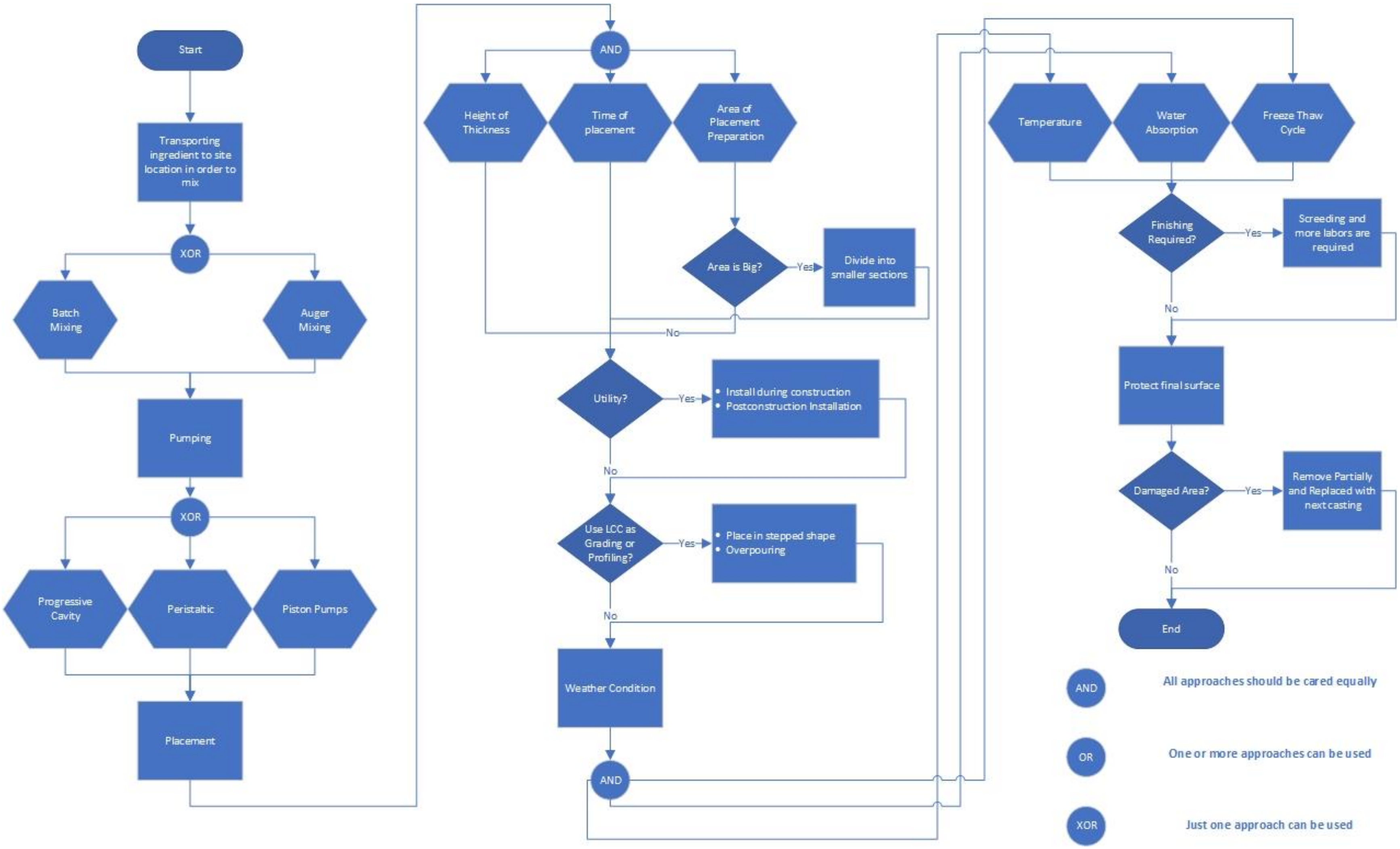


Figure 4.32. Flowchart guidelines for LCC operation.

4.1.2.2.5. Cost Information

Several studies have reported unit cost for LCC from \$50 to \$70/m^3 (\$67 to \$94/yd^3) (Harbuck 1993), \$55.00 to \$85.00 per m^3 according to the Ground Improvement Technical Summary I (Saboundjian 2008), and \$65.00 to \$95.00 per m^3 according to A Compendium of Ground Modification Techniques (Elias et al. 2001). However, the study by Schaefer et al. (2017a) went a step further and reported the cost of LCC separated based on delivered costs and in-place costs (Table 4.6). These amounts are different based on basic costs of the material at source, transportation cost, quantity of material, availability of material in addition to placement and/or compaction costs. Each of these factors can increase or decrease the final cost of the product. In the case of LCC products, the increased costs associated with mixing and producing this material on-site can compensate for the savings associated with minimal transportation costs relative to other lightweight materials. The other factors that may affect LCC price are the product's density and the quantity of the material. Moreover, placement of LCC requires sufficient curing time to reach to a specified compressive strength, which may add additional time to the project construction timeline and incur additional project costs.

Table 4.6 Typical range in costs for LCC at Source, Delivered, and In-Place (Schaefer et al. 2017a).

Lightweight Fill	Material Cost/yd^3 FOB at source	Delivered Material Cost/yd^3 FOB at Project	In-Place Cost/yd^3
Cellular concrete	n/d	\$70 to \$150	\$250 to \$340

4.1.2.3. Expanded Shale, Clay, and Slate (ESCS)

ESCS is a manufactured lightweight aggregate formed by expanding shale, clay, and slate with heat in a rotary kiln. Unlike the other lightweight fill technologies discussed in previous sections (EPS Geofoam, LCC), ESCS is similar in appearance to other aggregates used in geotechnical construction (e.g., granular fills), albeit the density is much lower. This means that in practice, ESCS is handled similarly to other aggregates during placement. The following sections discuss some of the specific issues regarding ESCS when considered as a design alternative for highway-related projects.

4.1.2.3.1. Typical Applications and Geometry

ESCS can be used in a number of highway-related infrastructure projects since it functions similar to typical aggregate fills. This includes use under roadways for frost insulation (Gustavsson et al. 2002), for load compensation and grading for fills (Holm and Ries 2007, Chapter 16), as a stabilized subgrade for pavement system (Holm and Ries 2007, Chapter 14), in embankments to prevent settlement (Saride et al. 2010a), and for bedding buried utilities (Holm and Ries 2007 Chapter 16).

For the case of roadways, a typical step by step process for construction is as follows: (1) Use a layer of geotextile or fabric layer to separate cohesive subgrade layer and ESCS from each other. (2) Place lifts of equal thickness. (3) Level and compact each lift thickness with designed compaction pressure and required number of passes. (4) Repeat steps 2 and 3 to reach the finished surface. (5) Operate designed bound surface layer over finished compacted ESCS.

