Project No. 24-11(02) **COPY NO. _____**

Guidelines for Geofoam Applications in Slope Stability Projects

FINAL REPORT

Prepared for
NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
Transportation Research Board
of
The National Academies

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Author Acknowledgements

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ABSTRACT

This report presents the results of a study performed to develop a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design guidance to facilitate the use of EPS-block geofoam for slope stabilization and repair. This report includes the following five primary research products: (1) summary of relevant engineering properties, (2) a comprehensive design guideline, (3) a material and construction standard, (4) economic data, and (5) a detailed numerical design example. In addition to these primary research products, an overview of construction tasks that are frequently encountered during EPS-block geofoam slope projects and a summary of case histories that provides examples of cost-effective and successful EPS-block geofoam slope stabilization projects completed in the U.S. is provided. The purpose of this report is to provide those who have primary involvement with roadway embankment projects, including the following five groups: design professionals, manufacturers/suppliers, contractors, regulators, and owners, with design guidance for use of EPS-block geofoam in slope stabilization and repair. The end users of the research include engineers, who perform the design and develop specifications, and owners, including the FHWA, state DOTs, and local county and city transportation departments that own, operate, and maintain roadways.

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EXECUTIVE SUMMARY

This report presents the results of a study performed under NCHRP Project 24-11(02) to develop a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design guidance to facilitate the use of expanded polystyrene (EPS)-block geofoam for slope stabilization and repair. This study is the second part of a two-part study on EPS-block geofoam. The first study was performed under NCHRP Project 24-11(01) titled "Guidelines for Geofoam Applications in Embankment Projects." The objective of the first study was to develop a recommended design guideline and a recommended material and construction standard for the use of EPS-block geofoam in stand-alone embankments and bridge approaches over soft ground.

The results of both NCHRP Project 24-11 studies demonstrate that EPS-block geofoam is a unique lightweight fill material that can provide a safe and economical solution to construction of stand-alone embankments and bridge approaches over soft ground, as well as an effective and economical alternative to slope stabilization and repair. Benefits of utilizing EPS-block geofoam as a lightweight fill material include: (1) ease of construction, (2) can contribute to accelerated construction, (3) ability to easily implement phased construction, (4) entire slide surface does not have to be removed because of the low driving stresses, (5) can be readily stored for use in emergency slope stabilization repairs, (6) ability to reuse EPS blocks utilized in temporary fills, (7) ability to be placed in adverse weather conditions, (8) possible elimination of the need for surcharging and staged construction, (9) decreased maintenance costs as a result of less settlement from the low density of EPS-block geofoam, (10) alleviation of the need to acquire additional right-of-way for traditional slope stabilization methods due to the ease with which EPS-block geofoam can be used to construct vertical-sided fills, (11) reduction of lateral stress on bridge approach abutments, (12) excellent durability, (13) potential construction without utility relocation, and (14) excellent seismic behavior.

The benefit of accelerated construction that use of EPS-block geofoam can provide was a key factor in the decision to use EPS-block geofoam in projects such as the I-15 reconstruction project in Salt Lake City, UT; the Central Artery/Tunnel Project (CA/T) in Boston, MA; and the I-95/Route 1 Interchange (Woodrow Wilson Bridge Replacement) in Alexandria, VA. EPS block utilized in slope stabilization and repair may not support a pavement system or heavy structural loads. Therefore, the potential to utilize EPS blocks with recycled EPS exists. The use of recycled EPS blocks would be an attractive "green" product that reduces waste by recycling polystyrene scrap and would also reduce raw material costs in production of EPS.

The NCHRP Project 24-11 studies revealed important analysis and design differences between the use of EPS-block geofoam in slope applications versus stand-alone applications over soft ground. The primary differences between slope applications versus stand-alone embankments over soft ground include the following issues: (1) Site characterization is usually much more complex and difficult because it typically involves explorations made on an existing slope and concomitant access difficulties; the slope cross-section often consists of multiple soil and rock layers that vary in geometry both parallel and perpendicular to the road alignment; and piezometric conditions may be very complex and even seasonal in variation. (2) The governing design issue is usually based on an ultimate limit state (ULS) failure involving the analysis of shear surfaces using material strength and limit-equilibrium techniques. Serviceability limit state (SLS) considerations involving material compressibility and global settlement of the fill are rarely a concern. (3) There is typically an unbalanced earth load, often relatively significant in magnitude, acting on the EPS mass that must be addressed as part of the design process. (4) Piezometric conditions are often a significant factor to be addressed in design. In fact, if the use of EPS geofoam is being considered to reconstruct a failed or failing area, piezometric issues typically contribute to the cause of the failure in the first place. (5) The volume of EPS placed within the overall slope cross-section may be relatively limited. Furthermore, the optimal location of the EPS mass within the overall slope cross-section is not intuitively obvious. (6) The road pavement may not overlie the portion of the slope where the EPS is placed. Therefore load conditions on the EPS blocks may be such that blocks of relatively low density or recycled EPS block can be used which can achieve economies in the overall design.

The main deliverables emanating from this project include: (1) a summary of relevant engineering properties, (2) a comprehensive design guideline, (3) a material and construction standard, (4) economic data, and (5) a detailed numerical design example. A summary of engineering properties of EPS-block geofoam that are relevant in the design of slopes is included in Chapter 3. A recommended design guideline is included in Appendix B. Chapter 4 provides the background to the design guideline and is the commentary to the design guideline. A recommended combined material and construction standard covering block-molded EPS for use as lightweight fill in slope stabilization and repair is included in Appendix F. Chapter 6 provides the basis of the recommended standard. Cost information related to the use of EPS-blocks geofoam in slope applications is included in Chapter 8. A detailed numerical example that demonstrates the recommended design guideline included in Appendix B and summarized in Chapter 4 is included in Appendix E.

In addition to the five primary research products listed above, an overview of construction tasks that are frequently encountered during EPS-block geofoam slope projects is included in Chapter 5, and four case histories are presented in Chapter 7 that provide examples of cost-effective and successful EPS-block geofoam slope stabilization projects completed in the U.S.

The purpose of this report is to provide those who have primary involvement with roadway embankment projects (including the following five groups: design professionals, manufacturers/suppliers, contractors, regulators, and owners), with design guidance for use of EPS-block geofoam in slope stability applications. The end users of the research include engineers, who perform the design and develop specifications, and owners, including the FHWA, state DOTs, and local county and city transportation departments that own, operate, and maintain roadways.

An example of extensive use of the Project 24-11(01) deliverables is the large use of EPS-block geofoam on the (CA/T) project in Boston, MA. This project is the first major project to use the Project 24-11(01) research results in practice. Another project that utilized the NCHRP results is the I-95/Route 1 Interchange (Woodrow Wilson Bridge Replacement) in Alexandria, VA. It is anticipated that the deliverables of this Project 24-11(02) study will also be used extensively, and will contribute to solving the major geologic hazard of landslides, which are expected to increase as new roadway alignments are constructed, and/or existing roadway embankments are widened as part of the effort to meet the growing demand of U.S highway capacity.

The general consensus that was reached at the first *International Workshop on Lightweight Geo-Materials* that was held on March 26-27, 2002, in Tokyo, Japan, is that although new weight-reduction techniques for decreasing applied loads have recently been developed, standardization of design and construction methods is required. The research results from both NCHRP 24-11 studies on EPS-block geofoam standardize the design and construction standards for the use of EPS-block geofoam in various U.S. highway applications.

The FHWA has designated EPS-block geofoam as a priority, market-ready technology with a deployment goal that EPS geofoam will be a routinely used lightweight fill alternative on projects where the construction schedule is of concern. The FHWA considers EPS-block geofoam an innovative material and construction technique that can accelerate project schedules by reducing vertical stress on the underlying soil and thus is a viable and cost-effective solution to roadway embankment widening and new roadway embankment alignments over soft ground. In summary, EPS-block geofoam is a market-ready technology that can contribute to solving the major highway problem in the U.S. of insufficient highway capacity to meet growing demand.

CHAPTER 1

BACKGROUND

INTRODUCTION

Geofoam is any manufactured material created by an internal expansion process that results in a material with a texture of numerous, closed, gas-filled cells using either a fixed plant or an in situ expansion process (Horvath, 1995). The predominant geofoam material used successfully from a technical and cost perspective as lightweight fill in road construction is expanded polystyrene-block (EPS-block) geofoam.

Geofoam is considered a type or category of geosynthetic. As with most types of geosynthetics, geofoam can provide a wide variety of functions including thermal insulation, lightweight fill, compressible inclusion, fluid transmission (drainage), damping, low earth pressure fill for retaining structures, and structural support. Each of these functions may have numerous potential applications. The focus of the present study is on the geofoam function of *lightweight fill* and the specific application of this function is slope stabilization and remediation of roadway embankments subjected to slope instability. The fact that geofoam can provide other functions, even if not intended or not necessarily desired in a particular project, should be considered in the design of geofoam for lightweight fills in road embankments. For example, in addition to the lightweight fill function, the functions of structural support and thermal insulation should be considered during the use of EPS-block geofoam as a lightweight fill material in slope stabilization and repair.

The first project to use block-molded EPS as a lightweight fill material was the Flom Bridge project in Norway in 1972. The EPS-block geofoam was used to rebuild a road over soft soil that had chronic settlement problems. In Europe, lightweight fills such as EPS-block geofoam are routinely used to construct embankments over soft foundation soils. In Japan, EPS-block geofoam is also extensively used for lightweight fill applications including in slope applications. Significant research and development of the use of EPS-block geofoam has been performed in Japan for seismic loading applications (Horvath, 1999).

Although EPS-block geofoam for road construction is an established technology, and despite the over 30 years of extensive and continuing worldwide use of EPS-block geofoam, it has been underutilized in U.S. practice because a comprehensive design guideline and a material and construction standard or specification for its use as lightweight fill in roadway embankments has been unavailable. Therefore, there was a need in the U.S. to develop formal and detailed design documents as well as an appropriate material and construction standard for use of EPS-block geofoam in roadway applications.

To meet this need, the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration (FHWA), funded National Cooperative Highway Research Program (NCHRP) Project 24-11(01) titled "Guidelines for Geofoam Applications in Embankment Projects." The NCHRP research was conducted from July 6, 1999 to August 31, 2002. The objective of this research was to develop a recommended design guideline and a recommended material and construction standard for the use of EPS-block geofoam in stand-alone embankments and bridge approaches over soft ground.

The results of this NCHRP project are presented in two reports. One report includes only the recommended design guideline and the recommended material and construction standard for use of geofoam in stand-alone roadway embankments (Stark et al., 2004a). The second report includes the background and analyses used to develop the recommended design guideline and material and construction standard, as well as a summary of the engineering properties of EPS-block geofoam and an economic analysis of geofoam versus other lightweight fill materials (Stark et al., 2004b).

The NCHRP Project 24-11(01) research confirmed that EPS-block geofoam can provide a safe and economical solution for construction of stand-alone roadway embankments on soft soils. Benefits of utilizing an EPS-block geofoam embankment include: (1) ease and speed of construction, (2) placement

in adverse weather conditions, (3) possible elimination of the need for preloading, surcharging, and staged construction, (4) decreased maintenance costs as a result of less settlement from the low density of EPS-block geofoam, (5) alleviation of the need to acquire additional right-of-way to construct flatter slopes because of the low density of EPS-blocks and/or the use of a vertical embankment because of the block shape of EPS, (6) reduction of lateral stress on bridge approach abutments, (7) use over existing utilities which reduces or eliminates utility relocation, and (8) excellent durability.

EPS-block geofoam is unique as a lightweight fill material because it has a unit weight that is only about 1 percent of the unit weight of traditional earth fill materials and also substantially less than other types of lightweight fills (16 kg/m^3 or 1 lb./ft^3 versus 1900 kg/m^3 or 120 lb./ft^3). In addition, geofoam is sufficiently strong to support heavy motor vehicles, trains, airplanes, lightly loaded buildings, and the abutments of bridges, if designed properly. The extraordinarily low unit weight of EPS-block geofoam results in significantly reduced gravity stresses on underlying foundation soils as well as reduced inertial forces during seismic shaking. Thus, the lower density of EPS-block geofoam may alleviate the costs of soft soil removal (which include the attendant disposal problems and costs), soil improvement techniques, and/or the possible need for an excavation support system, excavation widening, and extensive temporary dewatering.

DOTs are particularly interested in the benefit of accelerated construction that EPS-block geofoam can provide when constructing embankments over soft foundation soils. In June 2002, the FHWA, in a joint effort with AASHTO, organized a geotechnical engineering scanning tour of Europe (AASHTO and FHWA, 2002). The purpose of the European scanning tour was to identify and evaluate innovative European technology for accelerated construction and rehabilitation of bridge and embankment foundations. Lightweight fills is one of the technologies that was evaluated. One of the preliminary findings of the scanning project is that lightweight fills such as geofoam are an attractive alternative to surcharging soft soil foundations because the requirement of preloading the foundation soil can possibly be eliminated and therefore, construction can be accelerated.

The material cost per volume of EPS-block geofoam may initially seem greater than most other types of lightweight fills and conventional soil fill. However, if the intangible benefits of using geofoam are included in the cost analysis, e.g., reduced field installation and construction costs, shorter time roadway is not in service, and minimum field quality control testing, geofoam is then a cost-effective alternative to constructing roadway embankments over soft ground. On many projects, the overall immediate and long-term benefits and lower construction cost of using EPS-block geofoam more than compensate for the fact that its material unit cost is somewhat more than that of traditional earth fill materials.

An example of the extensive use of the NCHRP Project 24-11(01) deliverables is the large use of EPS-block geofoam on the Central Artery/Tunnel (CA/T) project in Boston, MA. This project is the first major project to use the NCHRP Project 24-11(01) research results in practice (Riad, 2005). Another project that utilized the NCHRP results is the I-95/Route 1 Interchange (Woodrow Wilson Bridge Replacement) in Alexandria, VA. These and other projects that have been completed in the United States, e.g., the I-15 Reconstruction Project in Salt Lake City, UT, demonstrate that EPS-block geofoam is a technically viable and cost-effective alternative to the construction or remediation of stand-alone embankments over soft ground. Additionally, Thompson and White (2005) conclude that EPS-block geofoam may be a stabilization technology that can be used as an alternative to the use of stability berms to minimize the impacts to environmentally sensitive areas where embankments cross soft or unstable ground conditions.

The FHWA has designated EPS-block geofoam as a priority, market-ready technology with a deployment goal that EPS geofoam will be a routinely-used lightweight fill alternative on projects where the construction schedule is of concern (FHWA, 2006). The FHWA considers EPS-block geofoam an innovative material and construction technique that can accelerate project schedules by reducing vertical stress on the underlying soil and thus, is a viable and cost-effective solution to roadway embankment widening and new roadway embankment alignments over soft ground. In summary, EPS-block geofoam

is a market-ready technology that can contribute to solving the major highway problem in the U.S. of insufficient highway capacity to meet growing demand.

PROBLEM STATEMENT

A major transportation problem in the U.S. is that current highway capacity is insufficient to meet the growing demand. Therefore, new roadway alignments and/or widening of existing roadway embankments will be required to solve the current and future highway capacity problem. As noted by Spiker and Gori (2003), roadway construction "often exacerbates the landslide problem in hilly areas by altering the landscape, slopes, and drainages and by changing and channeling runoff, thereby increasing the potential for landslides." Landslides occur in every state and U.S. territory, especially in the Pacific Coast, the Rocky Mountains, the Appalachian Mountains, and Puerto Rico (Spiker and Gori, 2003, Transportation Research Board, 1996). Active seismic activity contributes to the landslide hazard risk in areas such as Alaska, Hawaii, and the Pacific Coast. Spiker and Gori indicate that landslides are among the most widespread geologic hazard on earth and estimate damages related to landslides exceed $2 billion annually.

An additional application of EPS-block geofoam as a lightweight fill that has not been extensively utilized in the U.S., but has been commonly used in Japan, is in slope stabilization. The decades of experience in countries such as Norway and Japan with both soft ground and mountainous terrain have demonstrated the efficacy of using the lightweight fill function of EPS-block geofoam in both stand-alone embankments over soft ground and slope stabilization applications. The Japanese experience has also involved the use of EPS-block geofoam when severe seismic loading is a design criterion. The recommended design guideline and the standard included in the Project 24-11(01) reports are limited to stand-alone embankments and bridge approaches over soft ground. The experience in Japan has demonstrated that there are important analysis and design differences between the lightweight fill function for stand-alone embankments over soft ground and slope stabilization applications. Therefore, a need exists in the U.S. to develop formal and detailed design documents, design guideline, and an appropriate material and construction standard for use of EPS-block geofoam for slope stabilization projects. Slope stabilization projects include new roadways as well as repair of existing roadways that have been damaged by slope instability or slope movement. This need resulted in the current NCHRP Project 24-11(02), the results of which are described herein.

SOLUTION ALTERNATIVES

Slope stability represents one of the most complex and challenging problems within the practice of geotechnical engineering. The unique challenges presented by the interactions between groundwater and earth materials, the complexities of shear strength in earth materials, and the variable nature of earth materials and slope loadings can combine to make the successful design of a stable slope difficult, even for an experienced engineer. Over the years, a wide variety of slope stabilization and repair techniques have been used in both natural and constructed slopes. When implementing a slope stabilization and repair design, the strategy employed by the designer can usually be classified as: (1) avoid the problem altogether, (2) reduce the driving forces, or (3) increase the resisting forces. These broad categories are discussed in greater detail in Chapter 4, along with examples of each category. For any given project, the option of avoiding the problem is generally the simplest solution; however, it is typically not a feasible option, especially for roadways. In many cases, selecting an alternate site or removing and replacing the problematic earth material is simply not a viable option. This leaves designers with a choice between the remaining two strategies for constructing a stable slope. The resisting forces may be accepted as they are, and the design may be based on reducing the forces that drive instability; or, conversely, the driving forces may be accepted as they are, and the design may be based on improving the resisting forces sufficiently to prevent failure of the slope.

Some of the more common design alternatives to increase the resisting forces of a slope include: installation of butress fills, rock shear keys, deep foundations, e.g., piles and drilled shafts or other type of reinforcing material to assist in restraining the unstable slope material, construction of "toe berms" to add weight to the bottom portion of the slope, chemical or biotechnical soil improvement methods to increase the strength of earth materials, and/or installation of subsurface drainage to divert groundwater away from the slope and increase the effective stress which increases the soil resisting forces. Many of these procedures can be costly, both in terms of actual installation costs, as well as other indirect costs such as prolonged road closures, acquisition of additional right-of-way for the new construction, and long-term maintenance costs. However, some of these procedures do have the advantage of having a relatively long history of successful application. In many cases, designers and contractors are somewhat familiar with the approaches being used, enabling them to work more efficiently when using a well-established technology.

The simplest solution to reducing the driving forces within a slope is to simply reduce the slope inclination. This reduces the shear stress on the material in the slope, making the entire slope more stable. However, the costs of pursuing this solution can be considerable, including right-of-way acquisition, earth material removal costs, and lane or road closures during construction. For many slopes, particularly those in urban settings, flattening the slope is simply not a feasible option. Other alternatives that serve to reduce driving forces could be the installation of subsurface drainage (which can serve both to increase resisting forces and to reduce driving forces), installation of better surface drainage to reduce infiltration from storm water accumulation, and replacement of a portion of the natural slope material with lightweight fill.

The latter alternative to reducing the driving forces may encompass a wide variety of materials, both natural and man-made, that can significantly reduce the weight of the upper portion of the slope, thus reducing driving forces that tend to cause slope instability. A wide range of lightweight fill materials, such as shredded tires, wood fiber, saw dust, ash, pumice, air foamed stabilized soil, expanded-beads mixed with soil, and EPS-block geofoam, have been successfully used as lightweight fill both in the U.S. and globally. As might be expected, each type of lightweight fill has its own unique advantages and disadvantages which must be considered when evaluating alternatives for any design. The purpose of this report is to provide guidance for slope stabilization and repair utilizing EPS-block geofoam as a lightweight fill material.

RESEARCH OBJECTIVE

The overall objective of this research was to develop a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design guidance for engineers to facilitate use of EPS-block geofoam for the function of lightweight fill in slope stability applications. This document includes the design guideline as well as an appropriate material and construction standard.

The general consensus that was reached at the first *International Workshop on Lightweight Geo-Materials* that was held on March 26-27, 2002, in Tokyo, Japan, is that although new weight-reduction techniques for decreasing applied loads have recently been developed, standardization of design and construction methods is required (A Report on the *International Workshop on Lightweight Geo-Materials*, 2002). The research results from NCHRP Project 24-11(01) in conjunction with the results of this project standardize the design and construction standards for the use of EPS-block geofoam in various U.S. highway applications.

REPORT ORGANIZATION

The purpose of this report is to provide those who have primary involvement with roadway embankment projects, including the following five groups: design professionals, manufacturers/suppliers, contractors, regulators, and owners, with design guidance for use of EPS-block geofoam in slope stability applications. The end users of the research include engineers, who perform the design and develop

specifications, and owners, including the FHWA, state DOTs, and local county and city transportation departments that own, operate, and maintain the roadway.

This report is divided into two parts. The first part consists of nine chapters. This chapter, Chapter 1, provides a brief overview of EPS-block geofoam, a summary of the Project 24-11(01) study that focused on the use of EPS-block geofoam in stand-alone embankments over soft ground, the problem statement that led to the present study on the use of EPS-block geofoam in slope stabilization applications, and the research objective of this study. Chapter 2 provides a summary of the research approach used during this study. Chapter 3 provides an overview of EPS block engineering properties that are most relevant to the design of slopes stabilized with EPS block. Chapter 4 provides the design methodology developed herein for slopes incorporating EPS-block geofoam for the function of lightweight fill in slope stability stabilization and repair. Chapter 5 provides an overview of construction tasks that are frequently encountered during EPS-block geofoam slope projects. Chapter 6 presents the background for understanding the recommended EPS-block geofoam standard for slope stability applications included in Appendix F. Chapter 7 provides a summary of several case histories that have successfully incorporated EPS-block geofoam in slope stabilization applications. Chapter 8 provides cost information to allow a cost estimate for geofoam slope stabilization to be prepared during the design phase so that an optimal geofoam slope design can be selected. The designer can then use this optimal geofoam slope design to perform a cost comparison with other slope stabilization techniques. Finally, Chapter 9 provides a summary of the conclusions and recommendations of the findings from this study, as well as recommended areas of future research for EPS-block geofoam for the function of lightweight fill in slope stabilization and repair.

The second part of the report is composed of twelve appendices. Appendix A describes a geofoam usage survey that was developed and distributed during this study and also presents the responses to the survey. Appendix B presents the recommended design guideline for EPS-block geofoam slopes. Appendix C presents the details to the two procedures developed during this study for optimizing the volume and location of EPS blocks within the slope. One procedure is for landslides involving rotational slides and the second for translational slides. Appendix D provides the results of the study that was performed to determine the impact of typical centrifugal loads on an EPS-block fill mass. Appendix E presents a design example that demonstrates the design methodology included in Chapter 4 and outlined in the design guideline included in Appendix B. Appendix F presents the recommended standard for use of EPS-block geofoam, which should facilitate DOTs in specifying, and thus contracting, for the use of geofoam in slope stabilization and repair projects. Appendix G provides example design details and Appendix H provides example slope stabilization specifications. Appendix I includes a draft of a contract special provision for price adjustment for EPS-block geofoam that is similar to the special provisions that DOTs have utilized for other construction materials such as bituminous asphalt binder to minimize the impact of short-term oil price fluctuations on the cost of EPS-block geofoam during a multi-phased project. Appendix J includes the Phase I work plan and Appendix K includes the Phase II work plan. Finally, Appendix L provides an extensive bibliography of all references encountered during this study that relate to EPS-block geofoam.

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CHAPTER 2

RESEARCH APPROACH

OVERVIEW

The objective of this research is to develop a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design guidance to engineers, owners, and regulators for the use of EPS-block geofoam as lightweight fill in slope stability applications. Successful technology transfer and acceptance of a construction product or technique requires the availability of a comprehensive and useful design procedure and a material and construction standard. Additionally, knowledge of the engineering properties of materials that will be incorporated in a structure is also required to adequately design the structure. Designers also need cost data related to the proposed construction product or technique to perform a cost comparison with other similar alternatives. One of the lessons learned with the use of EPS-block geofoam on the CA/T Project in Boston is the need to include a detailed numerical design example to complement design guidelines.

Therefore, the five primary research products required to ensure successful technology transfer of EPS-block geofoam technology to slope stability applications in new and existing roadway projects that are included in this report are: (1) summary of relevant engineering properties, (2) a comprehensive and useable design guideline, (3) a material, product, and construction standard, (4) economic data, and (5) a detailed numerical example.

To develop these key research products, and thereby accomplish the research objective of developing a comprehensive document that provides design guidance to engineers for the use of EPS-block geofoam for the function of lightweight fill in slope stability applications, the research consisted of two phases. The Phase I work plan is included in Appendix J and the Phase II work plan is included in Appendix K. An overview of the research approach used in each phase is presented below.

PHASE I RESEARCH APPROACH

The objective of the Phase I research was to review, document, and synthesize the worldwide experience of using EPS-block geofoam as lightweight fill in new and existing slope stability applications and develop a recommended interim design guideline and material and construction standard. Phase I consists of the following six tasks: (1) perform literature search, (2) summarize design methods, (3) summarize geofoam construction practices, (4) review and update the NCHRP Project 24-11(01) recommended EPS-block geofoam material and construction standard, (5) perform an economic analysis of geofoam versus other lightweight fill material for slope stabilization purposes, and (6) prepare an interim report that summarizes the results of Phase I. As the Phase I work progressed, it was determined that it would be better to include a preliminary design algorithm as part of the interim design guideline instead of developing the algorithm during Phase II as was initially planned. Therefore, Task 8 (develop a design algorithm) was completed during Phase I.

Task 1 (perform literature search) consisted primarily of a literature review and a geofoam usage survey that was conducted via a questionnaire to obtain case history information, cost data, design details, and other geofoam related information for slope stabilization projects. The findings of Task 1 were used to accomplish Tasks 2 through 5. The results of the Phase I work were included in an interim report that was prepared as part of Task 6.

The interim design guideline that was included in the interim report was based on an assessment of existing technology and literature that involved primarily stand-alone embankments over soft ground. The Phase I research revealed important analysis and design differences between the use of EPS-block geofoam for the lightweight fill function in slope stability applications versus stand-alone applications over soft ground. The primary differences between slope applications versus stand-alone embankments over soft ground are:

- Site characterization is usually much more complex and difficult because it typically involves explorations made on an existing slope and concomitant access difficulties. The slope cross-section often consists of multiple soil and rock layers that vary in geometry both parallel and perpendicular to the road alignment, and piezometric conditions may be complex and seasonal in variation.
- The governing design issue is usually based on an ultimate limit state (ULS) involving the analysis of shear surfaces using material strength and limit-equilibrium techniques. Serviceability limit state (SLS) considerations involving material compressibility and global settlement of the fill are rarely a concern for slope stabilization versus stand-alone embankments over soft ground.
- There is typically an unbalanced earth load, often relatively significant in magnitude, acting on the EPS mass that must be addressed as part of the slope design process.
- Piezometric conditions are often a significant factor to be addressed in design. In fact, piezometric issues typically contributed to the cause of the failure in the first place. If the use of EPS geofoam is being considered to reconstruct a failed or failing area, drainage below and around the blocks should be considered.
- The volume of EPS placed within the overall slope cross-section may be relatively limited. Furthermore, the optimal location of the EPS mass within the overall slope cross-section is not intuitively obvious as it is with embankments.
- The road pavement may not overlie the portion of the slope where the EPS is placed. Therefore, load conditions on the EPS blocks may be such that blocks of relatively low density can be used throughout, which can achieve cost savings in the overall design, whereas high density blocks must be used near the top of stand-alone embankments.

Because a majority of the analysis and design methods available in the literature focus on stand-alone embankments over soft ground, further study was required to address various uncertainties in the current state-of-practice for analyzing various failure mechanisms included in the interim design procedure. The Phase II study addressed these uncertainties, refined the interim design procedure, and completed the comprehensive design guideline for the use of EPS-block geofoam in slope stabilization and repair applications.

PHASE II RESEARCH APPROACH

The objective of Phase II was to develop a comprehensive design methodology that optimizes both technical performance and cost for geofoam as lightweight fill in new and existing slope stability applications for highway projects. Phase II includes the following tasks: (7) pavement design considerations, (8) develop a design algorithm, (9) performance-based issues related to the material standard, (10) evaluate the applicability of the simplified seismic response methodology for stand-alone embankments to slope applications, (11) determine the impact of typical centrifugal loads on an EPS-block geofoam fill mass, (12) obtain higher density block test data, (13) update the design algorithms developed in Task 8, (14) develop routine design aids for slope stability applications, and (15) prepare a final report. As previously noted, Task 8 (develop a design algorithm) was completed during Phase I.

Unlike the use of EPS-block geofoam for stand-alone embankments over soft ground, the U.S. case history experience with EPS-block geofoam in slope stabilization is limited. Therefore, it was a more difficult and longer process to develop procedures to analyze the various slope failure mechanisms included in the design procedure for slope applications than to develop the NCHRP Project 24-11(01) design procedure for stand-alone embankments over soft ground.

KEY RESEARCH PRODUCTS

The five primary research products required to ensure successful technology transfer of geofoam information to slope stability applications in new and existing roadway projects are included in this report. First, a summary of engineering properties of EPS-block geofoam relevant to slope stabilization is included in Chapter 3. These properties include geofoam shear strength and density. Additionally, limit equilibrium methods of slope stability analysis are currently the most common approach for analyzing slopes, so an overview of the various approaches to modeling the strength of EPS block in limit equilibrium methods of slope stability analysis is also presented in Chapter 3. Second, a recommended design guideline developed herein for slopes incorporating EPS-block geofoam for the function of lightweight fill in slope stabilization and repair is included in Appendix B. Chapter 4 provides the background to the design methodology incorporated in the design guideline.

Third, a recommended material, product, and construction standard or specification for slope stability applications is included in Appendix F. Chapter 6 presents the background for understanding the recommended EPS-block geofoam standard included in Appendix F. Fourth, economic data is provided in Chapter 8. This chapter provides cost information to allow a cost estimate for the geofoam stabilization procedure to be prepared during the design phase so an optimal geofoam slope design can be selected. The designer can then use this optimal geofoam slope design to perform a cost comparison with other slope stabilization techniques. Fifth, a detailed numerical example is included in Appendix E that demonstrates the design methodology included in Chapter 4 and the design guideline included in Appendix B.

In summary, the results of this study, which are included in this report, include: (1) a summary of relevant geofoam engineering properties, (2) a comprehensive design guideline, (3) a material, product, and construction standard, (4) economic data, and (5) a detailed numerical example. These key research products facilitate the accomplishment of the overall research objective of this study, which is to develop a comprehensive document that provides design guidance to engineers, owners, and regulators for the use of EPS-block geofoam for the function of lightweight fill in slope stability applications.

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CHAPTER 3

ENGINEERING PROPERTIES OF BLOCK-MOLDED EPS RELEVANT TO SLOPE STABILIZATION

INTRODUCTION

Relevant engineering properties of block-molded EPS for the function of lightweight fill include physical, mechanical (stress-strain-time-temperature), and thermal. A comprehensive overview of these engineering properties of EPS is included in the NCHRP Project 24-11(01) report (Stark et al., 2004a). Additionally, the primary elements of the molding process are included in Project 24-11(01) report because the EPS block molding process can influence the quality and other performance aspects of EPS-block geofoam to include the physical, mechanical, and thermal properties. This chapter provides an overview of EPS block engineering properties that are most relevant to the design of slopes stabilized with EPS block. These properties include shear strength and density. Because limit equilibrium methods of slope stability analysis are commonly used for analyzing slopes, an overview of the various approaches available to model the strength of EPS block in limit equilibrium procedures of slope stability analysis is also presented.

SHEAR STRENGTH

Overview

In lightweight fill applications, two shear failure modes are of interest; Internal Shear Strength, related to the strength of individual EPS blocks, and External Shear Strength, which concerns the interface shear resistance between individual blocks or between EPS blocks and a dissimilar materials (soil, other geosynthetic, etc.).

Internal Shear Strength

The internal shear strength of EPS represents failure through the individual EPS block. The internal shear strength of EPS is measured by loading a test specimen fairly rapidly until the maximum shear stress is reached, whether or not this stress produces a physical rupture of the test specimen. ASTM test method C 273 (American Society for Testing and Materials, 2001) addresses internal shear strength of geofoam. However, this test method addresses the testing of cores of structural “sandwiches” or composites (Horvath, 1995), not an individual block. The correlation between shear strength of EPS block and EPS density is shown in Figure 3.1. The values of shear strength shown in Figure 3.1 were obtained by Horvath, (1995) from a manufacturer’s technical bulletin (BASF AG, 1991). However, specimen dimensions and testing strain rate are not included in the bulletin. Because the shear strength of EPS block exhibits a correlation with compressive strength, experience indicates that the shear strength test is rarely performed in practice for either Manufacturing Quality Control/Assurance (MQC/MQA) or engineering design.

Because the internal shear strength represents failure through an individual EPS block, the internal shear strength of a geofoam block can be represented by a cohesion value independent of the normal stress because the block has some shear resistance, even at zero normal stress. Because load bearing design, Step 10 of the proposed design procedure, is based on limiting strains of 1 percent, i.e. the elastic limit stress, cohesion can be estimated based on the elastic limit stress. Figure 3.2 shows a typical Mohr diagram for an unconfined compression test performed on soil. As shown, the undrained shear strength of a soil based on the Mohr-Coulomb shear strength criteria is one-half of the unconfined compression strength. For EPS block, the cohesion can be estimated as one-half of the elastic limit stress, as shown by Equation 3.1:

$$s_u = \frac{q_u}{2} = c = \frac{\sigma_e}{2} \qquad \textbf{(3.1)}$$

where

s_u	=	Undrained shear strength
q_u	=	Unconfined compressive strength
c	=	Cohesion
σ_e	=	Elastic limit stress

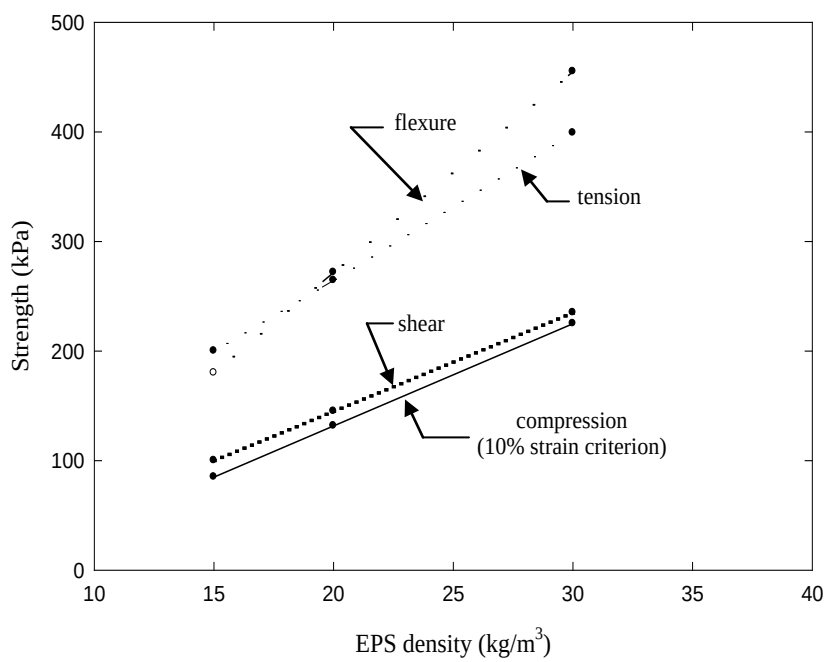

Figure 3.1 Strength of block-molded EPS in various test modes as a function of density (Horvath, 1995).

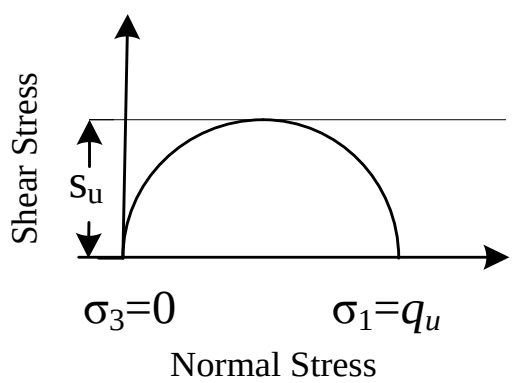

Figure 3.2. Typical UC test results.

External (Interface) Shear Strength

Interface friction, primarily along horizontal surfaces, is an important consideration in external and internal stability assessments under horizontal loads such as seismic shaking and shear stresses in slopes. Thus, tests to assess interface friction between the surface of EPS blocks and a variety of other materials are of interest in projects where significant horizontal design loads or internal sliding can occur. Two types of interfaces of interest for EPS-block geofoam in lightweight fill applications include an EPS/EPS interface, and an EPS/dissimilar material interface.

A discussion of EPS/EPS and EPS/dissimilar material interface shear resistance is included in the NCHRP Project 24-11(01) report (Stark et al., 2004a). An updated discussion is provided herein based on results reported in the literature since 2004. If the calculated resistance forces along the horizontal planes between EPS blocks is insufficient to resist the horizontal driving forces, additional resistance between EPS blocks is generally provided by adding mechanical inter-block connectors (typically prefabricated barbed metal plates) along the horizontal interfaces between the EPS blocks, or by adding a shear key. Therefore, a summary of available alternatives to increase the interface shear resistance between EPS blocks is also provided.

EPS/EPS Interface

The interface shear resistance of EPS/EPS interfaces has been studied by a number of researchers (Atmatzidis et al., 2001, EPS Construction Method Development Organization, 1993, Kuroda et al., 1996, Miki, 1996, Negussey et al., 2001, Norwegian Road Research Laboratory, 1992, Sanders and Seedhouse, 1994, Sheeley, 2000, Sheeley and Negussey, 2000). Unfortunately, the lack of a standard test method has meant that a range of test conditions (specimen size, specimen preparation, smoothness of specimen surface, test setup, loading rate, etc.) have been used. A summary of EPS/EPS interface shear resistance test results is included in Table 3.1.

Although there is no standard method for EPS/EPS interface shear tests, typical procedures that have been used involve placing two pieces of EPS in contact along a single horizontal surface, subjecting the contact to a vertical normal stress, then horizontally shearing one piece of EPS (typically the upper one) relative to the other while measuring the horizontal displacement and force required for movement. This process is similar to direct shear testing (ASTM D 5321) in soils and geosynthetics testing.

Based on a review of existing interface shear strength data between two pieces of EPS, the shearing resistance can be defined by the classical Coulomb (dry) friction equation:

$$\tau = \sigma_n \cdot \mu = \sigma_n \cdot \tan(\delta) \qquad \textbf{(3.2)}$$

where

τ	=	Shear strength
σ_n	=	Normal stress
μ	=	Coefficient of friction
δ	=	Interface friction angle

It is deemed that the interface has no adhesion, *a*, only a friction resistance.

Table 3.1 Summary of interface shear strength data for EPS/EPS interfaces.

Density (kg/m$^{3)}$	Confining Stress Range (kPa)	Reported Strength Properties	Testing Method	Sample Size (mm)	Cut Method	Surface	Shear Rate	References
20 30	σ_n = 14 - 55	δ_{Peak} = 40°, $\delta_{Residual}$ = 35° δ_{Peak} = 39°, $\delta_{Residual}$ = 33° δ_{Peak} = 40°, $\delta_{Residual}$ = 33° δ_{Peak} = 37°, $\delta_{Residual}$ = 33°	Direct Shear	100 x 100	Hot wire	Dry Wet Dry Wet	1.5 mm/min	Sheeley (2000)
20 30	σ_n = 14 - 55	*"Not Recommended"*			w. connector plates	NA		
12 15 20	σ_n = 10 - 52	δ_{Peak} = 42°, $\delta_{Residual}$ = 33° δ_{Peak} = 45°, $\delta_{Residual}$ = 33° δ_{Peak} = 48°, $\delta_{Residual}$ = 34°		175 x 375	Hot wire	NA	25 mm/min	Negussey et al. (2001)
18	σ_n = 10, 20, 30	δ_{Peak} = 43°, $\delta_{Residual}$ = 33° δ_{Peak} = 42°, $\delta_{Residual}$ = 29° δ_{Peak} = 32°, $\delta_{Residual}$ = 28° δ_{Peak} = 34°, $\delta_{Residual}$ = 32°			Hot wire Mfg. Skin	Dry Wet Dry Wet		
20	σ_n = 14 - 48	δ_{Peak} = 41°, $\delta_{Residual}$ = 33°		100 x 100 to 500 x 500	Hot wire	NA	NA	Sheeley and Negussey (2000)
20	σ_n = 0 - 50	δ = 27°		NA	NA	NA	NA	Miki (1996)
20	σ_n = 7.4 - 14.7	δ = 11 - 22°	Shake Table	500 x 1000	w. & w.o. Connect. Plates	NA	NA	Kuroda et al., (1996)
10	$\sigma_n \leq 17$ $\sigma_n > 17$	δ = 35° τ_{max} = 11 kPa	Direct Shear	100 x 300	Hot wire	NA	1.0 mm/min	Atmatzidis et al., (2001)
15	$\sigma_n \leq 31$ $\sigma_n > 31$	δ = 36° τ_{max} = 22 kPa						
20	$\sigma n \leq 29$ $\sigma n > 29$	δ = 40° τ_{max} = 26 kPa						
30	$\sigma_n \leq 75$ $\sigma_n > 75$	δ = 43° τ_{max} = 77 kPa						
NA	NA	*δ recommended = 30°*	NA	NA	NA	NA	NA	EPS Construction Development Org. (1993)
NA	NA	*δ recommended = 35°*	NA	NA	NA	NA	NA	Norwegian Research Laboratory (1992)
NA	NA	*δ recommended = 27°*	NA	NA	NA	NA	NA	Sanders and Seedhouse (1994)

Note: NA = Not Available, σ_n = normal stress, δ = interface friction angle, τ_{max} = maximum shear stress, w = with, w.o. = without

Because of variations in specimen dimensions, shear displacement rate, roughness of the EPS surfaces, and other factors, a range in EPS/EPS interface friction angles has been reported. For example, reported peak shear strength values, δ_{Peak}, range from 32 degrees to 48 degrees, and residual shear strength values, $\delta_{Residual}$, range from 27 degrees to 35 degrees. These ranges are based on normal stresses ranging from 10 to 80 kPa. Unfortunately, the stress range corresponding to the residual values is not included in the literature. Therefore, the value of δ = 30 degrees recommended as part of the NCHRP Project 24-11(01) study (Stark et al., 2004a) still appears reasonable for preliminary design. Barrett and Valsangkar (2009) also concluded that the NCHRP recommended value is appropriate for design purposes, albeit slightly conservative.

Because of a lack of uniformity in test procedures relating to the interface shear resistance of EPS-block geofoam, a summary of conclusions reached by various researchers about the interface resistance between EPS interfaces is provided below.

- **Density.** The density of EPS geofoam blocks does not appear to have a significant impact on the values of δ_{Peak} or $\delta_{Residual}$ (Atmatzidis et al., 2001, Negussey et al., 2001, Sheeley, 2000). The value of δ is independent of EPS density because shearing occurs on the surface of the specimen, and normal stress is assumed to be low enough that excessive deformation of the EPS did not occur during prior testing (EPS Construction Method Development Organization, 1993). However, Barrett and Valsangkar (2009) indicate that maximum shear resistance measured on EPS blocks is between 6 to 30 percent greater for blocks with a density of 30 kg/m^3 compared to the lower density block of 15 kg/m^3.
- **Moisture.** Values of δ_{Peak} may be decreased by roughly 10 percent with the presence of moisture at the interface between EPS blocks. Values of $\delta_{Residual}$ may be decreased by roughly 2 percent by the presence of moisture at the interface (Negussey et al., 2001; Sheeley, 2000). The duration of exposure to moisture does not appear to have a significant effect on either δ value (Sheeley, 2000). This conclusion is valid only with reference to liquid water at the block interface. In practice, there is a possibility that any moisture trapped between EPS geofoam blocks could freeze during a period of prolonged cold weather, thus potentially reducing the shear resistance at the EPS block interface because of sliding along the ice.
- **Sample Size.** The effects of laboratory specimen size does not appear to have a significant effect on either δ_{Peak} or $\delta_{Residual}$ (Sheeley, 2000).
- **Bead Type.** The type of resin used in the manufacturing process of the EPS block has more impact on interface friction than the density of EPS block (Negussey et al., 2001). However, the overall importance of bead type in determining the interface shear resistance of EPS-block geofoam is not clear (Negussey et al., 2001). No mention is made as to the importance of the size of the resin beads during the molding process as it relates to interface shear resistance.
- **Block Surface.** The value of δ_{Peak} for interfaces between EPS blocks having manufactured skin surfaces is slightly less than for blocks having hot wire cut surfaces. The values of $\delta_{Residual}$ are not significantly affected (Negussey et al., 2001). This may be significant because trimming laboratory samples using a hot wire cutter appears to be a common practice.

One new finding suggested by Atmatzidis et al. (2001) is the relationship between shear stress, τ, and normal stress, σ_n, for interfaces involving EPS blocks is best approximated using a bi-linear failure envelope with an initial segment indicating purely frictional behavior, and a secondary segment indicating a purely adhesive behavior, as shown in Figure 3.3 (Atmatzidis et al., 2001). This adhesional behavior is not technically a true adhesion, it is merely a maximum value for the shear resistance beyond which an increase in the applied normal stress ceases to produce a corresponding increase in the shear resistance. The term “adhesion” is used by Atmatzidis et al. (2001) merely as a descriptor rather than an actual technical explanation of the interface shear behavior. As indicated by the Atmatzidis et al. (2001) test data summary in Table 3.1, the transition normal stress for frictional and adhesive behavior increases with

increasing EPS density. Atmatzidis et al. (2001) indicate that a transitional segment of combined frictional and adhesive behavior may exist, but further research is needed to adequately describe the material behavior in this transitional range.

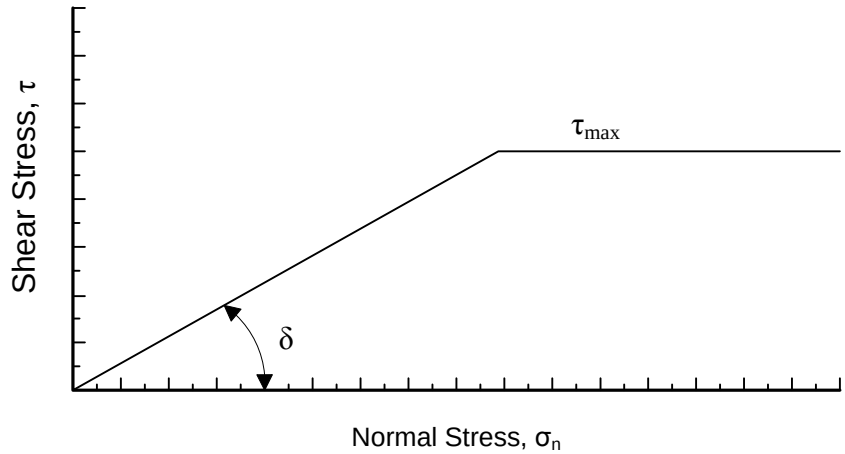

Figure 3.3 Conceptual bi-linear failure envelope for EPS-block geofoam interface shear resistance.

EPS/Dissimilar Material Interface

The interface friction between EPS blocks and a dissimilar material has also been studied by a number of researchers (Arellano, 2005, Atmatzidis et al., 2001, Bartlett et al., 2000, Jutkofsky et al., 2000, Negussey et al., 2001, Refsdal, 1987, Sheeley, 2000, Xenaki and Athanasopoulos, 2001). A summary of EPS/dissimilar interface strength results is included in Table 3.2. As shown in Table 3.2, the primary interface types, other than EPS/EPS typically encountered in EPS-block geofoam embankments include EPS/soil, EPS/concrete, and EPS/geosynthetics.

Two locations within the EPS-block fill mass where these dissimilar materials may be present include a separation layer between the pavement system and EPS block, and a separation layer between the EPS block and natural foundation material. Materials that are sometimes utilized between the pavement system and EPS block include a geotextile, geomembrane, a Portland cement concrete (PCC) slab, geogrid, geocell with soil or PCC fill, soil cement, and pozzolanic stabilized materials. Materials that are sometimes utilized between the EPS block and natural foundation soil include granular material such as sand and geotextiles to promote drainage.

Obviously, shear resistance at the interface between EPS-block geofoam and soil depends heavily on the type of soil in question. Testing has been performed to determine interface shear resistance between EPS blocks and clay, sand, and gravel (Atmatzidis et al., 2001). However, because most slope and embankment designs typically incorporate a layer of sand beneath EPS fill for drainage and leveling, the majority of research devoted to determining shear resistance between EPS block and soil has been focused on various types of sands. The shear resistance between EPS and sand is controlled by the number of contact points between sand grains and EPS blocks, and by the degree of penetration into EPS by the sand grains (Xenaki and Athanasopoulos, 2001). Testing has demonstrated that the shape and grain size distribution of the sand have a significant influence on the interface shear resistance (Xenaki and Athanasopoulos, 2001). Xenaki and Athanasopoulos (2001) stated that decreasing D_{50} of the sand sample from 2.17mm to 0.28mm caused a 20-50 percent increase in shear resistance. The void ratio of the sand was found to have no significant impact on interface shear resistance (Xenaki and Athanasopoulos, 2001).

The influence of EPS density on interface resistance of EPS/soil interfaces is not as well understood. Xenaki and Athanasopoulos (2001) conclude that density does have a significant effect on shear resistance. This conclusion seems to be in agreement with the influence of sand grain size and shape on EPS/sand interface resistance explanation described above, because a higher density EPS would have a

higher elastic modulus, making it harder for the sand grains to press into the EPS surface. However, Atmatzidis et al. (2001) conclude that density does not have a significant effect on the interface shear resistance between EPS and sand.

There is also disagreement on the relative strengths between EPS/EPS interfaces and EPS/sand interfaces. Atmatzidis et al. (2001) stated that the shear resistance of an EPS/sand interface may only exceed the interface resistance between an EPS/EPS interface if the sand grains are very angular. On the other hand, Negussey et al. (2001) concluded that EPS/sand interface shear strength is greater than that of an EPS/EPS interface. Additional investigation is needed to gain a thorough understanding of these relatively complex interactions.

The other major category of shear interfaces typically encountered in an EPS-block geofoam slope or embankment is that involving other geosynthetics. Because of the extreme diversity of these materials, each specific type of geosynthetic must be considered individually. Table 3.2 can be used to obtain a preliminary estimate of the interface shear resistance of EPS/geosynthetic material interfaces for use in preliminary design. However, shear strength tests should be performed with the specific geosynthetic that will be incorporated in the EPS-block geofoam slope system. The interface resistance behavior has been expressed by Equation 3.2. However, the same bi-linear relationship between shear stress and normal stress that was observed by Atmatzidis et al. (2001) for EPS/EPS interfaces was also observed for EPS/soil and EPS/geosynthetic interfaces.