Figure 4.33 shows a typical cross section of an embankment using ESCS. For the case of embankment applications: (1) The existing grade for fill area should be free of any objects. (2) A geotextile/fabric layer is installed at the prepared surface before placement of lifts. (3) Lifts of equal thickness are placed. (4) Level and compact each lift thickness with designed compaction pressure with the required number of passes. (5) If a deep foundation system is required, install vertical drains within the lift thickness (Figure 4.34). This process ensures that ESCS would not be close to locations intended for the deep foundation elements. (6) Repeat steps 3 and 4 with a typical slope to reach the finished surface. (7) All of the slopes will receive a layer of local clay as a soil cover for the purpose of erosion control. (8) Once the embankment has reached its designed stage and finished height, a flowable fill layer is placed with 1m thickness under the approach slab. (9) Placement of approach slab.

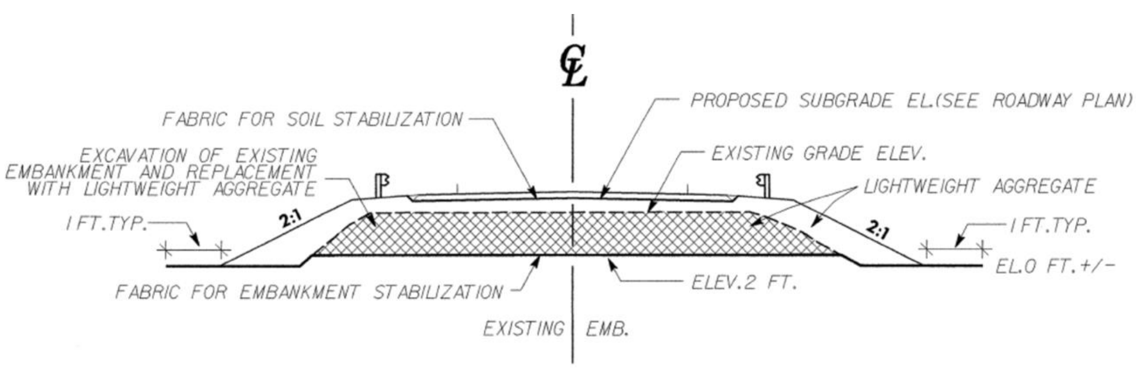


Figure 4.33. Typical cross section of an embankment using ESCS.

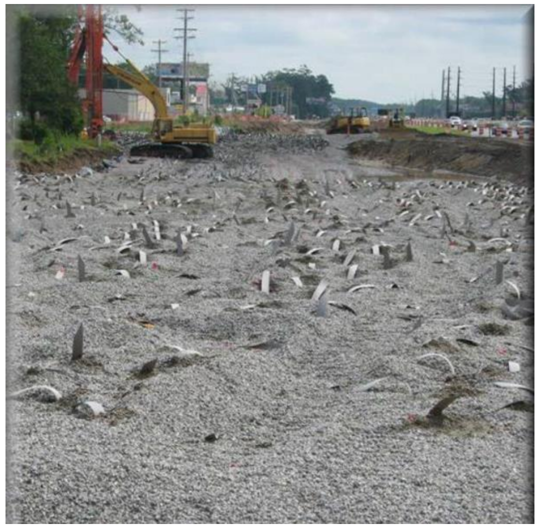


Figure 4.34. Installation of vertical drain for embankment cases.

For the case of buried utilities, ESCS can be used as a bedding layer or on top of any pipes to reduce overburden weight. The insulating properties of ESCS can allow engineers to reduce the trench depth from 11 ft. for normal weight aggregate to 7 ft. for cases using ESCS. This feature can provide safer working conditions for laborers, an easier excavation for probable pipe repair during winter, and reduced water supply disruption and street traffic by decreasing the construction time. For this purpose, three excavated trench types are possible depending on how embedded the utility pipe is into the ESCS (Figure 4.35). In type A, the pipe will be submerged into the lightweight aggregate and lay to the bottom of the trench. In contrast, in types B and C, lightweight aggregate is more used as a bedding layer with a different distance from the bottom of pipes to the bottom of the excavated trench. Through all the types, it is not necessary to remove the whole unstable soil. In such a manner, only sufficient material can be extracted to see the combination of weight of the pipe, water inside the pipe, and the foundation of material is less than that of the soil removed.

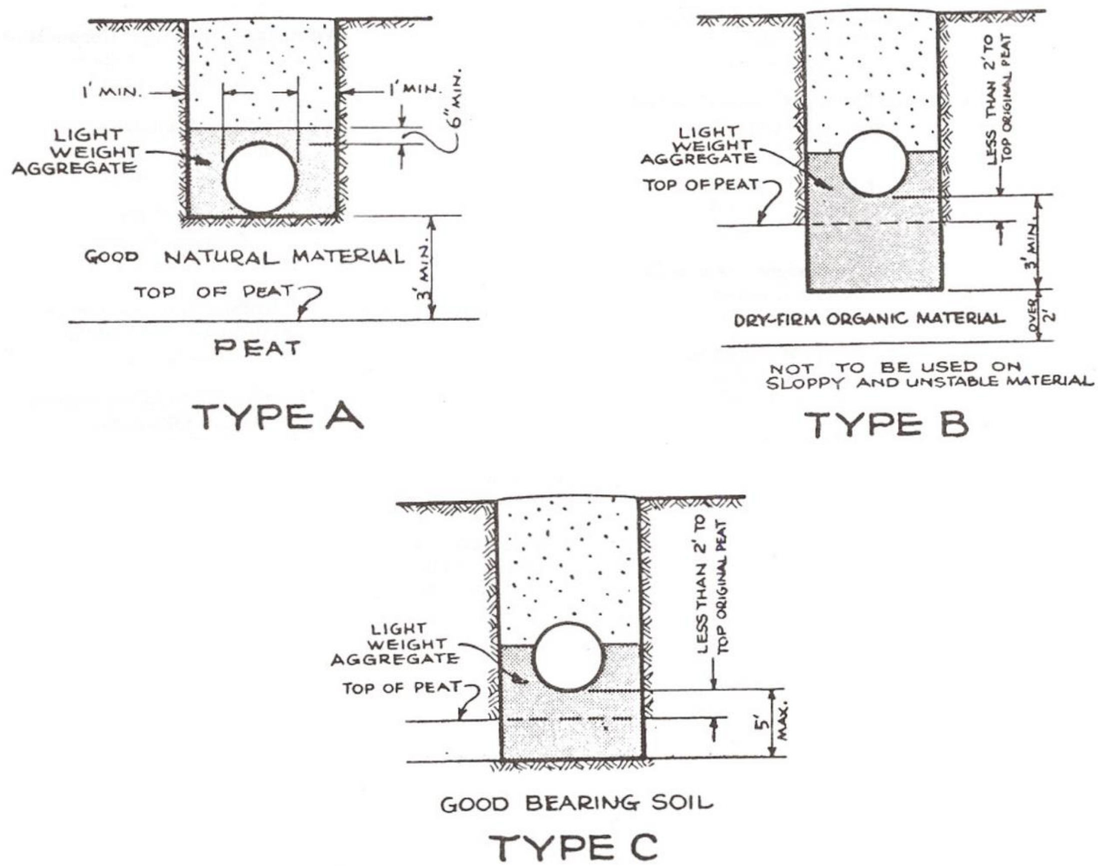


Figure 4.35. Different types of excavated trenches using ESCS.