One concern expressed recently involves the use of hydrocarbon-resistant geomembranes as a protection barrier over the geofoam blocks to guard against potential fuel spills in roadway applications. Liquid petroleum hydrocarbons (gasoline, diesel fuel/heating oil) will dissolve EPS if the EPS is inundated with liquid petroleum. As indicated in the NCHRP Project 24-11(01) report (Stark et al., 2004a), based on the German national design manual (Arbeitsgruppe Erd- und Grundbau, 1995; BASF AG, 1995), concern over protecting EPS-block geofoam from petroleum spills does not appear significant nor cost effective. However, consideration may be given to performing a risk analysis by obtaining petroleum spill occurrence data from a transportation agency or the Environmental Protection Agency (EPA). The concern with use of a hydrocarbon-resistant geomembrane is that the EPS/geomembrane interface shear resistance and/or load distribution PCC slab/geomembrane interface may not be sufficient to resist seismic loads produced by some seismic events. This was one concern during the Utah I-15 project (Bartlett and Lawton, 2008). Initial specifications specified use of a hydrocarbon geomembrane, but this requirement was subsequently removed because of interface strength concerns. If a hydrocarbon-resistant geomembrane is to be considered in the design of an EPS-block fill slope system, a seismic stability analysis based on direct shear interface strength test results using the specified geomembrane should be performed to determine the feasibility of incorporating a hydrocarbon resistance geomembrane.

Use of a PCC slab is primarily associated with the need to distribute vehicle loads as part of the pavement system to below the elastic limit stress levels of available EPS block types. Additionally, a PCC slab is typically required with vertical-sided embankments to support the upper part of the exterior protective facing. A secondary function is to provide anchorage for various highway hardware, such as safety barriers, signage, and lighting. The use of PCC slabs to function as barrier against potential petroleum spills is questionable due long-term development of cracks in PCC slabs. Table 3.2 provides test results of EPS/concrete interfaces. The Japanese typically require L-shaped reinforcing bar dowels cast into the slab that penetrate down into the EPS blocks to provide additional interface resistance between the PCC slab/EPS block interface during seismic loading.

One issue that evolved during the Utah I-15 project was the impact on interface shear resistance due to EPS block exterior surfaces that had become slightly degraded due to direct sun exposure. When exposed to ultraviolet (UV) radiation from sunlight, the surface of an EPS block will turn yellow in color and become somewhat brittle and chalky. This process takes months to develop and is limited to the surface (degradation does not progress into the block) so it is only necessary to protect EPS-block geofoam from long-term UV radiation. Relatively brief exposure such as during construction is typically not a problem. However, on large projects, such as the Utah I-15 project or on projects that are built in phases such as the Woodrow Wilson bridge project, protection of EPS blocks from direct sunlight may be

required. As shown in Table 3.2, the interface resistance of UV degraded EPS/ PCC slab interfaces is less than EPS surfaces that have not been degraded by ultraviolet radiation. However, power washing can be an effective method for removing UV degraded surfaces and restore the original interface resistance (Sheeley, 2000).

Alternatives to Increase Interface Shear Strength

If the calculated shear resistance forces along the horizontal planes between EPS blocks are insufficient to resist the horizontal driving forces, additional resistance between EPS blocks is generally provided by adding interblock mechanical connectors along horizontal interfaces between the EPS blocks. Experience in the U.K. (Sanders and Seedhouse, 1994) and elsewhere suggests that mechanical connectors are not required for typical gravity and vehicle-braking loads. However, mechanical connectors may be required where seismic or other lateral loads are deemed to be significant (EPS Construction Method Development Organization, 1993). It is typical practice in Japan to supplement the interface resistance between blocks by using some mechanical connectors between blocks because extensive research in Japan during the late 1980s to early 1990s demonstrated that the inherent inter-block friction between EPS blocks is insufficient to prevent lateral shifting between blocks during a "significant" seismic event. Although the Mohr-Coulomb interface friction angle, δ, for EPS/EPS interface sliding is comparable to that of sand ($\varphi \sim 30$ degrees), the shear resistance, $\sigma'_n * \tan \varphi$ is generally small in magnitude because the effective vertical normal stress, σ'_n, is relatively small. Consequently, EPS/EPS interface resistance may be insufficient to resist significant driving forces that result from seismic shaking.

The most common type of mechanical connector used in the U.S. is a prefabricated barbed metal plate, as shown in Figure 3.4. The use of mechanical connectors between layers of EPS blocks can be modeled by considering the horizontal interface between blocks according to the classical Mohr-Coulomb failure criterion:

$$\tau = c_a + \sigma'_n \tan \delta \qquad \textbf{(3.3)}$$

where

τ	=	Shear strength
c_a	=	Pseudo cohesion by connectors expressed as an average value per unit area
σ'_n	=	Effective vertical normal stress at the interface
δ	=	Interface friction angle of EPS/EPS

Equation 3.3 is illustrated conceptually in Figure 3.5 where it can be seen that mechanical connectors provide a pseudo cohesion to the otherwise frictional interface resistance. At the present time, all mechanical connectors available in the U.S. are of proprietary designs. Therefore, resistance provided by such connectors and placement location must be obtained from the supplier or via independent testing. For example, it is reported that each 102mm by 102mm (4 in. by 4 in.) plate exhibits a design pseudo cohesion of 267 N (60 lbs.). This resistance is based on tests performed on EPS block with a density of 16 kg/m³ (1 lbf/ft³) in accordance with ASTM C 578 and includes a factor of safety of two (AFM® Corporation, 1994). A mechanical connector such as the one shown in Figure 3.4 may not be the most efficient design for a mechanical connector. A better design may be a simple circular ring, because it provides a relatively larger contact area between the connector and EPS under horizontal loading compared to the pointed barbs in the square mechanical connector shown in Figure 3.4 (Horvath, 2001).

The effectiveness of barbed metal plate mechanical connectors, especially under reverse loading conditions, has been recently disputed. (Bartlett et al., 2000, Sanders and Seedhouse, 1994, Sheeley, 2000). Sheeley (2000) in particular indicates that when mechanical connectors are subjected to reverse loading conditions, such as those typically associated with strong ground motion during a seismic event, the connectors tend to plough a trench through the EPS whenever sliding occurs. Once this trench has been cut into the EPS, the mechanical connectors have essentially lost the value of C_a to resist sustained shaking. However, full-scale shake-table tests in Japan in the late 1990s demonstrate that while plowing of the mechanical connector barbs through the surface of EPS blocks does occur, the presence of mechanical connectors is nevertheless essential for the overall stability of an assemblage of EPS block under seismic loads. Therefore, the present state of knowledge suggests that a conservative approach is warranted so any EPS-block geofoam slope system designed for seismic loading should incorporate mechanical connectors on all horizontal surfaces between blocks, and the EPS/EPS interface shear resistance should be measured using laboratory shear tests for fills that do not utilize mechanical connectors. In addition to their role in resisting design loads, mechanical connectors have proven useful in keeping EPS blocks in place when subjected to wet, icy, or windy working conditions during construction (Horvath, 2001) and to prevent shifting under traffic loads when only a few layers of block are used (Duskov, 1994).

At present, there is no consensus on where mechanical connectors should be used. One recommended practice is to place the connectors across every horizontal joint between blocks (EPS Construction Method Development Organization, 1993; Horvath, 1995). Connectors are also used across horizontal joints on the outside face of the EPS block. Hotta et al. (2001) recommended a minimum of two metal connectors per square meter of EPS blocks if the blocks will be subjected to major seismic shaking. A minimum of two timber fasteners per block was specified by the WDOT for the SR 516 project. One supplier recommends a minimum of two plates for each 1.2m (4 ft.) by 2.4m (8 ft.) area of EPS be used per block (AFM® Corporation, 1994). The Utah Department of Transportation (UDOT) suggests a minimum of three connectors for each 1.2m by 2.4m section of geofoam material (UDOT, 2009). The UDOT technical provisions also suggest that connectors be firmly pressed into the geofoam block until the connector is flush with the surface, and that the upper block be seated firmly onto the connectors and lower block before placement of subsequent blocks.

An alternative to mechanical connectors is use of shear keys (Bartlett and Lawton, 2008). As shown in Figure 3.6, shear keys consist of half-height EPS blocks that are periodically installed within the fill mass to interrupt the horizontal joints typically present in EPS fills. A method of analyzing the number and location of shear keys is currently not available, but can be approximated by another material type in the stability analysis with the shear strength corresponding to the internal strength of the block.

Table 3.2. Summary of interface shear strength data for EPS/Dissimilar interfaces.

Interface Details	Confining Stress Range (kPa)	Reported Strength Properties	Sample Size (mm)	Shear Rate (mm/min)	References
		EPS- Soil Interface			
*EPS 20kg/m^3-Ottawa sand, (D_{50} = 0.28 & 2.17)	$\sigma_n \leq 35$ $\sigma_n > 35$	$\delta = 32°, c = 0$ $\delta = 15°, c = 13\ kPa$	100 x 100	0.4	Xenaki and Athanasopoulos (2001)
*EPS 10kg/m^3-Ottawa sand, (D_{50} = 0.28 & 2.17)	$0 < \sigma_n \leq 15$ $15 < \sigma_n \leq 30$ $\sigma_n > 30$	$\delta = 34°, c = 0$ $\delta = 19°, c = 5.5\ kPa$ $\delta = 0°, c = 16.5\ kPa$			
*EPS 20kg/m^3 -Clay	$\sigma_n \leq 25$ $\sigma_n > 25$	$\delta = 22.5°$ $\tau_{max} = 11\ kPa$	100 x 300	1.0	Atmatzidis et al. (2001)
*EPS 20kg/m^3 -Sand	$\sigma_n \leq 46$ $\sigma_n > 46$	$\delta = 38.5°$ $\tau_{max} = 41$			
*EPS 20kg/m^3 -Gravel	$\sigma_n \leq 27$ $\sigma_n > 27$	$\delta = 55.5°$ $\tau_{max} = 39\ kPa$			
Soil (Unspecified)	NA	$\delta = 27°$	NA	NA	Refsdal (1987)
Ottawa sand		$\delta = 27°$			Jutofsky et al. (2000)
Bedding sand	$25 \leq \sigma_n < 40$	$\delta = 33°$			Bartlett et al. (2000)
Sand; φSand = 35°	NA	$\delta_{Peak} \leq \varphi_{Peak\ of\ sand}$, $\delta_{Residual} = \varphi_{Residual\ of\ sand}$	100 x 100	1.5	Sheeley (2000)
*Sand	σ_n = 27.5, 45, & 70	$\delta_{Average} = 40°$			Negussey et al (2001)
		EPS - Concrete Interface			
Normal EPS, Cast-in-place PCC	NA	$\delta_{Peak} = 67°$, $\delta_{Residual} = 45°$	100 x 100	1.5	Sheeley (2000)
UV Degraded EPS, Cast-in-place PCC		$\delta_{Peak} = 41 - 67°$, $\delta_{Residual} = 35 - 45°$			

Note: Testing Method = Direct Shear unless otherwise indicated. PP = Polypropelene, HDPE = High Density Polyethylene, PVC = Polyvinyl Chloride
NA = Not Available, * = Hot Wire Cut, σ_n = normal stress, δ = interface friction angle, τ_{max} = maximum shear stress Page 1 of 2

Table 3.2 Summary of interface shear strength data for EPS/Dissimilar interfaces (cont.).

Interface Details	Confining Stress Range (kPa)	Reported Strength Properties	Sample Size (mm)	Shear Rate (mm/min)	References
		EPS- Geosynthetic Interface			
*Smooth tri-polymer alloy Geomembrane	σ_n = 12, 20, & 26	δ_{Peak} = 55°, $\delta_{Residual}$ = 43°	305 x 305	0.37	Arellano (2005)
*Smooth tri-ploymer alloy Geomembrane, Ring Shear			Dinside = 40, Doutside = 100		
*Nonwoven Geotextile		δ_{Peak} = 25°, $\delta_{Residual}$ = 18°	305 x 305		
*Nonwoven Geotextile, Ring Shear			Dinside = 40, Doutside = 100		
EPS 20kg/m^3 -Textured HDPE Geomembrane	σ_n = 24, 32 & 40	δ_{Peak} = 45°, $\delta_{Residual}$ = 45°	100 x 100	1.5	Sheeley (2000)
EPS 20kg/m^3 -Smooth HDPE Geomembrane		δ_{Peak} = 16°, $\delta_{Residual}$ = 13°			
EPS 20kg/m^3 -Textured PVC Geomembrane		δ_{Peak} = 31°, $\delta_{Residual}$ = 24°			
EPS 20kg/m^3 -Smooth PVC Geomembrane		δ_{Peak} = 35°, $\delta_{Residual}$ = 22°			
*Needle-punched nonwoven, staple fibers PP geotextile fibers	$\sigma_n \leq 40$ $\sigma_n > 40$	δ = 32°, τ_{max} = 28	100 x 300	1.0	Atmatzidis et al. (2001)
*Nonwoven continuous filament, thermally bonded PP geotextile	$\sigma_n \leq 55$ $\sigma_n > 55$	δ = 25°, τ_{max} = 26			
*Woven PP geotextile, (1.5mm tapes), negligible open area & asperities	$\sigma_n \leq 62$ $\sigma_n > 62$	δ = 21°, τ_{max} = 22			
*Woven PP geotextile, (1.9-2.3mm tapes), Negligible open area, 0.2mm, weaving asperities	$\sigma_n \leq 44$ $\sigma_n > 44$	δ = 34°, τ_{max} = 30			
*Smooth HDPE Geomembrane	$\sigma_n \leq 82$ $\sigma_n > 82$	δ = 16°, τ_{max} = 22			
*Smooth HDPE Geomembrane with 0.55mm microspikes	$\sigma_n \leq 32$ $\sigma_n > 32$	δ = 46°, τ_{max} = 32			
Smooth PP Geomembrane with 0.15 asperities (internal reinforcement)	$\sigma_n \leq 65$ $\sigma_n > 65$	δ = 23.5°, τ_{max} = 28			
*Smooth PVC Geomembrane	$\sigma_n \leq 45$ $\sigma_n > 45$	δ = 37°, τ_{max} = 32.5			

Note: Testing Method = Direct Shear unless otherwise indicated. PP = Polypropelene, HDPE = High Density Polyethylene, PVC = Polyvinyl Chloride
NA = Not Available, * = Hot Wire Cut, σ_n = normal stress, δ = interface friction angle, τ_{max} = maximum shear stress

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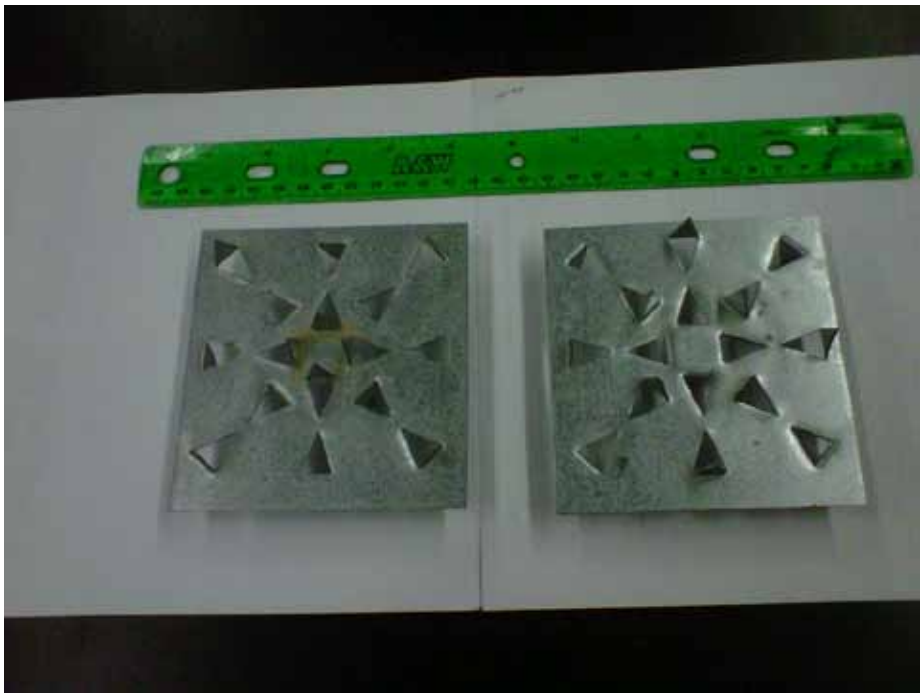


Figure 3.4. Photograph of mechanical connector plates.

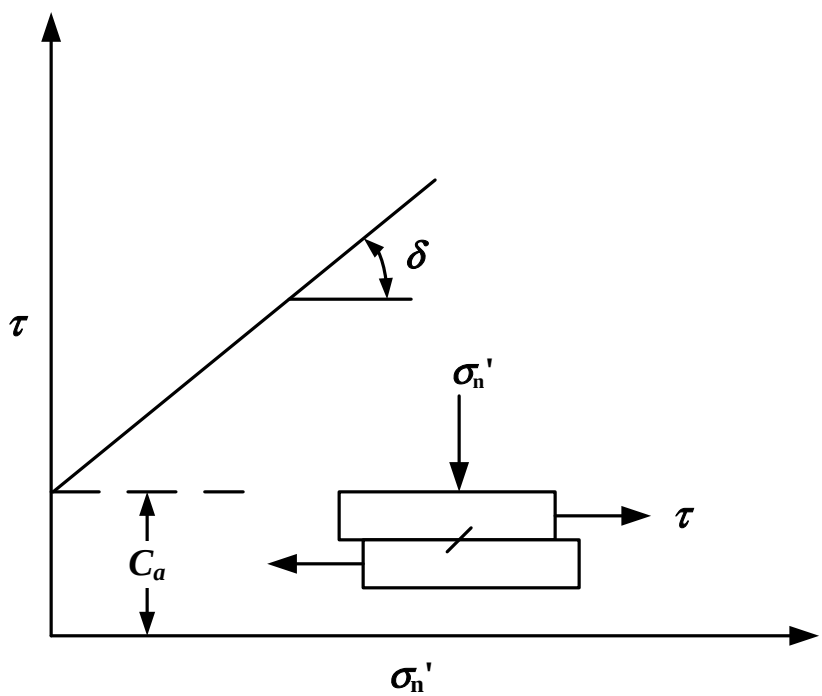


Figure 3.5 Interface shear strength of EPS blocks with mechanical connectors (Stark et al., 2004b).

Use of polyurethane adhesives used for roofing applications could be effective in providing additional shear resistance between EPS blocks (Barrett and Valsangkar, 2009). However, long-term durability testing is needed to verify that shear strength will not degrade with time. Adhesives provide a

purely adhesive (cohesive) connection so that if the adhesion is broken or lost, there will, theoretically, be zero residual resistance. Therefore, the potential loss of adhesion due to degradation of adhesives needs to be evaluated before adhesives can be incorporated in design of EPS-block geofoam slopes. The proper adhesive application process to include field verification such as surface preparation, application method and rate, and curing time, is also required before adhesives can be considered in design of EPS-block geofoam slopes.

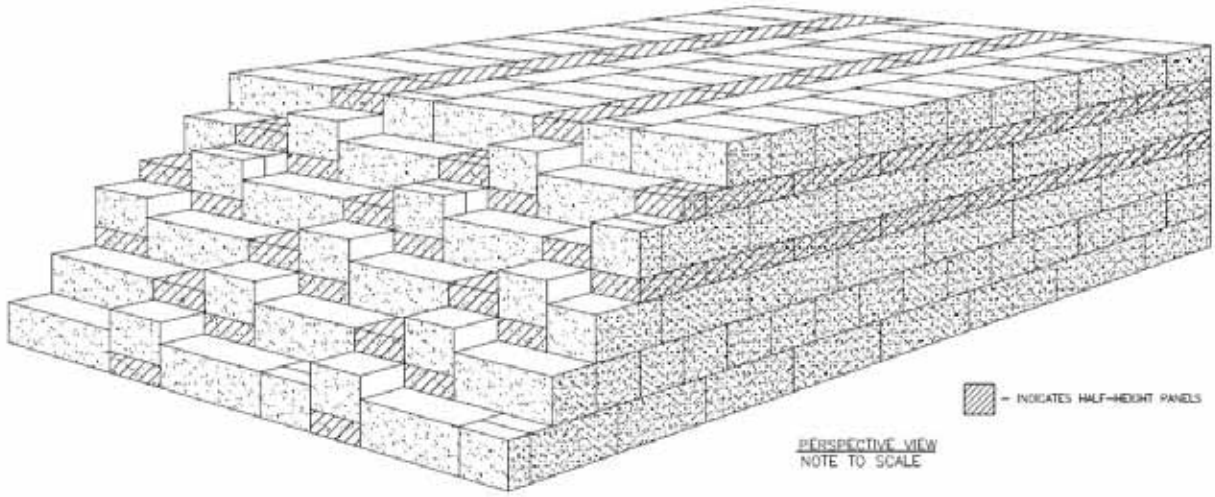


Figure 3.6 Geofoam shear key illustration (Courtesy of Insulfoam, a Carlisle Company).

INFLUENCE OF POTENTIAL WATER ABSORPTION ON EPS BLOCK DENSITY

The Japanese geofoam design manual recommends that the dead load of EPS block above ground water level be based on the specified dry density of the EPS block, while for EPS block in the vicinity or lower than ground water level be based on a density of 50 to 70 kg/m^3 (3.1 to 4.4 lb./ft^3) to account for water absorption (Public Works Research Institute, 1992). These recommended density values are based on laboratory tests performed on EPS block specimens with a density of 20 kg/m^3 (1.25 lb./ft^3).

Long-term water content tests performed in Norway on EPS blocks exhumed after more than 20 years in-ground from various project sites indicate water contents, by volume basis, in the range of 10 percent for blocks that were permanently submerged, 4 percent for blocks periodically submerged, and 1 percent for blocks located above the highest water level. (Frydenlund and Aabøe, 2001). Frydenlund and Aaboe (1994) do not provide the original block dry density values. However, they recommend that design dead loads be based on a density of 100 kg/m^3 (6.25 lb./ft^3) for submerged or semi-submerged EPS blocks, and a density of 50 kg/m^3 (3.1 lb./ft^3) for EPS blocks located above the highest water level.

Table 3.3 provides estimated EPS block densities based on the long-term water contents obtained in Norway for the standard material designations. The water content results in Table 3.3 are expressed on a volumetric basis and not on gravimetric basis which is customary for most geotechnical applications. The recommended design procedure is based on the use of a permanent drainage system. Therefore, the most applicable densities for use in determining dead loads are either the dry densities per the Japanese design recommendations or densities based on a water content of 1 percent by volume per the Norwegian test results. Although EPS-block geofoam can be manufactured to various densities, the preliminary design can be based on a density of 20 kg/m^3 (1.25 lbf/ft^3). Therefore, the dry unit weight of the EPS can be taken to be 200 N/m^3 (1.25 lbf/ft^3), or a unit weight at 1 percent water content by volume of 300 N/m^3 (1.9 lbf/ft^3) for preliminary design.

Table 3.3 Estimated EPS block densities for various water contents.

Material Designation	**Density kg/m^3 (lbf/ft^3)**			
	Minimum Allowable (*Dry*)	Permanently Submerged (*10%*)*	Periodically Submerged (*4%*)*	Above highest ground-water level (*1%*)*
EPS40	16 (1.0)	116 (7.2)	56 (3.5)	26 (1.6)
EPS50	20 (1.25)	120 (7.5)	60 (3.7)	30 (1.9)
EPS70	24 (1.5)	124 (7.7)	64 (4.0)	34 (2.1)
EPS100	32 (2.0)	132 (8.2)	72 (4.5)	42 (2.6)

* Water content by volume basis.

The use of the dry unit weight versus the unit weight at 1 percent water content by volume in design will depend on the failure mechanism being evaluated. For example, when evaluating certain failure mechanisms, such as internal load bearing of the EPS blocks, settlement, and bearing capacity failure of the foundation material, use of a higher unit weight for EPS-block geofoam would be conservative. However, this is not the case for all failure mechanisms. For example, when evaluating external slope stability, the higher unit weight would result in increased driving forces, which would make using the higher unit weight more conservative, and therefore more appropriate for design. By using a higher unit weight for the EPS, normal stresses along the slip surface are also increased, which, in turn, results in an increase in the shear strength. So the use of the unit weight at 1 percent water content by volume increases both the driving force and resisting forces in the slope analysis, making it difficult to discern which unit weight value is more conservative. For cases such as this, the best approach is to perform the analysis using both the dry unit weight and unit weight at 1 percent water absorption by volume and compare the results. The value for the unit weight that results in a lower factor of safety is the unit weight that should be used for that particular failure mechanism.

MODELING EPS BLOCK SHEAR STRENGTH

Overview

Conventional limit equilibrium methods can be used to evaluate the stability of potential slip surfaces involving EPS-block fill mass as part of external static slope stability (Step 5 in the design procedure). However, the literature search revealed some uncertainties in modeling of the shear strength of EPS blocks in slope stability analysis. Therefore, one current disadvantage of using limit equilibrium methods of analysis involving slip surfaces through the EPS blocks is the current uncertainty involved in modeling the shear strength of EPS blocks.

Much of the uncertainty surrounding modeling of EPS-block geofoam shear strength stems from the unusual material properties of EPS compared to soil. For example, an EPS block is a solid material with stress-strain behavior that is strain-hardening as well as time-dependent based on the relative stress levels within the material. The strain-hardening behavior cannot be modeled with traditional slope stability limit equilibrium methods of analysis that typically model soil using the Mohr-Coulomb shear strength model. Additionally, an EPS-block fill mass consists of discrete blocks. Overall interaction of these discrete blocks also cannot be modeled with traditional limit equilibrium methods of slope stability analysis. Finally, the issue of stress-strain incompatibility between EPS-block fill mass and underlying and adjacent natural material that can lead to progressive failure cannot be evaluated with limit equilibrium methods of analysis.

Five possible alternatives that are available to model shear strength of EPS blocks in limit equilibrium analysis of slip surfaces involving both the fill mass and existing slope material are summarized below:

1. Applying a surcharge to the surface of the foundation material that approximates the dead weight of the EPS block and any loads on top of the EPS block, such as those due to the weight of the pavement system, so the shear strength of EPS blocks does not have to be considered.
2. Modeling EPS blocks with a friction angle of one degree and a cohesion of zero so the EPS blocks do not contribute significantly to the factor of safety because of uncertainties in estimating how much shear resistance EPS blocks actually contribute in the field.
3. Assuming failure occurs between the EPS blocks, i.e., along EPS/EPS interfaces, and using an appropriate interface friction angle.
4. Assuming that failure occurs through the individual EPS blocks and using a cohesion value to represent the internal shear strength of a geofoam block.
5. Assuming that failure occurs through the individual blocks as well as between EPS blocks and using an appropriate cohesion and interface friction angle.

With the exception of the first alternative that models EPS-block geofoam fill mass as a surcharge, the primary difference between the various scenarios involves the assumption about the sliding mechanism through the EPS-block fill mass.

A summary of each of these five shear strength models is initially provided, followed by a discussion about the impact of EPS-block fill mass on groundwater and piezometric conditions that need to be considered in long-term stability analysis, is presented. The issue of progressive failure is also discussed.

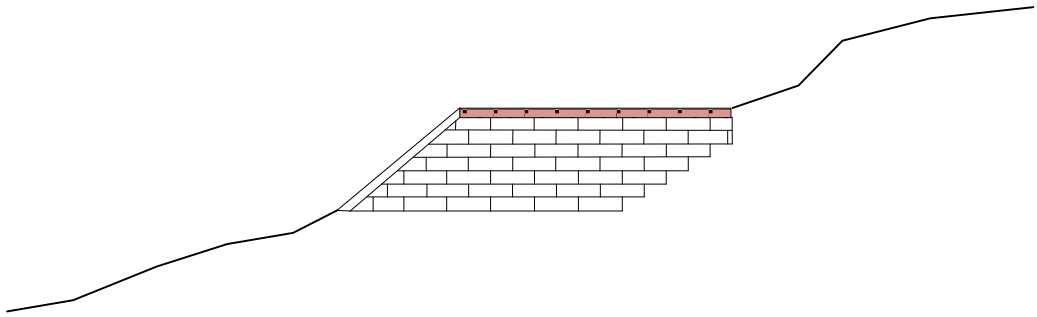
Alternative 1

Alternative 1 involves modeling the fill mass as a vertical surcharge and consists of applying a surcharge to the surface of the foundation material that approximates the weight of the fill mass, pavement system, and any additional loads applied to the pavement system so the strength of the geofoam does not have to be considered. Alternative 1 is the standard practice in Japan for stability analysis of EPS-block geofoam embankments over soft ground because no data is available about the sliding behavior of an EPS-block geofoam embankment system (Public Works Research Institute, 1992). Figure 3.7 depicts the surcharge loading recommended in the Japanese design manual.

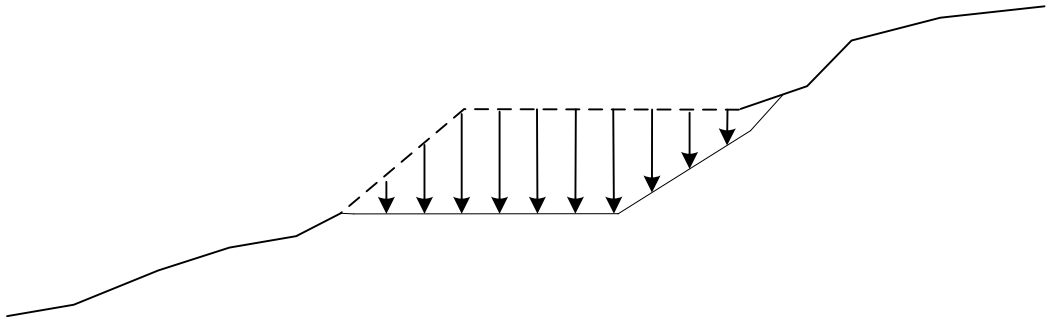
The New York State (NYDOT) used the surcharge scenario to model the shear strength of the EPS blocks in the analysis of the Route 23A slope stabilization project located in the Town of Jewett in Greene County (Jutkofsky, 1998; Jutkofsky et al., 2000). However, in a post-construction re-analysis of the slide, the geofoam was modeled using Approach 5 with a friction angle of 10 degrees and a cohesion of 400 lbs./ft^2 (Negussey, 2002). The surcharge scenario also appears to be the method of analysis used for the County Trunk Highway "A" landslide stabilization in Bayfield County, WI (Reuter, 2001).

The surcharge scenario is sometimes used to analyze stability of earth embankments consisting of a relatively strong cohesive soil overlying a weaker foundation. The cohesive embankment is modeled as a vertical surcharge that is applied to the surface of the foundation soil. The factor of safety obtained, assuming a vertical surcharge, is the same factor of safety obtained by incorporating a tension crack through the full height of the embankment. If a tension crack is assumed for the full height of the embankment, the shear strength of the embankment has no effect on the factor of safety (Duncan and Wright, 2005). However, Duncan and Wright (2005) indicate that modeling the embankment as a vertical surcharge load is not the same as assuming the embankment has no or little shear strength, i.e., Alternative 2, because the assumption of the embankment material having no or little shear strength may yield a significant horizontal thrust that is not represented by a vertical surcharge, as shown by P_f in Figure 3.8. Therefore, if the embankment is weak compared to the foundation soil, the resulting

horizontal thrust can be considered in the limit equilibrium calculations by using Alternative 2 whereby the fill is modeled as a low strength material with small or zero shear strength.



(a) Failure considering the shear strength of embankment materials.

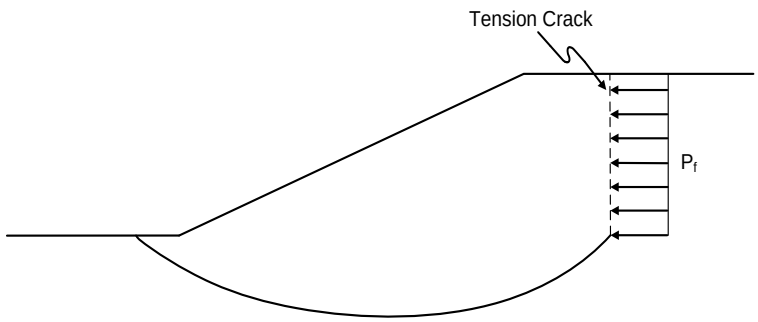


(b) Failure ignoring the shear strength of embankment materials and using a surcharge.

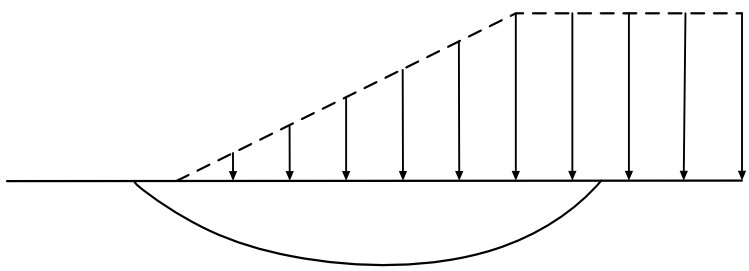
Figure 3.7 Surcharge scenario of modeling the shear strength of EPS-block geofoam fill mass.

Use of EPS-block geofoam as a lightweight fill in slope applications may not necessarily involve only soft soil, and the strength of the foundation material can be similar or greater than the strength of the EPS-block fill mass, especially if the slope material consists of soft or weathered rock. Therefore, there appears to be a need to further evaluate conditions for which modeling of EPS-block geofoam fill mass as a vertical surcharge load may or may not be applicable, and to further evaluate the behavior of EPS block overlying a stronger foundation.

The vertical surcharge model appears to be a convenient model to use if the existing or potential slip surface involves only the existing slope material and not EPS block. However, one disadvantage of modeling the fill mass as a vertical surcharge acting on the surface of the foundation slope material is this approach cannot be used for pseudo-static seismic slope stability analysis. The reason is that the seismic force must be applied at the center of gravity of the slide mass, and if the model has only a surcharge to replace the actual fill mass, then there is no center of gravity on which the seismic force may act. Thus, the surcharge approach to modeling the EPS-block geofoam fill mass is incompatible with the pseudo-static approach to seismic slope stability analysis. The pseudo-static seismic slope stability approach is presented in detail as part of Step 6 of the design procedure.



(a) Lateral thrust produced by weak fill.



(b) Vertical surcharge.

Figure 3.8. Load representations for embankments where shear strength is neglected.

Alternative 2

The second alternative involves modeling EPS blocks with a friction angle of one degree and a cohesion of zero so the EPS blocks do not contribute significantly to the factor of safety. The benefit of this scenario is that it minimizes uncertainties in estimating how much shear resistance the geofoam actually contributes in the field. As previously noted, this approach can account for the impact of horizontal thrust on stability if the foundation material is stronger than the proposed fill mass. Based on the NCHRP Project 24-11(01) study of stand-alone EPS-block geofoam embankments overlying soft foundation soil, this scenario may yield slip surfaces predominantly within the EPS-block fill mass and not through the foundation material or adjacent slope material because of the small shear strength assigned to the geofoam. As a result, this approach may not be suitable for static stability analysis because it may not result in failure through the foundation or adjacent slope material. It should be noted that because the external stability analyses performed during Project 24-11(01) study focused on soft, saturated foundation soil, circular failure surfaces through the foundation soil were assumed to be the appropriate failure mode for the stability analyses, and translational slip surfaces were not investigated unless a weak plane was found underneath the geofoam.

Alternative 3

The third alternative involves assuming failure occurs between the EPS blocks. Therefore, the analysis requires selection of an interface friction angle to represent sliding resistance along EPS/EPS interfaces. Based on the Project 24-11(01) study of stand-alone embankments overlying soft ground, this approach may also result in slip surfaces occurring predominantly through the EPS-block fill mass and not the foundation material or adjacent slope material, because the shear resistance provided by the geofoam, even with a typical EPS-EPS interface friction angle of 30 degrees, is small. The shear resistance is small because the normal stress, σ_n, applied to any failure surface passing through the EPS-

block fill mass is low due to the low unit weight of the geofoam. If the normal stress on the failure surface is low, the shear resistance, τ, is low, as shown in the following expression:

$$\tau = c + \sigma_n \tan\phi \tag{3.4}$$

where

τ	=	Shear resistance
c	=	Cohesion
σ_n	=	Normal stress
ϕ	=	Friction angle

It can be seen that the shear resistance is directly related to the applied normal stress and thus a low normal stress results in a small shear resistance. Shear resistance is further impacted because the normal stress is multiplied by the tangent of the friction angle. As a result, the impact of a high friction angle is reduced because the tangent of the friction angle is used to estimate shear resistance. In summary, modeling EPS-block fill mass using a friction angle may not result in the critical slip surface being located in the foundation or adjacent slope material, and may not be suitable for static stability analyses. If the original failure surface is still present in the native foundation material, the slope stability should force the failure surface along the observed failure surface modeled with a residual strength value to ensure the critical failure surface is identified.

Alternative 4

The fourth alternative assumes that failure occurs through individual EPS blocks. Therefore, as indicated in the internal shear strength discussion, a cohesion value is used to represent the internal shear strength of a geofoam block and the strength is independent of the normal stress.

Alternative 4 appears to be the approach used by Alabama (ALDOT) on a project involving use of EPS-block geofoam in a slope stabilization project on AL State Route 44 in the Town of Guin in Marion County. Figure 3.9 shows a cross-section of the slope prior to remediation, and Figure 3.10 shows a cross-section of the stabilized slide using EPS-block geofoam fill. As indicated in Figure 3.9, the two slip surfaces that were observed consisted of a shallow failure surface (primary slip surface) and a deep failure surface (secondary slip surface). Table 3.4 provides a summary of the material properties and Table 3.5 provides a summary of the results of slope stability analyses performed for both the primary and secondary slip surfaces. Photographs detailing the actual construction process used for this project are provided as part of the Construction Practices, Chapter 5, in Figures 5.1 through 5.26, 5.28, and 5.29.

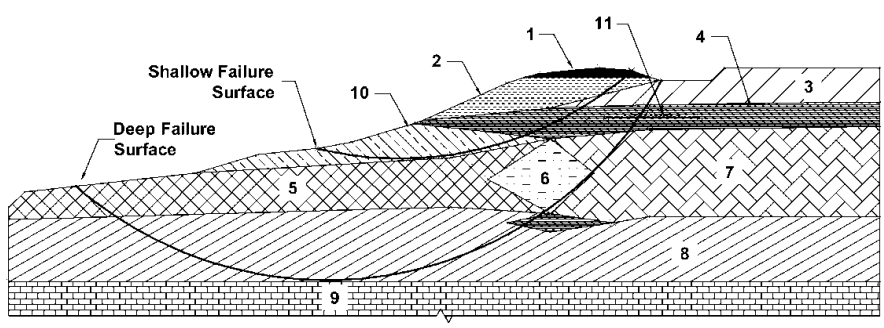


Figure 3.9. Subsurface profile of landslide from Guin, AL, case history prior to placement of EPS geofoam.

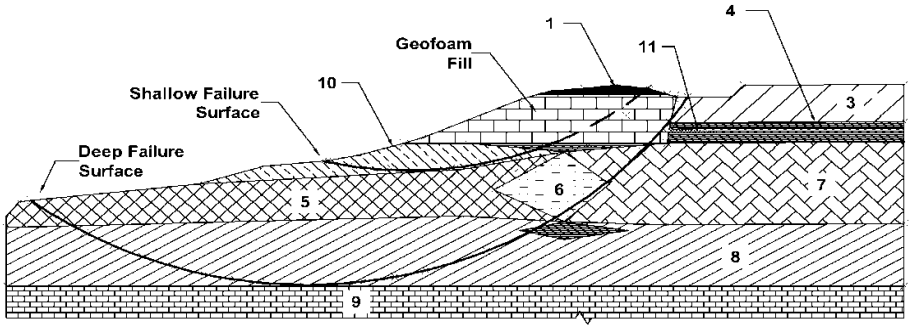


Figure 3.10 Subsurface profile of landslide from Guin, AL, case history following placement of EPS geofoam.

Table 3.4. Material properties used in Guin, AL, case history

Material #	Material Description	γ_{total} ($lb.ft^3$)	γ_{sat} ($lb.ft^3$)	c ($lb.ft^2$)	ϕ9 (deg)
	Geofoam Fill	*- See Table 3.5 -*			
1	Fill: Sandy Clay	120	130	2100	17
2	Fill: Sand-Clay-Gravel	111	125	0	30
3	Medium Moist-Wet Sandy Gravel	120	130	2900	17
4	Very Loose Wet Sand	94	115	0	29
5	Stiff Damp Silty Clay	120	130	1300	0
6	Medium Wet Silty Clay(ey Sand)	125	135	0	32
7	Hard Wet Silty Clay	133	140	4050	0
8	Very Stiff Damp Silty Clay	125	135	2000	0
9	Hard Moist Silty Clay	133	143	4000	0
10	Loose Wet Silty Gravelly Sand	111	125	0	29
11	"Hard Pan" Lens	133	143	6250	0

Table 3.5. Factors of safety for the primary & secondary landslides using various grades of EPS-block geofoam (Alabama Department of Transportation, 2004)

EPS Type	–	*EPS 40*	*EPS 50*	*EPS 70*	*EPS 100*
EPS Unit Weight	0.75 $lb./ft^3$	1.0 $lb./ft^3$	1.25 $lb./ft^3$	1.5 $lb./ft^3$	2.0 $lb./ft^3$
Strength Parameters	c = 360 $lb./ft^2$	c = 720 $lb./ft^2$	c = 936 $lb./ft^2$	c = 1080 $lb./ft^2$	c = 1800 $lb./ft^2$
F for Primary Landslide	1.16	1.16	1.16	1.17	1.17
F for Secondary Landslide	1.33	1.73	1.97	2.12	2.64

Note: "—" indicates no designation available.

As indicated in Table 3.5, although the factor of safety does not vary much with type of EPS for the deeper landslide, EPS type does influence the factor of safety for the shallower secondary landslide. As shown in Figure 3.10, the type of EPS does not have a great influence on the factor of safety for the

deeper landslide (primary landslide) because the slip surface extends through only a small portion of the EPS blocks. Therefore, only the unit weight of the EPS block has an influence on the factor of safety, and because the unit weight difference between EPS types is not great, the unit weight effect due to different EPS types is small. However, for the shallower, secondary landslide, the slip surface extends through a greater portion of the EPS blocks so the unit weight and shear strength of the block have an influence on the factor of safety. Consequently, EPS type will have a greater influence on the factor of safety for slip surfaces that extend through the EPS blocks, especially if a cohesion value is used to represent the shear strength of the blocks.

Alternative 5

Alternative 5 consists of assuming failure occurs through individual blocks as well as between EPS blocks and using an appropriate cohesion and interface friction angle. Although an EPS/EPS interface friction angle and a cohesion value can be estimated as discussed in Alternatives 3 and 4, respectively, such an estimate may not yield reasonable external static slope stability results because EPS-block geofoam is not continuous, thus the effect of joints or discontinuities between blocks may need to be considered to estimate a shear strength for EPS-block fill mass. Alternative 5 appears to have been the approach used by Negussey (2002) in a post-construction reanalysis of the NYDOT Route 23A slope stabilization project located in the Town of Jewett in Greene County. The slope stability reanalysis is based on a friction angle of 10 degrees and a cohesion of 19 kPa (400 lbs./ft^2) (Negussey, 2002).

Groundwater Considerations

Based on current design precedent, it is recommended that all EPS-block geofoam slope systems incorporate drainage systems to prevent water from accumulating above the bottom of EPS blocks and divert seepage water from adjacent upper slope material. Key groundwater issues related to stability of a EPS-block geofoam slope system is how to determine the impact of the drainage system on long-term groundwater conditions, and how to include the resulting piezometric conditions in slope stability analysis. The long-term groundwater regime can be obtained by performing a flow analysis based on a drainage system located below the EPS-block fill mass, and adjacent to the fill mass between the fill mass and upper slope material.

After the groundwater regime is determined the piezometric conditions need to be included in the slope stability analysis. The method of incorporating the piezometric conditions will be partially dependent on the model used to represent shear strength of EPS blocks in limit equilibrium analysis. For example, if Alternative 1, 2, 3, or 5 is used to model shear strength of the blocks, it is possible to perform a slope stability analysis of the EPS-block fill mass system using an effective-stress approach. However, if Alternative 4 is used to model shear strength of the blocks, it may be better to use a total-stress approach for EPS-block fill mass with boundary water pressures and an effective-stress approach for the surrounding natural material. However, it may not be possible to perform a dual total-stress and effective-stress slope stability analysis with currently available slope stability software. It is recommended that the issue of incorporating piezometric conditions be further evaluated as part of any research performed to develop an appropriate shear strength model.

Strain Incompatibility (Progressive Failure)

Figure 3.11 shows a schematic of stress-strain relationships for a compacted fill embankment and a soft foundation soil. It can be seen that failure through the compacted embankment results in a brittle failure and a post-peak strength loss at a small strain, while the foundation soil exhibits a plastic failure and a peak shear strength at a large strain. Therefore, if the strains mobilized in the embankment and foundation are equal, failure would occur through the embankment when only a fraction of the foundation

strength has been mobilized. Conversely, after the peak strength of the foundation soil has been mobilized, the strength of the embankment would correspond to a post-peak value. Thus, the peak strength of the compacted embankment should not be used in conjunction with the peak strength of the foundation soil to prevent progressive failure of the embankment. Progressive failure can occur when one material fails, e.g., the embankment, and the stresses that were being resisted by that material are transferred to another material, e.g., the foundation soil, which can result in overstressing of this material, especially if it does not mobilize its peak strength at the same strain as the failed material.

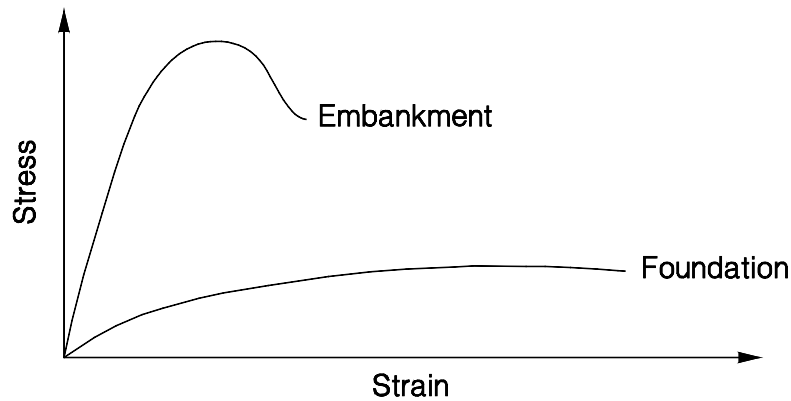


Figure 3.11. Typical stress-strain behaviors of compacted fill embankment and soft foundation soil (Chirapuntu and Duncan, 1975).

Therefore, the main EPS-block geofoam issue is determination of shear strength of the geofoam and foundation material as well as the geofoam and adjacent natural slope material that can be relied on, because the stress-strain behavior between EPS blocks and natural slope material may not be compatible. For the case of EPS-block geofoam in slope applications, further study is needed to evaluate the behavior of EPS-blocks overlying both a weaker and stronger soil and rock foundation material.

SUMMARY

In summary, uncertainty currently exists in modeling of shear strength of EPS blocks for external static slope stability. Therefore, further research using numerical modeling, physical testing, and/or observation of full-scale structures needs to be conducted to determine whether an external slope stability failure induces failure through individual EPS blocks or whether the blocks remain intact and displace as individual elements as a result of slope instability. This is important for modeling of EPS-block geofoam fill mass in a slope stability analysis. An appropriate shear-strength model may also be required to analyze the stability of embankments that may be widened with EPS block because the use of EPS block for lightweight fill function in embankment widening cases is similar to incorporating EPS block in slope applications. Thus, an accurate model for expressing the shear strength of EPS blocks is needed to ensure a safe and economical design, regardless of the condition of the surrounding natural materials.

Interface friction, primarily along horizontal surfaces, is an important consideration in external and internal stability assessments under horizontal loads such as slopes and seismic shaking. Tables 3.1 and 3.2 provide a summary of interface shear strength data for EPS/EPS interfaces and EPS/dissimilar material interfaces, respectively, which are the two types of interfaces that are of interest for EPS-block geofoam in lightweight fill applications.

If the calculated shear resistance along the horizontal planes between EPS blocks are insufficient to resist horizontal driving forces, additional resistance between EPS blocks is generally provided by adding interblock mechanical connectors along the horizontal interfaces between EPS blocks, or use of shear keys.

Use of polyurethane adhesives, used for roofing applications, could be effective in providing additional shear resistance between EPS blocks in the future once long-term durability testing is available that indicates that the shear strength will not degrade with time.

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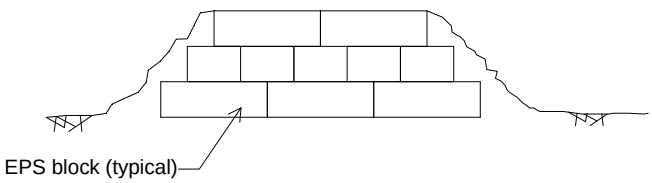
CHAPTER 4

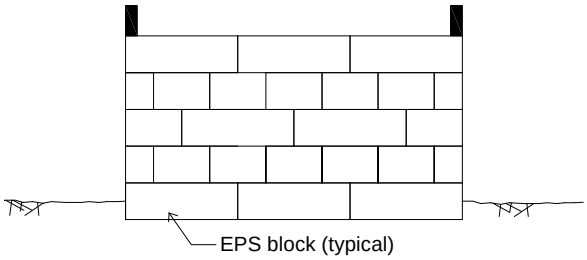
DESIGN METHODOLOGY

INTRODUCTION

This section presents background information on design methodology incorporated in abbreviated form in the recommended design guideline included in Appendix B. The recommended design guideline included in the NCHRP Project 24-11(01) reports is limited to stand-alone embankments that have a transverse (cross-sectional) geometry such that the two sides are more or less of equal height as shown conceptually in Figure 4.1. Slope stability applications (sometimes referred to as side-hill fills) are shown in Figure 4.2. As shown in Figure 4.2, use of EPS-block geofoam in slope applications can involve a slope-sided fill (Figure 4.2a) or a vertical-sided fill (Figure 4.2b). The latter application is sometimes referred to as a geofoam wall and this application is unique to EPS-block geofoam. Use of a vertical-sided fill will minimize the amount of right-of-way needed and impact of fill loads on nearby structures. For vertical-sided embankment walls, the exposed sides should be covered with a facing. The facing does not have to provide any structural capacity to retain the blocks because the blocks are self-stable, so the primary function is to protect the blocks from environmental factors.