4.1.2.3.2. Considerations Prior to Placement

Prior to placement on site, there are a number of considerations related to the shipping, unloading, and distribution of ESCS. After producing this material and screening it to produce the desired gradation, the materials must be transported to the site location. ESCS typically weighs about one half of normal aggregates, so a larger volume of material can be loaded per transport vehicle (Figure 4.36). Moreover, sideboards are often added to truck beds or bulk trailers to increase the hauling capacity and reduce the number of required trips. When the material is received at the site location, it is unloaded to the fill area with the aid of a tandem truck dumper or telescoping belts. After unloading the material, if a great volume of material needs to be moved, a Michigan loader, Caterpillar front end loader with a larger bucket, or any other lightweight equipment with a contact pressure of 4.5 psi or less can be used. Otherwise, laborers can easily handle the material manually with large grain scoops due to their low unit weight. In situations where the ESCS material will not be placed immediately, stockpiling may take place on site. However, ESCS is strongly capable of water absorption and can hold on to that moisture for several days. Consequently, the design should account for higher moisture contents if stockpiling is to take place.

Figure 4.36. Volume of normal weight aggregate compared with ESCS.

4.1.2.3.3. Considerations During Placement

As with any other aggregate material, various aspects must be considered during placement, including lift thickness, method, energy for compaction, and additional materials required for installation (e.g., geotextile layer as a separation layer between subgrade and ESCS material, or soil cover required for the slope area of embankments). Since the cost of ESCS production is quite high and ESCS is considered a high quality aggregate material, ESCS can be used as an aggregate in lightweight concrete, chipseal, mechanical, and chemical stabilized base course layer or it can be applied alone in roadway applications like embankments and pavements as a geotechnical fill material. The general process of ESCS operation is the same among different applications; however, in some parameters such as lift thickness, compaction devices, and their number of passes are different for each application.

The first step during construction is to ensure the condition of subgrade to carry the lightweight ESCS aggregate. If the subgrade does not have enough bearing capacity, it is recommended to be reinforced with a chemical additive like lime or cement. After preparing the bedding level of the fill area, a geotextile or fabric layer can be used to separate ESCS from the subgrade layer if it is cohesive material.

After the subgrade is prepared, a proper compaction procedure is required. This includes selection of appropriate compaction equipment and lift thickness, which depends on whether the ESCS will be mixed with other on-site soils or be placed in isolation. For example, in cases like mechanically stabilized base course layer where ESCS is used as a combined material, the only allowable compactor is pneumatic rollers because they can exert a contact pressure approximately equal to tire-inflation pressure which is a desired in-place density. In contrast, other compactors like steel rollers, sheepsfoot and grid rollers, in addition to slush rolling action, are not permitted as they may apply excessive force to the ESCS material. For example, steel rollers exert surface stresses of about 500 psi, which is higher than the crushing strength of most ESCS particles. This can leave a plane of weakness between successive layers and consequently increase the likelihood of degradation.

Several studies have reported different recommendations for the compaction method and the number of passes to handle these specifications. General recommendations specify the thickness of each layer and the number of passes for the compactor based on the compaction vehicles:

- In a high accessible area where a vibratory roller is used to compact the materials, the thickness of the layers should not exceed 12 inches. In this way, the vibratory rollers should have a static weight of no more than 12 tons, and a minimum of two passes are required.
- In low accessible areas where portable vibratory plate compactors (Figure 4.37) are allowed to be used, it is recommended that the maximum lift thickness of material be 6 inches with a minimum of two passes of compactors. In this case, a pass of at least 10 seconds is required before moving to the following location.

Schaefer et al. (2017a) reported that lift thickness of 3 ft or less is required and compaction should be performed by rubber-tired rollers using two to four passes for each layer. Moreover, the desired field density can be obtained in the lab by conducting a modified one-point AASHTO T272 density test. Saride et al. (2010a) constructed an embankment with equal lift thickness and acquired their desired density by using a combination of hauling dozers (Figure 4.37) and heavy 33 yd^3 capacity haul trucks with wide tires. Holm and Ries (2007) summarized a number of projects through the US. In each project, the thickness of lifts and the compaction method were different according to the condition of each project. For the case of runway repair, the recommended lift thickness and compaction methods were the 6-inch lift and vibratory plate, respectively. For bridge and embankment cases with the vaster area, it is reasonable to place 2 ft (0.61 m) thick lifts by using four passes of light 5 ton (4.5 Mg) vibratory roller (Dugan 1993). Additionally, considerations for using ESCS in roadway structures are as the following: The maximum of fill layers should be 0.6m (2 ft.), the maximum recommended contact pressure with compactor during leveling and compaction is 50 kN/m^2 (7.3 psi), a minimum of six passes after leveling is required, and the preferred temperature during the operation is above 0^oC (32^oF) (Saboundjian and Armstrong 2003).

Figure 4.37. Vibratory plate compactor (Left) and hauling dozers (Right) for ESCS placement.

Figure 4.38. Quality control of compaction efforts for ESCS based on steel boxes (Left) removed after compaction (Right).

In terms of quality control of compaction, use of typical methods like sand cones or nuclear gages for normal weight aggregates are not applicable for ESCS when used in isolation because laboratory compaction tests like ASTM D 698 or ASTM D 1557 can increase the amounts of fine particles which results in higher density than obtained in the field with the same compaction energy using rubber-tire equipment. Furthermore, there are systemic issues with the sand cone method (ASTM D 1556) and nuclear gage (ASTM D 2922) themselves that render them unsuitable for ESCS. The sand cone method has difficulties due to the instability of the test hole in such a granular material like ESCS. The nuclear gage is typically only calibrated against standardized blocks ranging from about 100 to 150 lbs/ft^3, rendering it ineffective for the low densities of ESCS. One approach highlighted in the literature is the use of steel boxes of 1 ft^3 and 3 ft^3 that are placed in the area to be compacted and dug out after compaction for weighing (Figure 4.38).

Afterward compaction, the designed bonding surface or concrete slab is typically placed as a finished surface. However, in sloped structures like embankments, the installation of vertical drains is typically necessary, as is the construction of slopes on the sides with a range of 1.5H:1V to 3H:1V, and adding a local clay with thickness of 0.75 m-1 m as a soil cover over the sloped area.

4.1.2.3.4. Summary of Considerations

The preceding discussion highlights several of the key considerations when using ESCS as a lightweight aggregate in highway construction. ESCS functions similar to other aggregate materials, except with much lower density. Consequently, the key considerations are narrower in scope relative to previously discussed lightweight fill technologies. Figure 4.39 presents a useful flowchart that highlights the considerations specific to ESCS aggregates.

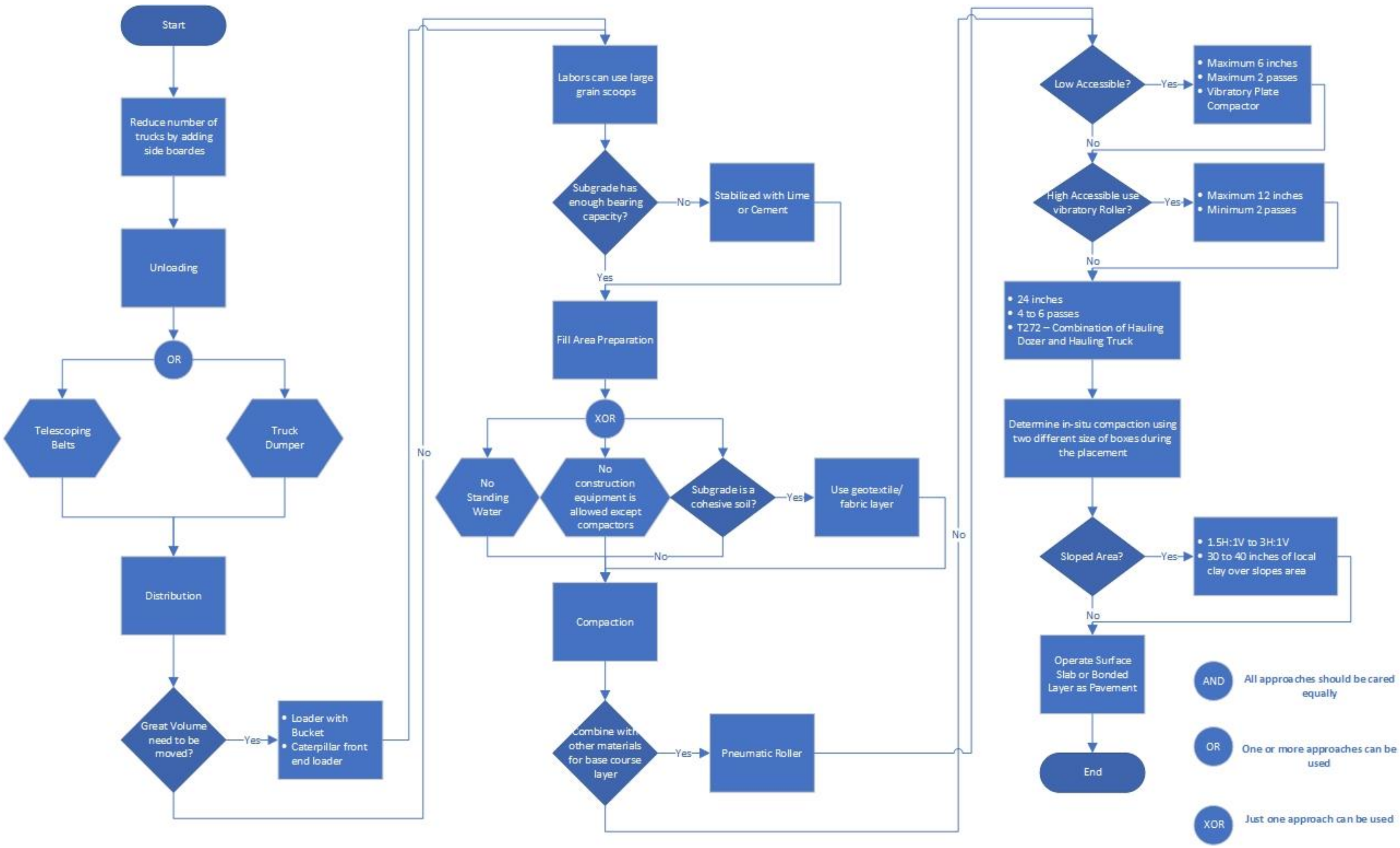


Figure 4.39. Flowchart guidelines for ESCS operation.

4.1.2.3.5. Cost Information

Although the costs of ESCS is at least twice that of common fill material, use of ESCS can indirectly yield overall cost savings in terms of transportation and maintenance costs. Schaefer et al. (2017a) presented typical cost ranges for ESCS (Table 4.7). A study by Saboundjian and Armstrong (2003) highlighted the costs of each component of an ESCS embankment project: $5-10/yd^3 for common fill, $20-25/ft with a $6,000 mobilization for piles, and $23.75/yd^3 for LWA. However, the cost of compaction was not included in that study, though it was reported that the cost of compaction for ESCS would be expected to be less than common fill materials. A few years later, a formula was developed with respect to price per ton of ESCS material, production plant, and trucking cost to the project as the following: $\$/yd^3 = [(X + Y) \times 55 \times 27] / 2{,}000$ (Holm and Ries 2007, Chapter 16) and $[\$(X+Y) \times 60 \times 27]/2{,}000$ where X is the price per ton; Y is FOB (freight on board) the production plant and trucking costs to the project location per ton, and 55 and 60 are the representatives of assumed unit weight of ESCS (Valsangkar, 1993). However, it should be noted that this was based on raw material costs at the time of the study and that the price of compaction was not considered in these formulas, nor should they be considered generalized for all site conditions. Nevertheless, they can serve as examples for how to estimate ESCS costs.

Table 4.7. Typical range in costs for ESCS (Schaefer et al. 2017a).

Lightweight Fill	Material Cost/yd^3 FOB at source	Delivered Material Cost/yd^3 FOB at Project	In-Place Cost/yd^3
Expanded shale, clay, and slate (ESCS)	$30 to $45	n/d	n/d

4.1.2.4. Foamed Glass Aggregate (FGA)

Foamed Glass Aggregate is another lightweight fill technology that is derived from post-consumer recycled glass. The glass is mechanically ground into a fine powder and mixed with an foaming agent before being kiln fired. The high temperatures cause a chemical reaction that expands the glass/foam mixture and creates closed-cell micropores. The foamed glass hardens as it cools and is broken down into aggregate pieces. The resulting product is approximately 15% of the bulk unit weight of typical aggregates. Like ESCS, FGA is delivered as an aggregate to the site and can be handled similar to other quarried aggregates. The following sections discuss some of the specific issues regarding FGA when considered as a design alternative for highway-related projects.

4.1.2.4.1. Typical Applications and Geometry

FGA was originally developed with considerations for its thermal insulating properties. Consequently, FGA has been used in a number of applications beyond highway-related infrastructure (e.g., foundation walls and slabs, greenroofs, plaza decks, etc.). In the case of highway geotechnics, FGA has been applied as an aggregate material for embankments, retaining walls and bridge abutments, slope repairs, and culverts (Figure 4.40). Previous case histories have highlighted some specific considerations relevant to each of these applications (Dettenborn et al. 2016; Auvinen et al. 2013; Frydenlund and Aabøe, 2002; Loux et al. 2018; Loux et al. 2019b). For example, the placement of FGA for embankments follows what can be considered a "typical" approach: (1) A separation cloth or geosynthetics is placed on the subsoil as well as along the sides and top. (2) The FGA is distributed using a crawler mounted dozer. (3) The FGA is compacted in lift thickness between 1-3 m. (4) Finally, a sub base and base layer are placed on top of the final lift of FGA (assuming the embankment will support a roadway).