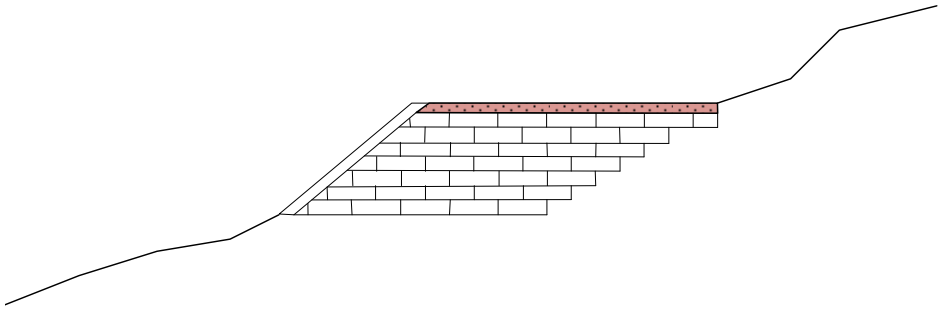


a) Slope-sided fill.

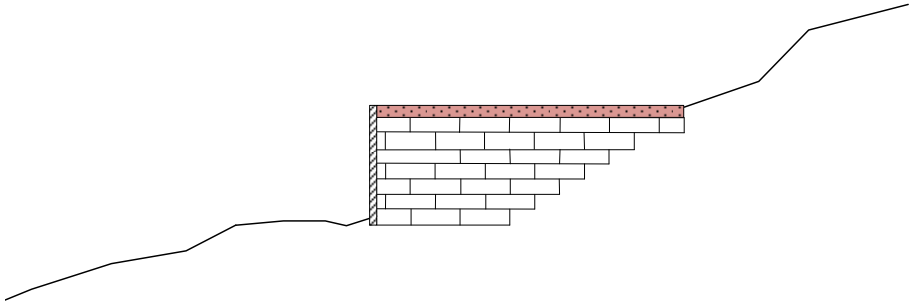


b) Vertical-sided fill (Geofoam wall).

Figure 4.1. Typical EPS-block geofoam applications involving stand-alone embankments (Horvath, 1995; Stark et al., 2004a).



a) Slope-sided fill.



b) Vertical-sided fill (Geofoam wall).

Figure 4.2. Typical EPS-block geofoam applications involving side-hill fills.

As shown in Table 4.1, approaches available for slope stabilization and repair can be categorized into the following three categories: avoid the problem, reduce the driving forces tending to cause slope movement, or increase the resisting forces resisting movement (Transportation Research Board, 1996). Use of lightweight fill is a slope stabilization procedure that can be used to reduce the weight of the sliding mass and thereby reduce the driving forces of the sliding mass. The recommended design methodology introduced in this chapter and included in the design guideline in Appendix B focuses on use of EPS-block geofoam as a lightweight fill material for slope stabilization and repair.

A review of current slope stability and landslide remediation textbooks (Abramson et al., 2002, Cornforth, 2005, Duncan and Wright, 2005, Transportation Research Board, 1996) revealed a lack of formal design guidelines to design slopes or remediate slides by reducing the weight of the slide mass using lightweight fill. Although a comprehensive design procedure is not available, some of the literature does provide general design guidance for use of geofoam in slope stability applications (Horvath, 1995, Negussey, 2002, Tsukamoto, 1996) and for use of lightly-cemented rubber tires (Lee et al., 2002).

Table 4.1. Slope stabilization and repair approaches (Transportation Research Board, 1996).

CATEGORY	PROCEDURE
Avoid Problem	-Relocate facility
Reduce driving forces	-Completely or partially remove unstable materials -Install bridge -Change line or grade -Drain surface -Drain subsurface -Reduce weight
Increase resisting forces by	**Applying external force** -Use buttress and counter-weight fills; toe berms -Use structural systems -Install anchors **Increasing internal strength** -Drain subsurface -Use reinforced backfill -Install in situ reinforcement -Use biotechnical stabilization -Treat chemically -Use electro osmosis -Treat thermally

Specific treatment of the use of EPS-block geofoam for slope stabilization work involved work done in Japan, largely in the mid-1980's to the mid-1990's time frame, with much of that work being discussed in various papers included in proceedings of the 1996 International Symposium on EPS held in Tokyo, Japan (EDO, 1996). The Japanese design procedure for use of EPS for slope stabilization, which is shown in Figure 4.3, includes many of the steps included in the NCHRP Project 24-11(01) recommended design guideline for stand-alone EPS-block geofoam embankments over soft soil. Therefore, the Project 24-11(01) recommended design procedure was used as the preliminary basis for the slope design guideline and was modified to incorporate slope design considerations. Although Tsukamoto (1996) introduced the design steps shown in Figure 4.3, he did not provide guidelines or procedures to perform these steps. Therefore, one challenge of this NCHRP Project 24-11 (02) study was to develop analysis procedures to perform the design steps.

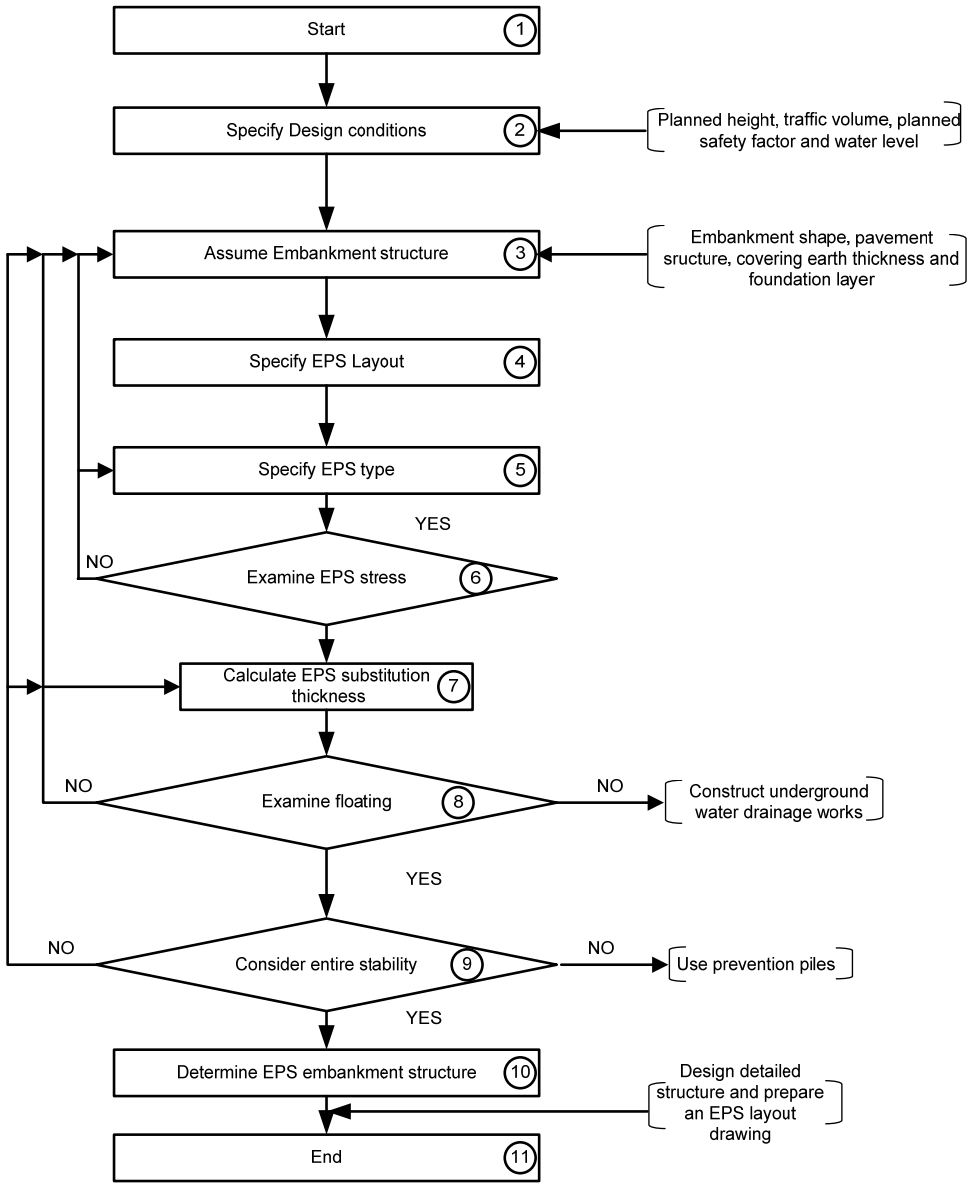


Figure 4.3. Planning procedure for EPS-block geofoam embankment at Japanese landslide sites (Tsukamoto, 1996).

Limiting conditions that must be considered in the design and analysis of geofoam slope systems and the two design approaches that can be used in design, i.e., Service Load Design (SLD) and Load and Resistance Factor Design (LRFD), are initially presented followed by a summary of the major components of an EPS-block geofoam slope system. It should be noted that Allowable Stress Design (ASD) and Service Load Design (SLD) are essentially the same design procedure (AASHTO, 1996). The AASHTO manual generally refers to the method as SLD, so this designation is used in the subsequent discussion; however, for the sake of clarity, it is important to point out that the two terms represent essentially the same thing. The recommended design procedure is presented by initially introducing the three primary failure modes, i.e., external instability, internal instability, and pavement system failure, and the design loads that need to be considered in the design. A general overview of the recommended design procedure is then provided, as well as a summary of each design step.

LIMIT STATES

The term "failure" as used in the recommended design guideline is a *loss of function*. This is the same definition incorporated in the Project 24-11(01) design guideline for stand-alone embankments over soft ground. Failure or loss of function of an EPS-block geofoam slope system may occur as either a *collapse failure* (the *ultimate* or *strength limit state*, ULS) or a s*erviceability failure* (the *service limit state*, SLS). Therefore analysis and design of geofoam slope systems must consider these two limiting conditions.

A geofoam slope system may undergo a ULS failure if the applied loads produce stresses that exceed the resistances provided by the whole geofoam slope system or any of its individual components. As shown in Table 4.2, ULS failure can occur as an external and internal failure mode. A geofoam slope system may undergo an external collapse failure as part of slope instability due to either static and/or seismic loads, sliding and overturning of the entire EPS-block geofoam mass due to seismic loads, and bearing capacity failure of the foundation material due to static and/or seismic loads. An internal ULS failure can occur as part of horizontal sliding between layers of blocks and/or between the pavement system and upper layer of blocks due to seismic induced loads.

A geofoam slope system may undergo an external serviceability failure if excessive total or differential deformation of the foundation soil develops over time due to static and/or seismic loads. An internal serviceability failure may occur if excessive vertical deformation of EPS blocks results from excessive initial (immediate) deformations under dead or gravity loads from the overlying pavement system, excessive long-term (for the design life of the fill) creep deformations under the same gravity loads, and/or excessive non-elastic or irreversible deformations under repetitive traffic loads. This type of failure associated with excessive vertical deformation of EPS blocks is also referred to as load-bearing capacity failure of EPS blocks. A load-bearing failure can also occur due to seismic-induced loads.

The geofoam slope system may also undergo a serviceability failure if premature failure of the pavement system occurs. Premature failure of the pavement system may include an uneven and often cracked pavement surface that may require frequent repaving, and possibly other maintenance.

The overall design objective for minimizing the potential against a collapse failure is to ensure that the resistance of the EPS-block slope system against failure exceeds the loads producing failure. Therefore, the ULS analysis must satisfy the following equation:

$$\text{ULS:} \quad \begin{array}{l}\text{resistance of EPS-block geofoam slope system to failure} > \\ \text{EPS-block geofoam slope system loads producing failure}\end{array} \qquad \textbf{(4.1)}$$

The overall design objective for minimizing the potential against a serviceability failure is to ensure that the estimated deformation of the EPS-block geofoam slope system does not exceed the maximum acceptable deformation. Therefore, the SLS analysis must satisfy the following equation:

$$\text{SLS:} \quad \begin{array}{l}\text{estimated deformation of EPS-block geofoam slope system} \leq \\ \text{maximum acceptable deformation}\end{array} \qquad \textbf{(4.2)}$$

The two primary approaches that are available to evaluate Equations 4.1 and 4.2 include SLD and LRFD. A summary of these two design approaches is subsequently presented.

Table 4.2. Summary of failure modes and mechanisms incorporated in the proposed design procedure for EPS-block geofoam as a lightweight fill in slope stability applications

FAILURE MODE	LIMIT STATE	FAILURE MECHANISM	ACCOUNTS FOR
External Instability	ULS	Static slope stability	Global stability involving a deep-seated slip surface and slip surfaces involving the existing slope material only (Figure 4.5). Also considers slip surfaces that involve both the fill mass and existing slope material (Figure 4.6).
	ULS	Seismic slope stability	Same as for static slope stability but considers seismic induced loads.
	SLS	Seismic settlement	Earthquake induced settlement due to compression of the existing foundation material (Figure 4.10) such as those resulting from liquefaction, seismic-induced slope movement, regional tectonic surface effects, foundation soil compression due to cyclic soil densification, and increase due to dynamic loads caused by rocking of the fill mass (Day, 2002).
	ULS	Seismic bearing capacity	Bearing capacity failure of the existing foundation earth material (Figure 4.9) due to seismic loading and, potentially, a decrease in the shear strength of the foundation material.
	ULS	Seismic sliding	Sliding of entire EPS-block geofoam fill mass (Figure 4.7) due to seismic induced loads.
	ULS	Seismic overturning	Overturning of the entire embankment at interface between the bottom of the assemblage of EPS blocks and underlying foundation material as a result of seismic forces (Figure 4.8).
	SLS	Settlement	Excessive and/or differential settlement from vertical and lateral deformations of underlying foundation soil (Figure 4.10).
	ULS	Bearing capacity	Bearing capacity failure of the existing foundation earth material (Figure 4.9) resulting in downward vertical movement of the entire fill mass into the foundation soil.
Internal Instability	ULS	Seismic sliding	Horizontal sliding between layers of blocks and/or between the pavement system and upper layer of blocks (Figure 4.12) due to seismic induced loads.
	SLS	Seismic load bearing (seismic rocking)	Excessive vertical deformation of EPS blocks (Figure 4.15) due to increase in vertical normal stress within EPS-block fill mass (Figure 4.13) due to the moment produced by seismic induced inertia force (Figure 4.14).
	SLS	Load bearing	Excessive vertical deformation of EPS blocks (Figure 4.15) due to excessive initial (immediate) deformations under dead or gravity loads from overlying pavement system, excessive long-term (for the design life of the fill) creep deformations under the same gravity loads, and/or excessive non-elastic or irreversible deformations under repetitive traffic loads.
Pavement System Failure	SLS	Flexible or rigid pavement	Premature failure of the pavement system (Figure 4.16), as well as to minimize the potential for differential icing (a potential safety hazard). Providing sufficient support, either by direct embedment or structural anchorage, for any road hardware (guardrails, barriers, median dividers, lighting, signage and utilities).

SLS = serviceability limit state
ULS = ultimate limit state

DESIGN APPROACHES

Introduction

Service Load Design (SLD) has been the traditional design approach in geotechnical engineering in the U.S. However, as of October 1, 2007, State Departments of Transportation are required to design substructures of bridges including shallow and deep foundations, earth retaining structures, and buried structures using the AASHTO Load and Resistance Factor Design (LRFD) methodology. This mandate also involves evaluation of overall stability of earth slopes with or without a foundation unit (AASHTO, 2007). Therefore, the literature search performed as part of this study attempted to evaluate information that would contribute to development of a procedure for design of EPS-block geofoam slopes based on both the SLD and LRFD approaches. A brief introduction of these two design approaches is provided.

Service Load Design (SLD)

In the traditional SLD approach, the "safety" or *factor of safety, F*, is defined as the extent that the resistances of a structure exceeds the applied loads and can be determined using Equation 4.3:

$$F = \frac{\sum \text{resistances to failure}}{\sum \text{applied loads producing failure}} = \frac{R}{P} \tag{4.3}$$

F obtained from Equation 4.3 is applicable for design of foundations and retaining structures. However, for the evaluation of slopes, *F* is defined with respect to the shear strength of the soil or:

$$F = \frac{\text{available shear strength of the soil}}{\text{mobilized shear stress}} = \frac{s}{\tau} \tag{4.4}$$

Rearranging Equation 4.4 results in:

$$\tau = \frac{s}{F} \tag{4.5}$$

As indicated by Equation 4.5, *F* represents the factor by which shear strength must be reduced so the reduced strength is just in equilibrium with the mobilized shear stress. Slope stability analyses methods based on Equation 4.5 are known as limit equilibrium procedures (Duncan and Wright, 2005).

In terms of a minimum allowable *F* required for design, F_{req}, such as the U.S. Army Corps of Engineer's required factor of safety of 1.5 for long-term conditions for slopes of dams, levees, and dikes (2003), Equation 4.4 becomes:

$$\mathrm{F_{req}} \leq \frac{s}{\tau} \tag{4.6}$$

The Japanese design procedure for use of EPS for slope stabilization, which is included in Figure 4.3, is based on the SLD procedure. As indicated previously, the Japanese design procedure includes many of the steps included in the NCHRP Project 24-11(01) recommended design guideline for stand-alone EPS-block geofoam embankments over soft soil. Therefore, a recommended design procedure for EPS-block geofoam slopes based on SLD was incorporated in the proposed design guideline included in Appendix B.

Load and Resistance Factor Design (LRFD)

As indicated in the AASHTO LRFD Bridge Design Specifications (2007), in LRFD the limit state must satisfy the following equation:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_i \tag{4.7}$$

where

η_i	=	load modifier: a factor relating to ductility, redundancy, and operational importance
	=	1.00 for the service limit state
γ_i	=	load factor: a statistically based multiplier applied to force effects
Q_i	=	force effect
ϕ	=	resistance factor: a statistically based multiplier applied to nominal resistance
R_n	=	nominal resistance
R_i	=	factored resistance = ϕR_n

For evaluation of slopes, the force effect, Q_i, can be determined from the loads imposed on a slip surface. These loads can typically be obtained by estimating vertical stress from the soil or rock material on the slip surface and any permanent loads on the slope. AASHTO LRFD specifications indicate that the evaluation of overall stability of earth slopes shall be investigated at the service limit state based on the Service I load combination and an appropriate resistance factor (AASHTO, 2007). Therefore, the load modifier to be used for evaluation of slope stability is 1.00. Equation 4.7 in terms of the relationship between the available shear strength and mobilized shear stress is:

$$\gamma_i \tau \leq \phi s \tag{4.8}$$

The AASHTO LRFD specification recommends the resistance factors shown in Table 4.3 "in lieu of better information." Also shown in Table 4.3 are the equivalent SLD factor of safety values. Note that these values are factors typically utilized in design of earth slopes based on SLD. AASHTO recommended resistance factors appear to have been derived by calibrating the LRFD resistance factor with the commonly accepted factors of safety used in SLD of earth slopes as demonstrated below. Rearranging Equation 4.8 results in:

$$\frac{\gamma_i}{\phi} \leq \frac{s}{\tau} \tag{4.9}$$

By combining Equations 4.6 and 4.9, we get a relationship between the LRFD relationship and the required factor of safety:

$$\frac{\gamma_i}{\phi} \leq \frac{s}{\tau} = \mathrm{F_{req}} \tag{4.10}$$

or

$$\frac{\gamma_i}{\phi} \leq \mathrm{F_{req}} \tag{4.11}$$

The load factor for the predominant Service I loadings that are typically considered in slope stability analyses, i.e., vertical stress from the dead load of the soil and permanent dead loads on the slope, is 1.00 (AASHTO, 2007). Therefore, a relationship between the resistance factor and minimum required factor of safety can be obtained as:

$$\phi = \frac{1}{F_{req}} \tag{4.12}$$

For a F_{req} of 1.5, ϕ is 0.67, or 0.65 if ϕ is rounded to the nearest 0.05. The rounded value of 0.65 agrees with the AASHTO ϕ for an F of 1.5 as shown in Table 4.3. Therefore, AASHTO recommended resistance factors included in Table 4.3 appear to have been derived by calibrating the resistance factor with the commonly accepted factors of safety used in SLD of earth slopes.

The approach demonstrated above of determining resistance factors based on SLD factors of safety is referred to as calibration of resistance factors by matching historical design procedures (Loehr et al., 2005). This approach, which consists of determining resistance factors that result in similar designs as current SLD methods, does not provide consistent levels of safety and reliability that are required to perform a design based on LRFD. However, resistance factors based on probabilistic calibrations are currently not available for earth slopes. Therefore, the overall objective of LRFD design of producing appropriate and consistent levels of safety and reliability regardless of the uncertainty in the input parameters cannot be currently fully achieved (Loehr et al., 2005). Although some work on developing resistance factors for regional soil conditions based on probabilistic calibrations has been started (Arellano and Anderkin, 2008; Loehr et al., 2005), one slope stability issue that has not been resolved yet is the inconsistency that occurs in limit equilibrium slope stability analysis when load factors are incorporated in an LRFD analysis.

Table 4.3. AASHTO recommended resistance factors for evaluation of overall stability of earth slopes and corresponding equivalent ASD factor of safety.

Slope Condition	Resistance Factor, ϕ **(AASHTO, 2007)**	**Equivalent Factor of Safety for Allowable Stress Design, *F***
Where geotechnical parameters are well defined and the slope does not support or contain a structural element.	**0.75**	**1.3**
Where geotechnical parameters are based on limited information or the slope contains or supports a structural element.	**0.65**	**1.5**

For example, if a slope stability analysis based on the ordinary method of slices is performed, the factor of safety is given by the following equation:

$$F = \frac{\Sigma M_r}{\Sigma M_d} = \frac{\Sigma c'_i \cdot \Delta\ell_i + \left(w_i \cos\alpha_i - u_i \Delta\ell_i\right)\tan\phi'_i}{\Sigma w_i \sin\alpha_i} \tag{4.13}$$

where

F	=	factor of safety
M_r	=	resisting moments
M_d	=	driving moments
c'_i	=	Cohesion
$\Delta\ell_i$	=	length along bottom of slice
w_i	=	weight of slice
α_i	=	angle along bottom of slice
u_i	=	pore water pressure
ϕ'_i	=	friction angle

This equation is based on the traditional SLD approach, which involves a factor of safety, *F*. The LRFD based equation depicting only the load factor and not the resistance factor is given by the following equations:

$$\Sigma M_d = \Sigma M_r \tag{4.14}$$

$$\Sigma \gamma_i w_i \sin\alpha_i = \Sigma c'_i \, \Delta\ell_i + \left(\gamma_i w_i \cos\alpha_i - u_i \Delta\ell_i\right)\tan\phi'_i \tag{4.15}$$

where

γ_i	=	load factor

As shown by Equation 4.15, the LRFD approach would involve applying a load factor to the weight of each slice, w_i. For ideal materials, the load is typically only applied to the load side of the equation, i.e., the left side of the LRFD equation. However, shear resistance of soil is dependent on the effective normal stress of the soil. Therefore, the load factor would also be introduced in the shear strength relationship on the right side of the LRFD equation. Therefore, applying a load factor to w_i would also result in an increase in the shear strength. This current dilemma of the load factor impacting the shear strength of soil has not been resolved. Consequently, the current state-of-practice is to design earth slopes based on SLD. Until this inconsistency in the use of LRFD in limit equilibrium slope stability analysis is resolved, design of EPS-block geofoam slopes will also continue to be based on SLD. Leshchinsky (2002) provides a more detailed discussion on the problems associated with use of LRFD in slope stability analysis.

Design Approach Incorporated in the Design Guideline

In summary, because the current state-of-practice of slope stability analysis is based on SLD, the recommended design guideline included in Appendix B is based on the SLD approach. Until the inconsistencies with applying the LRFD methodology to slope stability analysis are resolved, an LRFD based design procedure for EPS-block geofoam slopes cannot be developed.

MAJOR COMPONENTS OF AN EPS-BLOCK GEOFOAM SLOPE SYSTEM

As indicated in Figure 4.4, an EPS-block geofoam slope system consists of three major components:

- The existing slope material, which can be divided into the upper and lower slope. Slope material directly below the fill mass is also referred to as the foundation material.
- The proposed fill mass, which primarily consists of EPS-block geofoam. In addition, depending on whether the fill mass has sloped (slope-sided fill) or vertical (vertical-sided fill) sides, there is either soil or a protective structural cover over the sides of EPS blocks.
- The proposed pavement system, which is defined as including all material layers, bound and unbound, placed above EPS blocks.

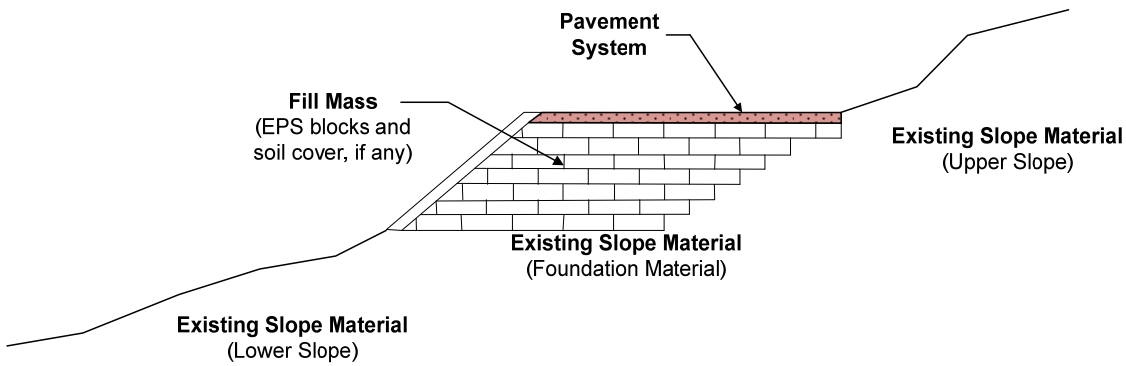


Figure 4.4. Major components of an EPS-block geofoam slope system.

FAILURE MODES

Introduction

Potential failure modes that must be considered during stability evaluation of an EPS-block geofoam slope system can be categorized into the same two general failure modes that a designer must consider in design of soil nail walls (Lazarte et al., 2003) and mechanically stabilized earth walls (Elias, et al., 2001). These failure modes are external and internal failure modes. EPS-block geofoam slope systems may also incorporate a pavement system. Therefore, to design against failure, the overall design process includes the evaluation of these three failure modes and must include the following design considerations:

- Design for external stability of the overall EPS-block geofoam slope system configuration.
- Design for internal stability of the fill mass.
- Design of an appropriate pavement system for the subgrade provided by the underlying EPS blocks.

Table 4.2 provides a summary of the three failure modes and various failure mechanisms that need to be considered for each failure mode. Each failure mechanism has also been categorized into either an ultimate limit state (ULS) or serviceability limit state (SLS) failure. The failure mechanisms are conceptually similar to those considered in the design of stand-alone EPS-block geofoam embankments over soft ground (Stark et al., 2004a, Stark et al., 2004b), as well as those that are considered in the design process of soil nail walls (Lazarte et al., 2003) and other types of geosynthetic structures used in road construction, e.g. mechanically stabilized earth walls (MSEWs) and reinforced soil slopes (RSS) (Elias et

al., 2001). Additionally, some of the failure mechanisms shown in Table 4.2 are also included in the Japanese design procedure depicted in Figure 4.3. The three failure modes are subsequently described in more detail.

External Instability Failure Mode

Design for external stability of the overall EPS-block geofoam slope system considers failure mechanisms that involve the existing slope material only as shown in Figure 4.5 as well as failure mechanisms that involve both the fill mass and existing slope material as shown in Figure 4.6. The latter potential failure surface is similar to the "mixed" failure mechanism identified by Byrne et al. (1998) for soil nailed walls, whereby the failure surface intersects soil outside the soil nail zone as well as some of the soil nails. Evaluation of the external stability failure mechanisms includes consideration of how the combined fill mass and overlying pavement system interacts with the existing slope material. The external stability failure mechanisms included in the Project 24-11(01) design procedure for stand-alone EPS-block geofoam embankments consisted of bearing capacity of the foundation material, static and seismic slope stability, hydrostatic uplift (flotation), translation and overturning due to water (hydrostatic sliding), translation and overturning due to wind, and settlement.

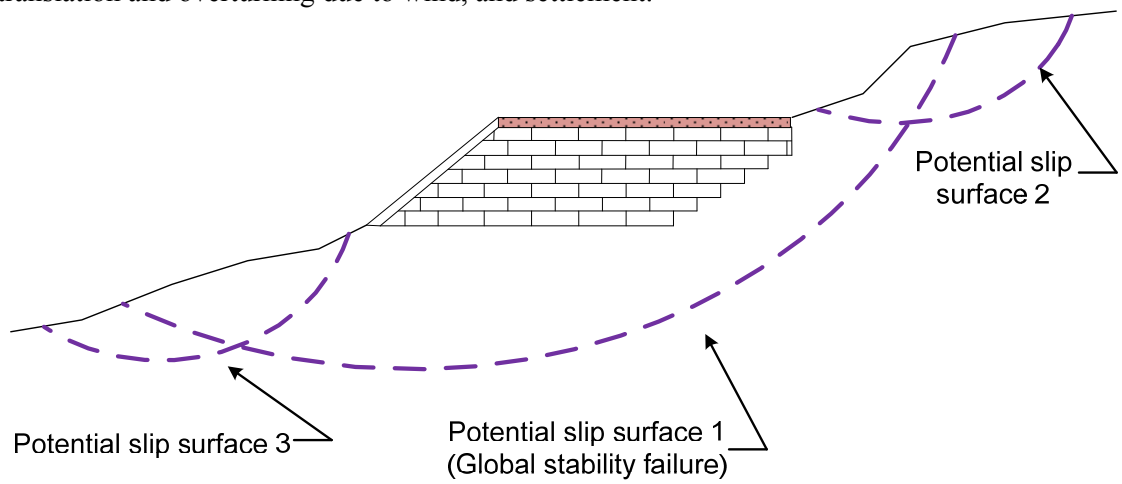


Figure 4.5. Static and seismic slope stability involving existing soil slope material only.

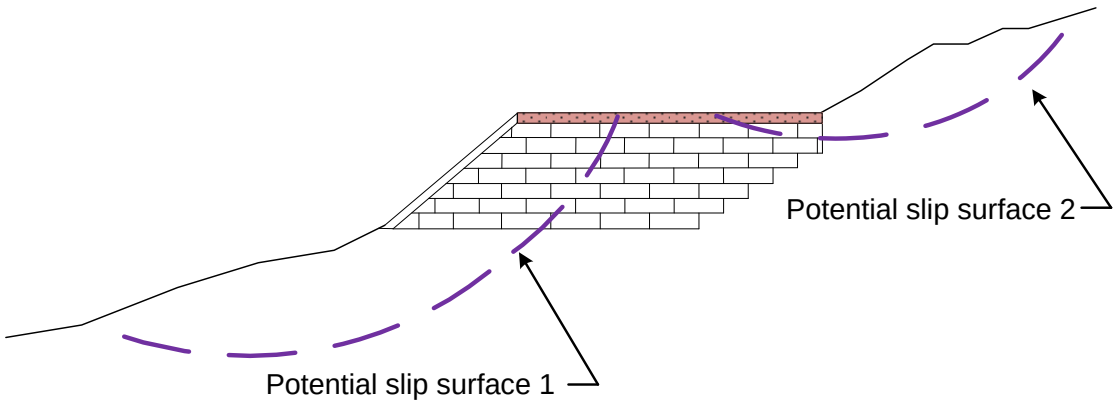


Figure 4.6. Static and seismic slope stability involving both the fill mass and existing soil slope material.

The Japanese design procedure specifically considers the hydrostatic uplift failure mechanism as part of Step 8 (See Figure 4.3). Many of the EPS-block geofoam slope case histories evaluated as part of this Project 24-11(02) research included use of underdrain systems below EPS blocks to prevent water from accumulating above the bottom of EPS blocks, and in some cases, incorporated a drainage system between the adjacent upper slope material and EPS blocks to collect and divert seepage water and thereby alleviate seepage pressures. Thus, based on current design precedent, it is recommended that all EPS-block geofoam slope systems incorporate drainage systems. If a drainage system is part of the design, then analyses for hydrostatic uplift (flotation) and translation due to water failure mechanisms that are included in the Project 24-11(01) design procedure for stand-alone EPS-block embankments are not required in slope applications. Therefore, hydrostatic uplift and translation due to water failure mechanisms are not included in current recommended design procedure for slope applications. It should be noted that in addition to a permanent drainage system, temporary dewatering and drainage systems need to be considered during construction.

Translation and overturning due to wind is a failure mechanism is considered in the Project 24-11(01) design of stand-alone embankments incorporating EPS blocks. Wind loading is not considered in the Japanese recommended design procedure for use of EPS blocks in slopes (Tsukamoto, 1996). In stand-alone embankments, the primary concern with wind loading is horizontal sliding of blocks. However, in slope applications, EPS blocks will typically be horizontally confined by the existing slope material on one side of the slope, as shown in Figure 4.2. Thus, wind loading does not appear to be a potential failure mechanism for EPS-block geofoam slopes. Therefore, the wind loading failure mechanism is not included in the current recommended design procedure. However, it is recommended that additional research be performed based on available wind pressure results on structures located on the sides of slopes to further evaluate the need to consider wind as a potential failure mechanism.

Potential failure mechanisms associated with external instability due to seismic loads include slope instability involving slip surfaces through existing slope material only as shown in Figure 4.5 and/or both the fill mass and existing slope material as shown in Figure 4.6, horizontal sliding of the entire EPS-block geofoam fill mass as shown in Figure 4.7, overturning of a vertical-sided embankment as shown by Figure 4.8, bearing capacity failure of the existing foundation earth material due to static loads and seismic loads and/or a decrease in shear strength of foundation material as shown in Figure 4.9, and earthquake induced settlement of the existing foundation material as shown in Figure 4.10.

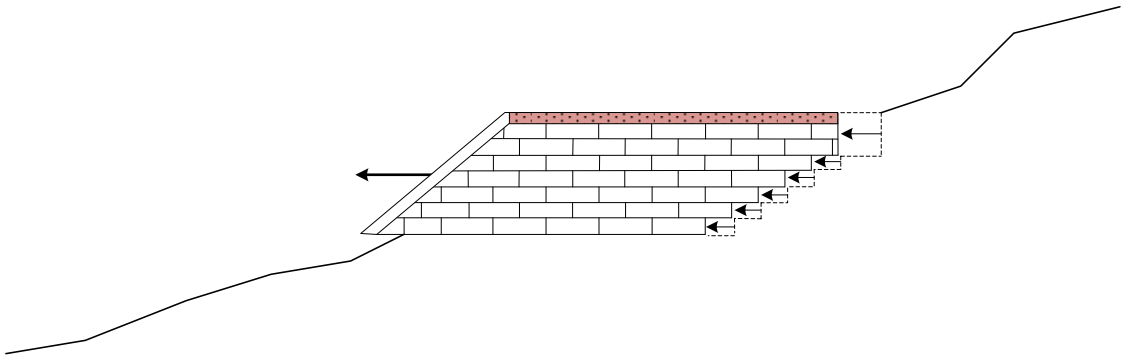


Figure 4.7. External seismic stability failure involving horizontal sliding of the entire embankment.

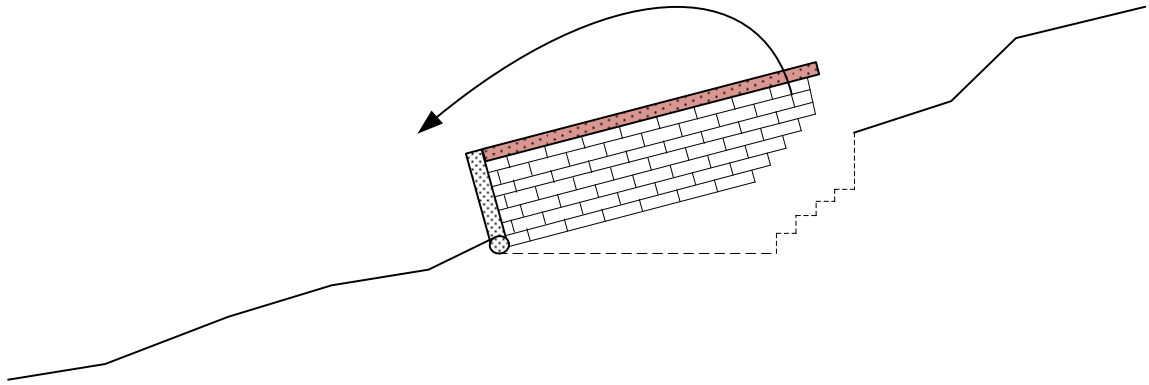


Figure 4.8. External seismic stability failure involving overturning of an entire vertical embankment about the toe of the embankment.

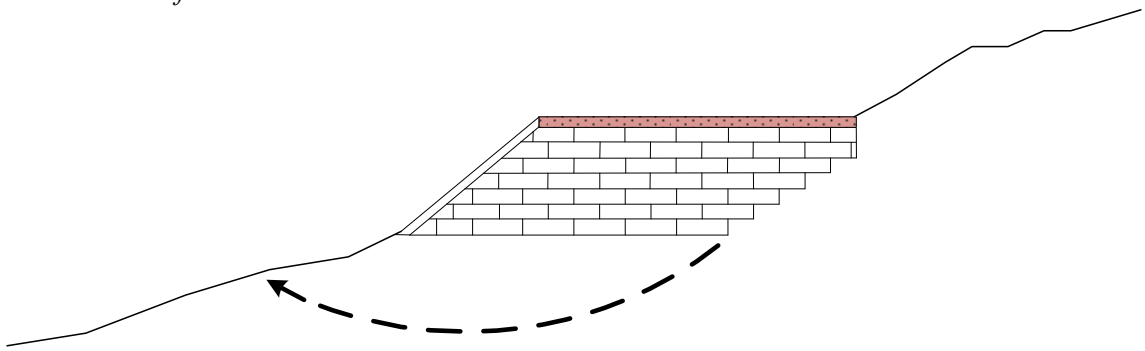


Figure 4.9. Bearing capacity failure of the embankment due to general shear failure or local shear failure.

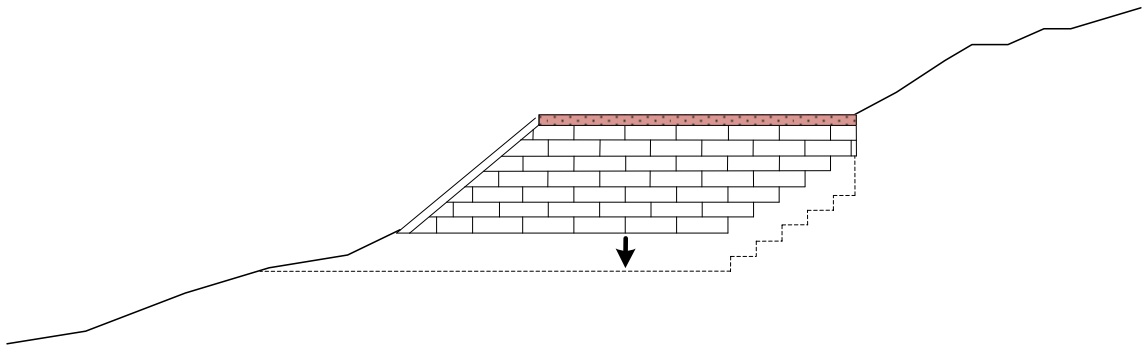


Figure 4.10. Excessive settlement.

In summary, as shown in Table 4.2, external stability failure mechanisms that are included in the proposed design procedure consist of static slope stability, settlement, and bearing capacity. Additional failure mechanisms associated with external seismic stability include seismic slope instability, seismic induced settlement, seismic bearing capacity failure, seismic sliding, and seismic overturning. These failure considerations, together with other project-specific design inputs, such as right-of-way constraints, limiting impact on underlying and/or adjacent structures, and construction time, usually govern the overall cross-sectional geometry of the fill. Because EPS-block geofoam is typically a more expensive

material than soil on a cost-per-unit-volume basis for the material alone, it is desirable to optimize the design to minimize the volume of EPS used, yet still satisfy external instability design criteria concerning settlement, bearing capacity, and static and seismic slope stability.

Internal Instability Failure Mode

Design for internal stability considers failure mechanisms within the EPS-block geofoam fill mass. Internal instability failure mechanisms included in the Project 24-11(01) design procedure for stand-alone embankments consisted of translation due to water and wind, seismic stability, and load bearing. As previously indicated in the external instability failure mode discussion, translation due to water and wind does not appear to be applicable to EPS-block geofoam slope systems. Therefore, seismic stability and load bearing of EPS blocks appear to be the primary internal instability failure mechanisms that need to be considered in EPS block slope systems.

It should be noted that static slope stability is not an internal stability failure mechanism for stand-alone embankments, and is not part of the internal stability design phase in the Project 24-11(01) design procedure for stand-alone embankments, because there is little or no static driving force within the EPS-block fill mass causing instability. The driving force is small because the horizontal portion of the internal failure surfaces is assumed to be along EPS block horizontal joints and completely horizontal while the typical static loads are vertical. The fact that embankments with vertical sides can be constructed demonstrates the validity of this conclusion.

For geofoam slope applications the potential for EPS-block fill mass to withstand earth pressure loads from adjacent upper slope material as depicted in Figure 4.4 was evaluated as part of this study. Horizontal sliding between blocks and/or between the pavement system and upper level of blocks due to adjacent earth pressures is a failure mechanism that needs to be considered if the adjacent slope is not self-stable. Since the mass of the EPS-block fill is typically very small, it may not be feasible for EPS fill to directly resist external applied earth forces from the adjacent slope material. Additionally, since the interface shear resistance of EPS/EPS interfaces is related to the normal stress, which is primarily due to the mass of the EPS blocks, the shear resistance between blocks may not be adequate to sustain adjacent earth pressures. Therefore, the design procedure is based on a self-stable adjacent upper slope to prevent earth pressures on the EPS fill mass that can result in horizontal sliding between blocks. Although the design procedure is based on a self-stable adjacent slope, it may be possible to design an EPS-block geofoam slope system that will support a portion of the upper adjacent slope by transferring the loads through the assemblage of EPS blocks to a structural wall that also acts as the protective facing cover of a vertical-sided fill.

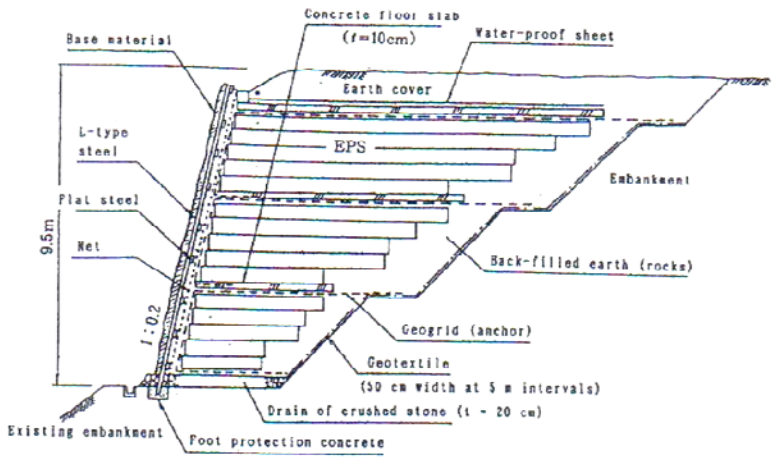
If adjacent slope material cannot be cut to a long-term stable slope angle, an earth-retention system must be used to resist the applied earth force. An anchored facing system can be used to support the adjacent earth forces as shown in Figure 4.11 (a). This approach, developed in Norway (Horvath, 1995) and also utilized in Japan (Tsukamoto, 1996), consists of placing one or more intermediate horizontal slabs of poured-in-place reinforced Portland cement concrete (PCC) within the EPS as the blocks are placed. The PCC slabs are connected to the facing system and anchored into the adjacent earth slope. The anchor system may consist of ground anchors or a geosynthetic such as a geogrid. A second earth retention system type consists of a gravity or cantilever retaining wall designed to retain both the EPS blocks and earth material. A retaining wall system is illustrated in Figure 4.11 (b). A key distinction between a facing wall system and a retaining wall system is that the purpose of a facing system is only to protect the EPS blocks from damage, not to support any lateral loads, whereas the primary purpose of a retaining wall system is to resist lateral loads imposed by the retained EPS block and adjacent earth material. A secondary purpose of a retaining wall system can be to function as a covering system and protect the EPS block. A third potential earth retention system consists of a reinforced soil slope system designed to retain the adjacent earth as shown by Figure 4.11 (c).

The primary evaluation of internal seismic stability involves determining whether the geofoam embankment will behave as a single, coherent mass when subjected to seismic loads. Since EPS block

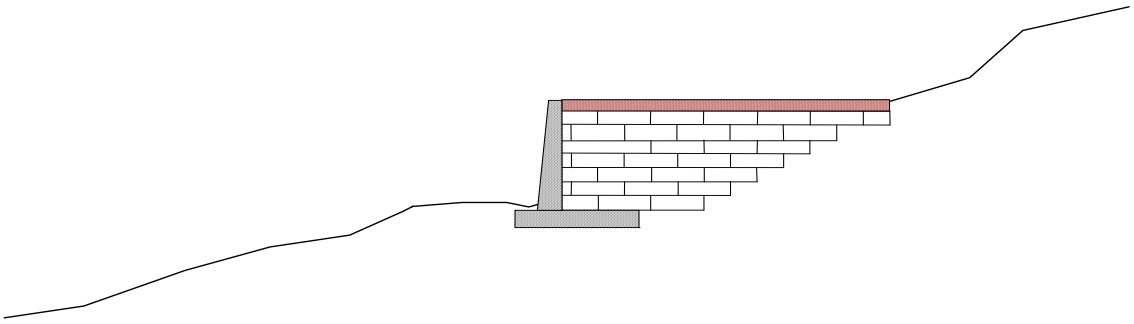
consists of individual blocks, the collection of blocks will behave as a coherent mass if the individual EPS blocks exhibit adequate vertical and horizontal interlock. The recommended standard included in Appendix F provides block placement guidelines that should provide adequate vertical interlocking. Therefore, the primary seismic internal stability issue is the potential for horizontal sliding along horizontal interfaces between blocks and/or between the pavement system and upper layer of blocks as shown by Figure 4.12. Another seismic internal stability failure mechanism that was recognized during the design of the Central Artery/Tunnel (CA/T) project embankments (Horvath, 2004a, Riad, 2005b, Riad and Horvath, 2004) is load-bearing failure due to the increase in the vertical normal stress within the EPS-block fill mass as shown in Figure 4.13 due to seismic rocking of the fill mass as depicted in Figure 4.14.

Load-bearing failure of the EPS block due to excessive dead or gravity loads from the overlying pavement system and traffic loads is the third internal stability failure mechanism. The primary consideration during load-bearing analysis is proper selection and specification of EPS properties so that the geofoam mass can support the overlying pavement system and traffic loads without excessive immediate and time-dependent (creep) compression that can lead to excessive settlement of the pavement surface (an SLS consideration) as shown in Figure 4.15.

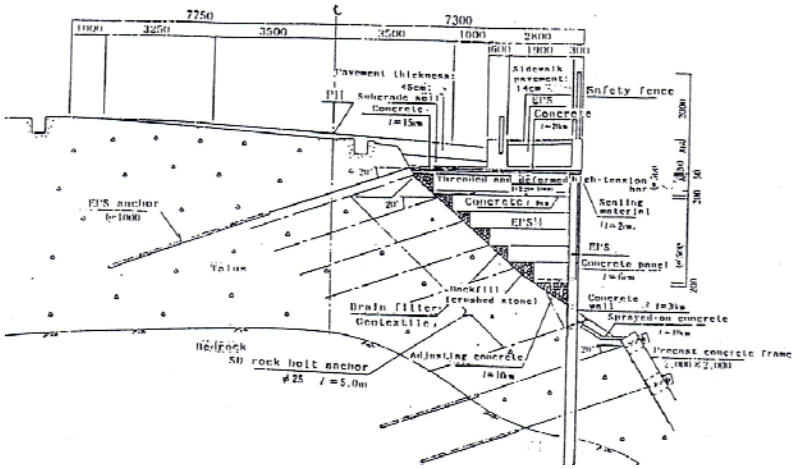
In summary, as shown in Table 4.2, the three internal instability failure mechanisms that are evaluated in the design guideline are seismic horizontal sliding, seismic load-bearing of EPS blocks, and static load-bearing of EPS blocks.



a) Anchored facing system.



b) Conventional retaining wall system.



c) Reinforced soil slope system.

Figure 4.11. Earth retention system alternatives. Figures a) and c) from Tsukamoto, (1996).

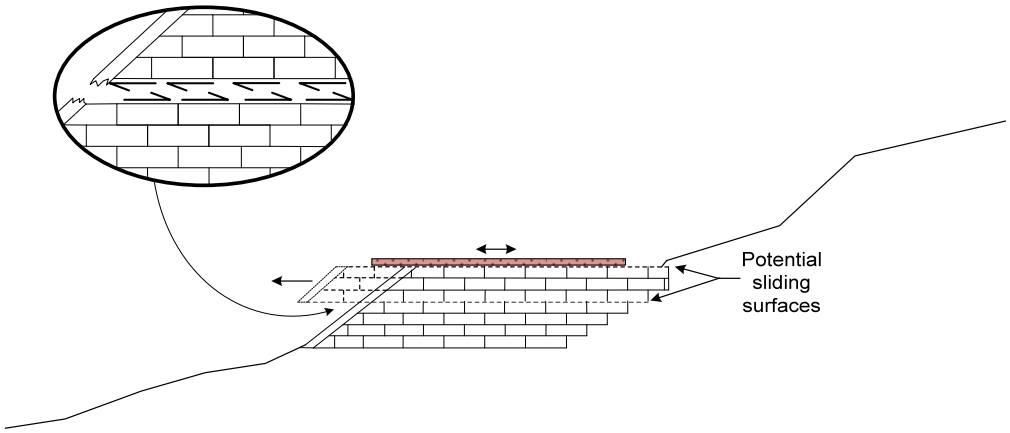


Figure 4.12. Internal seismic stability failure involving horizontal sliding between blocks and/or between the pavement system and upper layer of blocks due to seismic loading.

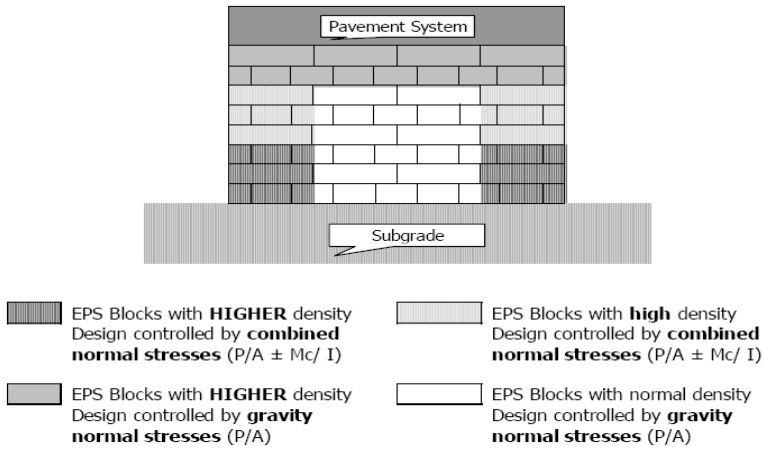


Figure 4.13. Relative compressive normal stresses in EPS blocks due to combined seismic and gravity loads (Riad, 2005a; used with permission from ASCE).