For slope repair, a geotextile layer is added at the bottom of the excavated area. Then the excavator with a maximum of 50 kN/m^2 pressure drives the foam glass to the area. Each layer of foam glass is compressed by 60-70 kg vibroplate. Subsequent layers are placed with the same method until reaching the final 1 m lift. For each lift, mesh reinforcement is wrapped around the foam glass with an overlap of 1 m. Then a new layer of mesh reinforcement is added from the inner edge of the excavated area. The slope is compacted lightly with excavator bucket. A geotextile is added at the top of the final lift and then covered with a 30 cm thick layer of crushed rock or at least 0.5 m of ordinary soil. An erosion net is added 1 meter within the slope edge. Finally, the layer of crushed rock/ordinary soil is compressed by a 500 kg vibroplate and then 20 cm of asphalt layer is added as the final layer.

In case of a roadway fill, it has been reported that the maximum allowable transmitted stress for long term heavy traffic loading over permanent and temporary pavement located on FGA fill layer are 1000 and 1500 psf. So it is necessary to design pavement thickness based on reported maximum allowable stress. Otherwise, the placement of FGA follows a similar approach to what was previously described for embankments.

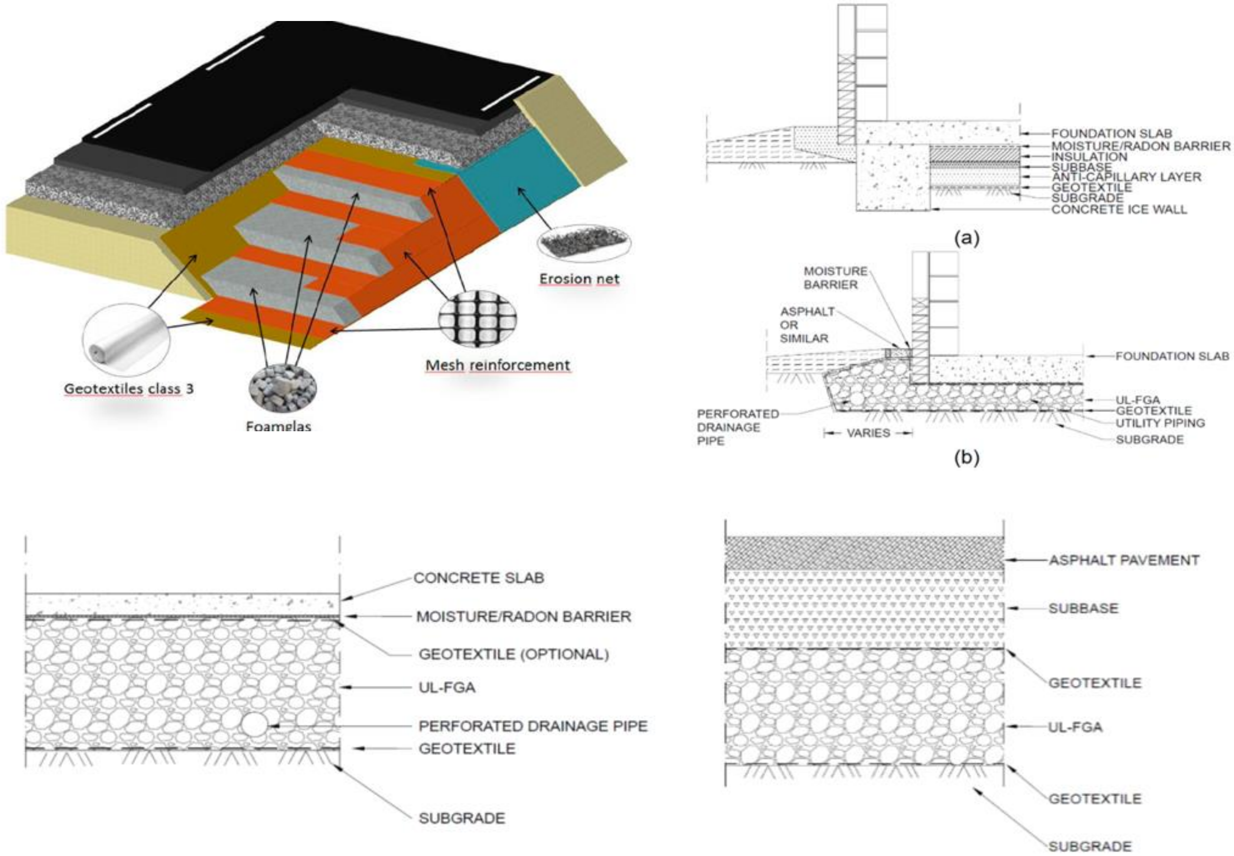


Figure 4.40. Example applications of FGA in highway construction: (Top-Left) slope repair; (Top-Right) retaining wall; (Bottom-Left) foundation slab; (Bottom-Right) pavements.

4.1.2.4.2. Considerations Prior to Placement

Unlike EPS and/or LCC, FGA does not really necessitate additional considerations prior to placement. For example, it is chemically inert and does not absorb water, which would otherwise necessitate additional care during stockpiling at the site. However, as with any lightweight fill technology, there may be some additional logistical considerations with respect to transport. FGA can be shipped in bulk in truckloads up to 100 cubic yards due to its low density. In this way the number of required truck deliveries can be reduced to save transportation costs and offset the higher material costs. The other approach is bagging FGA in bulk bags of 1.25 yd^3 to 3 yd^3 and ship them to the project. After shipping, the material can be dumped on site and placed using manual labors or other forms of construction equipment (e.g., backhoe) (Figure 4.41).

Figure 4.41. Delivery and unloading of FGA.

4.1.2.4.3. Considerations During Placement

Figure 4.42 shows an example of placement of FGA, which does not appear much different than typical non-lightweight aggregate fills. During placement, the area to be filled by FGA must be free of standing water. Additionally, construction equipment should not pass over the FGA except equipment relevant to placement and compaction due to the tendency for the aggregates to crush into smaller pieces when significant load is applied. A nonwoven geotextile should be placed directly on top of the prepared subgrade to separate the subgrade layer from the initial lift of FGA. Moreover, the FGA layer should be separated by geotextile from any adjacent material placed above the final lift and side slopes. In this way, geotextiles can minimize the intrusion of fine particles of subgrade or any other materials from the side and above the final lift into the FGA. The geotextile should have a 6 oz./yd^2 (minimum) needle punched nonwoven with a tensile strength of 160 lbs according to ASTM D4632. The geotextile seam should overlap 12 inches or greater and not be exposed for longer than 14 days.

In terms of general provisions for FGA placement, typical thickness of the FGA fill layer varies from 0.3 m -2 m according to the project condition. The minimum thickness of 0.3 m is required for places that are not readily accessible for big scale compactors like tracked excavators or dozers, while for structures like embankment, it is recommended to use minimum of 1 m up to 2 m.