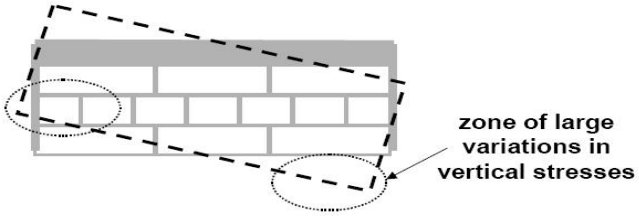


Figure 4.14. Seismic rocking due to seismic-inertia force (Horvath, 2004a; (Horvath, 2004a; From "Lessons learned from failure: EPS geofoam," Geotechnical Fabrics Report [now Geosynthetics magazine], Oct/Nov 2004, volume 22, number 8. Reprinted with permission).

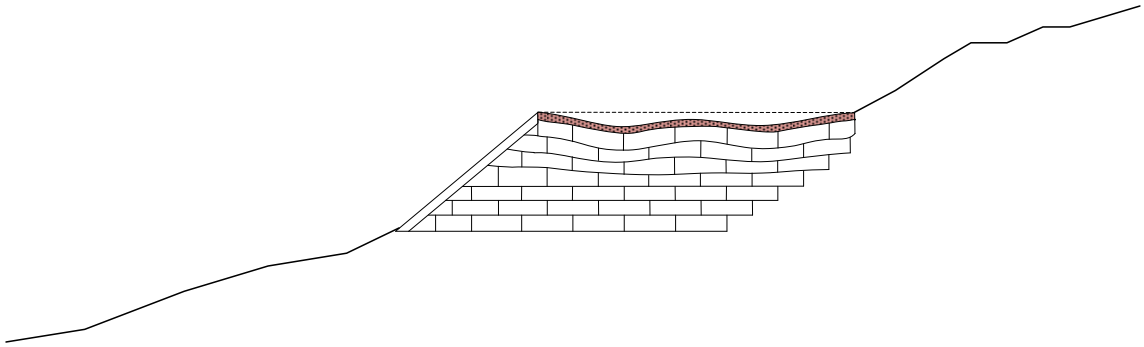


Figure 4.15. Load-bearing failure of the blocks involving excessive vertical deformation.

Pavement System Failure Mode

Design of an appropriate pavement system considers the subgrade provided by underlying EPS blocks. The design criterion is to prevent premature failure of the pavement system, such as rutting, cracking, or similar criterion, which is an SLS type of failure (Figure 4.16). Also, when designing the pavement cross-section, some consideration should be given to providing sufficient support, either by direct embedment or structural anchorage, for any road hardware such as guardrails, barriers, median dividers, lighting, signage and utilities.

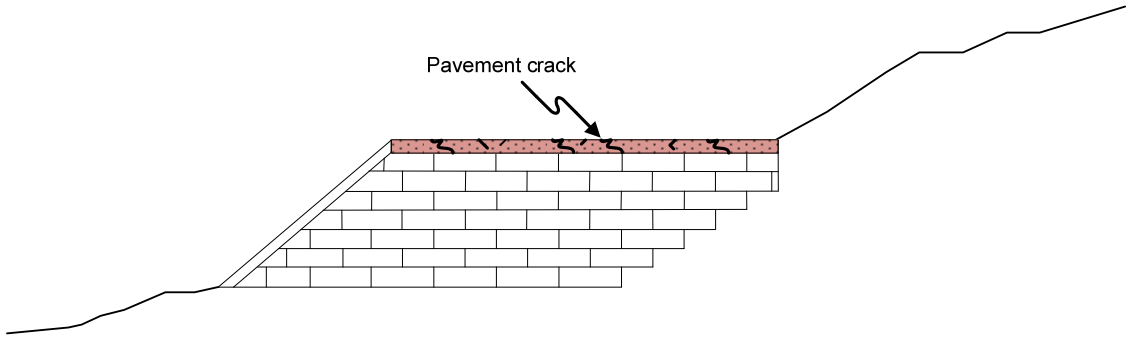


Figure 4.16. Pavement failure due to cracking.

Critical Failure Mechanisms in Practice

Most experience with EPS-block geofoam fills in the U.S. to date has been with use of stand-alone embankments over soft ground, and not with slope stabilization applications. Based on this experience and on preliminary information obtained as part of this Project 24-11(02) work, there have been a surprising number of failures of EPS-block geofoam earthworks. These failures appear to be primarily serviceability related. In at least two cases, the vertical displacement of the road surface due to unsatisfactory performance of EPS blocks was apparently so severe that the owning agency had to remove the EPS blocks and replace them with alternative materials. One of these failures was referenced by the NYDOT as part of their response to Question A5ii of the project questionnaire included in Appendix A. Therefore, the load-bearing failure mechanism deserves more attention during design as well during the manufacturing and construction quality assurance process than has been provided in the past. In particular, it is recommended that these failures be evaluated to determine if modifications to the load-bearing analysis procedure and/or the MQA/CQA process is required.

The primary failure mechanism that needs to be evaluated for EPS-block geofoam in slope stabilization applications is overall (external) slope stability because this is typically the primary reason for considering use of EPS blocks in slope applications. Based on the experience with mechanically stabilized earth walls (MSEWs), problems related with MSEWs have primarily involved global instability (Leshchinsky, 2002). Therefore, global stability of an EPS-block geofoam slope system may also prove to be a critical failure mechanism.

External stability failure mechanisms included in the proposed design procedure consist of static slope stability, settlement, and bearing capacity. Additional failure mechanisms associated with external seismic stability include seismic slope instability, seismic induced settlement, seismic bearing capacity failure, seismic sliding, and seismic overturning. The three internal instability failure mechanisms that are evaluated in the design guideline are seismic horizontal sliding, seismic load-bearing of EPS blocks, and static load bearing of EPS blocks.

In summary, the three failure modes that must be considered during stability evaluation of an EPS-block geofoam slope system include external instability, internal instability, and pavement system failure. Table 4.2 provides a summary of the failure mechanisms that are evaluated for each failure mode, as well as a summary of the limit state that is considered. The design procedure included in this report provides the recommended sequence for evaluating each of the failure mechanisms shown in Table 4.2. However, a summary of the design loads that need to be considered when designing an EPS-block geofoam slope system is presented next.

DESIGN LOADS

Introduction

Although the loads included in the AASHTO bridge specifications are intended for bridge structures and not for earth slopes, the various loads and forces included in the AASHTO specifications were used as a source of load types that may also need to be considered in EPS-block geofoam slopes. Since the design guideline is based on SLD, loads included in the 1996 AASHTO Standard Specifications for Highway Bridges (AASHTO, 1996) were considered. The current AASHTO LRFD Bridge Design Specifications (2007) were not referenced because this current specification is based on LRFD and not on SLD. Additionally, a variety of sources were considered, including design manuals related to the design of soil nail walls (Byrne et al., 1998, Lazarte et al., 2003), mechanically stabilized earth walls (Elias et al., 2001), and reinforced soil slopes (Elias et al., 2001).

Based on recommendations found in various earth structure design manuals noted above and the failure mechanisms that are considered in the proposed design procedure for EPS-block geofoam slopes, which are summarized in Table 4.2, the applicable load types for geofoam slopes are dead loads (D), live loads (L), and seismic loads (EQ). The following loads are typically not considered in the design of conventional slopes: rib shortening (R), shrinkage (S), temperature (T), stream flow pressure (SF), longitudinal force from live load (LF), and ice pressure (ICE). The following loads may be applicable to the design of EPS-block geofoam slopes: centrifugal force (CF), earth pressure (E), buoyancy (B), wind (W), and wind load on live load (WL). A synopsis of these potential applicable loads is subsequently provided.

CF loads are typically not considered in design of earth slopes. AASHTO requires that highway structures on curves be designed for a centrifugal horizontal radial force due to the acceleration of passing vehicles as they go around the curve. Each time a vehicle traverses the curve, the tires exert a lateral frictional force on the vehicle which causes the vehicle to travel along the curved path. The roadway provides a reaction force that gives vehicle tires "something to push against." It is this reaction force that, according to AASHTO specifications, must be accounted for in the design of highway structures. Design manuals for soil nail walls (Lazarte et al., 2003), mechanically stabilized earth walls and reinforced soil slopes (Elias et al., 2001), do not specifically address the issue of centrifugal force loads on earth structures. The reason for this exclusion is most likely the fact that the magnitude of the centrifugal force

is generally small compared to the weight of a typical earth structure. Thus, it appears that centrifugal forces are commonly neglected in design of conventional earth structures. However, this practice may not be suitable for roadways constructed over EPS-block geofoam. Because of the extremely low density of EPS, lateral forces imposed on a roadway curve by vehicle tires may become significant, and, therefore, may need to be accounted for in the design. This lateral force may tend to stabilize or destabilize the EPS-block geofoam slope, depending on the orientation of the slope and roadway curve. If the force is directed back into the slope, it will act as a stabilizing force. If the force is directed outward from the slope, it will act as a destabilizing force. For instances in which the lateral centrifugal force tends to destabilize the roadway, it should be taken into account in the design; however, if lateral force is oriented in such a way as to stabilize the slope, it should be ignored because the lateral force is only temporary and not permanent.

The importance of centrifugal loads in design of EPS-block geofoam slope systems is explained further in this chapter in the Centrifugal Loads section. E is not applicable for the design of EPS-block geofoam slopes that do not involve an earth retention system. The mass of EPS-block fill is typically very small and, consequently, it is not feasible for EPS fill to directly resist applied earth forces from adjacent slope material. As previously indicated in the Internal Instability Failure Mode section of this chapter, if the adjacent slope material cannot be cut to a long-term stable slope angle, an earth-retention support system must be included in the design of the slope. The proposed design procedure is based on the ability of the natural slope material adjacent to EPS-block geofoam fill mass to be self stable and does not consider design of earth-retention support systems.

B may also not be required for design of EPS-block slopes if a drainage system is incorporated in the design to prevent water from accumulating above the bottom of the EPS blocks and to collect and divert seepage water from the adjacent upper slope material. As previously indicated in the External Instability Failure Mode section of this chapter, based on current design precedent, it is recommended that all EPS-block geofoam slope systems incorporate drainage systems. Therefore, since the proposed design procedure is based on the use of a drainage system, B is not applicable.

As previously indicated in the External Instability Failure Mode section of this chapter, although wind loading is considered in the design of stand-alone embankments incorporating EPS blocks, wind loading does not appear to be a potential failure mechanism for EPS-block geofoam slopes in slope applications because the EPS blocks will typically be horizontally confined by the existing slope material on one side of the slope. However, it is recommended that additional research be performed of available wind pressure results on structures located on the sides of slopes to further evaluate the need to consider wind as a potential failure mechanism. In the interim, the proposed design procedure does not include wind loading as a potential failure mechanism. Thus, W and WL loads are not considered in the design procedure.

Barriers or guardrails are typically required with vertical-sided fills. Additional loads associated with design of barriers and railings, such as horizontal vehicle collision loads, may need to be considered. Experience with the design of the CA/T EPS-block geofoam stand-alone embankments is that these impact loads may be a significant design consideration.

In summary, the three primary loads applicable for design of EPS-block geofoam slopes are dead loads (D), live loads (L), and seismic loads (EQ). In addition to these three primary load types, centrifugal loads (CF) may also be significant for EPS-block geofoam structures. Additionally, loads associated with design of barriers and railings may need to be considered. A discussion of these loads is presented in greater detail below. For ultimate limit state calculations, the worst expected loadings are typically used, while for serviceability limit state calculations, the typical or average expected loadings are used.

Dead (Gravity) Loads

Components of the embankment system, which are depicted in Figure 4.4, that contribute to gravity loading and need to be considered in design include:

- The weight of the overlying pavement system, which includes any reinforced PCC slab that might be used at the pavement system and geofoam interface.
- The weight of soil cover placed on the sides of a slope-sided embankment or weight of the protective facing wall elements of a vertical-sided embankment.
- The net effective weight of any earth material placed between the existing ground surface (foundation material) and the bottom of EPS blocks.

Gravity loads can be calculated based on a preliminary assumed cross-section, including the pavement system, and any cover material over the sides of the embankment. To establish this preliminary cross-section of the embankment and to begin the design procedure, a minimum pavement system of 610 mm (24 in.) was recommended in the Project 24-11(01) report for design of stand-alone EPS-block geofoam embankments. This recommendation was based on limited case history data. This case history data indicated total pavement thicknesses ranging from 508 to 864mm (20 to 34 in.). This is an overall average of 660mm (26 in.). Case history data of EPS-block geofoam projects in Norway reported by Aabøe (1987) indicated an average pavement system thickness of 660mm (26 in.). Based on the minimum recommended pavement system thickness to minimize the potential for differential icing conditions from the Norwegian design guidelines (Horvath, 1995, Norwegian Road Research Laboratory, 1992) of 400 mm (16 in.) to 800mm (32 in.) and the Swedish guidelines of 400mm (16 in.) to 500mm (20 in.) (Gandahl, 1987, Horvath, 1995), a minimum pavement system thickness of 610mm (24 in.) was recommended in the Project 24-11(01) design guideline to be initially used in the preliminary design of stand-alone EPS-block embankments to minimize the potential for differential icing conditions. It should be noted that the Project 24-11(01) design procedure also included a separate pavement design step that included a structural pavement analysis to finalize the pavement system design. Thus, the initial recommended minimum pavement system of 610mm (24 in.) is only used to establish a preliminary cross-section of the embankment and to begin the design procedure,

The recommended design procedure for use of EPS-block geofoam in slopes is also based on starting the design procedure with a preliminary cross-section of the EPS-block fill mass. Therefore, if a pavement system will be placed over the fill mass, a pavement system thickness must be initially assumed. However, as with the design procedure for stand-alone embankments, the design procedure for slope stabilization also includes a separate pavement design step that includes a structural pavement analysis to finalize the pavement system design.

Use of EPS-block geofoam in lightweight fill applications since completion of the Project 24-11(01) study for stand-alone embankments has involved extensive use of vertical-sided embankments. This experience with vertical-sided embankments indicates that a thicker pavement system on the order of 1m (3 ft.) to 1.5m (5 ft.) may be more appropriate to accommodate any road hardware such as guardrails, barriers, median dividers, lighting, signage, and utilities. The design procedure for EPS-block slopes is based on obtaining a pavement system that provides the least amount of stress on top of the EPS-block geofoam fill mass to satisfy internal and external stability requirements. Therefore, it is recommended that the preliminary pavement system be assumed to be 1m (3 ft.) thick and the various component layers of the pavement system be assumed to have a total unit weight of 20 kN/m^3 (130 lbf./ft^3) for initial design purposes. The total unit weight value is based on findings obtained during the Project 24-11(01) work for stand-alone embankments. However, as will be later shown in the discussion of design steps, a pavement structural analysis is required to finalize the final pavement system design.

As indicated in Chapter 3, long-term density of EPS blocks that are permanently or periodically submerged in ground water may increase because of water absorption. The recommended design procedure is based on use of a permanent drainage system. Therefore, the most applicable densities for use in determining dead loads are either the dry densities per Japanese design recommendations or densities based on a water content of 1 percent by volume per Norwegian test results. Although EPS-block geofoam can be manufactured to various densities, preliminary design can be based on a density of 20 kg/m^3 (1.25 lbf./ft^3). Therefore, the dry unit weight of EPS can be taken to be 200 N/m^3 (1.25 lbf./ft^3) or a unit weight at 1 percent water content by volume of 300 N/m^3 (1.9 lbf./ft^3) for preliminary design.

Table 3.3 provides a summary of densities based on a water content of 1 percent by volume associated with various dry block densities for use in estimating dead loads.

Use of dry unit weight versus the unit weight at 1 percent water content by volume in design will depend on failure mechanism being evaluated. For example, when evaluating certain failure mechanisms such as internal load-bearing of EPS blocks, settlement, and bearing capacity failure of foundation material, use of a higher unit weight for EPS-block geofoam would be conservative; however, it should be noted that this is not the case for all failure mechanisms.

For example, when evaluating external slope stability, the higher unit weight would result in increased driving forces, which would make using the higher unit weight more conservative and therefore more appropriate for design. But, by using a higher unit weight for EPS, normal stresses along the slip surface are also increased, which, in turn, results in an increase in shear strength. So use of the unit weight at 1 percent water content by volume increases both the driving force and resisting forces in the slope analysis, making it difficult to discern which unit weight value is truly more conservative. For cases such as this, the best approach is to perform the analysis using both the dry unit weight and unit weight at 1 percent water absorption by volume and compare the results. The value for the unit weight that results in the lower factor of safety is the unit weight that should be used for that particular failure mechanism.

Live (Traffic) Loads

A live load surcharge pressure equal to 610mm (2 ft.) of earth is typically used in the design of soil nail walls, mechanically stabilized earth walls, and reinforced soil slopes. This surcharge pressure is probably based on the AASHTO requirement that when highway traffic can come within a horizontal distance from the top of the structure equal to one-half its height, an additional live load pressure equal to not less than 610mm (2 ft.) of earth shall be added (AASHTO, 1996). The AASHTO manual recommends a unit weight of 18.9 kN/m^3 (120 lbs./ft^3) for compacted sand, earth, gravel or ballast. Therefore, 610mm (2 ft.) of an 18.9 kN/m^3 (120 lbf./ft^3) surcharge material can be used to model traffic stresses at the top of the embankment.

The exception to use of a surcharge pressure to represent traffic loads is in evaluation of load-bearing capacity of EPS blocks. The basic procedure for designing against load-bearing failure is to calculate the maximum vertical stress at various levels within the EPS mass and select the EPS that exhibits an elastic limit stress that is greater than the calculated or required elastic limit stress at the depth being considered. Traffic loads are a major consideration in load-bearing capacity calculations. Therefore, the effects of traffic loading and traffic configuration are critical to the load bearing analysis and are explicitly estimated as a part of it. A more detailed discussion on explicitly estimating traffic loads is included in the load-bearing analysis section of this chapter.

Seismic Loads

Seismic loading is a short-term event that is considered in geotechnical problems including road embankments and slopes. Seismic loading can affect both external and internal stability of an EPS-block geofoam slope system. Considerations for seismic external stability analyses are similar for embankments constructed of EPS-block geofoam or earth materials. These considerations include various SLS and ULS mechanisms, such as seismic settlement and liquefaction, that are primarily independent of the nature of the embankment or fill mass material because they depend on the seismic risk at a particular site and nature and thickness of natural soil overlying the bedrock. Kavazanjian et al. (1997) provide a discussion of these topics. Mitigation of seismic induced subgrade problems by ground improvement techniques prior to embankment construction can be found in various publications (Elias et al., 1999, Holtz, 1989, Kavazanjian et al., 1997).

Question B.6 of the geofoam usage survey that was conducted as part of this project that is included in Appendix A revealed that 11 of the total 16 Departments of Transportation that responded to this question indicated that seismic loading is not a design consideration in the area, 2 of the 16 DOTs

indicated seismic loading is a design consideration in the geographical area but do not perform seismic slope stability analysis, and 3 of the 16 DOTs do perform seismic slope stability analysis. The project survey results appears to be in agreement with the findings of the NCHRP Report 611 that free-standing retaining walls and buried structures most often were not designed for seismic loading due in part to the lack of generally accepted design guidelines and the general costs associated with the implementation of additional design requirements.

Required seismic design policies vary between state DOTs. For example, the WSDOT policy on cut slopes in soil and rock, fill slopes, and embankments is that instability due to seismic events should be evaluated. However, mitigation of instability is not always required due to the high cost of requiring mitigation of cut and fill slopes and embankments statewide. However, stabilization is required for slopes that impact an adjacent structure if failure due to seismic loading occurs (Washington State Department of Transportation, 2006).

Failure mechanisms that are considered for external seismic stability analysis include slope instability, horizontal sliding of the entire EPS-block geofoam fill mass, overturning of a vertical sided embankment, bearing capacity failure of existing foundation earth material, and settlement of existing foundation material. The general external seismic analysis procedure consists of performing a pseudo-static analysis to evaluate slope instability. The effect of natural slope material on external horizontal sliding of the entire EPS-block fill mass and overturning of a vertical-sided embankment proposed in the design guideline in Appendix B consists of determining the magnitude of the seismic earth pressure based on the Mononobe-Okabe (M-O) method (Okabe, 1926, Mononobe, 1929). Bearing capacity failure and settlement of existing foundation material are also part of external seismic stability.

Failure mechanisms that are considered for internal seismic stability analysis include horizontal sliding between layers of blocks and/or between the pavement system and upper layer of blocks and load-bearing failure of EPS blocks. The general internal seismic analysis procedure consists of decoupling the determination of the overall seismic response acceleration of the EPS-block geofoam embankment into the determination of the seismic response of the natural slope material, followed by the seismic response of the EPS-block fill mass. The seismic response results of the adjacent natural slope material and EPS-block fill mass can then be used to evaluate each potential seismic failure mechanism separately. Both external and internal seismic analysis procedures are discussed in detail later in this chapter.

Centrifugal Loads

Although loads due to centrifugal forces are not typically considered in design of earth structures, they may prove significant in the design of slopes incorporating EPS-block geofoam. The AASHTO centrifugal load (CF) category is intended to account for the reaction forces exerted on a curved highway bridge as vehicles go around the curve. The vehicles' tires exert a force on the roadway to overcome the vehicles' inertia and accelerate it around the curve. This in turn produces a reaction force on the roadway surface which is eventually transmitted to the subgrade. It should be noted that the terminology commonly applied to this topic can be somewhat confusing because it is heavily dependent on the particular frame of reference being discussed. The force exerted by vehicle tires that overcomes the vehicle's inertia is technically a centripetal or "center-seeking" force; that is, it is oriented in such a way as to push the vehicle toward the center of the curve. The force of interest for the design of EPS-block slopes is the reaction force corresponding to this centripetal force. It is equal in magnitude and opposite in direction, pushing the roadway away from the center of the curve, hence the term centrifugal or "center-fleeing." Thus, the AASHTO designation is technically the proper way of describing the loads in question.

These centrifugal loads are dependent on the volume of traffic that the roadway is designed to carry, as well as the roadway design speed. An interstate highway designed to carry high traffic loads at high design speeds will obviously exert much greater centrifugal loads on its underlying subgrade than a low-traffic rural road. For most earth structures, the sum of the reaction forces developed at the roadway is so small compared to the mass of the underlying subgrade that centrifugal loading can be safely

ignored; however, because EPS-block geofoam has such an extremely low density, the inertia of the fill mass may not be large enough to justify neglecting the centrifugal forces developed at a curved roadway surface. A study on the impact of typical centrifugal loads on an EPS-block geofoam fill mass was performed as part of this Project 24-11(02) research. The results of this study are presented below. As with seismic loading, any lateral loads applied to the EPS-block geofoam fill must be given special consideration to prevent shifting and shearing at the interfaces between layers of blocks.

Because centrifugal loads on the roadway are directed away from the center of the curve, centrifugal forces acting on the roadway may not always tend to act as a destabilizing force. This is a key difference between use of EPS-block geofoam in slopes as opposed to stand-alone embankments. If the roadway curve is oriented in such a way that the center of the curve lies on the side of the roadway away from the slope, centrifugal forces generated by curving vehicles may actually push EPS fill back into the slope. This is, in essence, a stabilizing force acting on the slope. In view of this fact, it is recommended that centrifugal loads be considered only in the case of a curved roadway for which the center of the curve lies on the side of the roadway toward the slope. For instances where the roadway curve has its center on the side of the road opposite the slope, the effects of the centrifugal forces should not be taken into account. This practice will ensure a safe, conservative design for the slope and EPS fill.

According to AASHTO specifications (1996), this centrifugal force is calculated as a percentage of the live load associated with the roadway using Equation 4.16 shown below:

$$C = 0.00117S^2 D = \frac{6.68S^2}{R} \tag{4.16}$$

where

C	=	centrifugal force in percentage of the live load, without impact
S	=	design speed of roadway in miles per hour
D	=	degree of curve
R	=	radius of the curve in feet

Once C has been calculated by Equation 4.16, the magnitude of the force may be calculated using Equation 4.17 (AASHTO, 1996):

$$F_{CE} = \frac{C}{100} \times LL \tag{4.17}$$

If the roadway is superelevated, this fact must be accounted for by multiplying F_{CE} times the cosine of the angle of inclination of the roadway surface due to superelevation. This force F_{CE} is to be applied at a height of 1.8m (6 ft.) above the surface of the roadway (AASHTO, 1996). Therefore, the magnitude of the overturning moment may be calculated using Equation 4.18 shown below:

$$M_O = F_{CE}(H + 6) \tag{4.18}$$

where

M_O	=	overturning moment
F_{CE}	=	centrifugal force acting on roadway
H	=	height of the EPS-block fill + height of pavement system in feet

This M_O should be compared to the potential resisting moment due to the weight of the structure to determine the factor of safety against overturning. It should be noted that Equations 4.16, 4.17, and 4.18 are based on Imperial units.

In addition to evaluating the possibility of external slope instability, the nature of EPS-block geofoam construction is such that centrifugal loads may also tend to contribute to other failure mechanisms besides simply external slope instability of the overall EPS-block fill mass, such as sliding between block layers or internal instability. Centrifugal forces were included in the design of EPS-block geofoam stand-alone embankments utilized as part of the CA/T project (Parsons Brinckerhoff, undated).

Vehicle centrifugal forces are typically neglected during design of conventional soil fills because the magnitude of these forces is very small compared to the mass of a typical earth fill. However, since EPS-block geofoam has such a low unit weight (roughly 1 percent of conventional soil fill materials), it was not clear whether this practice of neglecting centrifugal forces in the design of EPS fill was appropriate. Therefore, the goal of centrifugal load analyses was to evaluate the significance of centrifugal force loads for EPS-block geofoam slope fills.

As will be reiterated later, two potential failure mechanisms relating to centrifugal force loads include horizontal sliding at some critical interface within the EPS fill system and overturning of the entire fill about its toe. These two failure mechanisms are also evaluated as part of external and internal seismic stability analysis. Therefore, it may be possible to compare centrifugal forces and seismic forces and perform the evaluation of horizontal sliding and overturning based on the larger of the two forces.

No literature directly relating to centrifugal forces in EPS-block geofoam slope fills was found during the literature search. However, the literature search did yield some useful information that served as a reference point for the analysis performed for this task. During the design of an EPS-block geofoam embankment as part of the CA/T project, centrifugal forces were evaluated using the method recommended by AASHTO for highway bridge design. However, in this case, the analysis focused on the failure mechanism of overturning of the entire embankment. No consideration was given to the possibility of centrifugal forces contributing to sliding between layers of EPS block within the fill, which is a failure mechanism that is typically considered for other design loads in the design of EPS-block geofoam slopes. Based on this precedent, the method proposed by AASHTO (1996) for calculating the magnitude of vehicle centrifugal force loads was used to evaluate the significance of these loads for the design of EPS-block geofoam slope fills.

The AASHTO method for calculating the magnitude of centrifugal force loads was originally developed to be used in design of highway bridges; however, because the method is based directly on the principles of dynamics and contains no limiting assumptions related specifically to highway bridge design, it can be readily applied to the design of almost any type of structure supporting a roadway. The AASHTO method calculates the centrifugal force exerted on the roadway by a vehicle's tires as the vehicle travels around a curve in the roadway as a function of the live load on the roadway. In other words, the magnitude of the centrifugal forces acting on the roadway is calculated as a percentage of the weight of the vehicle in question. In the case of the AASHTO method, this vehicle may be one of four standardized design trucks. Once calculated, the centrifugal force load acting on the roadway may be used to analyze each of the potential failure mechanisms.

Two potential failure mechanisms relating to centrifugal force loads were identified, namely that of horizontal sliding at some critical interface within the EPS fill system and that of overturning of the entire fill about its toe. A sensitivity study based on AASHTO (2002) recommendations regarding highway geometry was performed to evaluate the failure mechanisms of horizontal sliding. This study, described in detail in Appendix D, indicated that, for most EPS-block geofoam slope projects with roadways of two or more lanes that conform to AASHTO design standards for highway geometry, it is very unlikely that sliding due to centrifugal force loads would be critical. However, it is recommended that any projects involving a geosynthetic layer in the upper portion of the fill system, such as a geomembrane separation layer between EPS-block geofoam fill and the pavement system, should implement a testing program to determine the interface friction angle between EPS-block geofoam and the particular geosynthetic that is to be used.

It may also be advisable to perform a centrifugal force sliding analysis for projects involving very narrow roadways with a short radius curve. The analysis in Appendix D assumed that the shear force that resisted sliding resulted from the weight of a pavement block that was roughly 20 ft. (6m) wide. Thus, any roadway narrower than 20 ft. (6m) should be checked to ensure that the weight of the pavement block beneath the applicable AASHTO design truck has sufficient weight to develop the necessary shear resistance at the critical interface.

The second failure mechanism, overturning about the toe of EPS-block geofoam fill, could not be analyzed in any generalized way because analysis is so heavily dependent on the specific geometry of the EPS-block geofoam fill system. However, because of the similarities between this failure mechanism and the failure mechanism of seismic overturning, it was concluded that projects which include an analysis of seismic overturning in the design process may neglect the consideration of overturning due to vehicle centrifugal forces. This conclusion is based on the fact that the analysis for seismic overturning considers essentially the same failure mechanism using different loads, which, in almost every instance, will be greater in magnitude than the centrifugal force loads. Thus, if the factor of safety against seismic overturning is found to be acceptable, it can be inferred that the factor of safety against overturning due to vehicle centrifugal forces will also be acceptable as well. However, it is recommended that projects that do not include an analysis of seismic overturning in the design process take into account the possibility of overturning due to vehicle centrifugal forces. This is especially true for those projects involving vertical-sided EPS fill.

Barrier & Railing Loads

Road hardware such as guardrails, barriers, median dividers, lighting, signage and utilities can be incorporated in the EPS-block geofoam slope system by direct embedment or structural anchorage. The alternatives for accommodating shallow utilities and road hardware (barriers and dividers, light poles, signage) is to provide a sufficient thickness of the pavement system to allow conventional burial or embedment within soil or, in the case of appurtenant elements, provide for anchorage to a PCC slab or footing that is constructed within the pavement section. Barriers or guardrails are typically required with vertical-sided embankments. Design of traffic railings is addressed in Section 13 of the Bridge Design Specifications (AASHTO, 2007) and in the AASHTO Road Design Guide (2002).

OVERVIEW OF DESIGN PROCEDURE

Table 4.4 provides a summary of slope stabilization case histories involving use of lightweight fill. As is evident from Table 4.4, EPS-block geofoam has been widely used as a lightweight fill material to improve stability of both soil and rock slopes. In addition to geofoam, a wide variety of other lightweight fill materials, including shredded tires, wood chips, and pumice, have also been successfully incorporated into slope stability projects around the world. The use of lightweight fill materials to improve slope stability can be successful in both soil and rock slopes. As indicated in Table 4.4, rock types where lightweight fill has been utilized consist of colluvium, talus, and shale. Therefore, the general rock type most suitable for use of lightweight fill appears to consist of soft and weathered rock.

In addition to the type of soil or rock present in each slope, the type of slip surface evaluated in the slope stability analysis was also considered. Based on a review of available case histories, it appears that a circular (rotational) failure surface is more common for lightweight fills applied to soil slopes, while a noncircular (translational) failure surface is more frequently applied to rock slopes. This holds true for EPS-block geofoam as well as other types of lightweight fill material. However, both rotational (See Figure 4.17) and translational (See Figure 4.18) modes of sliding can occur in both soil and rock slopes.

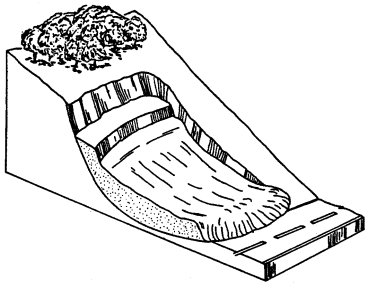
Rotational slides move along a surface of rupture that is curved and concave and generally exhibit a ratio of depth to length of the surface of rupture, D/L, between 0.15 and 0.33, as shown in Figure 4.19 (Abramson et al., 2002, Skempton and Hutchinson, 1969, Transportation Research Board, 1996).

Translational slides displace along a planar or undulating surface of rupture that is generally shallower than rotational slides and have D/L ratios less than 0.1 (Skempton and Hutchinson, 1969, Transportation Research Board, 1996). The displaced mass of a rotational slide typically reaches equilibrium, whereas a translational slide mass may remain active if the slip surface is sufficiently inclined (Transportation Research Board, 1996).

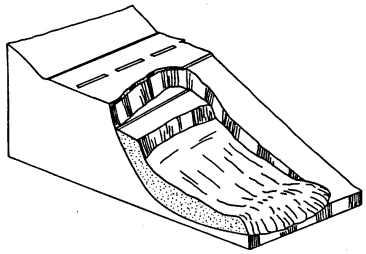
Table 4.4. Summary of lightweight fill in slope applications case histories.

Project Location	Lightweight Fill Type	Source	Slip Surface Material	Existing Slip Surface?	Type of Failure Surface	Lightweight Fill Modeled As:
Colorado: U.S. Highway 160	EPS-block geofoam	(Abramson et al., 2002, Transportation Research Board, 1996, Yeh and Gilmore, 1992)	Rock (*Weathered Shale overlying Mancos Shale*)	Yes	Translational	Surcharge
Japan	EPS-block geofoam	(Suzuki et al., 1996)	Rock (*Colluvium overlying tuff breccia*)	Yes	Translational	Surcharge
Yubari City, Japan: Naborikawa District	EPS-block geofoam	(Tsukamoto, 1996)	Rock (*Talus*)	Yes	Translational	Surcharge
Guin County, AL	EPS-block geofoam	(2006)	Soil	Yes	Rotational	Mohr-Coulomb (c = 1800 psf, φ = 0°)
Jewett, NY: Greene Co. Route 23A	EPS-block geofoam	(Jutkofsky, 1998, Jutkofsky et al., 2000, Negussey, 2002, Stark et al., 2004a)	Soil (*Layered silty clay overlying clayey silt, gravelly*)	Yes	Rotational	Surcharge
Bayfield County, WI: Trunk Highway A	EPS-block geofoam	(Reuter and Rutz, 2000, Stark et al., 2004a)	Soil (*Sand fill overlying silty clay*)	Yes	Rotational	Surcharge
Seattle, WA	EPS-block geofoam	(Stark and Mann, 2006)	Soil	Yes	Rotational	Surcharge
Washington State	Wood fiber	(Abramson et al., 2002, Nelson and Allen, 1974a, Nelson and Allen, 1974b, Transportation Research Board, 1996)	Soil (*Soft organic clay overlying various sand strata*)	Yes	NA	Mohr-Coulomb (*φ = 40°*)
Washington State: Stillaguamish Road	Wood fiber	(Permanent International Association of Road Congresses, 1997, Peterson et al., 1981)	NA	Yes	NA	Surcharge
Olympia, WA: Suiattle River Road	Wood fiber	(Permanent International Association of Road Congresses 1997; Peterson et al. 1981)	Rock	Yes	NA	Surcharge
Olympia, WA	Wood Fiber	(Kilian, 1984)	NA	Yes	Rotational	Mohr-Coulomb (*c = 3000 psf, φ = 25°*)
Oregon: U.S. 42	Shredded tires	(Read et al., 1991, Transportation Research Board, 1996)	NA	Yes	NA	Surcharge
Seattle, WA	Bottom ash	(Buechel and Yamane, 1989)	Soil (*Dense sand*)	Yes	Rotational	Mohr-Coulomb (*φ = 36°*)
Japan	Air foamed stabilized soil & expanded-beads mixed lightweight soil	(Miki, 2002, Nakano et al., 1999)	Soft Rock (*c = 5 tf/m^2, φ = 30°*)	No	Rotational	Surcharge
Salt Lake City, UT: I-15 Reconstruction	Pumice	(Sharma and Buu, 1992, Transportation Research Board, 1996)	Soil	No	Translational	Mohr-Coulomb
Winnipeg, Canada	Wood Fiber	(Coulter, 1975)	Soil	NA	NA (Settlement Only)	Mohr-Coulomb (*φ = 27.5°*)

Note: NA = Not Available

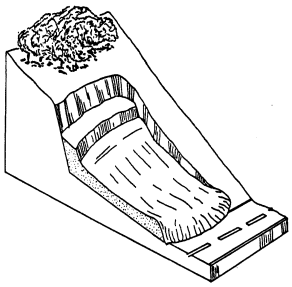


a) Rotational slide above roadway.

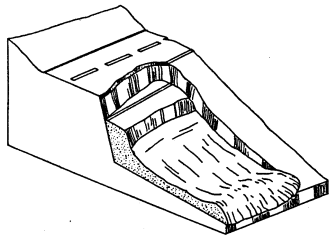


b) Rotational slide below roadway.

Figure 4.17. Rotational slides (Hopkins et al., 1988).



a) Translational slide above roadway.



b) Translational slide below roadway.

Figure 4.18. Translational slides (Hopkins et al., 1988).

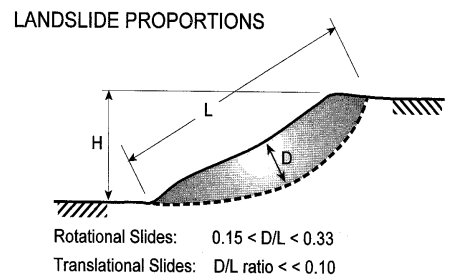


Figure 4.19. Landslide proportions. (From Slope Stability and Stabilization Methods, by Abramson et al., 2002; used by permission of John Wiley & Sons).

Slope instability may involve engineered fill and cut slopes. As shown in Figure 4.20, an engineered fill or embankment may cause instability by increasing the load, especially in the upper portion of the slide mass. If slope instability is anticipated due to proposed placement of a fill in the upper slope, use of EPS-block geofoam as a lightweight fill may be feasible.

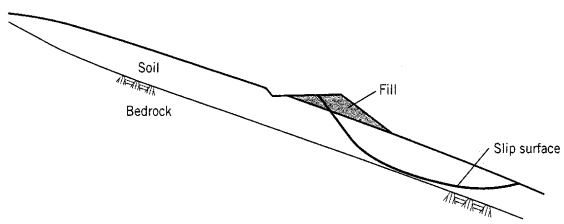


Figure 4.20. Landslide caused by fill placement. (From Landslides in Practice: Investigation, Analysis, and Remedial/Preventative Options in Soils, by Cornforth, D. H., 2005; used by permission of John Wiley & Sons).

Engineered cuts can cause instability by over-steepening the base of a slope on a soft foundation, causing the foundation to fail (Cornforth, 2005). Use of lightweight fill in the upper portion of the cut slope, as shown in Figure 4.21 prior to construction of the cut slope, may contribute to stability of an otherwise unstable cut slope. Use of EPS-block geofoam in a proposed cut slope may be especially beneficial if a steep cut slope is required because of right-of-way constraints.

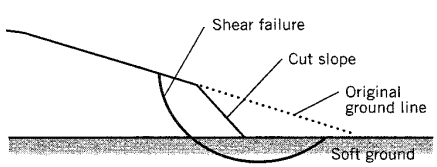


Figure 4.21. Landslide caused by a cut (From Landslides in Practice: Investigation, Analysis, and Remedial/Preventative Options in Soils, by Cornforth, D. H., 2005; used by permission of John Wiley & Sons).

The design requirements of EPS-block geofoam slope systems are dependent on the location of the existing or anticipated slip surface in relation to the location of the existing or proposed roadway. Figure 4.22 shows the recommended design procedure if the existing or proposed roadway is located within the existing or anticipated slide mass <u>and</u> the existing or anticipated slide mass is located below the roadway as shown in Figures 4.17(b) and 4.18(b), i.e., the roadway is near the head of the slide mass.

Figure.4.22. Recommended design procedure for the case of existing or proposed roadway located within the existing or anticipated slide mass and existing or anticipated slide mass is located below the roadway, i.e. roadway is near the head of the slide mass.

Figure 4.23 shows the recommended modified design procedure if the existing or proposed roadway is located outside the limits of the existing or anticipated slide mass and/or the existing or anticipated slide mass is located above the roadway as shown in Figures 4.17(a) and 4.18(a), i.e., the roadway is near the toe of the slide mass. It is anticipated that EPS-block geofoam used for this slope application will not support any structural loads, other than possibly soil fill above the blocks. Therefore,

the primary difference between the recommended design procedure in Figure 4.22 and the modified procedure in Figure 4.23 is that only failure mechanisms associated with external and internal instability failure modes, as shown in Table 4.2, need to be considered. The pavement system failure mode may not be an applicable failure mode, because if the roadway is near the toe of the slide mass, stabilization of the slide mass with EPS-block geofoam will occur primarily at the head of the slide and consequently, the EPS-block geofoam slope system may not include the pavement system. Therefore, Steps 7 and 8 of the full design procedure shown in Figure 4.22, which involves the pavement system, may not be required and is not part of the modified design procedure shown in Figure 4.23.

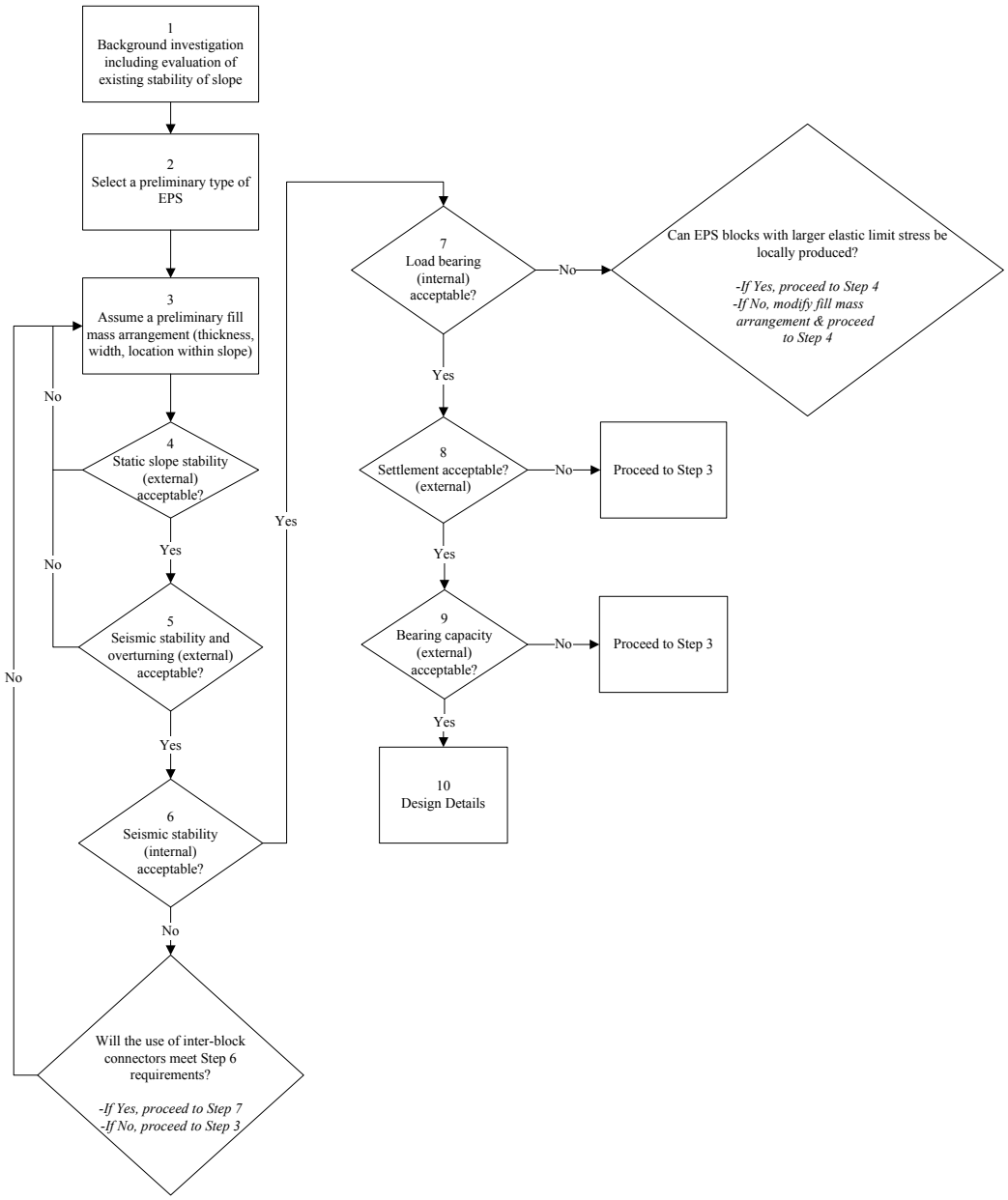


Figure 4.23. Modified design procedure for the case of the existing or proposed roadway located outside limits of the existing or anticipated slide mass and/or the existing or anticipated slide mass located above the roadway, i.e., roadway is near the toe of the slide mass.

Figure 4.24 shows a design selection diagram that can be used to determine whether to use the complete procedure shown in Figure 4.22 or the modified design procedure shown in Figure 4.23. Level I of the decision diagram indicates that the proposed design procedure is applicable to both remedial repair and remediation of existing unstable soil slopes involving existing roadways, as well as for design of planned slopes involving new roadway construction. Level II of the decision diagram indicates that for existing roadways the use of EPS-block geofoam will typically only involve unstable slopes. However, for new roadway construction, use of EPS-block geofoam may involve an existing unstable slope or an existing stable slope that may become unstable during or after construction of the new roadway. Level III categorizes the location of the existing or anticipated slide mass location in relation to the existing or proposed new roadway. Level IV indicates the location of the roadway in relation to the existing or anticipated slide mass.

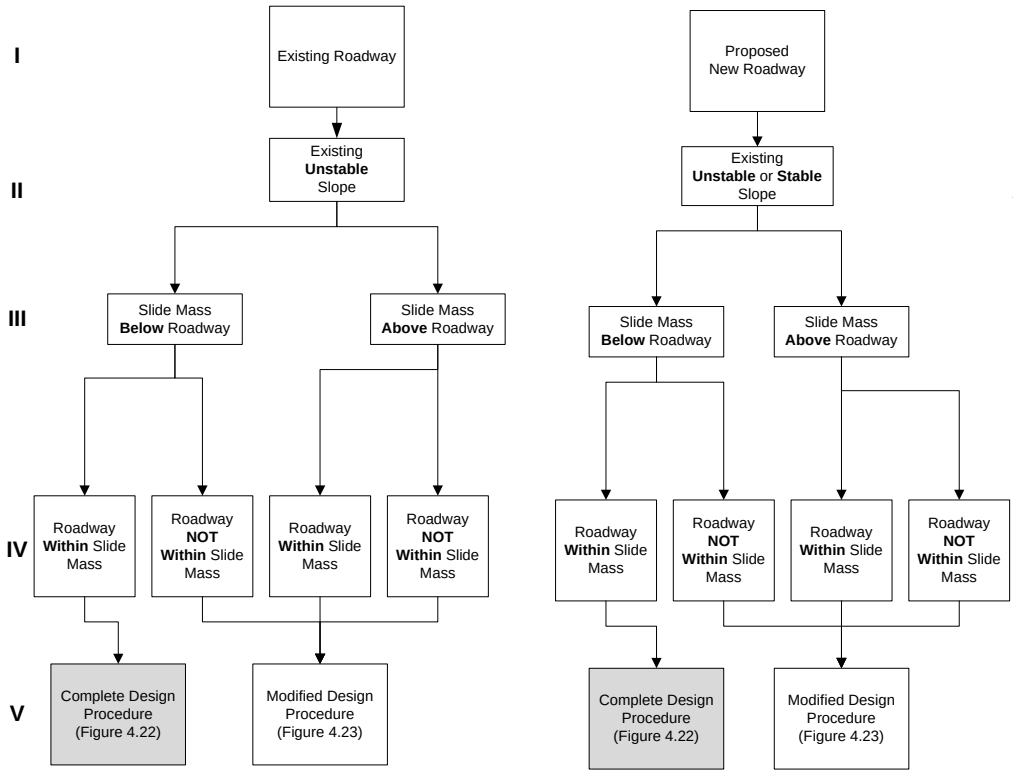


Figure 4.24. Design procedure selection diagram.

Level V indicates the recommended design procedure that can be used for design. As shown in Figure 4.24, the complete design procedure shown in Figure 4.22 is applicable if the existing or proposed roadway is located within the existing or anticipated slide mass <u>and</u> the existing or anticipated slide mass is located below the roadway as shown in Figures 4.17(b) and 4.18(b), i.e., the roadway is near the head of the slide mass. The modified design procedure shown in Figure 4.23 is applicable if the existing or proposed roadway is located outside the limits of the existing or anticipated slide mass <u>and/or</u> the existing or anticipated slide mass is located above the roadway as shown in Figures 4.17(a) and 4.18(a), i.e., the roadway is near the toe of the slide mass.

The proposed design procedure shown in Figures 4.17, 4.18, and Figures 4.22 through 4.24 was introduced in a presentation titled *"A Framework for the Design Guideline for EPS-Block Geofoam in Slope Stabilization and Repair"* at the 22nd Annual Meeting of the Tennessee Section of ASCE in 2009, and at the 89th Annual Meeting of the Transportation Research Board (TRB) held in January 2010. The corresponding TRB paper was included in the meeting compendium of papers. Additionally, TRB

accepted the paper for publication in the 2010 Transportation Research Record, Journal of the Transportation Research Board (Arellano et al., 2010).

As part of the effort to simplify the design procedures shown in Figure 4.22 and 4.23, the two design algorithms included in these figures were consolidated into a single algorithm, as shown in Figure 4.25. The differences between the two design procedures shown in Figures 4.22 and 4.23 are shaded in Figure 4.25 to facilitate understanding and usage. Therefore, the full design procedure, which is applicable if the existing or proposed roadway is located *within* the limits of the existing or anticipated slide mass and/or the existing or anticipated slide mass is located *below* the roadway as shown in Figure 4.17(b) and 4.18(b), consists of all the design steps. If the existing or proposed roadway is located *outside* the limits of the existing or anticipated slide mass and/or the existing or anticipated slide mass is located *above* the roadway as shown in Figure 4.17(a) and 4.18(a), the design procedure does not include Steps 8 and 9, which are directly related to design of the pavement system, because the EPS-block geofoam slope system may not include a pavement system. Steps 8 and 9, which are associated with the pavement system, are shaded in Figure 4.25 to help differentiate between the complete design procedure shown in Figure 4.22 that includes Steps 8 and 9 and the simplified procedure shown in Figure 4.23 that does not include Steps 8 and 9.

One challenge of slope stabilization design with lightweight fill is to determine the volume and location of EPS blocks within the slope that will yield the required level of stability or factor of safety at the least cost. Because EPS-block geofoam is typically more expensive than soil on a cost-per-unit-volume basis for the material alone, it is desirable to optimize the volume of EPS used yet still satisfy design criteria concerning stability. Therefore, to achieve the most cost-effective design, a design goal is to use the minimum amount of EPS blocks required to meet stability requirements. Therefore, Steps 3 and 4 were added that specifically include the optimization of the volume and location of the EPS blocks within the slope in the overall design procedure as shown in Figure 4.25.

In summary, Figure 4.25 shows the recommended design procedure for EPS-block geofoam slope fills. The following two key revisions were made to the two design algorithms included in Figures 4.22 and 4.23. First, the two design algorithms included in Figures 4.22 and 4.23 have been consolidated into a single algorithm to facilitate understanding and usage. Second, Steps 3 and 4 were added that specifically include optimization of the volume and location of EPS blocks within the slope.

The design of an EPS-block geofoam slope system requires consideration of the interaction between the three major components of an EPS-block slope system shown in Figure 4.4, i.e., existing slope material, fill mass, and pavement system. Because of this interaction, the design procedure involves interconnected analyses between the three components. For example, some issues of pavement system design act in opposition to some design issues involving external and internal stability of an EPS-block geofoam slope system, because a robust pavement system is a benefit for the long-term durability of the pavement system, but the larger dead load from a thicker pavement system may decrease the factor of safety of the failure mechanisms involving external and internal stability of the geofoam slope system. Therefore, some compromise between failure mechanisms is required during design to obtain a technically acceptable design.

However, in addition to the technical aspects of the design, cost must also be considered. Because EPS-block geofoam is typically a more expensive material than soil on a cost-per-unit-volume basis for the material alone, it is desirable to optimize the design to minimize the volume of EPS used, yet still satisfy the technical design aspects of the various failure mechanisms. It is possible in concept to optimize the final design of both the pavement system and the overall EPS block slope system considering both performance and cost so that a technically effective and cost efficient geofoam slope system is obtained. However, because of the inherent interaction between components, overall design optimization of a slope incorporating EPS-block geofoam requires iterative analyses to achieve a technically acceptable design at the lowest overall cost. In order to minimize the iterative analysis, the design algorithm shown in Figure 4.25 was developed. The design procedure depicted in this figure considers a pavement system with the minimum required thickness, a fill mass with the minimum thickness of EPS-block geofoam, and use of

an EPS block with the lowest possible density. Therefore, the design procedure will produce a cost-efficient design.

The recommended design procedure is applicable for both slope-sided fills and vertical-sided fills as depicted in Figures 4.2(a) and 4.2(b), respectively, except that overturning of the entire fill mass at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation material as a result of horizontal forces that is part of external seismic stability, Step 6, is applicable primarily for only vertical-sided fills. A summary of each design step of the proposed design procedure shown in Figure 4.25 is subsequently provided.

Figure 4.25.Complete design procedure for EPS-block geofoam slope fills.

STEP 1: BACKGROUND INVESTIGATION

The purpose of the background investigation step is to obtain and gather information required to determine the feasibility of using EPS-block geofoam as a lightweight fill alternative in the proposed slope, as well as to design and construct the EPS-block geofoam slope system. This step consists of project site evaluation and criteria selection.

The extent of site evaluation required on a project will be dependent on the type of project, i.e., new structure or existing structure. According to Turner and McGuffy (1996), field investigations of landslides may be conducted for two distinct purposes:

1. "When new facilities are planned, to identify areas that are potentially or currently subject to landsliding; in the case of transportation facilities, this investigation would be conducted during the route selection phase."
2. "When a landslide is adjacent to a facility, to define the landslide dimensions, features, and characteristics and to assess environmental factors that may contribute to the landsliding."

Details about the site investigation process for landslides can be found in various landslide texts (Abramson et al., 2002, Cornforth, 2005, Transportation Research Board, 1996). Additionally, the process of soil and rock property selection can be found in the FHWA Geotechnical Engineering Circular No. 5 (Sabatini et al., 2002). If the purpose of lightweight fill is to remediate an existing slide, the average shear strength along the existing slip surface can be back-calculated using back-analyses methods. If the project involves selection of a method of slope stabilization, Duncan and Wright (2005) recommend following factors be considered:

1. "What is the purpose of stabilizing the slope? Is it only to prevent further large movements, or is it to restore the capacity of the moving ground to provide firm support for structures or pavements? It is more difficult to restore the load-carrying capacity of the ground than merely to stop movements, particularly when the ground has already been disrupted by large movements."
2. "How much time is available? Is it essential that the repair be accomplished quickly: for example to open a blocked highway, railroad, or canal, or is time a less critical element? If time is of the essence, expeditious methods that can be undertaken without delay are the only ones appropriate. If time is not so critical, it may be possible to fine-tune the fix through study and to devise a less expensive solution for the problem. If it is possible to wait until the dry season before undertaking permanent repair, it may be feasible to use methods, such as excavation of the sliding mass and reconstruction of the slope, that make the slope temporarily steeper."
3. "How accessible is the site, and what types of construction equipment can be mobilized there? If the site is reachable only by small roads, or by water, or if steep terrain rules out the use of heavy equipment, considerations of access may limit the methods of stabilization that can be used."
4. "What would be the cost of the repair? If the costs exceed the benefits, can less expensive methods be used? Unless political factors dictate otherwise, it is illogical to stabilize a slope when the costs exceed the benefits."

When attempting to evaluate the feasibility of using EPS-block geofoam for a slope stabilization project, it is important to consider some of the unique characteristics of EPS-block geofoam as a construction material. For example, experience has demonstrated that EPS-block geofoam can be placed extremely quickly. Once the site is prepared, the actual process of moving and positioning EPS blocks requires minimal equipment and manpower. EPS-block geofoam blocks can be transported and placed easily, even at many project sites that would be inaccessible to heavy equipment. Although some specific safety measures may have to be implemented, placement of EPS blocks can be continued in almost any kind of weather, whereas many other slope stabilization methods may be delayed by rain or snow.

Another important consideration is the fact that EPS-block geofoam is a manufactured construction material that can be produced by the molder and then stockpiled at a designated site until it is needed. Therefore, a state DOT agency could potentially store a supply of EPS blocks that could be used for emergency landslide mitigation or repair. Also, EPS blocks can be molded in advance of the actual placement date and either transported immediately when needed, or stockpiled at the site for immediate use. Thus, use of EPS blocks in slope application projects can easily contribute to an accelerated construction schedule.

In addition to site evaluation, Step 1 includes criteria selection. Establishment of project criteria involves selecting a desired design life for the proposed EPS-block geofoam slope system, estimating design loads, selecting an appropriate factor of safety with respect to the various failure mechanisms that must be considered in the design, selecting desired settlement tolerances, and evaluating the geometric requirements of the proposed EPS-block fill mass.

It is recommended that similar design life ranges recommended for reinforced soil slopes also be adopted for EPS-block geofoam slope systems. Permanent EPS-block geofoam slope systems can be designed for a minimum service life of 75 years and temporary systems can be designed for a service life of 36 months or less (Elias et al., 2001). However, it should be noted that based on the current state of knowledge, actual design life may be greater than 75 years because EPS is inherently non-biodegradable and will not dissolve, deteriorate, or change chemically in the ground or ground water. The first project to use block-molded EPS as a lightweight fill material was the Flom Bridge project in Norway in 1972. The EPS-block geofoam was used to rebuild a road over soft soil that had chronic settlement problems. The rebuilt road performed flawlessly until 1996 when the road was relocated. The EPS-block geofoam was exhumed from the Flom Bridge project by the NRRL. The exhumed EPS showed no degradation after 24 years of in-ground service, including portions of the geofoam that were permanently submerged (Aabøe, 2000a, Aabøe, 2000b, Aabøe, 2007, Frydenlund and Aabøe, 2001). Therefore, EPS blocks can be reused indefinitely. For example, EPS blocks used as a temporary fill in one location may be reused as a temporary or permanent fill in another location. Also, based on the current state of knowledge, no maintenance of EPS blocks is required.

Guidelines for selection of design loads were presented in the Design Loads section of this chapter. Recommended minimum factors of safety for use in analyzing the various failure mechanisms of the design procedure shown in Figure 4.25 are included in the appropriate design step discussion.

An evaluation of the geometric requirements of the proposed EPS-block fill mass considers requirements of the local transportation agency and limitations due to site-specific restrictions. These requirements and restrictions will be used in Step 3 to select a preliminary fill mass arrangement and will also be considered throughout the remaining steps of the design process as various iterations of the fill mass arrangement are evaluated to obtain a fill mass arrangement that will satisfy the design criteria for the various failure mechanisms that are analyzed in each design step. Project-specific design inputs, such as right-of-way constraints, limiting impact on underlying and/or adjacent structures, and construction time, usually govern the overall cross-sectional geometry of the fill. For example, the use of a vertical-sided fill will minimize the impact to nearby structures including underground utilities. An assessment should be made of any adjacent structures, utilities and transportation facilities (roads, railroads), both existing and proposed, that may be affected by the loads imposed on the ground by the proposed EPS-block geofoam slope system.

STEP 2: SELECT A PRELIMINARY TYPE OF EPS AND ASSUME A PRELIMINARY PAVEMENT SYSTEM DESIGN

The second step of the design procedure is to select a preliminary type of EPS-block geofoam and to design a preliminary pavement system. As indicated in the Dead (Gravity) Loads section of this chapter, although EPS-block geofoam can be manufactured to various densities, the preliminary design can be based on a density of 20 kg/m^3 (1.25 lbf./ft^3). Therefore, the dry unit weight of the EPS can be taken to be 200 N/m^3 (1.25 lbf./ft^3) or a unit weight at 1 percent water content by volume of 300 N/m^3

(1.9 lbf./ft^3) for preliminary design. However, for some failure mechanisms, the unit weight based on long-term water absorption such as the unit weight at 1 percent water content instead of the dry unit weight may be more appropriate.

As noted in the Dead (Gravity) Loads section of this chapter, use of the dry unit weight versus unit weight at 1 percent water content by volume in design will depend on the failure mechanism being evaluated. The value for the unit weight that results in the lower factor of safety is the unit weight that should be used for that particular failure mechanism. Table 3.3 provides a summary of the densities based on a water content of 1 percent by volume associated with the various dry block densities for use in estimating dead loads. It should be noted that these water contents are expressed on a volumetric basis rather than a gravimetric basis. That is, the water content of the EPS is given as the ratio of the volume of air contained in the sample to its total volume. This volumetric expression of water content, although rarely used in geotechnical engineering, is common in the foam manufacturing industry.

Although the pavement system has not been designed at this point, the preliminary pavement system can be assumed to be 1m (3 ft.) thick and the various component layers of the pavement system be assumed to have a total unit weight of 20 kN/m^3 (130 lbf./ft^3) for initial design purposes. The basis for this initial assumption is included in the discussion of dead (gravity) loads. The final pavement system will be based on a pavement structural analysis as part of Step 8.

STEP 3: OPTIMIZE VOLUME & LOCATION OF EPS FILL

The third step of the design procedure is to determine a preliminary fill mass arrangement. Because EPS-block geofoam is typically more expensive than soil on a cost-per-unit-volume basis for the material alone, it is usually desirable to optimize the volume of EPS used, yet still satisfy design criteria concerning stability. Therefore, to achieve the most cost-effective design, a design goal for most projects is to use the minimum amount of EPS blocks possible that will satisfy the requirements for external and internal stability. The analyses of all external and internal stability failure mechanisms shown in Table 4.2 are based on verifying that the initial depth and extent of existing slope material removal and resulting EPS-block geofoam fill configuration will provide an overall stable slope.

The determination of optimal volume and location of EPS blocks will typically require iterative analysis based on various locations and thicknesses until a cross section that yields the minimum volume of lightweight fill is obtained. However, other factors will also impact the final design volume and location of EPS blocks such as:

- Construction equipment access to perform excavation work,
- Ease of accessibility for EPS block delivery and placement,
- Impact on traffic if lightweight fill will be incorporated below an existing roadway, and
- Right-of-way constraints and/or constraints due to nearby structures.

It should be noted that although minimization of EPS volume is the goal on most projects, for some projects it may be desirable to maximize the use of EPS. For example, economization of EPS volume may not be a concern in some emergency slope repair projects or projects with an accelerated construction schedule.

A minimum of two layers of blocks should be used beneath roads because a single layer of blocks can shift under traffic loads and lead to premature failure (Horvath, 1999). Block thicknesses typically range between 610mm (24 in.) to 1000mm (39 in.). Therefore, it is recommended that a minimum of two EPS blocks with a thickness of 610mm (24 in.) each or a total initial height of 1.2m (4 ft.) be considered for the EPS block height to determine the preliminary fill mass arrangement. Therefore, the preliminary fill mass arrangement can consist of the preliminary pavement system thickness of 1m (3 ft.) and the thickness of two EPS blocks of 1.2m (4 ft.). The thickness of EPS-block geofoam may change as various

iterations of the fill mass arrangement are evaluated to obtain a fill mass arrangement to satisfy the design criteria of various failure mechanisms analyzed in each supplemental design step shown in Figure 4.25.

For engineered fill embankments constructed on slopes, if the previously suggested initial preliminary fill mass arrangement will not yield the required finished grade and additional fill material is required, the preliminary fill mass arrangement can consist of the pavement system, EPS-block geofoam, and an underlying layer of natural fill. The preliminary height of natural fill is the total embankment height required based on the background investigation less the preliminary pavement system thickness of 1m (3 ft.) and less the thickness of two EPS blocks of 1.2 m (4 ft.).

The preliminary width and location of the EPS-block geofoam fill mass within the slope will be dependent on the results of the evaluation of the preliminary geometric requirements of the proposed EPS-block fill mass performed as part of Step 1. The most effective location of the lightweight fill mass will be near the head (upper portion) of the existing slide mass or proposed slope because reducing the load at the head by removing existing earth material and replacing it with a lighter fill material will contribute the most to reducing the destabilizing forces that tend to cause slope instability. The location of the fill mass within the slope selected in this step is only preliminary because the location of the fill mass, as well as the thickness, may change as various iterations of the fill mass arrangement are evaluated to obtain a fill mass arrangement to satisfy the design criteria of various failure mechanisms analyzed in each supplemental design step shown in Figure 4.25.

In some projects the volume and location of EPS blocks within the slope will be constrained by previously indicated factors. For example, for the case of the existing road that is located within the existing slide mass and existing slide mass located below the roadway as shown in Figures 4.17(b) and 4.18(b), i.e., the roadway is near the head of the slide mass, location of the EPS fill mass will typically be limited within the existing roadway location because of right-of-way constraints. However, in some projects the volume and location of EPS within the slope may not be obvious and may require that various iterations of the fill mass arrangement be evaluated to obtain a fill mass arrangement to satisfy the design requirements of various failure mechanisms analyzed in each design step shown in Figure 4.25. Therefore, as part of this Project 24-11(02), a study was performed to develop a procedure for optimizing the volume and location of EPS blocks within the slope to minimize the number of iterations that may be required to satisfy the design criterion.

Appendix C presents two procedures for optimizing the volume and location of EPS blocks within the slope. The optimization methods presented in Appendix C were inspired by the Moment Reduction Method developed by Negussey and Srirajan (2001). Although the Moment Reduction Method can be used to approximate the optimum volume and location for EPS fill, the procedure is relatively time-consuming to perform and it is applicable only to slides involving circular slip surfaces. In addition to these limitations, Negussey and Srirajan (2001) do not provide any rigorous explanation to demonstrate that this procedure identifies the true optimum volume and location for EPS fill. Thus, one of the goals for developing an alternative optimization method was to provide designers with a simple, easy-to-use method that could be used to identify the true optimum volume and location for an EPS-block geofoam fill to achieve a target factor of safety.

Two separate optimization methods were developed herein, one for slides involving rotational and the other for translational slides. Both methods are described in detail in Appendix C. The first method was developed based on the Ordinary Method of Slices, also known as Fellenius' Method. The second method was developed based on the Simplified Janbu Method. A suggested approach for implementing each procedure using Microsoft Excel® spreadsheet software is also provided in Appendix C.

The purpose of the optimization methods is only to obtain an approximate location within the slope where placement of EPS blocks will have the greatest impact in stabilizing the slope while requiring the minimum volume of EPS block. A separate static slope stability analysis must be performed as part of Step 5 of the design procedure as shown in Figure 4.25 with a better slope stability analysis method that preferably satisfies full equilibrium such as Spencer's method. Step 5 should be relied on to verify that the overall slope configuration meets the desired factor of safety.

Appendix C provides the results of a comparison study between the optimization procedures with actual case histories. The results of the comparison study revealed that optimizing the volume and location of the EPS-block geofoam slope fill can result in significantly lower material costs for a project. This is an advantage because EPS-block geofoam is typically more expensive than conventional fill materials. Thus, it is desirable to design EPS-block geofoam fill to be as efficient as possible in terms of the volume of EPS that needs to be used.

It should be noted that these cost comparisons are based on material costs only, which are usually higher for geofoam than other fill materials such as soil. On some projects, the higher geofoam material cost may be offset by project cost savings due to accelerated construction times, less construction equipment and personnel costs, and less field QA/QC costs made possible by use of EPS-block geofoam. However, even on these projects, optimizing the volume and location of EPS blocks can produce significant additional material cost savings. The proposed optimization procedures for slides involving both rotational and translational failure surfaces provide useful tools that can be used to identify the optimum volume and location for EPS-block geofoam fill, which designers can adopt to meet specific project requirements.

Another advantage of the optimization procedures presented in Appendix C is that ground water is explicitly considered in the derivation of each constraint equation. Thus, the optimization process not only accounts for the change in the weight of the slice due to the addition of geofoam, but it also accounts for lowering of the ground water level in that slice that occurs when drainage is installed behind or below the EPS and the EPS fill extends below the ground water table. The recommended design procedure requires a subsurface drainage system for EPS blocks placed below the ground water level to eliminate buoyancy and hydrostatic forces. Because this subsurface drainage is required by the design procedure, it was specifically accounted for in derivation of each of the optimization constraints.

Figure 4.25 shows a design algorithm that incorporates the proposed optimization methods as separate steps, i.e., Steps 3 and 4. Step 3 consists of performing one of the optimization procedures described in Appendix C. Step 4 consists of modifying the optimized EPS fill as needed for constructability.

The optimization procedures presented in Appendix C are optional within the proposed design procedure shown in Figure 4.25. In lieu of performing one of the optimization procedures, the designer can select a preliminary volume and location of EPS blocks within the slope and proceed with Step 5.

STEP 4: MODIFY OPTIMIZED EPS FILL AS NEEDED FOR CONSTRUCTABILITY

This step consists of evaluating the EPS block configuration obtained from the optimization procedure described in Appendix C and performing minor alterations to the fill mass configuration to ensure that the fill mass is constructible. As will be apparent upon examination of the optimization procedure described in Appendix C, it is unlikely that results of the optimization procedure can be used to obtain the final EPS block configuration because the configuration obtained from the optimization procedures described in Appendix C may be impractical to replicate in the field. In some cases, the geometry of the optimized EPS fill may simply be too complicated to be manufactured or constructed with a reasonable amount of time and effort. However, the optimized EPS-block geofoam fill geometry obtained will still provide a useful starting point from which to design an EPS geofoam fill that is both efficient and constructible. Minor alterations may be required to adapt the optimized EPS fill design to the specific project requirements and site restrictions.

STEP 5: STATIC SLOPE STABILITY (EXTERNAL)

Overview of External Static Slope Stability Analysis

Once a preliminary fill mass arrangement is determined, then the primary failure mechanism that needs to be evaluated is overall (external) slope stability, because this is typically the primary reason for

considering the use of EPS blocks in slope applications. Thus, as shown in Figure 4.25, overall external static slope stability is Step 5 of the proposed design procedure.

The purpose of this step is to analyze the EPS-block geofoam fill configuration that was developed in Steps 2 through 4 to ensure that the proposed slope system will have an acceptable factor of safety. In most cases, this required factor of safety will be specified by the policy of the supervising agency, such as the state DOT. Typical values for the required factor of safety range from as low as 1.15 for slide remediation projects to as high as 1.5 for new slope construction. Chapter 3 presents a summary of various methods available for modeling shear strength of EPS blocks for external static slope stability.

Design for external static stability considers potential slip surfaces involving the existing soil slope material only as shown in Figure 4.5, as well as potential slip surfaces that involve both the fill mass and existing slope material as shown in Figure 4.6. Of course, if the use of lightweight fill is to remediate an existing slide, the existing slip surface will be the critical slip surface that should be evaluated. However, slip surfaces such as those shown in Figures 4.5 and 4.6 should also be considered to evaluate the impact of the lightweight fill mass configuration on the short- and long-term stability of the slope.

Conventional limit equilibrium methods can be used to evaluate the stability of potential slip surfaces involving the existing soil slope material only as shown in Figure 4.5. Slip surfaces such as slip surfaces 1 and 3 evaluate the impact of the EPS block system on the overall stability of the slope and stability of the lower portion of the slope, respectively. Slip surface 2 evaluates both the short-term and long-term behavior of the upper slope immediately adjacent to the EPS-block fill system to ensure that no applied earth forces from the adjacent earth are applied to the EPS fill.

Limit equilibrium methods can also be used to evaluate the external static stability of slip surfaces involving both the fill mass and the existing slope material as shown in Figure 4.6. However, one current disadvantage of using limit equilibrium methods of analysis to evaluate the slip surfaces that extend through the fill mass and existing slope material is the uncertainty in modeling the strength of EPS blocks. This issue is discussed in Chapter 3.

As shown in Figure 4.6, potential slip surfaces that extend through both the fill mass and existing slope material to be evaluated include slip surfaces through the upper slope and EPS blocks (slip surface 2) as well as slip surfaces through the EPS blocks and lower slope material (slip surface 1). However, if the upper slope material is designed to be stable for both short-term and long-term conditions, an analysis of a potential slip surface such as shown by slip surface 2 in Figure 4.6 should yield a stable condition. The design procedure is based on a self-stable adjacent upper slope. If the adjacent upper slope material cannot be cut to a long-term stable slope angle, an earth-retention system must be used to resist the applied earth force. Various types of earth-retention systems are included in the Internal Instability Failure Mode discussion of this chapter.

Additionally, note that a slip surface entirely within the EPS blocks need not be considered and is not shown in Figure 4.6 because there is little or no static driving force applied along the horizontal portion of the internal failure surfaces, since the horizontal joints are assumed to be completely horizontal and typical static loads are vertical. The fact that embankments with vertical sides can be constructed demonstrates the validity of this conclusion. Therefore, the focus of external static stability analyses involving both the fill mass and existing slope material will be on slip surfaces involving EPS blocks and lower slope material, as depicted by slip surface 1 in Figure 4.6.

Only circular slip surfaces are shown in Figures 4.5 and 4.6. Based on a review of available lightweight fill case histories, it appears that circular (rotational) slip surfaces are more common for lightweight fills applied to soil slopes, while noncircular (translational) slip surfaces are more frequently associated with rock slopes. This holds true for EPS-block geofoam as well as other types of lightweight fill material. Table 4.4 provides a summary of slip surface geometries considered in various lightweight fill case histories.

The procedure for the evaluation of external static slope stability will typically consist of the following two phases:

- Evaluate existing slope conditions.
- Evaluate the proposed stability of the slope for various lightweight fill configurations and determine the optimum quantity and location of EPS-block geofoam that will yield the desired stability. The design objective in this phase is to determine an optimum EPS block configuration and location within the slope that will result in the lowest cost. Thus, it is desirable to minimize the volume of EPS used, yet still satisfy the technical design aspects of external static slope stability. If the desired stability is not obtained using lightweight fill alone, consider additional remediation alternatives in conjunction with lightweight fill.

Ground Water Considerations

As indicated in the External Instability Failure Mode section of this chapter, based on current design precedent, it is recommended that all EPS-block geofoam slope systems incorporate drainage systems to prevent water from accumulating above the bottom of EPS block and divert seepage water from the adjacent upper slope material. The key ground water issues related to stability of the EPS-block geofoam slope system is how to determine the impact of the drainage system on long-term ground water conditions and how to include the resulting piezometric conditions in slope stability analysis. The long-term ground water regime can be obtained by performing a flow analysis based on a drainage system located below the EPS-block fill mass and adjacent to the fill mass between the fill mass and the upper slope material.

After the ground water regime is determined, the piezometric conditions need to be included in the slope stability analysis. The method of incorporating piezometric conditions will be partially dependent on the model used to represent the shear strength of EPS blocks in limit equilibrium analysis. The various models available for modeling shear strength of EPS blocks for external static slope stability analysis are summarized in Chapter 3.

For example, if Alternative 1, 2, 3 or 5 is used to model the shear strength of the blocks, it is possible to perform a slope stability analysis of the EPS-block fill mass system using an effective-stress approach. However, if Alternative 4 is used to model the shear strength of the blocks, it may be better to use a total-stress approach for the EPS-block fill mass with boundary water pressures and an effective-stress approach for surrounding natural material. However, it may not be possible to perform a dual total-stress and effective-stress slope stability analysis with currently available commercial slope stability software. It is recommended that the issue of incorporating piezometric conditions be further evaluated as part of any research performed to develop an appropriate shear strength model.

STEP 6: SEISMIC STABILITY AND OVERTURNING (EXTERNAL)

Introduction

Although seismic stability analysis is not specifically included in the Japanese slope design procedure shown in Figure 4.3, it is a failure mechanism that is considered in Japanese practice (Miki, 2002, Nakano et al., 1999). Therefore, external seismic stability analysis is Step 6 and immediately follows external static stability analysis (Step 5). However, as previously indicated in the Seismic Loads section of this chapter, the requirement to mitigate slopes that may become unstable during a seismic event will depend on the state DOT seismic design policy. For example, the WSDOT policy is to only stabilize slopes that could impact an adjacent structure if failure due to seismic loading occurs (Washington State Department of Transportation, 2006).

Seismic loading is a short-term event that is typically considered in geotechnical problems including road embankments. Seismic loading can affect both external and internal stability of an embankment containing EPS-block geofoam. Most considerations for seismic external stability analyses for embankments constructed of EPS-block geofoam are the same as those for earth materials. These considerations include various SLS and ULS mechanisms, such as seismic settlement and liquefaction,

that are primarily independent of the nature of embankment material because they depend on the seismic risk at a particular site and the nature and thickness of the natural soil overlying the bedrock. A discussion of these topics can be found in the FHWA Geotechnical Circular No. 3 (Kavazanjian et al., 1997). A discussion on ground improvement to reduce potential seismic-induced subgrade problems can be found in various publications (Elias et al., 1999, Holtz, 1989, Kavazanjian, et al., 1997). However, several failure mechanisms are specific to EPS-block geofoam fills such as sliding, overturning, and internal load bearing. Additionally, the seismic behavior of EPS-block geofoam fills is different from earth fills.

The external seismic stability failure mechanisms include slope instability involving slip surfaces that include: the existing slope material only, as shown in Figure 4.5, and/or both the fill mass and the existing slope material, as shown in Figure 4.6, horizontal sliding of the entire EPS-block geofoam fill mass as shown in Figure 4.7, overturning of a vertical-sided embankment as shown in Figure 4.8, bearing capacity failure of existing foundation earth material as shown in Figure 4.9, and settlement of existing foundation material as shown in Figure 4.10.

The research related to the seismic stability of EPS-block geofoam slope systems consisted of two primary objectives: (1) to develop an analysis procedure that incorporates the impact of seismic inertial forces from adjacent slope material on the EPS-block geofoam fill mass slope system and (2) to evaluate the applicability of the simplified seismic response methodology used for geofoam stand-alone embankments to geofoam slope applications. The results of the former research objective are described as part of this design step, Step 6, while results of the latter research objective are described in Step 7.

Overview of Seismic Slope Stability Analysis

The pseudo-static stability analysis procedure that is typically used to evaluate the seismic stability of soil slopes and embankments can also be used to evaluate external seismic stability of EPS-block geofoam slopes. This method involves modeling the earthquake shaking with an equivalent static horizontal and/or vertical force (F_h and F_v, respectively) that acts permanently, not temporarily, on the slope. The horizontal and vertical forces (F_h and F_v, respectively) equal the slide mass or the mass of the vertical slice (m) multiplied by the appropriate seismic acceleration (a_h or a_v), i.e., $F = m \cdot a$, as shown by Equations 4.19 and 4.20. These equations also show the relation between the seismic accelerations and seismic coefficients, which are sometimes referenced in seismic stability literature, and may also be provided by local seismic design codes and guidelines:

$$F_h = \frac{a_h W}{g} = k_h W \tag{4.19}$$

$$F_v = \frac{a_v W}{g} = k_v W \tag{4.20}$$

where

F_h, F_v	=	equivalent static force in the horizontal or vertical direction, respectively
a_h, a_v	=	selected ground acceleration, usually some fraction of PGA
W	=	weight of EPS geofoam fill + weight of pavement system
g	=	acceleration due to gravity
k_h, k_v	=	horizontal and vertical seismic coefficients, respectively

The selected ground acceleration can be obtained by performing a seismic ground shaking hazard analysis. As noted by the WSDOT, the four types of seismic ground shaking hazard analysis include:

(1) use of a specification/code based hazard with specification/code based ground motion response, (2) use of a specification/code based hazard with site-specific ground motion response, (3) use of site-specific hazard with specification/code based ground motion response, and (4) use of site-specific hazard with site-specific ground motion response (Washington State Department of Transportation, 2010). The AASHTO (2010) specifications provide the conditions when a site-specific hazard analysis and/or a site-specific ground motion response analysis should be considered. Additionally, the AASHTO (2010) specifications also provide a procedure for performing a seismic ground shaking hazard analysis based on a specification/code based hazard with specification/code based ground motion response analysis.

The primary assumption of the pseudo-static analysis procedure is that the all of the effects of the ground motion produced by the earthquake can be accounted for by incorporating the static forces determined from Equations 4.19 and 4.20 into the analysis of the various potential failure mechanisms involving seismic stability of the EPS-block geofoam fill mass. A discussion of these failure mechanisms is included later within this Step 6 section.

As shown in Figures 4.5 and 4.6, potential slip surfaces involving the existing slope material only as well as slip surfaces involving both the fill mass and the existing slope material should be considered as part of external seismic slope stability. A pseudo-static seismic slope stability analysis involves application of a horizontal and/or vertical force to the center of gravity of the critical slide mass and in the direction of the exposed slope. If a stability method is used that involves dividing the slide mass into vertical slices, the horizontal and vertical forces are applied to the center of gravity of each vertical slice that simulates the inertial forces generated by the ground motion. The pseudo-static horizontal and vertical force must be applied to the slide mass that is delineated by the critical static failure surface. The steps in a pseudo-static slope stability analysis are summarized below:

1. Locate the critical static failure surface(s), i.e., the static failure surface with the lowest factor of safety that passes through the existing slope material only as well as a slip surface involving both the fill mass and the existing slope material, using a slope stability method that satisfies all conditions of equilibrium, e.g., Spencer's stability method (1967). This value of factor of safety should satisfy the required value of static factor of safety before initiating pseudo-static analysis.
2. Modify the static shear strength values for cohesive or liquefiable soils situated along the critical static failure surface to reflect a strength loss due to earthquake shaking.
3. Determine the equivalent horizontal and vertical force using Equations 4.19 and 4.20 that will be applied to the center of gravity of the critical static failure surface. Some slope stability software programs will calculate these static forces directly from the seismic accelerations. If a stability method is used that involves dividing the slide mass into vertical slices, the horizontal and vertical forces are applied to the center of gravity of each vertical slice.
4. Calculate the pseudo-static factor of safety, *F'*, for the critical static failure surface and ensure it meets the required value. The Naval Facilities Engineering Command (1986) indicates that for transient loads, such as earthquakes, safety factors as low as 1.2 or 1.15 may be tolerated. Day (2002) indicates that in southern California, a minimum factor of safety of 1.1 to 1.15 is considered acceptable for a pseudo-static slope stability analysis. A factor of safety between 1.0 and 1.2 is indicated by Kavazanjian (1997). The WSDOT recommends a minimum factor of safety of 1.1 for slopes involving or adjacent to walls and structure foundations and a minimum factor of safety of 1.05 for other slopes (cuts, fills, and landslide repairs) (Washington State Department of Transportation, 2010). The safety of factor required will most likely vary from state to state. Therefore, local Departments of Transportation factor of safety requirements for seismic stability should be used.

The vertical pseudo-static force is typically neglected in pseudo-static analysis of earth slopes because the vertical force generally alternates between reducing and increasing both the driving and resisting forces in the slope. Consequently, it generally has a significantly smaller influence on the resulting factor of safety than the horizontal force (Kramer, 1996).

In addition to the evaluation of seismic stability of the overall EPS-block fill mass, i.e., external seismic stability, the failure mechanisms of horizontal sliding of the entire EPS-block fill mass, overturning of vertical embankments, bearing capacity failure of the existing foundation, and settlement of the existing foundation material due to seismic loading should also be evaluated. However, before presenting these failure mechanisms, the procedure for determining the seismic inertia force from adjacent slope material is subsequently described because an estimate of seismic inertia forces is needed to evaluate the horizontal sliding and overturning failure mechanisms.

Seismic Inertia Forces from Adjacent Slope Material

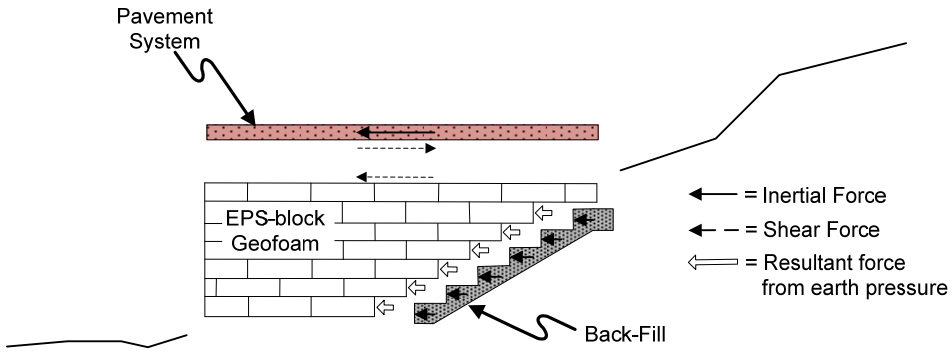
Existing seismic design procedures for the use of EPS-block geofoam are based on the work performed predominantly on stand-alone embankments over soft soils and does not consider the influence of the adjacent slope material. Therefore, the first seismic-related research objective was to develop an analysis procedure that incorporates the impact of seismic inertia forces from adjacent slope material on the EPS-block geofoam fill mass slope system. The Japanese manual (Public Works Research Institute, 1992) suggests use of external seismic forces as an approach to consider the seismic effect of the soil behind the EPS-block fill mass as shown in Figure 4.26. However, the Japanese manual does not provide guidance to specifically determine the inertia forces.

The recommended procedure for considering the effect of the natural slope material on the external and internal seismic stability of the EPS-block geofoam slope system proposed in the design guideline in Appendix B consists of determining the magnitude of the seismic earth pressure based on the Mononobe-Okabe (M-O) method.

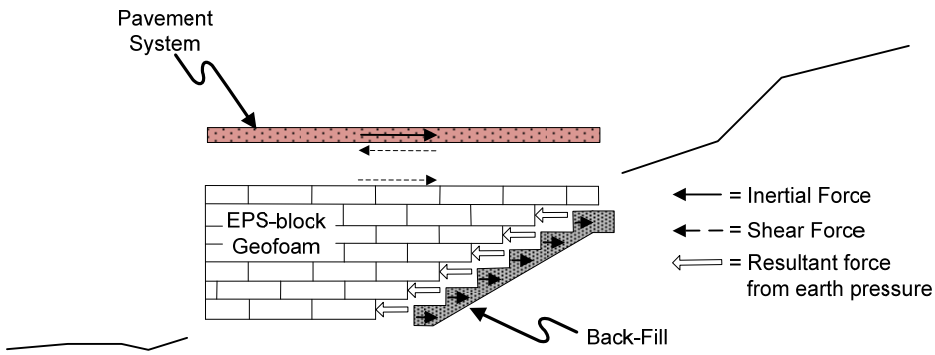
As noted in the FHWA GEC No. 3 (Kavazanjian et al., 1997), the most commonly used method for seismic design of retaining structures is the pseudo-static method developed by Okabe (1926) and Mononobe (1929), which is commonly known as the M-O method. This method is based on three key assumptions (Kavazanjian et al., 1997). First, the M-O method is based on the Coulomb earth pressure theory. Therefore, the wall is assumed to move sufficiently to induce active earth pressure conditions. Second, the backfill is completely drained and cohesionless. Third, the effect of earthquake motion is represented by a pseudo-static inertia force.

The first assumption that active earth pressure conditions exist is probably reasonable for geofoam slope applications because the EPS-block geofoam fill mass will behave more closely as a flexible retention system compared to conventional gravity and semi-gravity cantilever walls, for which active earth pressures are typically assumed in static and seismic design. Active earth pressure conditions are also assumed for the seismic external stability of MSE walls (AASHTO, 2010).

As previously noted in the Internal Instability Failure Mode section of this chapter, the recommended design procedure for geofoam slopes is based on a self-stable adjacent upper slope material under static conditions to prevent earth pressures from developing on the EPS fill mass that can result in horizontal sliding between blocks. However, backfill material, which typically consists of cohesionless material, is typically required between the EPS fill mass and adjacent natural slope material, as shown in Figure 4.27. Therefore, under static conditions, the EPS fill mass will be subjected to lateral earth pressures from the backfill material. Under seismic conditions, the EPS-block fill mass will be subjected to both static forces and inertia forces from both the backfill and natural slope material based on the M-O method. It should be noted that although the proposed design guideline in Appendix B is based on a stable adjacent slope, it is possible to design an EPS-block geofoam slope system that will support a portion of the upper adjacent slope by transferring the loads through the assemblage of EPS blocks to a structural earth retention system, as indicated in Figure 4.11.



a) Inertial force is oriented away from natural slope.



b) Inertial force is oriented toward natural slope.

Figure 4.26. Relationship between inertial forces and earth pressure (EDO, 1994).

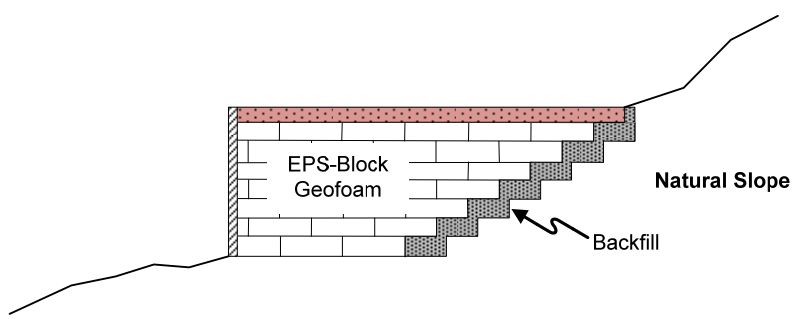


Figure 4.27. EPS-block geofoam fill mass, backfill, and adjacent natural slope material.

The second assumption regarding a completely drained and cohesionless backfill may not be strictly applicable to all site conditions. The recommended design procedure for EPS-block geofoam slopes is based on use of a drainage system to prevent water from accumulating above the bottom of EPS blocks and between the adjacent upper slope material and EPS blocks to collect and divert seepage water and thereby alleviate seepage pressures. If a cohesionless backfill material is utilized between the EPS fill mass and adjacent natural slope material in conjunction with a drainage system, the applicability of this assumption is based on the soil properties of the upper slope material adjacent to the backfill as well as

the post-construction seepage conditions within the adjacent upper slope material that will occur after the drainage system is operational.

Adherence to the third assumption that the effect of earthquake motion is represented by a pseudo-static inertia force is based on the assumption that the Coulomb active wedge within the adjacent backfill and upper slope material behind the EPS-block fill mass is a rigid block whereby the ground acceleration is fully transmitted. However, natural earth backfill material and natural soil slope material is not rigid. Additionally, the seismic peak acceleration only exists for a short time, and depending on the backfill and natural slope properties, amplification and deamplification effects may occur. The recent NCHRP 611 study suggests incorporating height-dependent effects to the seismic coefficients used in the pseudo-static analysis and design of retaining walls (Anderson et al., 2008). However, the proposed height-dependent modifications to the seismic coefficients are not incorporated in the most recent AASHTO Bridge Design Specifications (AASHTO, 2010). Therefore, the recommended approach for determining the earth pressures induced by the backfill and natural slope material on the EPS-block geofoam fill mass due to earthquakes is the M-O pseudo-static approach, which is the same approach recommended in the AASHTO Bridge Design Specifications (AASHTO, 2010) for design of free-standing abutments.

In addition to the above-described three assumptions inherent in the M-O method, another assumption is needed to apply the method to EPS-block geofoam slope systems. It is assumed that the EPS-block fill mass and materials directly above the EPS blocks together behave as a retaining wall system. Therefore, the M-O method can be used to estimate the seismic induced lateral forces imposed by the backfill and natural slope material onto the EPS-block fill mass and materials directly overlying the EPS blocks.

The Coulomb earth pressure theory, on which the M-O seismic analysis approach is based, only applies to walls with planar back surfaces. However, for the case of retaining walls with a stepped back wall, little error is involved if the earth pressure is assumed to act on an assumed planar surface extending from the top of the stepped wall to the heel of the wall (Huntington, 1948). Figure 4.28 shows the assumed back of wall of the EPS-block geofoam fill mass used in the M-O method. As shown, the back of wall is assumed to be along the bottom exterior edges of the EPS blocks. Figure 4.28 also shows the wall geometry, wedge force diagram, and forces associated with the M-O method.

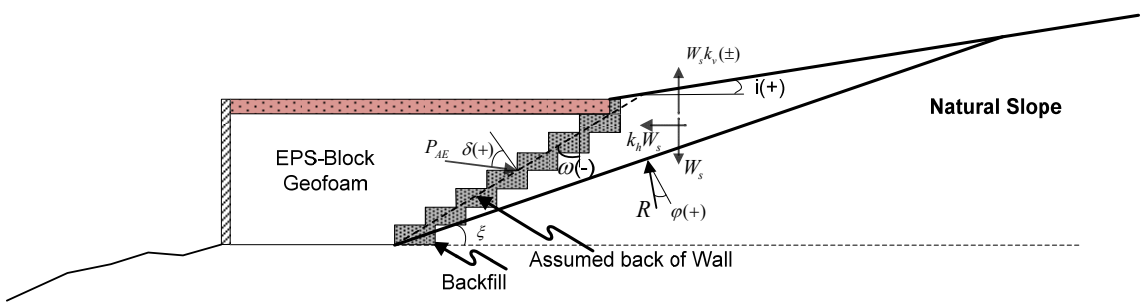


Figure 4.28. Forces behind the EPS-block geofoam mass in the Mononobe-Okabe (M-O) Method.

Equation 4.21 provides the total (static and dynamic) earthquake active earth pressure coefficient, K_{AE} and Equation 4.22 provides the M-O relationship for the total active earth pressure force.

$$K_{AE} = \frac{\cos^2(\phi - \psi - \omega)}{\cos\psi \cos^2\omega \cos(\delta + \omega + \psi)} \times \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \psi - i)}{\cos(\delta + \psi + \omega)\cos(i - \omega)}} \right]^{-2} \quad \textbf{(4.21)}$$

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v), \quad \textbf{(4.22)}$$

where

ϕ = friction angle of soil (deg),

δ = friction angle between soil and assumed wall (deg),

$\psi = \arctan(\frac{k_h}{1 - k_v})$,

ω = slope of the asummed back of wall to the vertical (deg) (Note that ω is negative in sign in this case.),

i = back of wall slope angle (deg),

k_h = horizontal acceleration coefficient,

k_v = vertical acceleration coefficient,

P_{AE}=total (static and dynamic) active earth pressure force,

R=resultant of the shear and normal forces on the surface of failure,

W_s=effective weight of the soil wedge,

H = Height of the soil face, and

γ = Unit weight of soil.

As shown in Figure 4.28, the seismic inertia force, which is represented by $W_s k_v (\pm)$, can act in an upward or downward direction, and both cases should ideally be considered. As noted in the NCHRP Report 611 (Anderson et al., 2008), the effect of vertical seismic loading is traditionally neglected because the rationale for neglecting vertical loading is generally attributed to the fact that the higher frequency vertical accelerations will be out of phase with the horizontal accelerations and will have positive and negative contributions to wall pressures, which on average can reasonably be neglected for design.

The M-O method provides the magnitude of the total (static and dynamic) active seismic earth pressure force but not a specific force location nor an equivalent pressure distribution behind the wall. The location of the resultant force and an equivalent earth pressure distribution are needed for external and internal seismic stability analysis of EPS-block geofoam slopes. The M-O total (static and dynamic) earthquake active earth pressure can be separated into a static and a seismic component. Richards and Elms (1979) indicate that Seed and Whitman (1970) suggested that the resultant of the static component of the active earth pressure acts at H/3 from the bottom of the wall, and that the dynamic effect can be taken to act at a height of 0.6H above the base. The recent AASHTO manual also indicates that the resultant of the static component of the active earth pressure with no earthquake effects may be taken as H/3 (AASHTO, 2010). The typical pressure distribution that is assumed for the static active earth pressure is a triangular pressure distribution that is 0 at the top and $K_A \gamma H$ at the bottom.

The WSDOT geotechnical design manual (Washington State Department of Transportation, 2010) also indicates that the resultant of the dynamic component is 0.6H from the bottom and that the pressure distribution for the dynamic effect is an inverted trapezoid with the pressure at the top of $0.8 \Delta K_{ae} \gamma H$ and the pressure at the bottom of $0.2 \Delta K_{ae} \gamma H$. Therefore, the locations of the static and seismic component of active earth pressure shown in Figure 4.29 are recommended for analysis of geofoam slope systems. Equation 4.23 provides the static component of the active earth pressure coefficient.

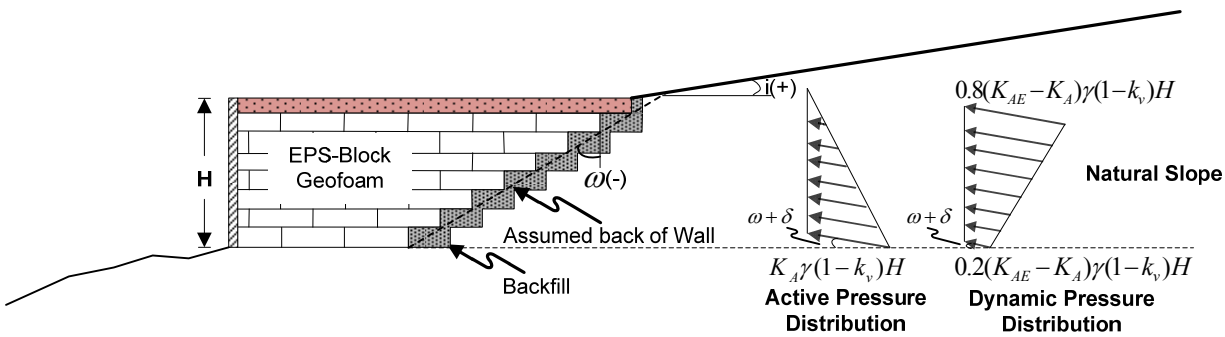


Figure 4.29. Static and dynamic components of active earth pressure.

$$K_A = \frac{\cos^2(\phi-\omega)}{\cos^2\omega\cos(\delta+\omega)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-i)}{\cos(\delta+\omega)\cos(i-\omega)}}\right]^2} \tag{4.23}$$

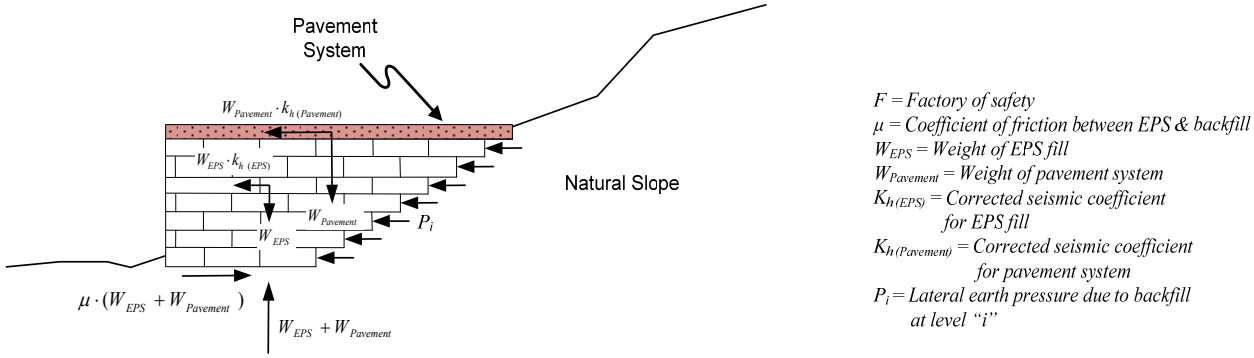
Note that in the M-O equation for calculating the total active force (see Equation 4.22), the unit weight of the soil is multiplied by $1\text{-}K_v$ to account for seismic acceleration effects. Therefore, to maintain consistency with the M-O equation, the pressure diagrams shown in Figure 4.29 includes the $1\text{-}K_v$ correction for both the static as well as dynamic components of earth pressure.

After the static and seismic (dynamic) components of active earth pressure are determined, the external seismic stability failure mechanisms can be analyzed.

Horizontal Sliding

Horizontal sliding analysis considers potential sliding of the entire EPS-block geofoam fill mass as shown by Figure 4.7. Figure 4.30 shows the recommended model for evaluating external sliding stability. The Japanese design manual indicates that if the width of the bottom of the EPS-block fill mass is small compared with the height and if the area is susceptible to rocking, the friction coefficient should be reduced and the required factor of safety against sliding must be increased (Public Works Research Institute, 1992). However, no further guidance is provided about the magnitude of friction coefficient reduction and factor of safety increase to utilize.

The horizontal earth pressure from any backfill material placed between the natural slope material and the EPS-block fill mass is considered in the sliding, overturning, and bearing capacity analysis if the width of the backfill material for each step of EPS fill mass is relatively smaller than the height (Public Works Research Institute, 1992). Figure 4.26 depicts the inertia effects of the backfill material on the EPS-block geofoam fill mass. As shown in Figure 4.26, the inertial earth pressure will be less than the static earth pressure if the inertia force is applied in the direction of the EPS blocks, and the inertial earth pressure will be greater if the inertia force is applied in the direction of the adjacent natural slope material. Therefore, for design it appears that the dynamic earth pressure should be based on the condition of the inertia force applied in the direction of the adjacent slope material. Based on the results of shake-table tests performed by the Japanese Public Works Research Institute, the height of backfill may not affect the dynamic earth pressure (Nomaguchi, 1996). Mechanical metal connectors were included between layers of EPS blocks in these dynamic tests.