In special conditions like filling material over utilities, it is recommended to operate an initial natural aggregate around the utilities before FGA placement unless the exterior material of pipe is PVC, PE, coated cast iron, coated steel (Polyurethane coating) and concrete. A vibrating plate can be used as a QC/QA to ensure compaction. As part of the FGA lift process, placing a stable slope may be necessary in certain applications (e.g., embankment). For this purpose, it is recommended that the FGA lifts are placed with a slope of 1H:1V without additional reinforcement. However, a cover material over geotextile must be stable at this angle. Otherwise, a slope stabilization system may be used on a 1H:1V slope, or a cover soil may be graded at a 2H:1V Slope. At the end of the construction process, a capping layer is usually used. This layer aims to avoid direct contact with traffic load and breakage of material; however, a high level of accuracy is required to not damage any geotextile layers placed during the operation.

Figure 4.42. Placement and compaction of FGA.

4.1.2.4.4. Summary of Considerations

As noted previously, FGA functions similar to other aggregate materials, except with much lower density. Consequently, like ESCS, the number of key considerations both prior to and during placement are smaller. Figure 4.43 shows a summary of these considerations related to FGA operations at a site.

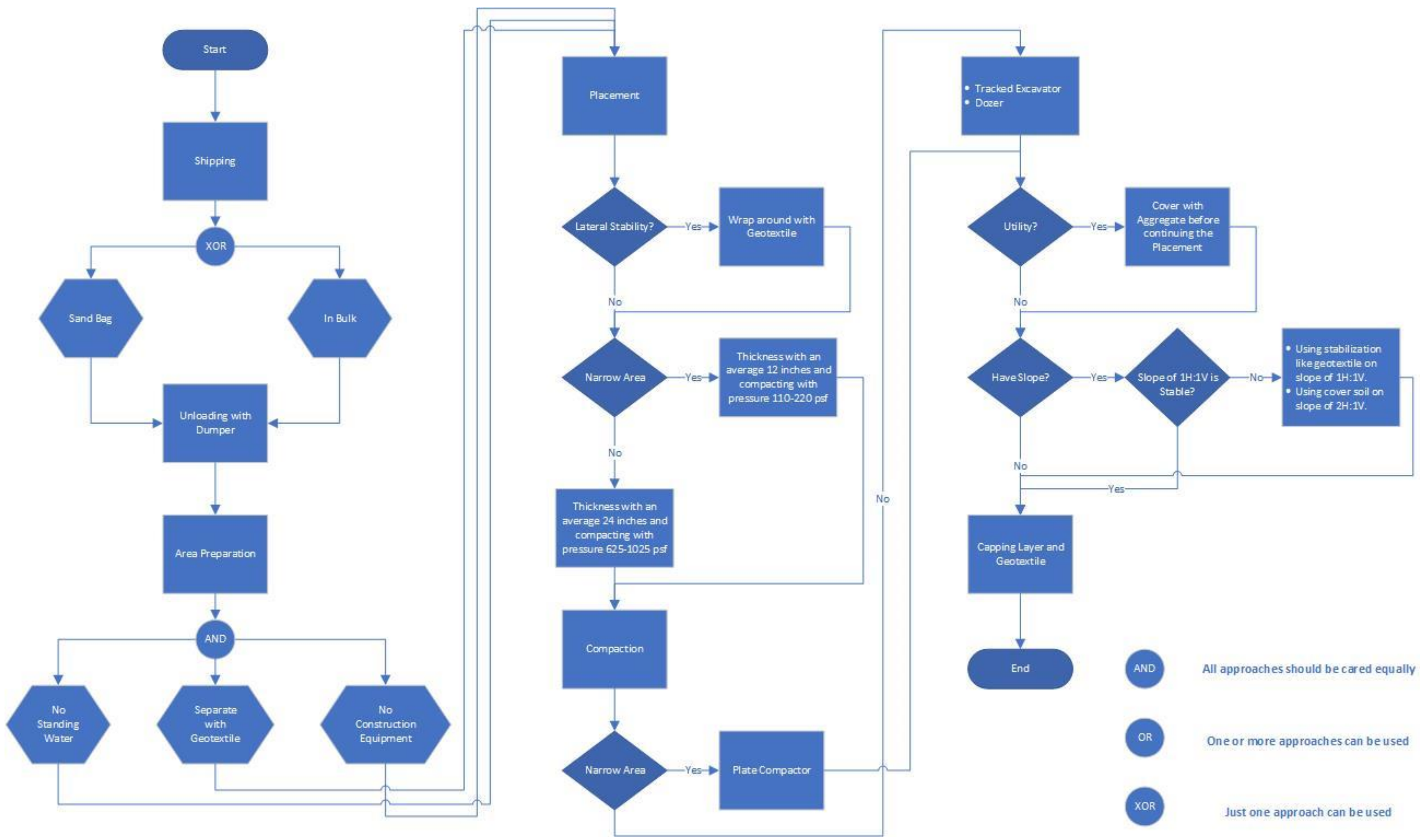


Figure 4.43. Flowchart guidelines for FGA operation.

4.1.2.4.5. Cost Information

Since FGA is a relatively new material (full scale production did not start until the 1980s in Europe), detailed cost information is not readily available for this material compared to other lightweight technologies. However, a few researchers in different countries have reported unit costs of FGA of \$35-40 per m^3 in Norway (Frydenlund and Aabøe, 2003), and \$50 and \$76.47 per m^3 corresponding to raw material and total cost including installation in Canada (Schneider, 2017). Schneider (2017) performed a comprehensive Life-Cycle Cost Analysis (LCA) comparison between foamed glass aggregate and EPS Geofoams. Table 4.8 presents this comparison based on design equivalent single-axis loads (ESAL) and demonstrates that the overall project costs are lower for FGA.

Table 4.8. LCA comparison between FGA and EPS geofoam (Schneider 2017).

(All values below are in \$ CAD.)

Design Artificial Subgrade	LWA	EPS	LWA	EPS	LWA	EPS
Design Lifetime ESALs	1×10^6	1×10^6	10×10^6	10×10^6	60×10^6	60×10^6
Depth Hot Mix Asphalt (mm)	127.0	317.5	190.5	482.6	304.8	711.2
Depth Granular Base (mm)	152.4	152.4	152.4	152.4	152.4	152.4
Depth Granular Subbase (mm)	152.4	152.4	228.6	215.9	393.7	368.3
Depth Lightweight Fill (mm)	5568.2	5377.7	5428.5	5149.1	5149.1	4768.1
Road Length (m)	1000	1000	1000	1000	1000	1000
Road Width (m)	15	15	15	15	15	15
Volume Hot Mix Asphalt (m^3)	1905.0	4762.5	2857.5	7239.0	4572.0	10668.0
Volume Granular Base (m^3)	2286.0	2286.0	2286.0	2286.0	2286.0	2286.0
Volume Granular Subbase (m^3)	2286.0	2286.0	3429.0	3238.5	5905.5	5524.5
Volume Lightweight Fill (m^3)	83523.0	80665.5	81427.5	77236.5	77236.5	71521.5
Cost Hot Mix Asphalt	\$492,443	\$1,231,106	\$738,664	\$1,871,282	\$1,181,862	\$2,757,678
Cost Granular Base	\$96,698	\$96,698	\$96,698	\$96,698	\$96,698	\$96,698
Cost Granular Subbase	\$80,582	\$80,582	\$120,872	\$114,157	\$208,169	\$194,739
Cost Lightweight Fill	\$6,387,053	\$8,927,109	\$6,226,809	\$8,547,627	\$5,906,321	\$7,915,158
TOTAL COST - CONSTRUCTION	**\$7,056,775**	**\$10,335,494**	**\$7,183,043**	**\$10,629,764**	**\$7,393,049**	**\$10,964,273**