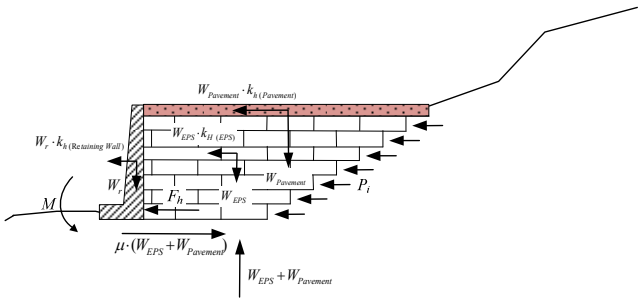
$$F = \frac{\mu(W_{EPS} + W_{Pavement})}{W_{EPS} \cdot k_{h\,(EPS)} + W_{Pavement} \cdot k_{h\,(Pavement)} + \Sigma P_i} \geq Allowable\ \ value$$

Figure 4.30. External seismic stability for sliding (Public Works Research Institute, 1992).

An earth retention system such as a retaining wall can also be included in the design to increase the stability against sliding and overturning. Figure 4.31 provides a model that can be used to analyze a retaining wall system subjected to seismic loading. A detailed discussion of the resulting static pressure diagram on an abutment or retaining wall is included in Horvath (1995) and in the Project 24-11(01) report (Stark et al., 2004a).

Overturning

For tall and narrow vertical embankments, overturning of the entire embankment at the interface between the bottom of the assemblage of EPS blocks and underlying foundation material as a result of seismic forces should be considered, as depicted in Figure 4.8. Figure 4.32 shows the recommended model for evaluating external overturning. The Japanese design manual indicates that if the width of the bottom of the EPS-block fill mass is small compared with the height, and if the area is susceptible to rocking, the required factor of safety against overturning must be increased (Public Works Research Institute, 1992). However, no further guidance as to the degree to which the factor of safety should be increased is provided.

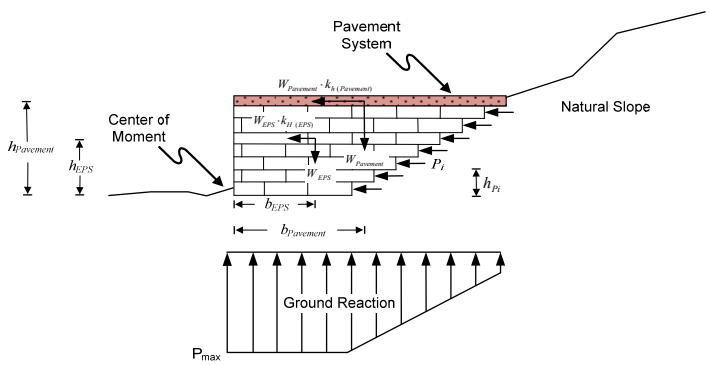


M = Moment acting on retaining wall
F = Factory of safety
M = Coefficient of friction between EPS & backfill
W_{EPS} = Weight of EPS fill
b_{EPS} = Horizontal distance from vertical face to center of gravity of EPS fill mass
h_{EPS} = Distance from bottom of EPS fill to center of gravity of EPS fill mass
$K_{h\,(EPS)}$ = Corrected seismic coefficient for EPS fill
$W_{Pavement}$ = Weight of pavement system
$b_{Pavement}$ = Horizontal distance from vertical face to center of gravity of pavement system
$h_{Pavement}$ = Vertical distance from bottom of EPS fill to center of gravity of pavement system
$K_{h\,(Pavement)}$ = Corrected seismic coefficient for pavement system
W_r = Weight of retaining wall
h_r = Distance from bottom of EPS fill to center of gravity of retaining wall
$K_{h\,(Retaining\ Wall)}$ = Corrected seismic coefficient for retaining wall
F_h = Horizontal force acting on retaining wall
P_i = Lateral earth pressure due to backfill at level "i"
h_{Pi} = Distance from bottom of EPS fill to resultant force from lateral earth pressure at level "i"
P_{max} = Maximum ground reaction force

$$M = (W_{EPS} \cdot k_{h\,(EPS)} \cdot h_{EPS}) + (W_{Pavement} \cdot k_{h\,(Pavement)} \cdot h_{Pavement}) + (W_r \cdot k_{h\,(\mathrm{Retaining\ Wall})} \cdot h_r) + (\Sigma P_i \cdot h_{Pi}) - (W_{EPS} \cdot b_{EPS})$$

$$F_h = (W_{EPS} \cdot k_{h\,(EPS)}) + (W_{Pavement} \cdot k_{h\,(Pavement)}) + (W_r \cdot k_{h\,(\mathrm{Retaining Wall})}) + \Sigma P_i - \mu (W_{EPS} + W_{Pavement})$$

Figure 4.31. Seismic stability of EPS supported by retaining wall (Public Works Research Institute, 1992).



F = Factory of safety
W_{EPS} = Weight of EPS fill
b_{EPS} = Horizontal distance from vertical face to center of gravity of EPS fill mass
h_{EPS} = Distance from bottom of EPS fill to center of gravity of EPS fill mass
$K_{h\,(EPS)}$ = Corrected seismic coefficient for EPS fill
$W_{Pavement}$ = Weight of pavement system
$b_{Pavement}$ = Horizontal distance from vertical face to center of gravity of pavement system
$h_{Pavement}$ = Vertical distance from bottom of EPS fill to center of gravity of pavement system
$K_{h\,(Pavement)}$ = Corrected seismic coefficient for pavement system
P_i = Lateral earth pressure due to backfill at level "i"
h_{Pi} = Distance from bottom of EPS fill to resultant force from lateral earth pressure at level "i"
P_{max} = Maximum ground reaction force

$$F = \frac{W_{EPS} \cdot b_{EPS} + W_{Pavement} \cdot b_{Pavement}}{W_{EPS} \cdot k_{h\,(EPS)} \cdot h_{Pavement} + W_{Pavement} \cdot k_{h\,(Pavement)} \cdot h_{Pavement} + \Sigma P_i \cdot h_{Pi}} \geq Allowable\ value$$

Figure.4.32. Seismic stability for overturning (tipping) and bearing capacity (Public Works Research Institute, 1992).

Bearing Capacity

Bearing capacity failure of the existing foundation earth material as shown in Figure 4.9 due to seismic loading and, potentially, a decrease in the shear strength of foundation material can be considered using the model shown in Figure 4.32. The Japanese design manual indicates that if the width of the bottom of the EPS-block fill mass is small compared with the height, and if the area is susceptible to rocking, the required factor of safety against bearing capacity must be increased (Public Works Research Institute, 1992). However, no further guidance is provided about the magnitude of factor of safety increase to utilize.

Settlement

Potential settlement of the existing foundation material as shown by Figure 4.10 should also be considered. The settlement that is performed as part of external seismic stability analysis considers earthquake-induced settlements. These earthquake-induced settlements include those resulting from liquefaction, seismic-induced slope movement, regional tectonic surface effects, foundation soil compression due to cyclic soil densification, and increase due to dynamic loads caused by rocking of the fill mass (Day, 2002). Methods for considering these earthquake-induced settlements can be found in various references (Day, 2002, Kavazanjian et al., 1997).

Summary

The general external seismic analysis procedure consists of performing a pseudo-static analysis to evaluate overall slope instability. The effect of the natural slope material on external horizontal sliding of the entire EPS-block fill mass and overturning of a vertical-sided embankment proposed herein consists of determining the magnitude of seismic earth pressure based on the M-O method. Bearing capacity failure and settlement of the existing foundation material are also part of external seismic stability.

Geotechnical engineering seismic analysis and design is currently going through a state of flux. For example, the FHWA is currently updating the FHWA Geotechnical Circular No. 3 (Kavazanjian et al., 1997). The FHWA and AASHTO are currently updating the guidance for seismic design to promote the use of Generalized Limit Equilibrium (GLE) and limit the use of M-O method. For conditions where the M-O method is not applicable, such as for cases with nonhomogeneous soils and complex slope geometry, the GLE procedure can be used with conventional limit-equilibrium slope stability programs (Kavazanjian et al., 1997). Additionally, AASHTO is currently considering an update to the seismic design guidance for retaining walls that is included in the 2010 AASHTO Bridge Design Specifications. Therefore, the general seismic analysis procedure for EPS-block geofoam slope systems may also require updating as changes to FHWA and AASHTO seismic design guidance are implemented.

STEP 7: SEISMIC STABILITY (INTERNAL)

Introduction

Internal seismic stability (Step 7) follows external seismic stability (Step 6). The main difference between internal and external seismic stability is that in internal seismic stability analysis, sliding is assumed to occur only within the EPS fill mass. Two failure mechanisms that involve internal stability of the EPS-block geofoam fill mass include horizontal sliding between blocks and/or between the pavement system and the upper layer of blocks, as shown by Figure 4.33, and load-bearing failure of the EPS blocks as shown in Figure 4.15. The load-bearing failure due to seismic loading is caused by rigid body seismic rocking, as shown in Figure 4.14, that can lead to excessive compressive normal stresses as shown in Figure 4.13. Therefore, the failure mechanisms that are considered for internal seismic stability analysis include horizontal sliding between blocks and/or between the pavement system and upper layer of blocks, and load-bearing failure of EPS blocks.

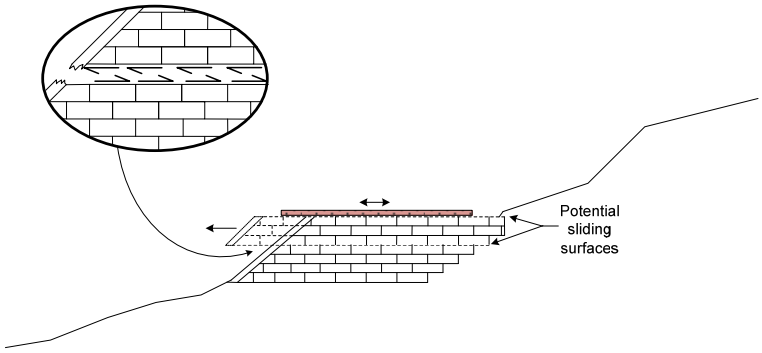


Figure 4.33. Internal seismic stability failure involving horizontal sliding between blocks and/or between the pavement system and upper layer of blocks due to seismic loading.

Internal seismic stability design of EPS-block geofoam slopes consists of determining the seismic-response acceleration of the existing natural slope material and EPS-block geofoam fill mass and evaluating the various potential failure mechanisms indicated above. The current state-of-practice of internal seismic stability analysis is to decouple the determination of the overall seismic response acceleration into the determination of the seismic response of natural slope material, followed by the seismic response of the EPS-block fill mass. Additionally, it is current state-of-practice to evaluate each potential seismic failure mechanism separately. Therefore, internal seismic stability analysis and design of EPS-bock geofoam slope systems can be separated into the following three primary steps: (7i) estimating the seismic-response acceleration at the existing ground surface or base (subgrade level) of the EPS fill mass by performing a site-specific assessment, (7ii) estimating the seismic-response acceleration at the top of the EPS fill mass, (7iii) performing pseudo-static stability analyses of the various failure mechanisms.

The Japanese procedure for seismic design of EPS-block geofoam slopes is based on the assumption that the natural adjacent slope material remains stable during and after an earthquake (Public Works Research Institute, 1992). Therefore, the Japanese seismic design procedure considers only the behavior of the EPS-block geofoam fill mass and any backfill material that is placed between the EPS-block fill mass and adjacent natural slope material. Additionally, the Japanese design procedure does not appear to consider the seismic inertial forces from the adjacent slope material, i.e., seismic interaction between the adjacent natural slope material and EPS-block geofoam fill mass, nor of any potential protective facing material that may incorporated in the fill mass design. However, the procedure for considering the effect of the natural slope material on external stability that consists of determining the magnitude of the seismic earth pressure, based on the M-O method presented in Step 6, can also be used for internal seismic stability. A procedure for incorporating the impact of the protective facing material is provided by Riad and Horvath (2004) and is presented herein. An overview of the three-step seismic analysis procedure is subsequently presented.

Estimating the Seismic-Response Acceleration of the Existing Ground Surface or Base (Subgrade Level) of the EPS fill mass (Step 7i)

The seismic-response acceleration of the existing ground surface can be determined by performing one of the four types of seismic ground shaking hazard analysis described in the Step 6 in the Overview of Seismic Slope Stability Analysis discussion.

Estimating the Seismic-Response Acceleration at the Top of the EPS Fill Mass (Step 7ii)

After estimating the free surface motion acceleration in Step 7i, the seismic-response acceleration at the top of the EPS-block geofoam fill mass must be estimated. The majority of published information related to seismic response of EPS-block geofoam embankments has been focused on stand-alone embankments with vertical sides, and not on side-hill embankments. A majority of studies were performed in Japan and involved both small-scale and full-scale shake-table tests and numerical analyses, as well as observation of the actual behavior of EPS-block geofoam embankments (Hotta et al., 1998, Nishi et al., 1998, Nomaguchi, 1996a).

More recent work on the seismic behavior of stand-alone embankments has been performed in the U.S. as part of the CA/T Project (Horvath, 2004a, Horvath, 2004b, Riad and Horvath, 2004) and the I-15 Reconstruction Project in Salt Lake City, UT (Bartlett and Lawton, 2008). A numerical simulation study was also performed by Kalinski and Pentapati (2006) and Kojima and Maruoka (2002) on the dynamic behavior of EPS-block geofoam embankments over soil susceptible to liquefaction. A study on the dynamic properties of EPS-block geofoam was performed by Athanasopoulos et al., (1999).

A careful review and interpretation of the extensive Japanese research indicates that the response of an assemblage of EPS blocks can exhibit a complex combination of flexible and rigid behavioral characteristics depending on the specific overall geometry. The present state of knowledge is to uncouple the flexible and rigid behavioral components and analyze them separately. If the EPS geofoam fill mass was rigid, the acceleration at the top of the fill mass would equal the acceleration at the base of the embankment. If the EPS-block fill mass is flexible, acceleration at the top of an EPS-block fill mass will generally not equal the acceleration at the base of the fill mass. The acceleration at the top of the fill mass could be greater or less than the base acceleration, depending on the response of the EPS-block fill mass. However, the acceleration at the top of an EPS-block fill mass will typically be greater than the free surface motion acceleration at the base of the fill mass.

The two approaches available for determining seismic-response acceleration of the EPS-block geofoam fill mass can be categorized into a simplified method and a detailed method (Horvath, 1995). The research on which both the simplified and detailed methodologies are based on has focused on applications where the fill mass is composed predominantly of EPS blocks. As it is possible for EPS blocks to be limited to a portion of a slope cross-section, applicability of both the simplified and detailed seismic response methods to cross-sections that consist partly of EPS blocks requires further study.

Simplified Seismic Response Method

For situations in which EPS-block geofoam is used as a lightweight fill and placed beneath a pavement structure, the pavement system comprises the vast majority of the overall lightweight system's mass, resulting in a structure that is extremely top-heavy. Thus, when a cyclic lateral load is applied, such as that produced by an earthquake, the fill mass structure has a tendency to sway, as shown in Figure 4.34. This means that the ground motion experienced by the foundation soil can be amplified at certain frequencies to such a degree that the EPS-block fill and the pavement system it supports may actually "feel" a vibration of much greater amplitude than that "felt" by the soil immediately beneath it. Therefore, it is current state-of-practice to model the EPS-block geofoam fill mass as a classical single-degree-of-freedom (SDOF) system (Horvath, 1995, Horvath, 2004a, Riad, 2005b, Riad and Horvath, 2004):

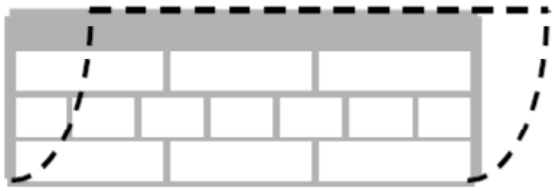


Figure 4.34. Lateral sway due to seismic-inertia force (Horvath, 2004a; From "Lessons learned from failure: EPS geofoam," Geotechnical Fabrics Report [now Geosynthetics magazine], Oct/Nov 2004, volume 22, number 8. Reprinted with permission).

Figure 4.35 depicts the SDOF model. As shown, the EPS-block fill structure can be modeled as an equivalent fixed-end, cantilevered beam. The Japanese design manual indicates that for vertical-sided fills in slope applications, the height of the equivalent cantilevered beam to be used in Figure 4.35 should be selected so that the model has the same cross-sectional area as the EPS-block geofoam fill in the actual fill mass arrangement, as shown below in Figure 4.36.

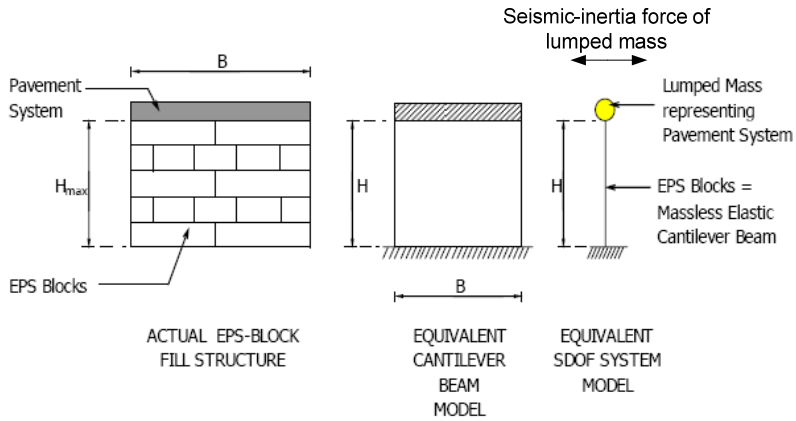
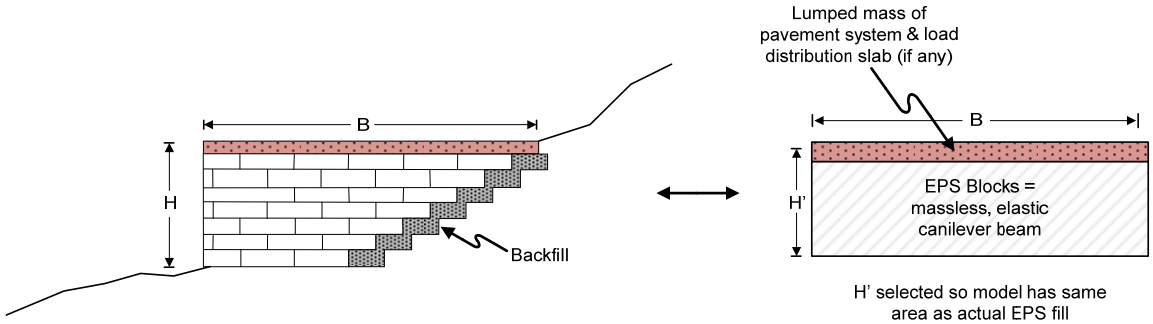


Figure 4.35. SDOF model for EPS (Riad, 2005a; used with permission from ASCE).



a) Actual structure *b) Model*

Figure 4.36. Approximate (simplified) seismic modeling of an EPS fill (Horvath, 1995, Public Works Research Institute, 1992).

The simplified method of determining the seismic-response acceleration of the EPS-block geofoam fill mass consists of the following steps: a) calculate the fundamental period of the EPS-block geofoam fill mass system, b) determine the site-response acceleration spectrum of the EPS-block geofoam fill mass system, c) determine the horizontal acceleration of the lumped mass, d) calculate the seismic-inertia force produced by horizontal acceleration acting on the lumped mass (Horvath, 1995, Riad and Horvath, 2004).

The fundamental period of the equivalent SDOF system model can be obtained from Equation 4.24. Any consistent set of units may be used with this equation. As noted by Horvath (2004b), both flexural and shear components of bending are considered in the derivation of the equation.

$$T_0 = 2\pi \left\{ \frac{\sigma'_{v_0} H}{E_{t_i} g} \left[4\left(\frac{H}{B}\right)^2 + \left(\frac{12}{5}\right)(1+\upsilon) \right] \right\}^{1/2} \quad \textbf{(4.24)}$$

where

T_0 = resonant period of the SDOF system

H = height of embankment

E_{t_i} = initial tangent Young's modulus of the EPS

g = gravitational constant = 9.81 m/s^2 = 32.2 ft./s^2

B = embankment width

υ = poisson's ratio for the EPS (typically taken to be ≈ 0.1 within the elastic range as is applicable for lightweight-fill applications)

The resonant frequency is the reciprocal of the fundamental period and can be obtained from the equation below:

$$f_0 = \frac{1}{T_0} \quad \textbf{(4.25)}$$

where

f_0 = resonant frequency of the SDOF system

The Japanese design manual suggests that the SDOF model can also be used to estimate the seismic-response acceleration of the EPS fill mass in slope applications by converting the sloped EPS cross-section shown in Figure 4.37(a) to a stand-alone embankment cross section that has an equivalent EPS cross-sectional area equal to the actual cross sectional area of the EPS-block geofoam slope system as shown in Figure 4.37(b), (Horvath, 1995, Public Works Research Institute, 1992). As indicated in Figure 4.37(b), the equivalent model cross-section is based on determining an equivalent height, H', that will yield the same sloped cross sectional area using the actual top width, B, of the actual slope structure shown in Figure 4.37(a). The Japanese manual does not provide a reason as to why the equivalent model cross-section depicted in Figure 4.37(a) is based on the actual top width of the EPS and an equivalent height instead of the actual height, H, and an equivalent width, B', as depicted in 4.37(c). Therefore, a parametric analysis was performed as part of this NCHRP study to compare the estimated period of the EPS fill mass based on the two models, i.e., Figure 4.37(b) versus 4.37(c).

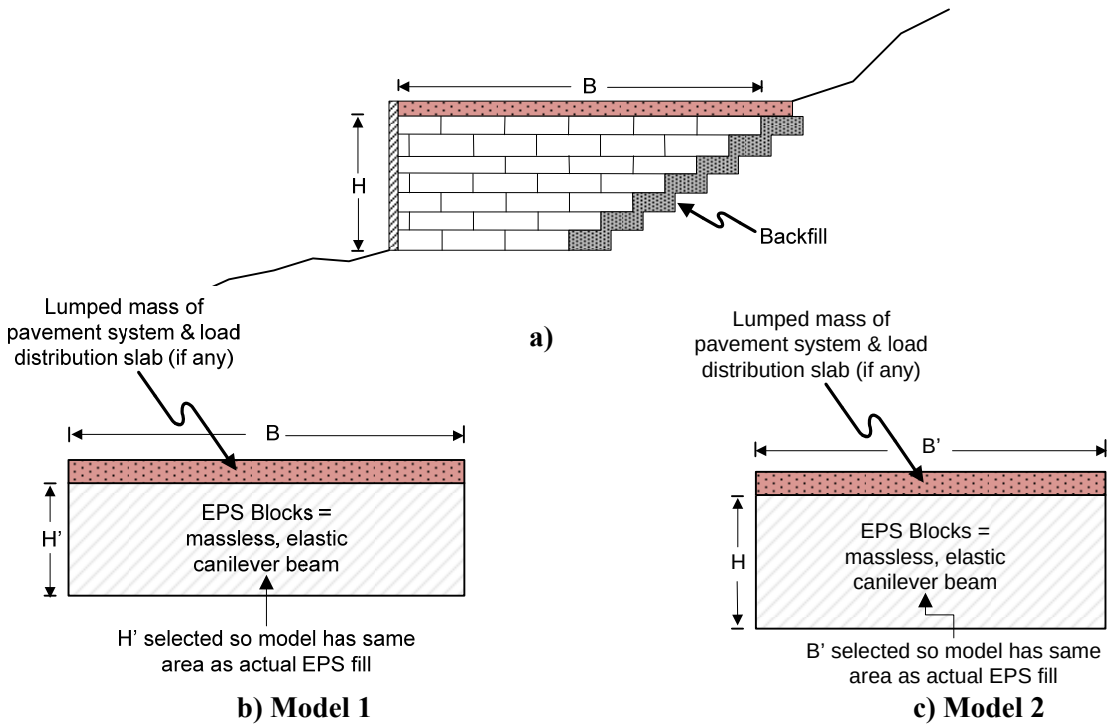


Figure 4.37. Approximate (simplified) seismic modeling of an EPS fill.

The comparison between the two models is based on the ratio between the period of Model 1 to Model 2, T_1/T_2, as shown by the below equations:

$$\begin{cases} T_1 = 2\pi[(\dfrac{\sigma'_{V0}H'}{E_{t_i}g})(4(\dfrac{H'}{B})^2 + (\dfrac{12}{5})((1+\nu))]^{0.5} \\ T_2 = 2\pi[(\dfrac{\sigma'_{V0}H}{E_{t_i}g})(4(\dfrac{H}{B'})^2 + (\dfrac{12}{5})((1+\nu))]^{0.5} \end{cases} \Rightarrow \frac{T_1}{T_2} = \frac{[(H')(4(\dfrac{H'}{B})^2 + (\dfrac{12}{5})((1+\nu))]^{0.5}}{[(H)(4(\dfrac{H}{B'})^2 + (\dfrac{12}{5})((1+\nu))]^{0.5}} \quad \textbf{(4.26)}$$

where

$A = constant = H'B = HB'$

$\nu = 0.1$ assumed

In addition to the actual height (H) and width (B) of the EPS fill mass, we also considered the effect of slope magnitude on T_1/T_2 in the parametric study. Figure 4.38 shows results of the parametric study.

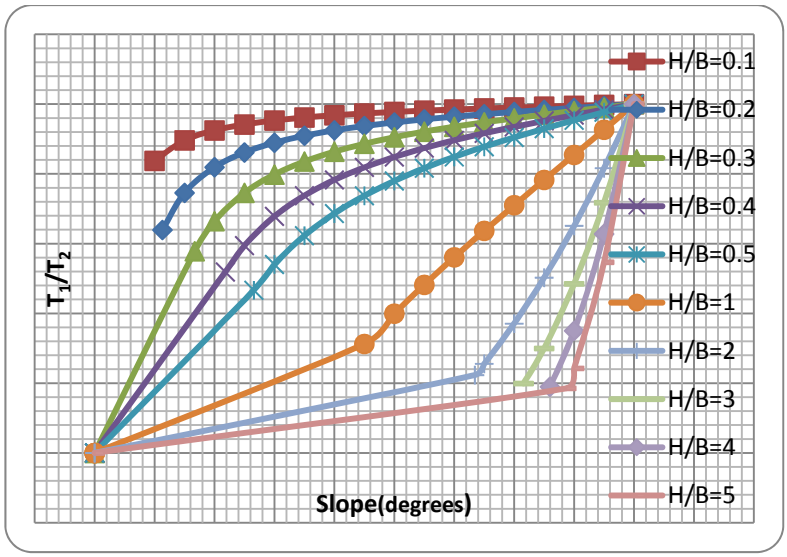


Figure 4.38. Parametric study results.

At a slope angle of 90 degrees, which represents a vertical-sided and stand-alone embankment, the periods obtained for both Model 1 and 2 are the same, i.e., $T_1/T_2=1$, at any H/B ratio. For slopes less than 90 degrees, T_1/T_2 is less than 1, which implies that the estimated period of Model 1 is less than Model 2. Therefore, for a typical site-modified acceleration response spectrum such as the one shown in Figure 4.39, the two models will yield different spectral accelerations. For example, the model with a lower period may yield high spectral accelerations, while the model with a high period may yield low spectral accelerations. Therefore, for design, it is recommended that the period based on both Models 1 and 2 be determined, and that the higher resulting spectral acceleration value, S_a, that is obtained from the site-modified acceleration response spectrum be used to determine the seismic coefficient, i.e., $k=S_a/g$. This larger seismic coefficient can be used in pseudo-static slope stability analysis.

Figure 4.38 shows that T_1/T_2 increases linearly with increase in slope for an H/B of 1. However, T_1/T_2 is especially sensitive for EPS fill mass slopes with H/B greater than 1, i.e., tall and narrow EPS fill mass slopes, but not as sensitive for EPS fill mass slopes with H/B less than 1. Therefore, selecting the seismic coefficient based on the model that will provide the higher spectral acceleration value is especially important for tall and narrow EPS fills slopes with H/B greater than or equal to 1.

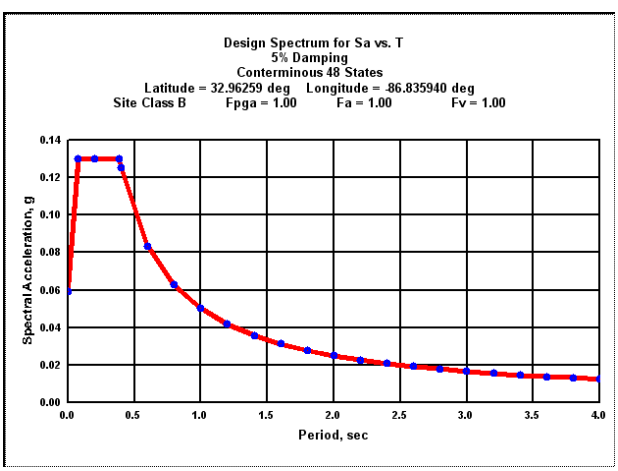


Figure 4.39. Example of a site-modified acceleration response spectrum.

What was the basis of the recommendation in the Japanese design manual to utilize the single-degree of freedom model that is applicable to stand-alone embankments also for slope applications? The results on small-scale shake table tests indicated that the slope of the EPS blocks had only a small effect on amplification of the EPS fill mass, and the resulting amplifications were similar to amplifications obtained in the stand-alone model tests (EPS Construction Method Development Organization, 1994). Therefore, it appears that the Japanese practice to also consider an EPS fill in slope applications as a flexible structure with a concomitant amplification of surface motion that is frequency dependent stems from the observation that the slope of the EPS fill mass in slope applications displayed only a minor influence on the resulting amplification in the small-scale shake table tests. However, results of full-scale shake table tests or from instrumentation on actual EPS fills in slopes is required to fully validate the use of the single-degree of freedom model for slope applications.

Instrumentation results of a full-scale slope that was supported by a reinforced concrete retaining wall backfilled with EPS blocks in Yokosuka City in Japan indicated that at a given elevation, the measured acceleration based on 16 seismic events, was larger within the EPS fill than the adjacent natural slope material. This difference increased at higher foundation soil accelerations as measured at a depth of approximately 10m below the bottom of the retaining wall foundation. Although these full scale observations are based on an EPS fill mass in a slope application supported by a retaining wall, results of these full-scale observations appear to support the observations made on the small-scale model shake table test results of EPS blocks on slopes without retaining walls, that the EPS fill mass does undergo amplification effects. Accelerations measured within the EPS fill mass near the center of the EPS fill mass cross-section and near the retaining wall were similar. These measurements suggest that at a given elevation, the EPS fill mass will have the same acceleration throughout the cross section.

In summary, it is proposed herein that the SDOF model be used to estimate the resonant period, T_o, of the EPS fill mass in slope applications by converting the sloped EPS cross section shown in Figure 4.37(a) to a stand-alone embankment cross section that has an equivalent EPS cross-sectional area that is equal to the actual cross sectional area of the EPS-block geofoam slope system as shown by Model 1 in Figure 4.37(b) and Model 2 in 4.37(c). Equation 4.24 can be used to obtain T_o for each of the two models. The higher resulting spectral acceleration value, S_a, obtained from the site-modified acceleration response spectrum between the two T_o values, can be used to determine the seismic coefficient, i.e., $k=S_a/g$, which can be used in a pseudo-static slope stability analysis.

Although T_o can be used with the site-appropriate response spectrum to determine S_a of the lumped mass, additional factors will influence S_a of the EPS-block geofoam fill mass. These factors include amplification effects of the flexible EPS fill mass, system damping due to energy losses within the EPS-block fill mass, the impact of exterior covering systems typically required for vertical-sided fills on the horizontal acceleration obtained from the SDOF analysis, the impact of adjacent natural slope material on horizontal acceleration obtained from the SDOF analysis, and the influence of including the vertical component of seismic acceleration in the seismic design of EPS-block geofoam slopes.

Based on research performed predominantly in Japan, it is Japanese practice to consider an EPS fill to respond as a flexible structure with a concomitant amplification of surface motion that is frequency dependent, especially for relatively high, narrow fills (Horvath, 1995). The SDOF model previously described considers this amplification and will provide the amplified acceleration of the lumped mass.

System damping within the EPS fill mass is assumed to be the result of energy losses within the assemblage of EPS blocks. These energy losses result from both internal material damping within the EPS blocks as well as inter-block sliding friction along joints (Horvath, 1995). In a study that included laboratory resonant column tests and cyclic uniaxial tests on block molded EPS geofoam specimens, Athanasopoulos et al., (1999) concluded that that the damping ratio values for EPS-block geofoam specimens are less than 1.5 percent for strains less than 1 percent, but increase to a value of about 10 percent for strains on the order of 10 percent. Based on shake-table tests performed in Japan, damping coefficients due to inter-block friction varied between 2.5 percent and 8 percent (Horvath, 1995). These shake-table tests were performed on fills with vertical faces only and not on slope-sided fills at

accelerations of up to 0.3g. Apparent damping increased with increasing acceleration and was greater for side-hill fills compared to free-standing fills (Horvath, 1995).

The analysis and design experience of EPS-block geofoam embankments for seismic loading on the CA/T Project revealed that use of heavy exterior side protective coverings of vertical-sided embankments will increase the mass of the lumped mass and will, therefore, increase the fundamental period of the fill mass system. Approximately 25 percent of the mass of the side panels was added to the lumped mass in analysis of the CA/T fills. The mass of the panels was incorporated in the model shown in Figure 4.35 by including the mass of the side panels as a distributed mass along the entire length (height) of the cantilever beam (Riad and Horvath, 2004).

One issue that requires further investigation is the impact of adjacent natural slope material on the horizontal acceleration obtained from the SDOF analysis. Based on results of shake-table tests performed by the Japanese Public Works Research Institute, the impact of adjacent natural slope material on the resonant frequency and amplification factor of the EPS fill mass was small (Nomaguchi, 1996). However, this conclusion is not fully supported by Nomaguchi (1996). Therefore, further investigation will be required to determine if this conclusion is valid for most typical EPS-block geofoam fill mass systems.

The SDOF model shown in Figure 4.35 is typically used to obtain the horizontal acceleration of the lumped mass, and only horizontal accelerations are typically considered in analyses of the various seismic instability failure mechanisms. However, based on numerical analyses, Barlett and Lawton (2008) concluded that for cases where interlayer sliding is just initiating, the vertical component of acceleration is important because an analysis based on only the horizontal component of acceleration may yield unconservative results. However, the vertical component of motion is less important when the interlayer sliding is well developed. Therefore, a simple model similar to the SDOF model that is used to estimate horizontal seismic acceleration is needed to obtain the vertical component of acceleration that can be incorporated in the simplified seismic response method of analysis.

Rigorous Response Method

For EPS-block geofoam slopes located such that failures have the potential to be especially catastrophic, a more rigorous analysis of seismic stability may be required. The detailed or rigorous approach consists of performing numerical analysis with the input earthquake motion consisting of either an acceleration response spectrum (frequency-domain analysis) or an actual time-varying earthquake record (time-domain analysis) (Horvath, 1995, Public Works Research Institute, 1992).

Because a considerable amount of time is typically required to develop an accurate numerical model, the detailed seismic analysis method is generally restricted to major projects. The simplified seismic response method is useful as a screening tool to evaluate whether or not a full numerical analysis is necessary to evaluate the seismic stability of the slope. A detailed explanation of a numerical analysis procedure is beyond the scope of this report; however, details of a case history that involved numerical analysis are provided to assist in applying this method to seismic stability analysis.

This case history involved performing a numerical analysis of an EPS-block geofoam stand-alone, vertical-sided embankment constructed as part of the I-15 Reconstruction Project in Salt Lake City, UT, and was presented by Bartlett and Lawton (2008). The goal of the numerical study was to model the behavior of the embankment under seismic loads generated during a nearby M7.0 earthquake. The analysis was performed using the finite difference program *FLAC*™ (Fast Lagrangian Analysis of Continua).

Figure 4.40 shows the configuration of the 8m high by 20m wide embankment that was analyzed. The model used in the analysis consisted of a 10m thick clay foundation, 8m high EPS-block geofoam, and a 1m thick lumped mass that combined the masses of the concrete load distribution slab, untreated base course, and PCC pavement.

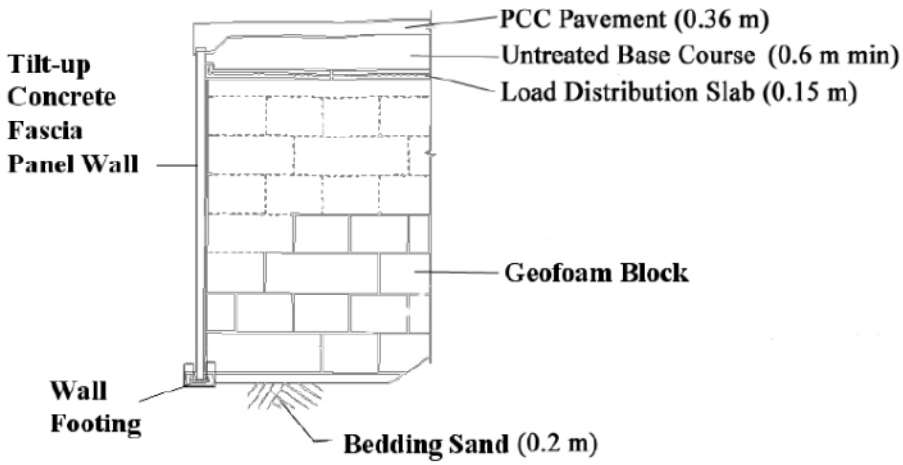


Figure 4.40. Typical EPS-block geofoam cross-section used for the I-15 Reconstruction Project (Bartlett and Lawton, 2008a).

Several unique aspects of the numerical analysis included (1) allowance of two degrees of freedom (horizontal and vertical) movement, (2) use of nonlinear stress-strain relations for all materials, except for the lump mass, which was treated as an elastic material, (3) allowance of horizontal sliding between geofoam layers by including interfaces nodes, and (4) input of both the horizontal and vertical components of the strong motion records into the model to explore their combined effects on dynamic response and potential sliding (Bartlett and Lawton, 2008).

The input ground motion used for the analysis consisted of eight different horizontal acceleration time histories and three different vertical acceleration time histories obtained from the Pacific Earthquake Engineering Research (PEER) Center. These time histories corresponded to the 2,500-yr. Design Basis Earthquake (DBE). Both horizontal and vertical components of the ground motion were considered as fully coupled for the purposes of the analysis; that is, the effects of both components were considered as acting in conjunction with one another rather than separately.

The initial elastic material properties of each of the materials in the actual embankment are summarized in Table 4.5. The foundation soil, whose material properties are noted in Table 4.5, consisted of medium to medium-stiff clay. In addition to these initial elastic properties, the hysteretic damping option of *FLAC* was used to model the nonlinear, strain-dependent modulus and damping of the foundation soil and EPS geofoam. The use of elastic material properties instead of Mohr-Coulomb material properties ensured that yielding and plastic behavior did not occur during interblock sliding and to capture only the interface sliding behavior. The protective panel wall and influence of mechanical connectors were not included in the analysis to simplify the numerical analysis. In the actual embankment, a 0.2m gap was left between the facing panel wall and EPS fill. Because this gap was wide enough to prevent any interaction between the EPS fill and facing wall, the effects of the facing wall on the system were ignored in the numerical analysis.

Table 4.5. Initial elastic material properties used for numerical model (Bartlett and Lawton, 2008a).

Material	ρ (kg/m^3)	E MPa	υ	K MPa	G MPa
Foundation Soil	1840	174	0.4	290	62.1
EPS Geofoam	18	10	0.103	4.2	4.5
Untreated Base Course	2241	570	0.35	633.3	211.1
Load Distribution Slab	2401	30000	0.18	15625	12711.9
PCC Pavement	2401	30000	0.18	15625	12711.9

Figure 4.41 shows the shear modulus degradation curves for the foundation soil and EPS blocks. The *sig3* model curve included in *FLAC* is used to utilize the hysteretic damping option in the analysis. In addition to accounting for the hysteretic damping behavior of the EPS fill, a 5 percent Rayleigh damping was applied to frequencies of 200Hz to prevent numerical errors in modeling vibrations of the fill mass. Table 4.6 provides interface properties for the model interfaces.

Barlett and Lawton (2008) concluded the following from the numerical analysis:

- Interlayer sliding displacement is a highly nonlinear process and is influenced by the frequency content and long period displacement pulses present in the input time histories.
- For cases where interlayer sliding is just initiating, the vertical component of acceleration is important because an analysis based on only the horizontal component of acceleration may yield unconservative results. However, the vertical component of motion is less important when the interlayer sliding is well developed.
- Horizontal sway and rigid-body rocking can cause local tensile and compressive yielding of blocks near the base of the embankment. In several cases, tensile yielding may propagate upwards and may result in decoupling of the EPS blocks and load distribution slab.

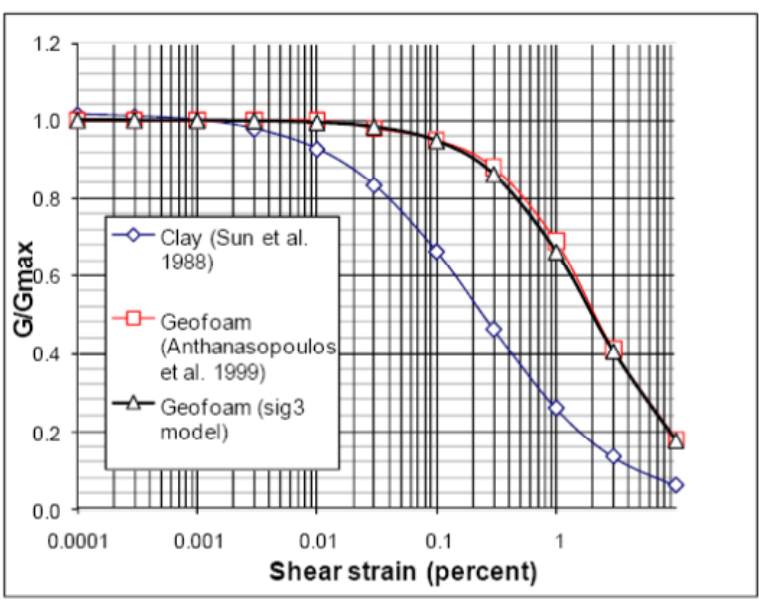


Figure 4.41. Shear modulus degradation curves used in FLAC's hysteretic damping option (Bartlett and Lawton, 2008a).

Table 4.6. Interfacial properties used for sliding evaluation in FLAC model (Bartlett and Lawton, 2008a).

Contact Surface	Interface # (Top to Bottom)	Normal & Shear Stiffness ($k_n = k_s$) (MPa)	Friction Angle (degrees)
Geofoam-Soil	1	102	31*
Geofoam-Geofoam	2-8	102	38
Geofoam-Lump Mass	9	102	38**

** A glued interface was used for interface 1 in FLAC because the geofoam is abutted against the panel wall footing and cannot slide.*
*** Neglects any tensile or shear bonding that may develop between the top of geofoam and base of load distribution slab.*

In summary, the seismic-response acceleration at the top of the EPS fill mass can be determined using the simplified seismic response method. This method consists of using the SDOF model to estimate the resonant period, T_o, of the EPS fill mass in slope applications by converting the sloped EPS cross-section shown in Figure 4.37(a) to a stand-alone embankment cross section that has an equivalent EPS cross-sectional area equal to the actual cross-sectional area of the EPS-block geofoam slope system as shown by Model 1 in Figure 4.37(b) and Model 2 in 4.37(c). Equation 4.24 can be used to obtain T_o for each of the two models. The higher resulting spectral acceleration value, S_a, obtained from the site-modified acceleration response spectrum between the two T_o values, can be used to determine the seismic coefficient, i.e., $k=S_a/g$, which can be used in a pseudo-static slope stability analysis. For EPS-block geofoam slopes located such that failures have the potential to be especially catastrophic, a more rigorous analysis of seismic stability, e.g., numerical analysis, may be required.

Performing Pseudo-Static Limit Equilibrium Stability Analyses of the Various Failure Mechanisms (Step 7iii)

Internal Horizontal Sliding

The primary evaluation of internal seismic stability involves determining whether the geofoam embankment will behave as a single, coherent mass when subjected to external loads. This is determined by the interface shear resistance between the pavement system and the upper surface of the EPS mass and the interface friction between adjacent EPS blocks. Therefore, a discussion of methods that can be used to ensure adequate block interlock is initially presented prior to describing the seismic internal stability analysis procedure for horizontal sliding.

Internal stability of an EPS-block fill mass is maintained if it acts as a single, coherent mass when subjected to external loads. Since the fill mass consists of individual blocks, the collection of blocks will behave as a coherent mass if the individual EPS blocks exhibit vertical and horizontal interlock. Sufficient interlock between blocks involves consideration of the overall block layout (which primarily controls interlocking in the vertical direction) and inter-block shear (which primarily controls interlocking in the horizontal direction).

Guidelines for an appropriate layout of EPS blocks to obtain adequate interlocking in the vertical direction are included in the recommended standard included in Appendix F. As indicated in Chapter 3, there are two modes of shear of interest in lightweight fill applications: (1) internal shear strength within a specimen of EPS, and (2) external shear strength (interface sliding resistance) between EPS blocks or between an EPS block and a dissimilar material (soil, other geosynthetic, etc.). The latter mode is the

primary shear mode of interest in internal stability assessment under horizontal loads such as seismic shaking.

Two types of shear interfaces that are of interest for EPS-block geofoam in lightweight fill applications include an EPS to EPS interface and an EPS to a dissimilar material interface. If the calculated resistance forces along horizontal planes between EPS blocks are insufficient to resist horizontal driving forces, additional resistance between EPS blocks is generally provided by adding mechanical inter-block connectors (typically prefabricated barbed metal plates) along the horizontal interfaces between the EPS blocks, or by adding a shear key. A discussion of EPS/EPS and EPS/dissimilar material interface shear resistance is included in Chapter 3.

Since it is standard practice to include the use of mechanical connectors between layers of blocks to prevent sliding between blocks during earthquake motions, the Japanese seismic design procedure does not evaluate sliding between blocks as an internal failure mechanism. However, it is possible to perform a sliding stability analysis by incorporating the additional resistance that is provided by mechanical connectors or shear keys as part of a horizontal sliding analysis at various depths of the EPS fill. Figure 4.42 shows a photo of mechanical connectors between two layers of EPS blocks.

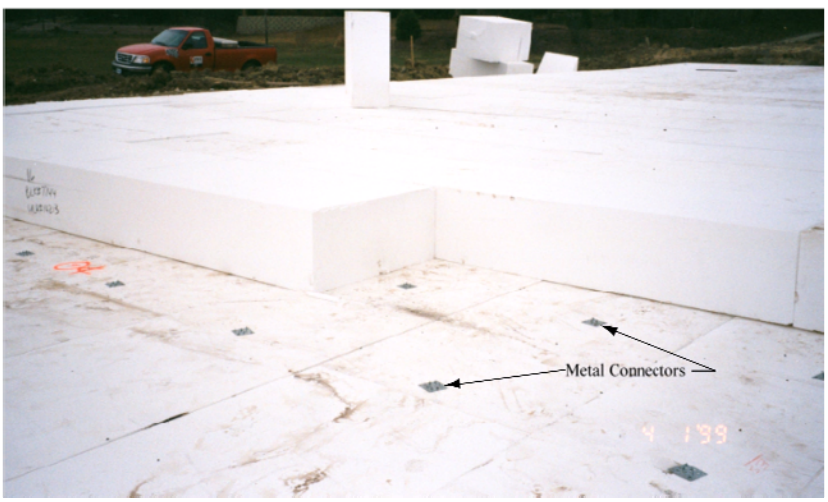


Figure 4.42. Photograph showing use of mechanical connector plates within EPS fill.

The internal seismic response is more complex than the overall external response. This is caused by the EPS-block geofoam acting as a flexible, not rigid, structure and slippage possibly occurring between blocks. This slippage may result in the mobilization of a post-peak interface strength, which did not have to be considered in the external stability analyses because the shear resistance of geofoam was assumed to be negligible. Therefore, a post-peak or residual EPS/EPS interface resistance value may be more appropriate for seismic stability analysis.

The mass of an EPS-block geofoam slope system is concentrated at the top of the fill mass because the mass of EPS fill is negligible compared to the mass of the overlying pavement system. Therefore, the pavement system will be subjected to amplified accelerations compared to the bottom of the fill mass and sliding between the pavement system and EPS blocks may occur. If the sliding resistance between the pavement system and EPS-blocks is not sufficient to withstand seismic loading, a Portland cement concrete slab can be incorporated at the interface of the pavement system and EPS blocks and the slab can be anchored into the adjacent slope material to obtain additional resistance against sliding. Figure 4.43 provides a model that can be used to determine the required anchor resistance. Supplemental guidance on the design of ground anchors is available in the FHWA Geotechnical Engineering Circular No. 4 (Sabatini et al., 1999). Use of ground anchors can also increase the stability of the fill mass to overturning. In addition to ground anchors, the Japanese typically require L-shaped

reinforcing bar dowels cast into the slab that penetrate down into the EPS blocks to provide additional interface resistance between the PCC slab/EPS block interface during seismic loading.

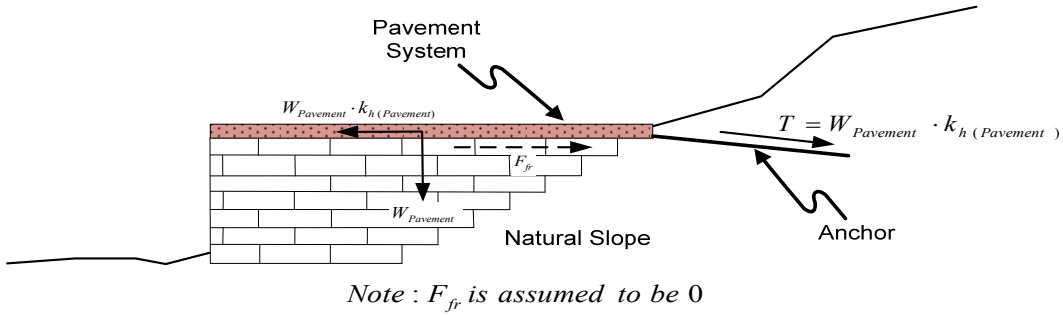


Figure 4.43. Analysis of ground anchors within the pavement system (Public Works Research Institute, 1992)

Seismic Load-Bearing

A new mode of seismic behavior called seismic rocking was recognized during the design of the CA/T Project embankments and was supported by shake-table research performed in Japan around the same time (Horvath, 2004a, Nishi et al., 1998, Riad, 2005b, Riad and Horvath, 2004). Seismic rocking is defined as rigid-body rotation of the entire embankment about its longitudinal axis due to the moment created by the relatively concentrated, elevated mass of the pavement system (Riad, 2005b). The seismic rocking behavior mode is shown in Figure 4.14. This rotation would occur in the plane perpendicular to the cross-section shown in Figure 4.14 and caused by the seismic-inertia force. Seismic rocking was critical in the design of CA/T embankments because of their combination of a vertical-sided cross-section coupled with a relatively slender cross-sectional geometry with a height-to-width ratio of about one.

The seismic rocking will produce an increase in the vertical normal stress within the EPS-block geofoam fill mass due to the moment produced by the seismic-inertia force associated with the lumped mass. This seismic-inertia force is assumed to act at the center of gravity of the EPS-block fill mass, which, because the majority of the system's mass is concentrated in the pavement system, should be located near the top of the fill mass at the horizontal center of the embankment cross-section. The stress that this moment produces is obtained by:

$$\sigma_{vm} = \frac{Mc}{I} \tag{4.27}$$

where

σ_{vm}	=	dynamic vertical stress produced by the moment
M	=	moment about center of gravity produced by horizontal seismic force
c	=	distance from centroid to location at which normal stress is to be calculated
I	=	moment of inertia

The importance of dynamic vertical stress produced by the moment is that for tall and narrow vertical-sided embankments, this dynamic stress may yield high stresses near the lower and exterior portions of the fill mass, as shown by Figure 4.13. The key contribution of the Boston CA/T design experience is that these additional dynamic stresses must be considered in selection of an appropriate type of EPS block in load-bearing analysis. These dynamic stresses must be combined with the gravity normal stresses in Step 10 of the design procedure shown in Figure 4.25. It should be noted that if the vertical

component of acceleration is significant enough to be considered in the design, additional stress from the vertical acceleration will need to be considered in the load-bearing analysis and selection of geofoam type.

The CA/T project involved stand-alone embankments. Therefore, currently no research or design data is available on the impact of the seismic rocking behavior mode on EPS fills used in slope systems. Therefore, it is recommended that the seismic rocking behavior mode of EPS-block geofoam slope systems be investigated.

Summary

The general internal seismic analysis procedure consists of decoupling the determination of the overall seismic response acceleration of the EPS-block geofoam embankment into the determination of the seismic response of natural slope material followed by the seismic response of the EPS-block fill mass. The seismic response results of adjacent natural slope material and the EPS-block fill mass can then be used to evaluate each potential seismic failure mechanism separately. The failure mechanisms that are considered for internal seismic stability analysis include horizontal sliding between layers of blocks and/or between the pavement system and upper layer of blocks and load-bearing failure of EPS blocks.

Seismic analysis and design of EPS-block geofoam slopes can be separated into the following three primary steps: (1) estimating the seismic-response acceleration at the existing ground surface or base (subgrade level) of the EPS fill mass by performing a site-specific assessment, (2) estimating the seismic-response acceleration at the top of the EPS fill mass, (3) performing pseudo-static limit equilibrium stability analyses of the various failure mechanisms.

STEP 8: PAVEMENT SYSTEM DESIGN

The purpose of Step 8 is to perform a pavement structural analysis that considers the subgrade support provided by EPS blocks. The estimated pavement section determined in Step 2 is preliminary only, and served to facilitate estimation of dead loads for evaluation of Steps 3 through 7 of the design procedure shown in Figure 4.25. Step 8 provides the pavement configuration based on the anticipated loading conditions.

Steps 8 and 9 will typically be required if the existing or proposed roadway is located within the existing or anticipated slide mass and the existing or anticipated slide mass is located below the roadway as shown in Figures 4.17(b) and 4.18(b), i.e., the roadway is near the head of the slide mass. However, a pavement system design will typically not be required if the existing or proposed roadway is located outside the limits of the existing or anticipated slide mass and/or the existing or anticipated slide mass is located above the roadway, as shown in Figures 4.17(a) and 4.18(a), i.e., the roadway is near the toe of the slide mass. It is anticipated that EPS-block geofoam used for this latter slope application will not support any structural loads, other than possibly soil fill above the blocks. Therefore, as shown in Figure 4.25, Steps 8 and 9 will not be required and only failure mechanisms associated with the external and internal instability failure modes, as shown in Table 4.2, need to be considered. The pavement system failure mode may not be an applicable failure mode because if the roadway is near the toe of the slide mass, stabilization of the slide mass with EPS-block geofoam will occur primarily at the head of the slide and, consequently, the EPS-block geofoam slope system may not include the pavement system.

For the case of the existing or proposed roadway located within the existing or anticipated slide mass and the existing or anticipated slide mass located below the roadway, pavement system design (Step 8) follows seismic stability. The pavement system is defined as including all materials, bound and unbound, placed above EPS blocks. The objective of pavement system design is to select the most economical arrangement and thickness of pavement materials for the subgrade provided by the underlying EPS blocks. The design criteria are to prevent premature failure of the pavement system (Figure 4.16), as well as to minimize potential for differential icing (a potential safety hazard) and solar heating (which can lead to premature pavement failure) in those areas where climatic conditions make these potential problems. Also, when designing the pavement cross-section overall, consideration must be given to

providing sufficient support, either by direct embedment or structural anchorage, for any road hardware (guardrails, barriers, median dividers, lighting, signage and utilities).

A unique aspect of pavement design over lightweight fill is that the design must also consider the potential failure mechanisms associated with external and internal stability of the overall EPS-block geofoam slope system. The benefits of using a thicker pavement system include increased pavement life, increased internal load bearing capacity of the EPS-block fill mass, reduced potential for differential icing, reduced potential for solar heating, and better accommodation of shallow utilities and road hardware. The drawbacks of a thicker pavement system include increased weight, which will decrease the factor of safety of the external stability failure mechanisms, decreased seismic internal stability of the EPS-block geofoam fill mass, and higher total cost for the project. Thus, some compromise is required to optimize the final design of both the pavement system and overall fill mass. The benefits and drawbacks of utilizing a thicker pavement system as well as procedures for design of pavement systems over EPS-block geofoam embankments are further discussed in the Project 24-11(01) report (Stark et al., 2004a).

The literature search revealed a lack of current research results that focus on design of pavement systems overlying EPS blocks, especially design parameters for use in design of pavement systems based on the AASHTO Mechanistic-Empirical Pavement Design Guide (AASHTO, 2008) and recommendations for design of pavements systems that include a separation layer between the top of EPS blocks and the overlying pavement system. A separation layer can have two functions: (1) to enhance the overall performance and life of the pavement system by providing reinforcement, separation, and/or filtration, and (2) to enhance the durability of EPS blocks both during and after construction.

The use of a 100 to 150mm (4 to 6 in.) thick reinforced PCC slab is currently the state of practice, primarily because it is considered a necessity for providing sufficient lateral confinement of unbound pavement layers and load distribution when using EPS-block geofoam, and because of historical usage of PCC slabs dating back to the earliest EPS-block geofoam lightweight fills in Norway in the 1970s. The original function of the PCC slab was for pavement reinforcement, and the intent was to allow use of a minimum pavement system thickness. In later designs, the PCC slab was also used for the function of a barrier against potential petroleum spills. However, use of a PCC slab for this function is questionable due to the usual long-term development of cracks in PCC slabs.

PCC slabs generally represent a significant relative cost, so PCC slabs should only be used if specifically required as determined during design of the pavement system in Step 8 and in load-bearing analysis in Step 10. Examples of EPS slope stabilization projects that did not have a PCC slab above EPS blocks include the AL 44 project near Guin, AL, and the County Trunk Highway "A" project in Bayfield County, WI. A summary of these two projects is provided in Chapter 7.

One application where a PCC slab is typically required is when an embankment with vertical sides, i.e., geofoam wall, is used (Horvath, 2001). However, this standard practice may be changing because the Project 24-11(01) research revealed that the cost of a PCC slab represents a significant cost to an EPS-block geofoam embankment. For example, a 9 to 12m (30 to 40 ft.) high by 152m (500 ft.) long vertical-sided embankment that is currently being constructed as part of the Topaz Bridge project on U.S. 30 at Topaz, ID, does not include a PCC slab. The pavement system above the EPS blocks consists of a galvanized steel mesh mechanically stabilized earth wall system backfilled with gravel that extends the full width of the four lane highway.

The primary function of the PCC slab in an embankment with vertical sides is to support the upper part of the exterior facing system. A secondary function is to provide anchorage for various highway hardware, such as safety barriers, signage, and lighting. A PCC slab used for these functions will act primarily as a structural member for the benefit of other embankment system components and not the EPS. Therefore, the PCC slab should be designed for the intended function.

A reinforced geomembrane that will resist hydrocarbon spills has also been used on top of EPS blocks as an alternative to a PCC slab to protect EPS blocks against fuel spills. An example specification for a hydrocarbon-resistant geomembrane is included in Appendix H.

Negussey and Huang (2006) and Huang and Negussey (2007) provide preliminary test results for estimating a composite resilient modulus and composite modulus of subgrade reaction for the design of

pavement systems that include a PCC slab separation layer. However, they noted that further verification and confirmation from field performance monitoring and field tests on actual pavements overlying EPS blocks is needed to confirm the preliminary test results. Studies that provide the potential benefits of using separation layers other than a PCC slab such as a geogrid, geocell with soil or PCC fill, soil cement, and Pozzolanic stabilized materials are also needed.

STEP 9: EVALUATION OF PAVEMENT SYSTEM DESIGN ON PREVIOUS FAILURE MECHANISMS ALREADY ANALYZED

A unique aspect of pavement design over lightweight fill is that the design must also consider the affect of the final pavement system design on static and seismic slope stability because the thickness and type of pavement system materials will impact static and seismic stability. Therefore, Step 9 consists of determining if the pavement system design obtained in Step 8 results in a change in the overburden stress compared to the preliminary pavement system initially utilized in Step 2. If the overburden stress between the pavement system obtained in Step 8 is the same as the overburden stress from the preliminary pavement system initially utilized in Step 2, Step 10 (load-bearing analysis) can be performed. If the overburden stress between the Step 8 and Step 2 pavement systems are different, static and seismic slope stability must be rechecked and, therefore, the design procedure reverts back to Step 5.

STEP 10: LOAD-BEARING (INTERNAL)

Summary of Load-Bearing Analysis Procedure

Step 10 involves load-bearing analysis. A load-bearing analysis consists of selecting an EPS type with suitable properties to be able to support the overlying pavement system and traffic loads without excessive EPS compression that could lead to excessive settlement of the pavement surface. In order to ensure adequate performance of EPS blocks, three design goals must be achieved. First, the initial (immediate) deformations under dead or gravity loads from the overlying pavement system must be within acceptable limits. Second, long-term (for the design life of the fill) creep deformations under the same gravity loads must be within acceptable limits. Third, non-elastic or irreversible deformations under repetitive traffic loads must be within acceptable limits.

The elastic limit stress, σ_e, is the parameter used to evaluate the three deformation issues presented above. The elastic limit stress of EPS geofoam is defined as the compressive stress at 1 percent strain as measured in a standard rapid-loading compression test (Stark et al., 2004a). The basic procedure for designing against load-bearing failure is to calculate maximum vertical stresses at various levels within the EPS mass (typically the pavement system/EPS interface is most critical) and select the EPS type that exhibits an elastic limit stress that is greater than the required elastic limit stress at the depth being considered. If EPS blocks with a higher elastic limit stress than what is currently available locally are required, consideration can be given to modifying the pavement system design such as adding a separation layer to further distribute live loads and decrease the stresses at the top of the EPS-block fill mass. For example, a PCC load distribution slab can be included in the pavement system to decrease stresses within EPS blocks. Table F.2 in Appendix F provides the minimum recommended values of elastic limit stress for various EPS densities.

Because the applied vertical stress decreases with depth under the pavement and side slopes, it is possible to use multiple densities of EPS blocks in an embankment. For example, lower density blocks can be used at greater depths and/or under side slopes, and higher density blocks can be used under the pavement system to obtain an economical design fill mass arrangement. The exception to using lower density blocks with depth may involve consideration of the seismic load bearing failure mechanism, as noted in Step 6 and 7. The reason for not wanting to use an excessively high density of EPS throughout the EPS fill mass is that the manufacturing cost of EPS block is significantly linked to the relative amount

of raw material (expanded polystyrene) used. As noted in the Project 24-11(01) report, use of *EPS40* directly below the pavement system is not recommended (Stark et al., 2004a).

The procedure for evaluating the load bearing capacity of EPS as part of internal stability is outlined in the following thirteen steps (Stark et al., 2004a, Arellano and Stark, 2009):

10i. Estimate traffic loads.
10ii. Add impact allowance to traffic loads.
10iii. Estimate traffic stresses at top of EPS blocks.
10iv. Estimate gravity stresses at top of EPS blocks.
10v. Calculate total stresses at top of EPS blocks.
10vi. Determine minimum required elastic limit stress for EPS under pavement system.
10vii. Select appropriate EPS block to satisfy the required EPS elastic limit stress for underneath the pavement system, e.g., *EPS50*, *EPS70*, *EPS100, EPS130,* or *EPS160*.
10viii. Select preliminary pavement system type and determine if a separation layer is required.
10ix. Estimate traffic stresses at various depths within the EPS blocks.
10x. Estimate gravity stresses at various depths within the EPS blocks.
10xi. Calculate total stresses at various depths within the EPS blocks.
10xii. Determine minimum required elastic limit stress at various depths.
10xiii. Select appropriate EPS block to satisfy the required EPS elastic limit stress at various depths in the embankment.

The load-bearing design procedure can be divided into two parts. Part 1 consists of Steps 10i through 10viii and focuses on the determination of traffic and gravity load stresses applied by the pavement system to the top of the EPS blocks, and selection of the type of EPS that should be used directly beneath the pavement system. Part 2 consists of Steps 10ix through 10xiii, and focuses on the determination of traffic and gravity load stresses applied at various depths within the EPS-block fill mass and selection of appropriate EPS for use at these various depths within the embankment. Each of the load bearing analysis design steps are described in detail in the Project 24-11(01) report (Stark et al., 2004a).

An evaluation and comparison of various procedures to estimate the dissipation of live load traffic vertical stresses through the pavement system so an estimate of traffic stresses at the top of the EPS blocks can be obtained was performed as part of this Project 24-11(02) study. The vertical stress at the top of the EPS is used to evaluate the load bearing capacity of the blocks directly under the pavement system as part of Step 10iii of the procedure for evaluating the load bearing capacity of the EPS blocks. The purpose of the live load traffic stress distribution investigation was to address the concern that the wheel load stresses recommended in the reports for stand-alone embankments over soft ground (Stark et al., 2004a, Stark et al., 2004b) were overly conservative. This concern was expressed by one of the respondents to the project questionnaire included in Appendix A. A summary of the stress distribution study is included below.

Live Load Traffic Stress Distribution Study

In the Project 24-11(01) reports, Burmister's elastic layered solution (Burmister, 1943) was recommended to estimate the stress distribution through the pavement system to obtain the applied vertical stress at the top of the EPS-block fill due to a wheel load applied to the pavement surface. Burmister's elastic layered solution is based on a uniform pressure applied to the surface over a circular area on top of an elastic half-space mass. Each layer has a finite thickness, except for the lowest layer, which is assumed to be infinite in thickness and each layer is assumed to be homogeneous, isotropic, and linearly elastic. The primary advantage of Burmister's theory is that it considers the influence of layers with different elastic properties within the system being considered. The primary disadvantage is that vertical stress calculations are time consuming if not performed by computer.

To facilitate estimation of stresses on top of EPS blocks from traffic loads, stress design charts (see Figures 41 through 43 of the NCHRP Report 529) were developed during the Project 24-11(01) study for various vehicle tire loads and pavement systems (Stark et al., 2004b). The computer program KENLAYER (Huang, 1993), which is based on Burmister's solution, was used to calculate vertical stresses on top of EPS blocks through various thicknesses of the following types of pavement systems: asphalt concrete, Portland cement concrete (PCC), and a composite pavement system. A composite pavement system is defined here as an asphalt concrete pavement system with a PCC slab separation layer placed between the asphalt concrete pavement system and EPS-block geofoam.

KENLAYER is based on an elastic multilayer system under a circular load. The main assumption in the KENLAYER analysis is that the interface of various pavement system layers and the interface between the pavement system and EPS blocks are frictionless. This assumption yields more conservative values of applied vertical stress on top of the EPS. The vertical stress charts in Figures 41 through 43 of the NCHRP Report 529 can be used to estimate the applied vertical stress on top of the EPS due to a wheel load on top of an asphalt concrete, PCC, and composite pavement system, respectively.

The three supplemental stress distribution methods evaluated during this study to compare with the stresses obtained with the KENLAYER computer program included the 1(horizontal):2(vertical) (1(H):2(V)) stress distribution solution, the varied stress distribution solution, and Odemark's method. A summary of each of these three stress distribution methods is subsequently presented, followed by a comparison of results obtained with each of these three methods with KENLAYER computer program results.

The 1(H):2(V) stress distribution solution is based on the assumption that the applied vertical stress on the pavement surface is distributed over an area of the same shape as the loaded area on the surface, but with dimensions that increase by an amount equal to the depth below the surface, as shown in Figure 4.44 (U.S. Army Corps of Engineers, 1994). For example, for a rectangular shaped loaded area with dimensions of B x L at the surface, the vertical stress at a depth z is assumed to be distributed over an area (B + z) by (L + z). The vertical stress is assumed to be uniform over the stressed area, and is determined by dividing the total applied loads at the surface by the stressed area.

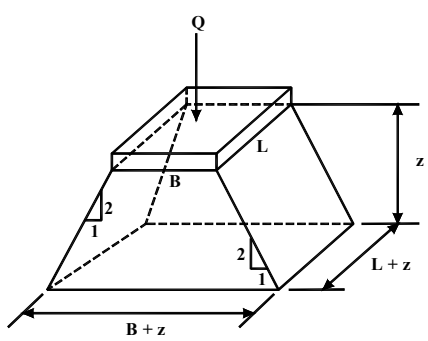


Figure 4.44. Approximate stress distribution by the 1(horizontal):2(vertical) method (U.S. Army Corps of Engineers, 1994).

The vehicle loading included in the 1(H):2(V) stress distribution analysis consists of a 100 kN load on a truck rear axle load that consists of two sets of dual tires. Therefore, each dual tire set consists of a 50 kN load. For the case of a single axle with dual tires, the contact area can be estimated by converting the set of duals into a singular circular area by assuming that the circle has an area equal to the contact area of the duals as indicated by Equation 4.28. The radius of contact is given by Equation 4.29. Equation 4.28 yields a conservative value, i.e., smaller area, for the contact area because the area between the duals is not included.

The contact pressure is typically assumed to be equal to the tire pressure (Huang, 1993) and the tire and pavement surface interface is assumed to be free of shear stress. A tire pressure of 689 kPa (100

lbs/in²) was used in this analysis. Although typical tire pressures for legal highway trucks with single and dual tires range from of 414 to 621 kPa (60 to 90 lbs./in²) (Schroeder, 1984) a tire pressure of 689 kPa (100 lbs./in²) is used for analysis purposes by transportation software such as ILLI-PAVE (Raad and Figueroa, 1980). As indicated by Equations 4.28 and 4.29, a contact area of 0.0726 m^2 and a contact area radius of 0.152m was obtained for the 50 kN dual tire load. Note that the dual tire load is half of the 100 kN total axle load:

$$A_{CD}=\frac{Q_D}{q}=\frac{50kN}{689kPa}=0.0726m^2 \quad \textbf{(4.28)}$$

$$r=\left(\frac{A_{CD}}{\pi}\right)^{\frac{1}{2}}=0.152m \quad \textbf{(4.29)}$$

where

A_{CD} = contact area of dual tires
Q_D = live load on dual tires
q = contact pressure on each tire = tire pressure
r = radius of contact area.

For the 1(H):2(V) method as well as the varied distribution solution, the circular loaded area must be converted to a rectangular loaded area. As discussed in the NCHRP Report 529 (Stark et al., 2004b), the Portland Cement Association 1984 method as described in Huang (1993) can be used to convert the circular loaded area to an equivalent rectangular loaded area, as shown in Figure 4.45. The rectangular area shown is equivalent to a circular contact area that corresponds to a single axle with a single tire, A_C , or a single axle with dual tires, A_{CD}. Use the values of A_C or A_{CD} to calculate the value of L′ in Figure 4.45 by equating A_C or A_{CD} to $0.5227L'^2$ and solving for L′. After solving for L′, the dimensions of the rectangular loaded area in Figure 4.45, i.e., 0.8712L′ and 0.6L′, can be calculated and Figure 4.45 can be used to determine the vertical stress on top of the geofoam based on the 1(H):2(V) stress distribution solution. This stress is called the live load stress from the traffic wheel load, σ_{LL}.

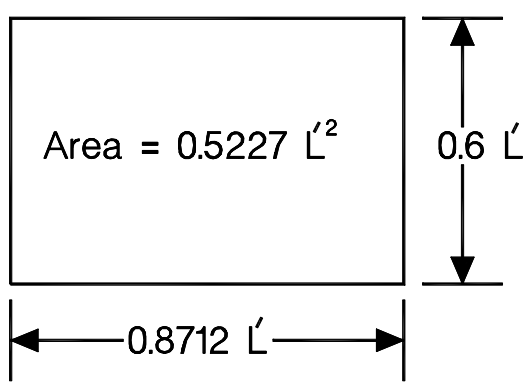


Figure 4.45. Method for converting a circular contact area into an equivalent rectangular contact area (adapted from Huang, 1993).

Stresses resulting from the gravity load or dead load stresses from the weight of the pavement system, σ_{DL} , was determined using Equation 4.30:

$$\sigma_{DL}=\sum h*\gamma \quad \textbf{(4.30)}$$

where

h=thickness of the layer

γ=unit weight of layer

The total vertical stress at the top of EPS blocks immediately underlying the pavement system from traffic and gravity loads, σ_{total}, is given in Equation 4.31:

$$\sigma_{total} = \sigma_{LL} + \sigma_{DL} \quad \textbf{(4.31)}$$

The determination of live-load stresses from traffic loads and dead-load stresses from the weight of the pavement system was performed for various assumed pavement layer system configurations that consisted of an asphalt concrete pavement system, as well as a Portland cement concrete pavement system.

For the asphalt concrete pavement system configurations, an asphalt thickness ranging from 76 to 178mm (3 to 7 in.) was utilized with a corresponding crushed stone base thickness equal to 610mm (24 in.) less the thickness of the asphalt. This provides a pavement system thickness of 610mm (24 in.). A minimum pavement thickness of 610mm (24 in.) was recommended in the Project 24-11(01) reports to minimize the potential for differential icing conditions. The analysis was limited to this pavement thickness because this was the total pavement thickness utilized for the KENLAYER computer program analysis results shown in Figures 41 through 43 in the NCHRP Report 529 (Stark et al., 2004b). As previously indicated, one key difference in the current state-of-practice of pavement systems in the U.S. compared to the state-of-practice at the time the project for stand-alone embankments was ongoing is that currently minimum pavement system thicknesses are on the order of 915mm (36 in.).

For the asphalt concrete, a typical unit weight of 23 kN/m³ (148 lbf./ft³) was used. For the crushed stone base, a unit weight of 22 kN/m³ (138 lbf./ft³). Figure 4.46 shows the vertical stress values obtained for the various asphalt thicknesses.

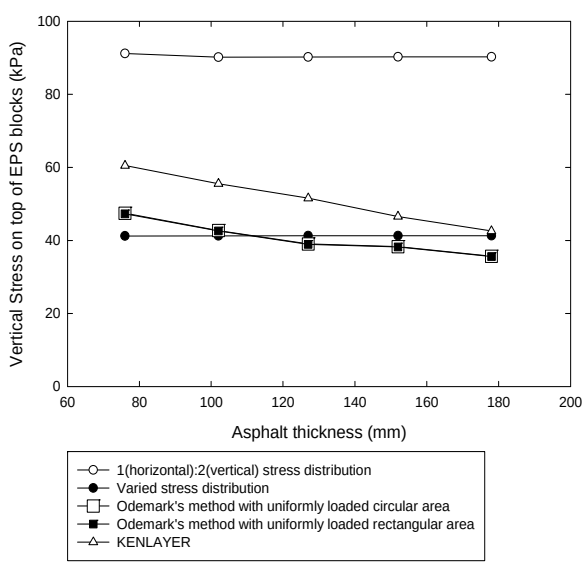


Figure 4.46. Comparison of vertical stress results through an asphalt concrete pavement system based on the various stress distribution methods.

For PCC pavement system configurations, a PCC thickness ranging from 127 to 229mm (5 to 9 in.) was utilized with a crushed stone base thickness equal to 610mm (24 in.), less the thickness of the PCC. This provides a pavement system thickness of 610mm (24 in.). For the PCC, an average unit weight of 23.5 kN/m³ (150 lbf./ft³) was used. The same properties for the crushed stone base and EPS fill used in the analysis of an asphalt concrete pavement system were utilized to develop the vertical stress applied by a PCC pavement system. Figure 4.47 shows the vertical stress values obtained for various PCC thicknesses.

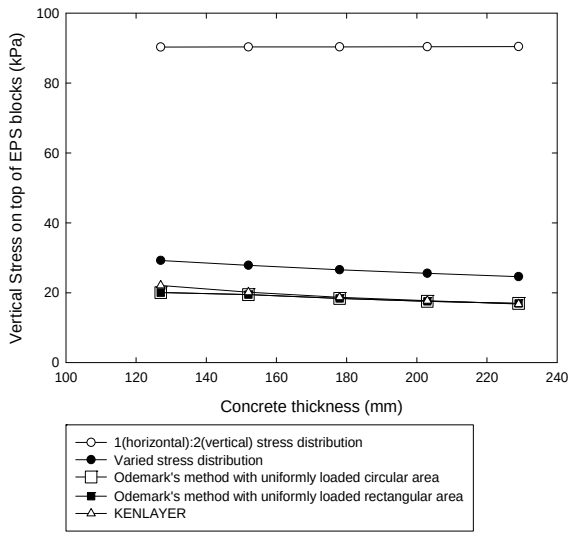


Figure 4.47. Comparison of vertical stress results through a Portland cement concrete pavement system based on the various stress distribution methods.

The varied stress distribution solution considers that load distribution through typical pavement system materials (asphalt concrete, Portland Cement Concrete, granular materials) will generally exceed the distribution of 1(H):2(V) or 26.6 degrees from the vertical. Hunt (1986) indicates that Sowers (1979) suggests an angle of 30 degrees within relatively weak soil, and 45 degrees for relatively strong soil. Greater load-spreading in the range of 35 to 45 degrees may be obtained through stiffer materials such as well-compacted granular fill over soft clay (Jewell, 1996). Therefore, a 1(H):1(V) or 45 degree load distribution can be assumed through pavement materials except for concrete. Concrete can be substituted for granular material using a 1 concrete to 3 gravel ratio (Permanent International Association of Road Congresses, 1997, Refsdal, 1987). This is equivalent to a 3(H):1(V) or 72 degree load distribution. These load distributions of 1(H):1(V) or 45 degrees through asphalt concrete and granular base pavement materials and 3(H):1(V) or 72 degrees for concrete is called the varied stress distribution in this study.

The varied stress distribution analyses were performed using the same truck axle load, tire pressure, conversion of circular loaded area to an equivalent rectangular loaded area procedure, dead load from the weight of the pavement system determination procedure, and total vertical stress determination procedure, and assumed pavement layer system configurations used in the 1(H):2 (V) analyses. Figure 4.46 shows the vertical stress values obtained for varied stress distribution analyses for the various asphalt thicknesses, and Figure 4.47 shows the vertical stress values obtained for various PCC thicknesses.

Odemark's method was the third method evaluated (Odemark, 1949). The Odemark procedure described by Ullidtz (1998) was used in this analysis and a brief summary is subsequently provided. Odemark's method is based on the assumption that the stresses and strains at a given depth is a function of the stiffness of the layer immediately underlying the depth of interest. Therefore, stresses and strains at the depth of interest can be determined by transforming the layers above the depth of interest into a single layer that has the same stiffness as the layer below the depth of interest.

The stiffness of a layer is defined by Equation 4.32 (Ullidtz, 1998):

$$\frac{h^3 E}{1-\nu^2} \tag{4.32}$$

where

h = thickness of the layer
E = modulus of elasticity of the layer
ν = Poison's ratio

A two-layered system can be transformed into a single layer by determining an "equivalent" thickness of the upper layer that will yield the same stiffness as the lower layer, as shown in Figure 4.48. Equation 4.33 or 4.34 can be used to determine the equivalent thickness of the upper transformed layer.

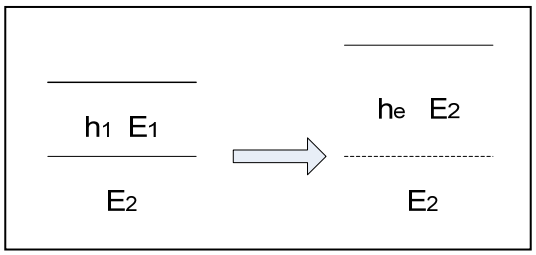


Figure 4.48. Transformation of a two-layered system into a single layer using Odemark's method.

$$\frac{h_1^3 E_1}{1-\nu_1^2} = \frac{h_e^3 E_2}{1-\nu_2^2} \tag{4.33}$$

where

h_e = "equivalent" thickness of the transformed layer.

Solving for h_e, Equation (4.33) becomes

$$h_e = h_1 \sqrt[3]{\left(\frac{E_1}{E_2}\right)\left(\frac{1-\nu_2^2}{1-\nu_1^2}\right)} \tag{4.34}$$

The transformed system shown in Figure 4.48 can only be used for determining stresses, strains, and displacements at the interface of the untransformed, original two layers. Ullidtz (1998) indicates that a correction factor can be introduced into Equation 4.34 to obtain results that will be in better agreement with the theory of elasticity. The correction factors are based on the assumption that Poisson's ratio is the same for all layers. Therefore, Equation 4.35 shows the modified Equation 4.34 with the correction factor, but without Poisson's ratio.

$$h_e = (f)(h_1)\sqrt[3]{\left(\frac{E_1}{E_2}\right)} \tag{4.35}$$

where

f = correction factor = 0.8 except for the first interface where a value of 0.9 is used for a two-layer system and 1.0 for a multi-layer system.

The correction factor values above will give answers reasonably close to theory of elasticity provided that the modulus values are decreasing with depth ($E_i/E_{i+1}>2$) and that the equivalent thickness of each layer is larger than the radius of loaded area (Ullidtz, 1998). The pavement systems evaluated during this study satisfied both of these requirements.

Odemark's method can also be used for multi-layer systems, i.e., systems with more than two layers, by consecutively transforming the top most layer with the layer immediately underlying it into a single layer that has the same modulus as the underlying layer and a modified thickness for the topmost layer. For example, for a three-layer system, the transformation process involves two steps as shown in Figure 4.49:

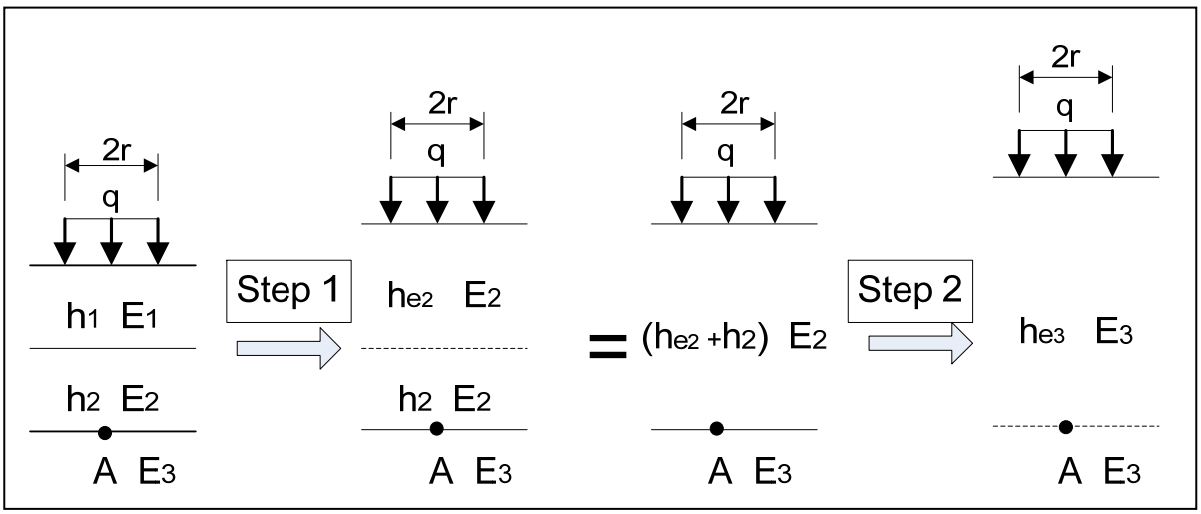


Figure 4.49. Transformation of a three-layered system into a single layer using Odemark's method.

In Step 1, Layer 1 is transformed using Equation 4.36:

$$h_{e2} = (f)(h_1)\left(\sqrt[3]{\frac{E_1}{E_2}}\right) \tag{4.36}$$

where

f = 1.0, because it is the first interface of the multi-layer system.

In Step 2, the transformed layer of Step 1, i.e., the results of transforming Layer 1 and 2 , are transformed using Equation 4.37:

$$h_{e3} = (f)(h_{e2} + h_2)\left(\sqrt[3]{\frac{E_2}{E_3}}\right) \tag{4.37}$$

where

f = 0.8.

Once the layer transformation procedure is completed, the Boussinesq solution (Boussinesq, 1885) for a uniformly loaded circular area is used in conjunction with the final transformed single layer obtained using Odemark's method to obtain the vertical stress due to traffic live load on top of EPS blocks. Equation 4.38 below provides the Boussinesq solution in terms of Odemark's transformed equivalent thickness of the three pavement layers shown in Figure 4.49:

$$\sigma_A = q[1 - \frac{(h_{e3})^3}{(r^2 + (h_{e3})^2)^{1.5}}] \tag{4.38}$$

where

q = uniform pressure=tire pressure
r = radius of contact area from Equation 4.29
z = depth from the top of the pavement

Analyses based on the Odemark method were performed using the same truck axle load, tire pressure, conversion of circular loaded area to an equivalent rectangular loaded area procedure, dead load from the weight of the pavement system determination procedure, and total vertical stress determination procedure, and assumed pavement layer system configurations used in the 1(H):2(V) and varied stress distribution solution analyses. Additionally, for the asphalt concrete pavement system configuration, a modulus of elasticity of 689 MPa (100 x 10^3 lbs./in²) was used for the asphalt concrete, a modulus of elasticity of 21 MPa (3,000 lbs./in²) was utilized for the crushed stone base, and a modulus of elasticity of 9,997 kPa (1,450 lbs./in²) was used for EPS blocks. A modulus of elasticity of 20,684 MPa (3 x 10^6 lbs./in²) was used for the PCC in the PCC pavement system configuration. The material moduli values used in the Odemark analyses are the same values used in the KENLAYER analyses included in the NCHRP Report 529 (Stark et al., 2004b). Figure 4.46 shows the vertical stress values obtained for the various asphalt thicknesses, and Figure 4.47 shows the vertical stress values obtained for various PCC thicknesses based on Odemark's method.

Figure 4.46 shows a comparison of the estimated vertical stress on top of EPS blocks through an asphalt pavement system based on the four methods considered in this study, which are the 1(H):2(V), varied stress distribution solution, Odemark's method, and KENLAYER. The 1(H):2(V) method yielded vertical stress values that were much higher than the other three methods. The varied stress distribution solution and Odemark's methods provide the lowest vertical stress values, and the KENLAYER values were slightly above the varied stress distribution solution and Odemark's method values.

Figure 4.47 shows a comparison of the estimated vertical stress on top of EPS blocks through a PCC pavement system based on the four methods considered in this study. As with the asphalt pavement system results, the 1(H):2(V) method yielded vertical stress values that were much higher than the other three methods. Both the Odemark and KENLAYER results were similar and provided the lower bound values. The varied stress distribution solution yielded values slightly above the Odemark and KENLAYER results.

The Odemark and KENLAYER results are based on a circular loaded area with a radius determined from Equation 4.29 and the 1(H):2(V), and the varied stress distribution solution are based on converting the circular loaded area to an equivalent rectangular contact area as shown in Figure 4.45. In order to determine the potential influence of the rectangular loaded area conversion on stress distribution results, a supplemental analysis was performed using Odemark's method, but instead of using a circular loaded area provided by Equation 4.38, Figure 4.45 was used to convert the circular loaded area to an equivalent rectangular contact area, and Equation 4.39 was used to determine stress distribution. Equation 4.39 provides the increase in stress below the corner of a rectangular loaded area (Budhu, 2007):

$$\Delta\sigma_z = \frac{q_s}{2\pi}\left[\tan^{-1}\frac{LB}{zR_3} + \frac{LBz}{R_3}\left(\frac{1}{R_1^2} + \frac{1}{R_2^2}\right)\right] \quad \textbf{(4.39)}$$

where

$B = width$

$L = length$

$R_1 = \left(L^2 + z^2\right)^{1/2}$

$R_2 = \left(B^2 + z^2\right)^{1/2}$

$R_3 = \left(L^2 + B^2 + z^2\right)^{1/2}$

As shown in Figures 4.46 and 4.47, both the rectangular loaded area and circular loaded area results based on Odemark's method yielded similar results. Thus, the procedure for converting a circular loaded area into a rectangular loaded area depicted in Figure 4.46 and utilized in the 1(H):2(V) and the varied stress distribution solution does not contribute to the differences in stress results shown in Figures 4.46 and 4.47. Therefore, the stress differences can be attributed to the assumed stress dissipation inherent in the four methods.

Based on the results of Figures 4.46 and 4.47, the 1(H):2(V) method is very conservative compared to the other three methods evaluated. The KENLAYER results, which are based on Burmister's elastic layered solution and is the method recommended in the Project 24-11(01) reports (Stark et al., 2004a, Stark et al., 2004b) to estimate stress distribution through the pavement system to obtain the applied vertical stress at the top of EPS blocks due to a loads applied to the pavement surface, is in general agreement with Odemark's method and the varied stress distribution solution. Therefore, the KENLAYER or Burmister's elastic layered solution does not appear to be too conservative. The results of this study suggest that the Odemark solution and varied stress distribution solution may also be viable methods of analysis for preliminary design. However, a definitive recommendation as to which method is best for determining stresses on top of EPS blocks due to wheel loads on the surface of pavement systems as part of the load bearing analysis step cannot be made until data from actual stress wheel load stresses through pavement systems are available.

STEP 11: SETTLEMENT

Step 11 consists of estimating settlement of the proposed EPS-block geofoam slope system. Total settlement of an EPS-block geofoam fill mass embankment, S_{total}, consists of five components as shown by Equation 4.40:

$$S_{total} = S_{if} + S_i + S_p + S_s + S_{cf} \quad \textbf{(4.40)}$$

where

S_{total} = total settlement,
S_{if} = immediate or elastic settlement of the fill mass,
S_i = immediate or elastic settlement of the foundation soil,
S_p = end-of-primary (EOP) consolidation of the foundation soil,
S_s = secondary consolidation of the foundation soil, and
S_{cf} = long-term vertical deformation (creep) of the fill mass.

A summary of settlement analysis procedures to estimate settlement is provided in the NCHRP Project 24-11(01) report (Stark et al., 2004a).

Two sets of settlement tolerances will typically need to be considered: one set of settlement tolerances for the pavement system overlying the EPS-block fill mass, and one for the protective facing panels, if utilized. As indicated in the Project 24-11(01) report (Stark et al., 2004a), tolerable settlements for highway embankments are not well established in practice, nor is information concerning tolerable settlements readily available in the geotechnical literature. Post-construction settlements of 0.3 to 0.6m (1 to 2 ft.) during the economic life of a roadway are generally considered tolerable provided the settlements are uniform, occur slowly over a period of time, and do not occur next to a pile-supported structure (Transportation Research Board, 1975). If post-construction settlement occurs over a long period of time, any pavement distress caused by settlement can be repaired when the pavement is resurfaced. Although rigid pavements have performed well after 0.3 to 0.6m (1 to 2 ft.) of uniform settlement, flexible pavements are usually selected where doubt exists about the uniformity of post-construction settlements and some states utilize a flexible pavement when predicted settlements exceed 150mm (6 in.) (Transportation Research Board, 1975).

If precast facing panels will be used in conjunction with a vertical-sided fill, settlement tolerances for various types of precast facing panels can be obtained in the Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines (Elias et al., 2001).

If the estimated settlement is not feasible, the fill mass arrangement can be revised by changing the thickness, width and/or location of EPS blocks within the slope such that the stresses tending to cause settlement are decreased to obtain the desired settlement. It may be beneficial to partially excavate a portion of the foundation material and replace the excavated material with EPS-block geofoam to limit the final effective vertical stress to a tolerable level.

STEP 12: BEARING CAPACITY (EXTERNAL)

Step 12 consists of evaluation of bearing capacity failure of the foundation material as a potential external failure mode of an EPS-block geofoam embankment. Bearing capacity failure occurs if the applied stress exceeds the bearing capacity of the foundation material, which is related to its shear strength. Failure is only considered through the foundation material because Step 10 addresses internal load bearing failure through the EPS-block geofoam fill.

The general expression for the ultimate bearing capacity of soil, q_{ult}, is shown in Equation 4.41 below (Kimmerling, 2002):

$$q_{ult} = c(N_c) + q(N_q) + 0.5(\gamma)(B_f)(N_\gamma) \quad \textbf{(4.41)}$$

where

q_{ult} = ultimate gross bearing capacity

c = cohesion of soil

N_c = bearing capacity factor for the cohesion term

q = surcharge at the base of the footing

N_q = bearing capacity factor for the surcharge term

B_f = footing width

γ = unit weight of soil beneath the footing

N_γ = bearing capacity factor for soil unit weight

Values for the bearing capacity factors N_c, N_q, and N_γ are based on the friction angle, φ, of the foundation soil and may be obtained from various references, including the FHWA Geotechnical Engineering Circular No. 6 (Kimmerling, 2002).

The presence of ground water near the bottom of the footing may reduce the shear strength of the foundation soil, thus reducing its ultimate bearing capacity. To account for these effects, correction factors, C_{W_q} and C_{W_γ}, can be calculated using Equations 4.42a and 4.42b, as shown below (Kimmerling, 2002):

$$C_{W_q} = 0.5 + 0.5\left(\frac{D_W}{D_f}\right) \leq 1.0 \qquad \textbf{(4.42a)}$$

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{D_W}{1.5B_f + D_f}\right) \leq 1.0 \qquad \textbf{(4.42b)}$$

where

D_W = depth of groundwater
D_f = depth of embedment of footing
B_f = footing width

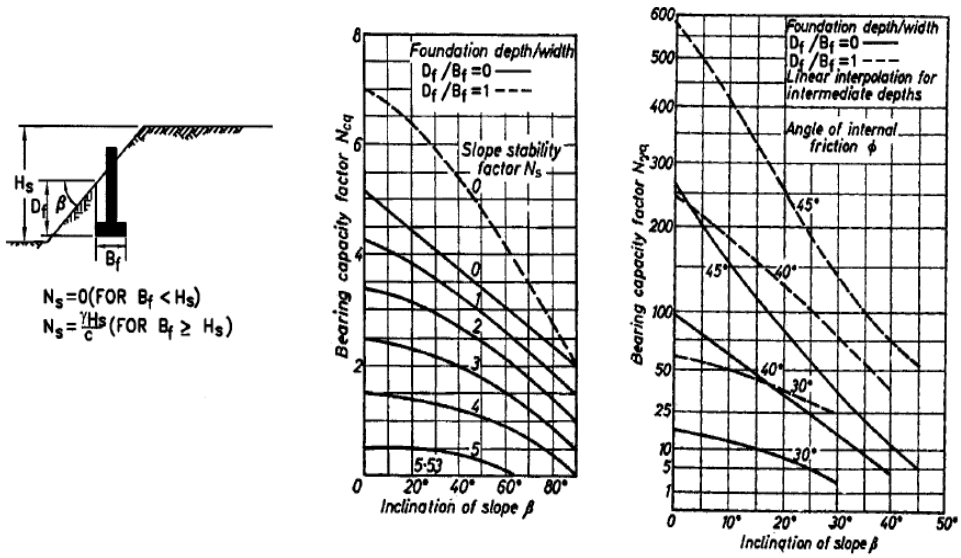
Incorporating the correction factors from Equations 4.42a and 4.42b into Equation 4.41 yields Equation 4.43, shown below, which can be used to calculate the ultimate bearing capacity of the soil, taking into account the effects of the groundwater table.

$$q_{ult} = c(N_c) + q(N_q)(C_{W_q}) + 0.5(\gamma)(B_f)(N_\gamma)(C_{W_\gamma}) \qquad \textbf{(4.43)}$$

Placing a footing on or near a slope, instead of on level ground, can also have an effect on the bearing capacity. The effects of the sloped ground are accounted for by replacing bearing capacity factors, N_c and N_γ, with corrected bearing capacity factors, N_{cq} and $N_{\gamma q}$. The resulting equation for bearing capacity of a shallow footing on sloped ground is shown below (Kimmerling, 2002):

$$q_{ult} = c(N_{cq}) + 0.5(\gamma)(B_f)(N_{\gamma q}) \qquad \textbf{(4.44)}$$

Values for N_{cq} and $N_{\gamma q}$ must be obtained using Figures 4.50 and 4.51, respectively.



a) Geometry *b) Cohesive ($\phi = 0^{\circ}$) soils* *c) Cohesionless ($c = 0$) soils*

Figure 4.50. Modified bearing capacity factor for surcharge, Ncq, for footings on sloping ground (From Standard Specifications for Highway Bridges, 1996, by the Association of State Highway and Transportation Officials, Washington, D.C., used by permission; and Meyerhof (1957) used by permission of Elsevier Limited; Kimmerling (2002)).

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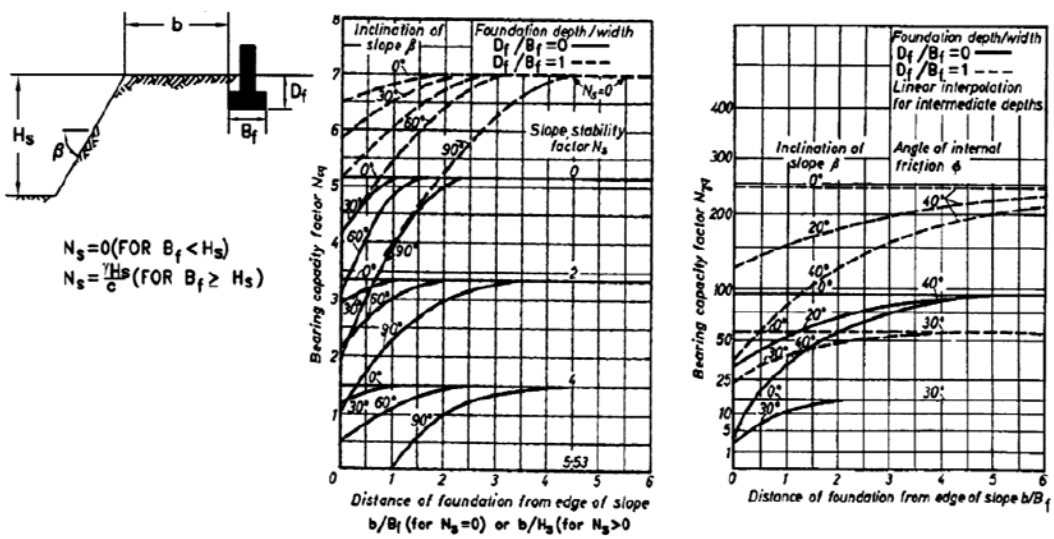

a) Geometry *b) Cohesive ($\phi = 0^{\circ}$) soils* *c) Cohesionless ($c = 0$) soils*

Figure 4.51. Modified bearing capacity factor for surcharge, Nγq, for footings on sloping ground (From Standard Specifications for Highway Bridges, 1996, by the Association of State Highway and Transportation Officials, Washington, D.C., used by permission; and Meyerhof (1957) used by permission of Elsevier Limited; Kimmerling (2002)).

In some cases the friction angle of the soil is not in the range of values provided by Figures 4.50 and 4.51. The general expression for the ultimate bearing capacity of soil, q_{ult}, as given by Coduto (2001), is shown below in Equation 4.45 and can be used when Figures 4.50 and 4.51 are not applicable. For the vast majority of cases involving EPS-block geofoam slope fills, the *s*, *d*, *i*, and *b* factors will all be equal

to 1.0. For special circumstances where this is not the case, Coduto (2001) provides guidance for calculating the appropriate correction factors:

$$q_{ult} = c' N_c s_c d_c i_c b_{ci} g_c + \sigma'_{vD} N_q s_q d_q i_q b_q g_q + 0.5\gamma'_{soil} BN_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma \quad \textbf{(4.45)}$$

where

c' = effective cohesion of the foundation material

σ'_{vD} = vertical effective stress due to soil overburden at bottom of EPS fill

B = width (perpendicular to long axis) of the EPS fill

N_c, N_q, N_γ = bearing capacity factors

s_c, s_q, s_γ = shape factors = 1.0 for most EPS slope fills

d_c, d_q, d_γ = depth factors = 1.0 for most EPS slope fills

i_c, i_q, i_γ = load inclination factors $\leq$ 1.0 for most EPS slope fills

b_c, b_q, b_γ = base inclination factors = 1.0 for most EPS slope fills

$g_c = 1 - \frac{\beta}{147^\circ}$ where β = slope inclination

$g_q = g_\gamma = (1 - \tan\beta)^2$

γ'_{soil} = effective unit weight of soil

In order to calculate the effective unit weight of the soil, γ'_{soil}, for use in Equation 4.45, the location of the ground water table must be identified as falling under one of three different cases:

Case 1 applies when the depth from the ground surface to the groundwater table, D_w, is located at or above the bottom of the EPS-block geofoam fill, i.e., $D_w \leq D$ where D is the depth of the EPS-block geofoam fill. Equation 4.46 is applicable for Case 1.

Case 2 applies when the depth from the ground surface to groundwater table, D_w, falls between the bottom of the EPS-block geofoam fill and lower limit of the zone of influence, which is defined as $D + B$, where D is the depth of the EPS-block geofoam fill and B is the width of the EPS-block geofoam fill. In other words, Case 2 is defined as $D < D_w < D + B$. Equation 4.47 is applicable for Case 2.

Case 3 applies when the ground water table is below the zone of influence (i.e., $D + B \leq D_w$). For Case 3, no ground water correction is needed, as shown in Equation 4.48.