4.1.3. Consideration of Lightweight Fill Technologies During Design

As mentioned previously, engineering design of various highway-related infrastructure (e.g., embankments, bridge abutments, retaining walls, etc.) often starts with a consideration of a number of "typical" design alternatives assuming site conditions are favorable. For example, the starting point for design of a retaining wall may be a precast concrete gravity wall with a shallow foundation and an approved aggregate backfill. In many cases, problematic subgrade soils may preclude the use of these design options and alternatives are sought. Lightweight fill materials have been often neglected in comparison to other options such as use of deep foundations and various ground improvement techniques as highlighted in section 2.1. This is despite the fact that lightweight fill materials can provide stability, decrease lateral earth pressure, and control excessive settlements while being more cost effective under certain project conditions. The purpose of this section is therefore to highlight how the lightweight fill technologies in this chapter can be considered during the engineering design process for highway-related infrastructure. It is assumed in the proceeding discussion that specific site conditions have rendered simpler (and often more inexpensive) design alternatives as inadequate and lightweight fill technologies are being compared to deep foundations and ground improvement techniques. The discussion therefore centers on how to prioritize the appropriate selection of the lightweight fill technology relative to themselves and to other design alternatives. This will be accomplished by examining the merits of each lightweight fill technology relative to unit weight reductions, costs, availability, and construction considerations.

4.1.3.1. Unit Weight

The first major consideration when prioritizing lightweight fill materials is the amount of weight reduction possible when using a particular technology. Table 4.9 summarizes the typical range of in-place unit weights for the lightweight materials specifically discussed in this chapter. As noted, EPS Geofoam exhibits the smallest overall unit weight and would decrease the applied loads the most. The remaining technologies compare favorably with each other though ESCS has a narrower range given the constituent materials and manufacturing process from which it is derived. It should be noted that the wide range in unit weights for LCC and FGA are due to differences in manufacturing, mix design, and/or compaction efforts that also coincide with other materials properties. For example, LCC can be differentiated into different classes depending on the ratio of cement, water, foaming agent, and admixtures (Table 4.10). The different mixes result in different ranges of unit weight as well as compressive strength, which allows some tailoring of the LCC to the specific needs of the project. Table 4.9 highlights that if the overwhelming factor that governs selection of a design alternative is a reduction in applied loads, EPS Geofoam may be the best suited given the very low unit weight.

Table 4.9. Range of unit weights for lightweight fill technologies.

Material Type	Range in Unit Weight (pcf)
EPS Geofoam	0.7 – 3.0
LCC	25 – 90
ESCS	40 – 65
FGA	30 - 90

Table 4.10. Caltrans classification of LCC (Rollins et al. 2019).

Cellular Concrete Class	Cast Density kN/m^3 (lb/ft^3)	Minimum 28-day Compressive Strength kPa (psi)
I	3.8-4.6 (24-29)	69 (10)
II	4.7-5.5 (30-35)	276 (40)
III	5.7-6.4 (36-41)	550 (80)
IV	6.6-7.7 (42-49)	830(120)
V	7.9-12.4 (50-79)	1100 (160)
VI	(12.6-14.1) 80-90	2070 (300)

4.1.3.2. Costs

As noted previously, it is very difficult to compare costs for different lightweight fill technologies because of differences in transport costs, equipment needs for placement, quality control, and similar considerations discussed in previous sections. Nevertheless, it can be beneficial to examine the differences between EPS Geofoam, LCC, ESCS, and FGA based on raw material costs. Table 4.11 gathers these unit costs based on availability in the literature. These results demonstrate that FGA and ESCS compare favorably in terms of raw material costs, while LCC and EPS Geofoam tend to be more expensive depending on where or from whom the material is sourced. However, this can be offset by the fact that less EPS Geofoam may be needed to cover the volume of fill needed for the project. Moreover, less specialty equipment may be necessary relative to LCC. It should also be noted that the costs for LCC already incorporate transport costs because LCC is mixed on-site and only the components are shipped to the project location. When compared to other design alternatives described in previous sections, lightweight fill technologies can compare favorably with respect to costs (Table 4.12). In fact, some case histories have documented significant cost savings as detailed in previous sections of this report.

Table 4.11. Range of cost for lightweight fill technologies.

Material Type	Material Costs FOB at Source (per yd^3)
EPS Geofoam	\$40 – \$80
LCC	\$35 – \$70 (*FOB at Project*)
ESCS	\$30 – \$45
FGA	\$25 – \$40

Table 4.12. Comparative unit costs by ground modification technology as of November 2016 (Schaefer et al. 2017a).

Category	Technology	Unit Cost
Vertical Drains and Accelerated Consolidation	PVDs, with and without fill preloading	\$0.50–\$4/lft
Lightweight Fills	Compressive Strength Fills: Geofoam; Foamed Concrete	\$75–\$150/yd^3
Lightweight Fills	Granular Fills: Wood Fiber; Blast Furnace Slag; Fly Ash; Boiler Slag; Expanded Shale, Clay and Slate; Tire Shreds	\$3–\$15/yd^3
Deep Compaction	Deep Dynamic Compaction	\$10–\$30/yd^2
Deep Compaction	Vibro-Compaction	\$5–\$9/lft
Aggregate Columns	Stone Columns and Rammed Aggregate Piers	\$15–\$60/lft
Column Supported Embankments	Column Supported Embankments	\$9/ft^2 + cost of the column
Column Supported Embankments	Columns: Non-compressible	\$30–\$80/lft
Column Supported Embankments	Columns: Compressible	\$20–\$100/lft
Soil Mixing	Deep Mixing (dry)	\$60–\$125/lft
Soil Mixing	Mass Mixing	\$15–\$75/yd^3
Grouting Technologies	Chemical Grouting	\$20/ft + \$0.65/qt
Grouting Technologies	Compaction Grouting	\$75–\$750/yd^3
Grouting Technologies	Bulk Void Filling	\$50–\$150/yd^3
Grouting Technologies	Slabjacking	\$6.50–\$9.30/ft^2
Grouting Technologies	Jet Grouting	\$250–\$750/yd^3
Grouting Technologies	Rock Fissure Grouting	\$25–\$80/ft^2
Pavement Support Stabilization Technologies	Mechanical Stabilization	\$1–\$5/yd^2
Pavement Support Stabilization Technologies	Chemical Stabilization	\$2–\$5/yd^2
Pavement Support Stabilization Technologies	Moisture Control	\$3–\$12/lft
Reinforced Soil	Reinforced Embankments	\$2–\$12/yd^2
Reinforced Soil	MSE Walls	\$30–\$65/ft^2
Reinforced Soil	Reinforced Soil Slopes	\$3–\$25/ft^2
Reinforced Soil	Soil Nailing	\$20–\$50/lft

4.1.3.3. Availability

Another major aspect that affects selection of an appropriate lightweight fill technology is its availability and the degree to which general contractors have experience working with a particular technology. Some lightweight fill technologies are only manufactured by a limited number of manufacturers who are not always geographically distributed throughout the United States. Consequently, transport costs may increase dramatically if sourcing a lightweight fill technology from a manufacturer or retailer that is far away from the project location. Moreover, availability plays a role in whether local contractors are familiar with placing the lightweight material across a range of site conditions and project types. It may be necessary to subcontract a specialty contractor who has experience with a particular lightweight fill technology in order to ensure long term performance and minimize construction issues.

A review of the different lightweight fill technologies in this chapter highlights that EPS Geofoam has one of the largest supplier network across the United States, including several locations in the Northeast and in Pennsylvania specifically (e.g., see list of manufacturers asscociated with the EPS Industry Alliance at http://epsindustry.org/other-applications/geofoam). This is not surprising given the extensive history of EPS Geofoam in highway construction as well as other commercial applications across a wide range of industries. ESCS also has an extensive network of producers/suppliers (https://www.escsi.org/memberlist/), again with several located in the mid-Atlantic and Northeast that can supply projects in Pennsylvania. It should be noted that it can still be quite difficult to obtain ESCS for PennDOT project and it regionally has to come from New York or North Carolina, which increases transport costs. ESCS is often sold under various trade names such as Solite, Stalite, Norlite, Utelite, and/or Haydite. The fact that LCC is created on the project site and that many proprietary and/or trademarked products fall under the umbrella category of cellular concrete complicates the identification of the extent of LCC suppliers/contractors. Nevertheless, it is clear from previous PennDOT experience as well as the . For example, PennDOT District 6-0 projects have previously partnered with the Elastizell Corporation on the installation of LCC for I-95 wingwalls. The Elastizell Corporation maintains a list of approved applicators of LCC for engineered fills (https://elastizell.com/approved-engineered-fill-applicators/). Other known US contractors with an extensive network of suppliers and installers of LCC include Cell-Crete Corporation (https://www.cell-crete.com/locations/) and Pacific International Grout Company (https://pigcoinc.com/). Finally, FGA has a more extensive network of manufacturers/suppliers in Europe where FGA first originated in the 1980s. However, there are only a limited number in the United States with two based in the Northeast: AeroAggregates located in Eddystone, Pennsylvania (https://aeroaggregates.com/); and Glavel located in Burlington, Vermont (https://www.glavel.com/). Despite a smaller network, the proximity to Pennsylvania for both of these manufacturers encourages the consideration of FGA in highway-related construction across Pennsylvania. FGA is currently being used in projects across the mid-Atlantic all the way up to the Northeast from North Carolina to Maine.

4.1.3.4. Constructability

As noted in earlier sections, each of the lightweight fill technologies have specific considerations related to placement and constructability that affect their suitability for different kinds of highway-related infrastructure. Given that each project has a unique set of logistical constraints, site conditions, and geometry, the role of constructability can vary across different projects. In other words, what may be more desirable for fill placement in one project may be undesirable for another depending on the unique circumstances of that particular project. Generally speaking, ESCS and FGA will be the most similar to regular aggregate in terms of construction efforts since they share similar gradation characteristics, compaction efforts, and material handling. However, extra care must be taken with respect to compaction of FGA given the tendency for particle breakage. LCC is self-leveling and self-compacting, which allow engineers to use this material in limited work space and places where compaction is difficult to perform. The flowability of this material also allows it to reach areas where EPS, ESCS, and FGA may be impractical. However, LCC necessitates additional equipment on site to mix the components and pump the slurry into place. Additionally, the curing time needed between lifts may increase the project timeline relative to the other lightweight fill technologies. Finally, placement of EPS Geofoam is the most dissimilar to typical aggregates used in highway construction due to its large rectangular form factor and the lack of necessary compaction efforts. The rectangular shape and standard blocks sizes also limit their usage in narrow and irregular areas. Though cutting the blocks to size is possible, it does introduce some additional construction efforts. Consequently, EPS Geofoam has more commonly been used in larger scale applications such as embankments which can take advantage of the block form factor and size.

4.1.3.5. Sustainability

One final aspect worthy of consideration when selecting between appropriate lightweight fill technologies is whether the material is derived from recycled or waste products and can increase the sustainability of a particular project. The push towards more sustainable highway construction has encouraged the use of alternative materials and designs that can reduce or offset carbon emissions. Within that context, FGA is derived from 100% post-consumer recycled glass and diverts a waste stream away from landfills. FGA can also be reused and recycled and its light weight allows for more efficient transport relative to other lightweight fill technologies such as ESCS (e.g., 100 yd^3 trailer for FGA versus triaxle truck for ESCS), which can further add to its sustainability. EPS Geofoam blocks are 100% recyclable and some studies have explored the performance of Geofoam blocks derived from recycled EPS and found that the mechanical properties of recycled-content EPS can meet current ASTM D6817 minimum compressive resistance and flexural strength requirements (Wang and Arellano 2014). However, the availability of recycled EPS Geofoam appears limited and it is unclear what percentage of the EPS is from recycled sources when examining information provided by manufacturer/supplier websites and brochures. ESCS and LCC do not use recycled materials or waste byproducts in their manufacture.

4.2. Summary

This chapter highlighted the conditions under which lightweight fills can be considered for various highway projects. Additionally, it discussed the specific considerations that must be accounted for when designing with EPS Geofoam, LCC, ESCS, and FGA. Recommendations were made regarding what aspects should be prioritized when considering these specific lightweight fill technologies as a design alternative, including unit weight, cost, availability, constructability, and sustainability. No particular technology can ever be recommended in all cases, and careful consideration should be placed on navigating the specific constraints presented by each project and set of site conditions.

5. Conclusions

Lightweight fill materials can be used for highway-related infrastructure when specific site conditions demand decreased applied loads, reductions in settlements, elimination or reduction of the need for surcharge loading, reductions in lateral loads, and reductions in construction time. However, lightweight fill technologies are often neglected in favor of other design options such as deep foundations and/or ground improvement methods. This is despite the fact that documented case histories have demonstrated successful implementation of lightweight fills at reduced costs and construction times. Several examples of such case histories were presented in previous chapters of this report, including some projects implemented by PennDOT. Based on the literature review and case histories, this report provided guidelines regarding appropriate selection of lightweight/sustainable fill technologies, focusing on EPS Geofoam, LCC, ESCS, and FGA. As part of these guidelines, recommendations were made to consider the following items when deciding between these four lightweight/sustainable fill materials: unit weight, cost, availability, constructability, and sustainability. Consideration of these parameters as well as the unique constraints presented at each site can ensure successful implementation of lightweight/sustainable fill technologies across a wide range of highway projects.

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