Case 1:
$$\gamma'_{soil} = \gamma_{soil} - \gamma_w \quad \textbf{(4.46)}$$

Case 2:
$$\gamma'_{soil} = \gamma_{soil} - \gamma_w \left(1 - \frac{D_w - D}{B}\right) \quad \textbf{(4.47)}$$

Case 3:
$$\gamma'_{soil} = \gamma_{soil} \quad \textbf{(4.48)}$$

If the desired factor of safety against bearing capacity failure is not feasible, the fill mass arrangement can be revised by changing the thickness, width and/or location of EPS blocks within the slope such that the stresses on the foundation material can be reduced.

STEP 13: FINAL DESIGN DETAILS

The final step of the design procedure, Step 13, consists of preparing final design details of the EPS-block geofoam slope system. In addition to cross-sectional and longitudinal geometry details of the slope, additional details that may require consideration and further analysis include drainage system, road hardware (guardrails, barriers, median dividers, lighting, signage and utilities), and facing system.

The transition zone between geofoam and the natural foundation material in the longitudinal direction along the centerline of the road, as well as between the geofoam and the adjacent upper slope material, should be gradual to minimize differential settlement. The EPS blocks should be stepped as shown in Figure 4.52 as the slope transitions from geofoam fill to natural foundation material. However, a minimum of two layers of blocks is recommended to minimize the potential of blocks to shift under traffic loads. The only exception to this is the final step of the geofoam embankment, which can consist of one block as shown in Figure 4.52.

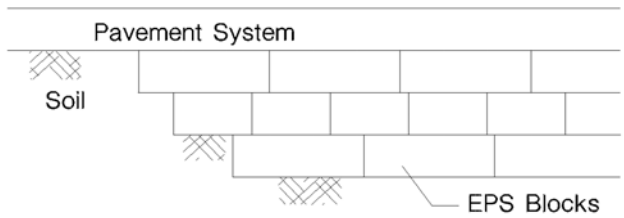

Figure 4.52. Cross-sectional view along longitudinal axis of roadway showing a typical EPS block transition to a soil subgrade (Horvath, 1995).

The drainage system is a critical component of the EPS-block geofoam slope system because the recommended design procedure is based on inclusion of an effective permanent drainage system to prevent hydrostatic uplift (flotation) and translation of the EPS-block geofoam fill mass due to water. Many of the EPS-block geofoam slope case histories evaluated as part of this research included the use of underdrain systems below EPS block to prevent water from accumulating above the bottom of the EPS blocks, and in some cases, incorporated a drainage system between the adjacent upper slope material and EPS blocks to collect and divert groundwater and thereby alleviate seepage pressures.

Road hardware such as guardrails, barriers, median dividers, lighting, signage and utilities can be incorporated in the EPS-block geofoam slope system by direct embedment or structural anchorage. The alternatives for accommodating shallow utilities and road hardware (barriers and dividers, light poles, signage) is to provide a sufficient thickness of the pavement system to allow conventional burial or embedment within soil or, in the case of appurtenant elements, provide for anchorage to a PCC slab or footing that is constructed within the pavement section. Barriers or guardrails are typically required with vertical-sided embankments. Design of traffic railings is addressed in Section 13 of the Bridge Design Specifications (AASHTO, 2007) and in the AASHTO Road Design Guide (AASHTO, 2002). Several road hardware details are provided in the Construction Practices section of this chapter. Additional loads such as vehicle impact loads may need to be considered. The experience with the design of the CA/T EPS-block geofoam stand-alone embankments is that these impact loads were a significant design consideration.

If a vertical-sided fill is used, a facing wall system will be required to protect the EPS blocks. The facing does not have to provide any structural capacity to retain blocks because the blocks are self-stable.

The primary function of the facing wall is to protect blocks from damage caused by environmental factors. A summary of various types of protective facing systems is included in Chapter 5. Regardless of the type of facing system used, the resulting vertical stress on the foundation soil must be considered in calculations for both settlement and global stability. The weight of the facing elements should be obtained from a supplier or estimated to ensure that the correct weight is used in calculations for that specific type of facing system.

The figures included in Chapter 5 and design details included in Appendix G will facilitate development of final design details.

SUMMARY

This section presented background information on the design methodology incorporated in abbreviated form in the recommended design guideline included in Appendix B. Design of an EPS-block geofoam slope system considers the interaction of three major components as shown in Figure 4.4: existing slope material, the fill mass, and the pavement system. The three potential failure modes that can occur due to the interaction of these three primary components of an EPS slope system that must be considered during stability evaluation of an EPS-block geofoam slope system include external instability of the overall EPS-block geofoam slope system configuration, internal instability of the fill mass, and pavement system failure.

Design for external stability of the overall EPS-block geofoam slope system considers failure mechanisms that involve existing slope material only, as shown in Figure 4.5, as well as failure mechanisms that involve both the fill mass and existing slope material, as shown in Figure 4.6. The external stability failure mechanisms included in the proposed design procedure consist of static slope stability, settlement, and bearing capacity. Additional failure mechanisms associated with external seismic stability include seismic slope instability, seismic induced settlement, seismic bearing capacity failure, seismic sliding, and seismic overturning.

Design for internal stability considers failure mechanisms within the EPS-block geofoam fill mass. The three internal instability failure mechanisms evaluated in the design guideline are seismic horizontal sliding, seismic load-bearing of EPS blocks, and static load-bearing of EPS blocks.

The objective of pavement system design is to select the most economical arrangement and thickness of pavement materials for the subgrade provided by the underlying EPS blocks. The design criteria are to prevent premature failure of the pavement system, as well as to minimize the potential for differential icing (a potential safety hazard) and solar heating (which can lead to premature pavement failure) in those areas where climatic conditions make these potential problems. Also, when designing the pavement cross-section overall, consideration must be given to providing sufficient support, either by direct embedment or structural anchorage, for any road hardware (guardrails, barriers, median dividers, lighting, signage and utilities).

Figure 4.25 shows the recommended design procedure for EPS-block geofoam slope fills. All steps are required if the existing or proposed roadway is located *within* the limits of the existing or anticipated slide mass <u>and/or</u> the existing or anticipated slide mass is located *below* the roadway as shown in Figure 4.17(b) and 4.18(b). If the existing or proposed roadway is located *outside* the limits of the existing or anticipated slide mass <u>and/or</u> the existing or anticipated slide mass is located *above* the roadway as shown in Figure 4.17(a) and 4.18(a), the design procedure does not include Steps 8 and 9, which are directly related to design of the pavement system, because the EPS-block geofoam slope system may not include a pavement system. Procedures to analyze each step are summarized in this chapter.

The recommended design procedure is applicable for both slope-sided fills and vertical-sided fills as depicted in Figures 4.2(a) and 4.2(b), respectively, except that overturning of the entire fill mass at the interface between the bottom of the assemblage of EPS blocks and underlying foundation material as a result of horizontal forces that is part of external seismic stability, Step 6, is applicable primarily for only vertical-sided fills.

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CHAPTER 5

CONSTRUCTION PRACTICES

INTRODUCTION

This chapter provides an overview of construction tasks frequently encountered during EPS-block geofoam slope projects. This discussion serves as a supplement to the proposed standard and commentary included in Appendix F and construction practices discussion included in the NCHRP Project 24-11(01) report (Stark et al., 2004). Lessons learned from case histories are also provided. The construction task list presented herein is by no means exhaustive, nor will all of these tasks be applicable to every project. However, based on a review of available case history information, the tasks discussed herein represent common issues that may be encountered in EPS-block geofoam slope construction.

Most figures used in this report consist of construction photographs of a slope repair project on AL 44 near Guin, AL, and a bridge abutment project associated with the Route 1 bridge over I-95 that was completed as part of the Woodrow Wilson Bridge Replacement project in Alexandria, VA. Photographs of the exterior insulating and finishing system (EIFS) incorporated in EPS-block geofoam ramps as part of the Central Artery/Tunnel (CA/T) project, also known as the Big Dig, in Boston, MA, are also included. These photographs are used to illustrate topics discussed in the text; however, efforts were also made to arrange photographs to show the construction sequence of each project. Thus, the photographs may not be referenced in numerical order in the text.

Appendix G includes design details from the CA/T project, the Route 1/I-95 project, and the I-15 Reconstruction Project in Salt Lake City, UT. Although the Route 1/I-95 and I-15 projects do not involve slope stabilization or repair, some design details may be applicable to EPS-block geofoam slope stabilization systems and may be useful in developing site-specific drawings or details. Appendix H includes the Virginia Special Provision for Block-Molded Expanded Polystyrene Lightweight Fill (EPS-block fill) that was incorporated in the Route 1/I-95 project specifications and the Alabama Special Provision for Geofoam Blocks utilized as part of the specifications for the AL 44 slope stabilization project. Additionally, the EIFS protective facing specifications utilized for the Big Dig EPS block ramps and shotcrete protective facing specifications utilized as part of the Topaz, ID, bridge project are also included.

Construction topics subsequently presented include site preparation; drainage; EPS block shipment, handling, and storage; CQA/CQC of EPS blocks; block placement; backfill between EPS blocks and adjacent earth slope; phased construction; accommodation of utilities and roadway hardware; facing wall; earth retention system; pavement construction; and post-construction monitoring. The figures are located at the end of this chapter instead of within the text because of the large quantity of figures.

SITE PREPARATION

The first step in the construction process is site clearing, grubbing, and excavation. Figures 5.1 and 5.2 show the effects of slope movement prior to repair of a slope on AL 44 near Guin, AL. Figure 3.9 provides a cross-section of the landslide prior to placement of EPS-block geofoam and Figure 3.10 provides a cross-section of the repaired slide. Figure 5.3 shows the initial site clearing and grubbing, and Figure 5.4 shows the initial excavation of slope material. An overview of the slope excavation is provided in Figures 5.5 and 5.6.

Once the excavation has commenced, it is crucial that the excavated slope remains stable. Figure 5.7 shows the beginnings of cracks forming near the top of the cut slope. Cracks such as these serve as an indication that a portion of the slope is under tension and may be in need of temporary stabilization to prevent sloughing. In the case of the slope shown in Figure 5.7, local instability did not become serious enough to require additional stabilization measures. However, local sloughing of the exposed slope can be relatively sudden. Loose fill and/or highly weathered material that are typically encountered at shallow

depths are conditions especially conducive to local instability (Byrne et al., 1998). Byrne et al. (1998) suggest that local instability of the excavation facing during soil nail wall construction is not amenable to conventional analysis, and is typically addressed during design by a field test cut to demonstrate that the face can stand unsupported for sufficient time to allow nail and facing installation. Local instability of the excavated slope during an EPS-block geofoam slope stabilization project can also be addressed during design by a field test cut to verify that the slope can stand unsupported for sufficient time to allow placement of EPS blocks. As in soil nail wall construction, local stability of the slope facing during excavation can be an important consideration during EPS-block geofoam slope stabilization.

A stable adjacent slope is required not just for the construction phase, but for the entire life of any EPS-block geofoam slope designed according to the recommended design guideline provided in this report. Because the mass of EPS-block fill is typically very small, it may not be feasible for the EPS fill to directly resist external applied earth forces from the adjacent slope material. Therefore, the design procedure is based on a self-stable adjacent upper slope to prevent earth pressures on the EPS fill mass that can result in horizontal sliding between blocks. If adjacent slope material cannot be cut to a long-term stable slope angle, an earth-retention system must be used to resist the applied earth force. A discussion of earth-retention systems is included later in this construction practices section.

Once the slope has been excavated, the next step is to prepare the foundation material to provide a planar platform for placing the first layer of EPS blocks. Prior to placement of the first layer of blocks, the foundation surface must be prepared such that it provides a smooth, planar surface. Failure to adequately remove plants, rocks, and other obstructions from the placement area may lead to major difficulties when EPS blocks are placed. Ensuring that the first layer of EPS block is smooth and stable is essential to constructing the remainder of the fill. A sand bedding layer is sometimes used to ensure a stable working platform. If a granular drainage layer is utilized as part of a drainage system, the granular layer can also aid in providing a stable working platform. Figures 5.17 and 5.18 show the process of checking the planarity of the drainage layer prior to placement of the first row of EPS blocks during the AL 44 project. Figure 5.34 shows the use of a granular leveling layer overlying a geotextile as part of the Route 1/I-95 project.

DRAINAGE

Many of the EPS-block geofoam slope case histories evaluated as part of this research included the use of underdrain systems below EPS blocks to prevent water from accumulating above the bottom of the EPS block, and in some cases, incorporated a drainage system between the adjacent upper slope material and EPS blocks to collect and divert seepage water and thereby alleviate seepage pressures. Therefore, based on current design precedent, it is recommended that all EPS-block geofoam slope systems incorporate drainage systems. The recommended design guideline is based on use of an adequate drainage system and, consequently, the design procedure does not specifically consider the hydrostatic uplift or translation due to water accumulation failure mechanisms.

An adequate permanent drainage system is a critical component of an EPS-block geofoam slope system, and is one key difference between EPS-block geofoam applications in slopes versus stand-alone embankments. For example, Figures 5.8 and 5.9 show the presence of free water due to ground water seepage during the AL 44 project excavation. Therefore, a permanent drainage system that consisted of perforated underdrains within a granular layer overlying a geotextile was utilized, as shown by Figures 5.10 and 5.11. The drainage system should ensure water collected within the drainage system is adequately diverted away from the slope. Figures 5.28 and 5.29 show the completed concrete drainage channel constructed as part of the AL 44 project drainage system that diverts water collected in the underdrains away from the completed EPS-block geofoam stabilized slope.

For projects in colder climates, it may be necessary to consider the effects of prolonged periods of freezing weather on the subsurface drainage system. The exposed ends of drainage pipes, such as those used for the Guin, AL, slide repair shown in Figure 5.29, could potentially become clogged by ice during a prolonged freeze and contribute to the development of hydrostatic and seepage pressures beneath the

EPS-block geofoam fill. Therefore, the drainage system should be designed to minimize the potential for clogging during prolonged freezing weather.

Drainage of water seepage from the adjacent upper slope should also be considered. Figures 5.25 and 5.26 show the use of granular backfill being placed between the EPS-block fill and adjacent upper slope material. The use of a granular backfill may assist in draining any water seepage to the underlying underdrains and, thus, alleviate any seepage pressures on the EPS-block fill mass.

It should be noted that in addition to a permanent drainage system, a temporary dewatering and drainage system may be required during construction due to natural seepage from the excavated slope. During construction, inadequate temporary drainage may lead to water collecting in and around the area where EPS blocks are being placed. Because the density of EPS is much lower than that of water, buoyancy forces on EPS blocks can potentially cause major shifting of the blocks, as well as erosion of foundation material from beneath the bottom layer of EPS block, as shown in Figure 5.44. Figure 5.44 shows the consequences of improper temporary drainage and the effects of block shifting during a heavy rain event during the Route 1/I-95 project. Figure 5.45 shows the temporary drainage system implemented after problems with the heavy rain event occurred. Although problems such as those depicted in Figure 5.44 are not particularly difficult to avoid with proper planning, they are not uncommon on EPS-block geofoam projects. Therefore, it is recommended that careful attention be given to providing an adequate drainage system at the job site to help prevent the additional costs, both in terms of time and money, of floating and shifting of EPS blocks during construction.

EPS-BLOCK GEOFOAM SHIPMENT, HANDLING, AND STORAGE

Similar to other types of geosynthetics, the period between manufacturing and installation completion is when EPS blocks are most vulnerable to damage. However, unlike some geosynthetics, e.g., geomembranes, EPS blocks cannot be repaired once damaged. Therefore, precautions should be taken to prevent damage to EPS blocks from the time they are removed from the mold until they have been safely placed and covered at the project site. Although proper handling and storage of EPS blocks is generally not difficult or expensive, it is frequently neglected on projects simply because of carelessness or poor planning. In many cases, the responsibility for protecting EPS blocks prior to their arrival at the job site rests with the manufacturer. However, it is important for the agency responsible for construction oversight to set forth some specific requirements regarding block quality and condition in the planning phase of the project, and to enforce these requirements throughout the duration of the project. Once EPS blocks arrive on site, the agency responsible for overseeing construction should ensure that the blocks are handled and stored properly. There are many different approaches and techniques to handling that have been tried over the years, with varying degrees of success. The purpose of this report section is to clarify what is considered good practice for shipping, handling, and storing EPS blocks so agencies involved in these activities can make informed decisions regarding their own standards of practice.

The EPS blocks may be shipped in enclosed trailers as shown in Figure 5.12 or on open flat-bed trailers, as shown in Figure 5.35. The advantage of using enclosed trailers to ship blocks to the project site is that the potential for damage is less compared to open trailers. However, the disadvantage of using enclosed trailers is that unloading blocks may be more labor intensive, because the blocks may need to be moved to the exit of the trailer prior to unloading, as shown in Figure 5.12. This is not a large inconvenience because of the light weight of the blocks. The use of open trailers allows for easier access to the blocks during loading and unloading, as shown in Figure 5.35 and 5.36, because crews or equipment can simply lift blocks off from all sides of the trailer.

If blocks are shipped via open flat-bed trailers, it is recommended that the edges of corners of exposed blocks be covered by structural angles or some other similar protective material to prevent damage from the straps used to secure the EPS blocks to the trailer, as shown in Figure 5.35. The use of open flat-bed trailers appears to be more common because of the shift in the state-of-practice to use longer EPS blocks. Regardless of what type of trailer is used to transport the blocks, advanced planning is

required to ensure that the project site is accessible to delivery trailers, and that a plan is established to transport the blocks from the delivery trailers to the placement location within the fill mass of the slope.

Various methods can be utilized to unload blocks from delivery trailers. For example, Figures 5.12 through 5.14 demonstrate use of a specialized gripper lifting device. Figures 5.35 and 5.36 show the use of a forklift to unload the blocks. Figure 5.37 shows use of straps to unload the blocks. The bottom edges of the blocks contain a steel angle to minimize damage to the blocks. However, no steel angle was used along the top. Thus, the top block was damaged due to the stress from the strap, as well as the swinging that occurred due to use of only one strap. If straps such as this are used to unload blocks, angles or other guards along the top and bottom block edges should be used to prevent block damage, which is irreversible.

The blocks are either immediately placed at the planned location within the fill mass or may be temporarily stockpiled in a staging area while the proposed fill mass location is prepared for block placement. If a staging area is used, it should have adequate temporary drainage to prevent flotation of the blocks. Additionally, adequate overburden such as the use of "soft" weights should be applied to the top of the blocks to prevent the blocks from being picked up or displaced by high winds. While this might not immediately be obvious as a significant hazard, there is at least one documented case of a field technician being struck and killed by an unsecured EPS geofoam block that had been picked up by a strong wind gust on the order of 80 km/hr (50 miles/hr). (MIFACE, 2007). The blocks at this job site also shifted due to inadequate temporary drainage. Therefore, it is recommended that provisions be made to secure or ballast EPS blocks, whether in the staging area or construction area, to prevent shifting or other movement until they are permanently secured. The extent of ballast required to secure blocks should be based on specific project site conditions during the duration of EPS block placement. The Michigan Case Report of the wind fatality incident recommends the following practices. First, contractors should secure/ballast geofoam block edges in accordance with manufacturer specifications for installation and storage. Second, construction employers should conduct a daily hazard assessment to determine if environmental working conditions have changed or will change. They should inform their employees of their findings and how the changing conditions may affect the work to be performed. Third, trade groups involved in the manufacture and installation of geofoam should develop a guideline for geofoam applications as foundation material in excavations. The third recommendation was made because the incident involved the use of EPS blocks within an excavation for support of a building foundation.

The incident report indicates that after the incident, the employer involved in the incident implemented a wind hazard assessment program that consists of the following: (1) the Beaufort scale (wind speed scale) was distributed to all field employees and kept in their toolbox, and (2) laptops were used to monitor wind speed. If the wind speed is in excess of 32 km/hr (20 miles/hr), geofoam blocks are not to be placed.

If the construction schedule is such that any EPS blocks will be exposed to direct ultraviolet (UV) rays from sunlight for a long period of time, the blocks should be temporarily protected to prevent UV damage to the EPS due to sunlight exposure. Figure 5.46 shows the use of plastic sheeting secured with sandbags to protect the EPS blocks from direct sunlight exposure. Prolonged exposure to sunlight can cause the surface of the EPS geofoam blocks to discolor and begin to crumble. In the case of the I-15 reconstruction project in Salt Lake City, UT, UV damage became a problem during construction. The practice of removing the degraded surface of the EPS blocks using a pressure washer to remove the degraded surface was successfully implemented and allowed the damaged blocks to still be used (Bartlett et al., 2000). The I-15 experience with surficial degradation was quite atypical and, in fact, is the only known case history, going back to 1972, when EPS-block geofoam was first used as a lightweight-fill material where such degradation was a problem that needed to be addressed. Therefore, currently there is no established methodology for temporarily protecting EPS blocks from UV deterioration during construction. The use of a dark-colored geomembrane may not be the best practice for the purpose of EPS protection against UV rays because temperatures under the geomembrane due to solar heating may physically damage the EPS even more than direct UV radiation would.

CQC/CQA OF EPS-BLOCK GEOFOAM

An important component of any construction plan involving EPS-block geofoam is the provision for monitoring and verifying the properties of EPS blocks that are used. This process involves manufacturing factors, many of which are not directly related to the construction process. Because this chapter deals with construction, this section is intended to address only those issues related to the construction process. A more detailed discussion of manufacturing quality control (MQC) and manufacturing quality assurance (MQA), as well as additional commentary on current practices relating to EPS geofoam construction quality control and quality assurance (CQC/CQA) is provided in the recommended material and construction standard included in Appendix F and in Chapter 6.

The primary goal of the CQC/CQA plan for an EPS-block geofoam construction project is generally to ensure that the EPS blocks that are used meet the design requirements. An overview of various methods used to comply with the requirements of CQC/CQA that have been utilized in various projects is provided in this section.

Each EPS block should be labeled to indicate the name of the molder (if there is more than one supplying the project), the date the block was molded, the mass/weight of the entire block, dimensions of the block, and actual dry density/unit weight. Proper labeling is especially important on projects that will require multiple block types within the fill mass. Figures 5.15 and 5.42 illustrate several types of labels. Figure 5.16 demonstrates the use of a scale to check the weight and density of EPS blocks in the field. Photographs showing EPS blocks being checked to ensure they meet dimensional tolerances are provided in Figures 5.38 and 5.39.

In addition to ensuring that the correct EPS block type is placed, it is also important to ensure that methods being used by the contractor to construct the overall EPS-block geofoam slope produce an acceptable slope system that complies with the assumptions inherent in the recommended design. For example, the design procedure included in Appendix B assumes that the adjacent slope is self-stable to prevent earth loads from developing on the EPS-block fill mass and that an adequate drainage system is provided to prevent hydrostatic and seepage forces from developing within the EPS fill mass. Therefore, it is necessary to monitor the construction process to ensure that the adjacent slope is stable, e.g., monitoring with slope inclinometers, and that the drainage system is constructed properly.

BLOCK PLACEMENT

The size of blocks will typically be dependent on the capability of the molder that will supply the blocks. During the EPS geofoam product demonstration showcase held on July 25, 2006 at the US Route 1 North Abutment of the I-95 Interchange of the Woodrow Wilson Bridge (WWB) project, the subcontractor that placed the EPS blocks indicated that the use of shorter blocks, such as 2.4 m (8 ft.) length instead of the 4.8 m (16 ft.) lengths that were actually used, may have resulted in more rapid placement because shorter blocks are easier to handle during placement.

Various methods can be used to place blocks within the proposed fill mass. For example, blocks may be moved by hand using an installation crew, as shown in Figure 5.19. For large blocks, the blocks can be placed using straps to move each block into place, as shown in Figure 5.40. Figure 5.20 shows a gripper carrying device that allowed two people to carry a single block instead of four. Figure 5.21 is a close-up view of the gripper device. However, the recommended standard included in Appendix F does not allow willful damage to an EPS block for any purpose during handling and placement. Therefore, use of carrying tools such as those shown in Figure 5.21 is not recommended unless no damage will occur to the block. The use of commercially-available scissor clamps to place each block, as shown in Figure 5.22, (typically used to move bundles of timber railroad ties), will not damage the block. The scissor clamp shown in Figure 5.22 was used to lift and place blocks weighing up to 2.2 kN (500 lbs).

Blocks should be placed tightly against adjacent blocks on all sides. Every effort should be made to eliminate gaps at vertical joints between blocks. Blocks should generally be placed so the long dimension of the blocks in each layer is turned perpendicular to that of the long dimension of the blocks

in the underlying layer, as shown in Figure 5.47. This vertical interlocking minimizes vertical shifting and sliding between blocks. Photographs of the EPS-block geofoam placement process is provided in Figures 5.23 and 5.47.

Common practice for cutting and trimming blocks consists of the use of wire saws, chain saws, and hot wire cutters, as shown in Figure 5.41. However, field cutting of blocks should be minimized, because field cutting impacts the block placement rate and may be more expensive than cutting blocks at the plant.

If the calculated resistance forces along the horizontal planes between EPS blocks are insufficient to resist the horizontal driving or imposed forces, additional resistance between EPS blocks is required to supplement the inherent inter-block friction. This is generally accomplished by adding mechanical inter-block connectors (typically prefabricated barbed metal plates, along the horizontal interfaces between EPS blocks. Because of the relative cost of these plates, they should only be used where calculations indicate their need. A discussion on the interface frictional resistance between EPS blocks, as well as alternatives to increase the interface shear strength, is provided in Chapter 3.

Negussey et al. (2001) and Sheeley (2000) indicate that mechanical plate connectors have been ineffective in resisting seismic loads based on laboratory tests. However, full-scale shake-table tests in Japan in the late 1990s demonstrate that while plowing of the mechanical connector barbs through the surface of EPS blocks does occur, the presence of mechanical connectors is nevertheless essential for the overall stability of an assemblage of EPS blocks whenever seismic loads are to be resisted. Therefore, the present state of knowledge suggests that a conservative approach is warranted so any EPS-block geofoam slope system designed for seismic loading should incorporate mechanical connectors on all horizontal surfaces between blocks. Further information regarding placement of mechanical connectors as well as other potential alternatives to increase the interface shear resistance between blocks is included in Chapter 3.

In addition to their role in resisting horizontal design loads, mechanical connectors have been useful as a constructability tool to keep EPS blocks in place when subjected to wet, icy, or windy working conditions (Horvath, 2001a) and to prevent shifting under traffic where relatively few layers of blocks are used (Duskov, 1994).

Photographs of mechanical connectors used in practice are provided in Figures 5.24 and 5.43. One potential alternative to the use of mechanical connectors to minimize sliding during seismic events is the inclusion of geofoam shear keys (Sheeley, 2000). A shear key is essentially a block or portion of a block of EPS geofoam that is placed at various locations within the fill mass so as to interrupt the horizontal failure plane between layers of EPS blocks. The use of polyurethane adhesives, which are used for roofing applications, can be effective in providing additional shear resistance between EPS blocks in the future, once long-term durability testing is available that indicates that the shear strength will not degrade with time. Additional information on shear keys and adhesives is included in Chapter 3.

Surfaces of EPS blocks should not be directly traversed by any vehicle or construction equipment during or after placement of the blocks. This issue is subsequently addressed in the section titled Pavement Construction.

BACKFILL BETWEEN EPS BLOCKS AND ADJACENT EARTH SLOPE

Once EPS blocks have been placed, the area between the EPS-block geofoam fill and existing slope should be backfilled. Granular backfill material, as shown in Figure 5.25 and 5.26, is the most suitable backfill material type because of the narrow fill area that the backfill material must be placed. Additionally, granular material may also function as part of the drainage system along the EPS fill and adjacent earth slope interface.

One issue raised as part of the AL 44 slide correction project involved payment quantity of EPS block versus backfill material at the interface between EPS blocks and the adjacent cut slope. In this project, the contractor initially extended the EPS blocks into the adjacent cut slope instead of cutting the blocks and backfilling the void between the blocks and adjacent cut soil slope interface. The contractor requested payment for the additional volume of EPS block required to extend the blocks to the cut slope.

The requested cost was higher than the estimate made based on limiting the extent of EPS blocks to within 1 foot of the cut slope, because of the greater cost associated with the additional volume of EPS blocks required to extend the blocks to the cut slope. To alleviate this potential pay quantity discrepancy, it is recommended that the drawings specifically show the limits of EPS block placement along the EPS block and adjacent earth slope.

PHASED CONSTRUCTION

When necessary, an EPS-block geofoam fill can be constructed in phases, allowing one portion of the fill to be completed before beginning construction on the next portion. The advantage of this approach is that it can eliminate the need to completely close an existing roadway to repair the unstable portion. As shown by Figure 5.48, temporary sheeting was used as part of the New York State Route 23A slope stabilization project to support the excavation during soil removal and replacement due to the depth of soil removal required, and the need to maintain one lane of traffic at all times adjacent to the excavation (Jutkofsky, 1998, Jutkofsky et al., 2000).

Another case history involving phased construction of an EPS-block geofoam fill was a bridge approach on a section of Payne Road near South Portland, ME. In this instance, the initial layout of the EPS-block geofoam fill did not consider the need for phased construction. To enable traffic to continue using a portion of the road during construction, sheet piles were driven to separate the project area into sections which could be excavated separately and filled with EPS-block geofoam. A photograph of a section of sheet piles is included in Figure 5.49. The empty space between the edge of the EPS block and the sheet pile wall was filled with concrete. The sheet piles were left in place after the completion of each section because of concerns that extracting them would damage the geomembrane liner that was utilized to protect the EPS blocks against fuel spills (Maguire, 2007).

Use of sheet piles in phased construction work does not allow for the EPS blocks to be interlocked between the various EPS-block fill mass sections that are separated by sheet piles. Although the resulting continuous vertical joint does not appear to have had any negative impact on the completed NY State Route 23A and the Maine Payne Road EPS block embankments, in seismic-prone areas, the seismic stability analysis should consider the presence of any continuous vertical joint(s) on the overall EPS-block fill mass behavior subjected to seismic shaking.

One alternative approach to use of sheet piles in phased construction of a roadway embankment with EPS-block geofoam proposed in Japan is depicted in Figure 5.27. This method has the advantage of allowing sections of EPS-block geofoam fill to interlock with each other. Figure 5.27 shows that it may be possible to leave a “saw-tooth” pattern of blocks at the edge of each layer within a given section to allow blocks placed as part of a subsequent section to fit in between them. This stepped-construction approach is currently being used on the Topaz Bridge project in Idaho, which is the first use of EPS-block geofoam on a public road in that state.

Phased construction was utilized as part of an EPS vertical sided embankment ramp during the CA/T Project in Boston. The temporary EPS ramp was constructed to occupy nearly the same general alignment as the permanent ramp, except that the temporary portion of the ramp consisted of additional EPS blocks placed on the sides of the permanent ramp location (Riad, 2005). Once the temporary additional extended ramp width was no longer needed, the temporary EPS blocks were removed and an EIFS covering was installed on the exposed surface of the permanent EPS blocks. Use of a temporary EPS ramp that occupied nearly the same alignment with a permanent EPS ramp is believed to be the first ever implemented on an EPS transportation structure (Riad, 2005).

ACCOMMODATION OF UTILITIES AND ROAD HARDWARE

Road hardware such as guardrails, barriers, median dividers, lighting, signage and utilities can be incorporated in the EPS-block geofoam slope system by direct embedment or structural anchorage. The alternatives for accommodating shallow utilities and road hardware (barriers and dividers, light poles,

signage) is to provide a sufficient thickness of the pavement system to allow conventional burial or embedment within soil or, in the case of appurtenant elements, provide for anchorage to a PCC slab or footing constructed within the pavement section. Barriers or guardrails are typically required with vertical-sided embankments. Design of traffic railings is addressed in Step 13 of the recommended design guideline discussion in Chapter 4. Several road hardware details are provided in the design details included in Appendix G as well as in the Project 24-11(01) report (Stark et al., 2004).

FACING WALL

If a vertical-sided fill is used, a facing system will be required to protect EPS blocks. The facing does not have to provide any structural capacity to retain the blocks because the blocks are self-stable, especially if mechanical connectors are used. The primary function of the facing wall is to protect the blocks from damage caused by environmental factors. The selection of the type of facing system is based on three general criteria: (1) facing must be self-supporting or physically attached to EPS blocks, (2) architectural/aesthetic requirements, and (3) cost. The following materials have been successfully used for facing geofoam walls:

- prefabricated metal (steel or aluminum) panels,
- precast PCC panels, either full height or segmental (such as used in mechanically stabilized earth walls, MSEWs),
- segmental retaining wall (SRW) blocks, which are typically precast PCC,
- shotcrete,
- geosynthetic vegetative mats, and
- exterior insulation finish systems (EIFS).

Other materials that might be suitable for facing geofoam walls include:

- wood panels or planks, and
- EPS-compatible paint for temporary fills

Shimanuki et al. (2001) report use of a vegetation EPS-block geofoam protection cover system consisting of a steel net with attached planting sheets that contain plant seeds. Soil-filled sand bags are placed between the planting sheets and EPS blocks to support vegetative growth. Construction details included in Appendix G provide details for various facing systems. Details for the I-15 Reconstruction Project in Salt Lake City, UT include precast PCC panel details. The details for the CA/T project include details for an EIFS facing wall. Figures 5.30 through 5.33 include photos of the EIFS and EPS block mock-up wall used for the CA/T project, and Appendix H provides the EIFS specifications. The EIFS consisted of a mesh-reinforced, two-part coating system field applied over a mounting EPS 40 board adhered to the EPS embankment exposed surface (Riad, 2005). The advantages of the EIFS panels compared to the initially proposed precast concrete curtain walls is summarized below (Riad, 2005):

- Significantly lighter EIFS panels reduced applied loads on the existing subgrade.
- EIFS with an EPS mounting board is compatible with EPS blocks used to create fill from the standpoint of dead loads, stiffness, deformations and other mechanical and material properties. This minimized the potential for differential movement between the two elements.
- It simplified design, construction and maintenance through elimination of the pinned connections tying the exterior panels and the load-distribution slab located at the top of the EPS blocks.
- EIFS could be applied at any time after the EPS blocks are in place, thereby providing a more flexible schedule that would allow structures to open sooner to traffic.

- The results of fire tests indicated that structural damage to EPS block substrate is limited and the structural integrity of EPS blocks was not adversely impacted. Thus, the EIFS panels provide significant protection for EPS blocks. The EIFS satisfied the 30-minute fire resistance requirement established by the Boston Fire Department.

Shotcrete protective facings are also economical and can be quickly applied. Approximately 929m^2 (10,000 ft^2) of EPS blocks were covered with a dry-mix process shotcrete during the CA/T project. Jamieson (2003) provides details of the shotcrete system used and application procedure. Shotcrete protective facing will also be used as part of the Topaz, ID Bridge project. A copy of the shotcrete specifications from the Topaz, ID project are included in Appendix H. Bartlett et al. (2009) provide results of bond strength and impact resistance tests performed on a proprietary shotcrete system applied to EPS blocks.

EARTH RETENTION SYSTEM

If the adjacent slope material cannot be cut to a long-term stable slope angle, an earth-retention system must be used to resist the applied earth force. An anchored facing system can be used to support the adjacent earth forces as shown in Figure 4.11(a). This approach, developed in Norway (Horvath, 1995) and also utilized in Japan (Tsukamoto, 1996), consists of placing one or more intermediate horizontal slabs of poured-in-place reinforced Portland cement concrete (PCC) within the EPS as the blocks are placed. The PCC slabs are connected to the facing system and anchored into the adjacent earth slope. The anchor system may consist of ground anchors or a geosynthetic, such as a geogrid. A second earth retention system consists of a gravity or cantilever retaining wall designed to retain both the EPS blocks and earth material. A retaining wall system is illustrated in Figure 4.11(b). A key distinction between a facing wall system and a retaining wall system is the purpose of a facing system is only to protect the EPS blocks from damage, not to support any lateral loads, whereas the primary purpose of a retaining wall system is to resist lateral loads imposed by the retained EPS blocks and adjacent earth material. A secondary purpose of a retaining wall system can be to function as a covering system and protect the EPS blocks. A third potential earth retention system consists of a reinforced soil slope system designed to retain the adjacent earth as shown by Figure 4.11(c).

PAVEMENT CONSTRUCTION

The pavement system is defined for the purposes of the design standard in Appendix F as all material placed above EPS blocks within the limits of the roadway, including any shoulders. Care must be exercised when constructing the pavement system so the separation layer (if one is used) and/or EPS blocks are not damaged.

A separation layer between the top of EPS blocks and the overlying pavement system can have two functions. First, the separation layer can function to enhance the overall performance and life of the pavement system by providing reinforcement, separation, and/or filtration. A separation layer used for these purposes is technically part of the pavement system. Second, the separation layer can enhance the durability of EPS blocks both during and after construction. A detailed discussion about separation materials is included in the Project 24-11(01) report (Stark et al., 2004).

Use of a 100 to 150mm (4 to 6 in.) thick reinforced PCC slab is currently the state of practice primarily because it is considered a necessity for providing sufficient lateral confinement of unbound pavement layers and load distribution when using EPS-block geofoam, and because of historical usage of PCC slabs dating back to the earliest EPS-block geofoam lightweight fills in Norway in the 1970s. The original function of the PCC slab was for pavement reinforcement, and the intent was to allow use of a minimum pavement system thickness. In later designs, the PCC slab was also used to function as a barrier against potential petroleum spills. However, use of a PCC slab for this function is questionable due to usual long-term development of cracks in PCC slabs. PCC slabs generally represent a significant relative

cost, so PCC slabs should only be used if specifically required as determined during design. Examples of EPS slope stabilization projects that did not have a PCC slab above the EPS blocks include the AL 44 project near Guin, AL and the County Trunk Highway "A" project in Bayfield County, WI. Alternative separation layers for reinforcement that can be considered in pavement design include a geogrid, a reinforced geomembrane that will also resist hydro-carbon spills, geocell with soil or PCC fill, and soil cement.

One application where a PCC slab is typically required is when an embankment with vertical sides, i.e., geofoam wall, is used (Horvath, 2001b). However, this standard practice may be changing because the Project 24-11(01) research revealed that the cost of a PCC slab represents a significant cost to an EPS-block geofoam embankment. For example, a 9 to 12m (30 to 40 ft.) high by 152m (500 ft.) long vertical-sided embankment currently being constructed as part of the Topaz Bridge project on U.S. 30 at Topaz, ID, does not include a PCC slab. The pavement system above the EPS blocks consists of a galvanized steel mesh mechanically stabilized earth wall system backfilled with gravel that extends the full width of the four-lane highway.

The primary function of the PCC slab in an embankment with vertical sides is to support the upper part of the exterior facing system. A secondary function is to provide anchorage for various highway hardware, such as safety barriers, signage, and lighting. A PCC slab used for these functions will act primarily as a structural member for the benefit of other embankment system components and not the EPS. Therefore, the PCC slab should be designed for the intended function.

In general, the pavement system can be constructed in the normal manner with only a few cautions related to the presence of EPS blocks. The most critical phase of pavement construction is placement and compaction of the initial lift or layer of soil on the separation layer or EPS blocks. Vehicles and construction equipment such as earthmoving equipment must not directly traffic on the EPS blocks or separation layer (even if a PCC slab is used, as it is still possible to overstress the underlying EPS). The type and size of construction equipment should be limited to wheel, track, or roller loads that produce maximum applied stresses that do not exceed the elastic limit stress of the EPS, i.e., in no case should vehicle loads exceed the elastic limit stress of the EPS. After completion of the pavement system, vehicle loads should not exceed the design vehicle load.

It may be possible to utilize the various traffic stress distribution procedures included in Chapter 4 to estimate the thickness of soil or aggregate that may be required to ensure the applied stresses from specific construction equipment types that will be used on a project do not exceed the elastic limit stress of the EPS blocks. However, as indicated in Chapter 4, a definitive recommendation as to which stress distribution method is best for determining stresses on top of the EPS blocks due to construction equipment loads through soil and/or aggregate cannot be made until data from actual vehicle stresses through pavement systems are available.

POST-CONSTRUCTION MONITORING

The majority of EPS-block geofoam projects in the U.S. have not utilized any post-construction monitoring program, including major projects such as the EPS-block geofoam ramps constructed as part of the Big Dig project in Boston, MA. However, most of these projects involved the construction of EPS-block geofoam stand-alone embankments rather than slope stabilization fills. In determining the need for and/or planning a post-construction monitoring program, the first step is to define the parameter(s) to be measured that will aid in answering specific questions about the behavior and long-term performance of the EPS-block geofoam slope system. As noted by Elias et al. (2001), every instrument on a project should be selected and placed to assist in answering a specific question. "If there is no question, there should be no instrumentation."

For example, the recommended design guideline is based on an adequate drainage system to minimize hydrostatic and seepage uplift pressures below the EPS-block fill mass as well as adjacent to the EPS-block fill mass. Therefore, if there is concern about the long-term performance of the proposed

drainage system, post-construction monitoring of the long-term behavior of the drainage system, such as visual observation at outflow points or piezometers, may be useful.

Parameters of interest in EPS-block geofoam slope stabilization and repair projects that may be measured can be categorized into parameters that measure the characteristics of the subsurface conditions including conditions below and adjacent to the EPS-block fill mass, behavior of the EPS-block fill mass, behavior of the pavement system, performance of the overall drainage system, and performance of the facing system. Measured parameters can be further subcategorized into the following purposes (Elias et al., 2001): (1) confirm design stress levels and monitor safety during construction, (2) allow construction procedures to be modified for safety or economy, (3) control construction rates, (4) enhance knowledge of the behavior of EPS-block geofoam slope systems to provide a base reference for future designs, with the possibility of improving design procedures and/or reducing costs, and (5) provide insight into maintenance requirements, by long-term performance monitoring.

Information about instrumentation for post-construction monitoring can be found in various textbooks (Dunnicliff, 1988).

SUMMARY

This chapter provides an overview of construction tasks that are frequently encountered during EPS-block geofoam slope projects. The construction topics presented include site preparation; drainage; EPS block shipment, handling, and storage; CQA/CQC of EPS blocks; block placement; backfill between EPS blocks and adjacent earth slopes; phased construction; accommodation of utilities and road hardware; facing wall; earth retention system; pavement construction; and post-construction monitoring. This chapter is a supplement to the material and construction standard included in Appendix F of this report, and construction practices discussion included in the NCHRP Project 24-11(01) report (Stark et al., 2004).

Figures and photographs that may aid in preparation of bid and construction documents are included as part of this construction practice overview. Additionally, Appendix G includes various design details and Appendix H includes example specifications utilized in geofoam projects. The construction details included in Appendix G, which were obtained from actual geofoam construction drawings used in projects throughout the United States, can be used as a guide for developing site-specific drawings or details. The details presented relate to a variety of geofoam issues, such as configuration of the EPS blocks, inclusion of utilities and roadway hardware, construction of a load distribution slabs over the EPS, and construction of facing walls.

FIGURES

Figure 5.1. Pavement cracking due to slope movement, AL 44 near Guin, AL (Alabama Department of Transportation).

Figure 5.2. Scarp due to slope movement, AL 44 near Guin, AL (Alabama Department of Transportation).

Figure 5.3. Site clearing and grubbing, AL 44 near Guin, AL (Alabama Department of Transportation).

Figure 5.4. Excavation to prepare for placement of EPS-block geofoam (Alabama Department of Transportation).

Figure 5.5. Excavated slope at site of slide repair, AL 44 near Guin, AL (Alabama Department of Transportation).

Figure 5.6. Overview of slope excavation (Alabama Department of Transportation).

Figure 5.7. Cracks developing from potentially sloughing material within excavation (Alabama Department of Transportation).

Figure 5.8. Presence of water at the bottom of cut (Alabama Department of Transportation).

Figure 5.9. Extent of water collected in bottom of excavation (Alabama Department of Transportation).

Figure 5.10. Placement of separation geotextile layer and installation of subsurface drainage system (Alabama Department of Transportation).

Figure 5.11. Placement of pipe drains to divert water away from the area where EPS-block geofoam is to be placed (Alabama Department of Transportation).

Figure 5.12. Preparing to unload EPS-block geofoam from trailer upon arrival on site (Alabama Department of Transportation).

Figure 5.13. Using specialized lifting device to lift EPS-block geofoam down from trailer upon arrival on site (Alabama Department of Transportation).

Figure 5.14. Lifting an EPS-block geofoam using specialized lifting device (Alabama Department of Transportation).

Figure 5.15. EPS-block geofoam with manufacturer's tag to identify block properties and date of manufacture (Alabama Department of Transportation).

Figure 5.16. Preparing to weigh EPS block as part of CQA/CQC to check density of EPS blocks being placed on site (Alabama Department of Transportation).

Figure 5.17. Checking to ensure levelness of granular drainage layer prior to placement of EPS blocks (Alabama Department of Transportation).

Figure 5.18. Leveling and checking the prepared granular drainage layer to ensure the area is ready for EPS blocks to be placed (Alabama Department of Transportation).

Figure 5.19. Placement crews moving EPS blocks by hand (Alabama Department of Transportation).

Figure 5.20. Placement crews lifting and moving EPS blocks using specialized lifting tool (Alabama Department of Transportation).

Figure 5.21. Close-up photo of specialized lifting tool (Alabama Department of Transportation).

Figure 5.22. Use of a scissor clamp to place blocks as part of the new Topaz Bridge project in Idaho (Horvath).

Figure 5.23. Overview of EPS-block geofoam being placed into prepared site (Alabama Department of Transportation).

Figure 5.24. Use of mechanical connector plates between EPS blocks (Alabama Department of Transportation).

Figure 5.25. Placement of granular backfill behind EPS-block geofoam fill (Alabama Department of Transportation).

Figure 5.26. Placing and leveling granular backfill being placed behind EPS (Alabama Department of Transportation).

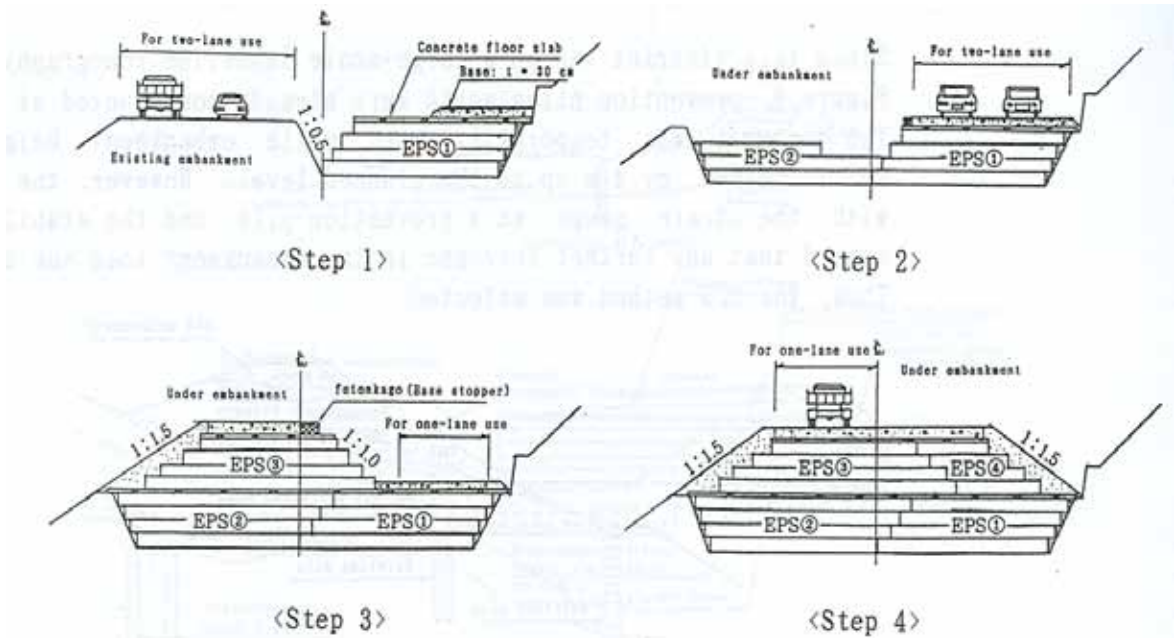

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Figure 5.39. Checking to ensure EPS blocks meet dimensional tolerances (Virginia Department of Transportation).

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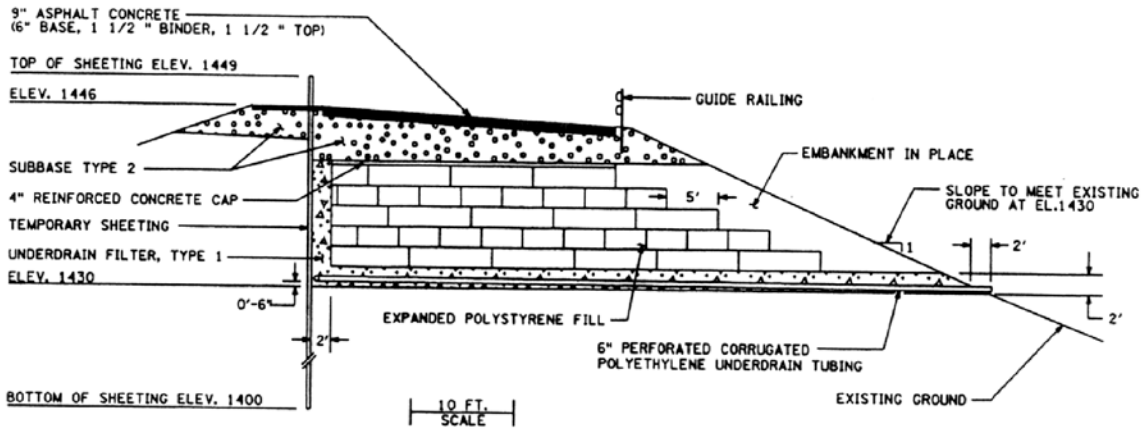

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CHAPTER 6

RECOMMENDED EPS-BLOCK GEOFOAM STANDARD FOR SLOPE STABILITY APPLICATIONS

INTRODUCTION

A recommended combined material, product, and construction standard covering block-molded EPS for use as lightweight fill in stand-alone road embankments and related bridge approach fills on soft ground was developed during NCHRP Project 24-11(01) (Stark et al., 2004a; Stark et al., 2004b). The recommended standard was intended to be used to create a project-specific specification. Specifications for two major, high-profile projects were based on the recommended Project 24-11(01) standard. These projects are the EPS-block geofoam ramps constructed as part of the Boston Central Artery/Tunnel (CA/T) project, also known as the Big Dig, in Boston, MA and the Woodrow Wilson Bridge Replacement project in Alexandria, VA. Additionally, the specification for a slope repair project on AL 44 near Guin, AL, was also based on the recommended Project 24-11(01) standard.

The objective of this current Project 24-11(02) study was to modify the Project 24-11(01) standard to make it specific to geofoam usage in slope stability applications. To understand the basis of the recommended standard for use of EPS-block geofoam for lightweight fill in slope stabilization included in Appendix F, it is necessary to understand the basis of the Project 24-11(01) standard for stand-alone embankments and developments related to EPS-block geofoam technology since development of the Project 24-11(01) standard. Therefore, this chapter provides an overview of the basis of the NCHRP Project 24-11(01) standard. Also included is a historical overview of EPS-block geofoam standards since the Project 24-11(01) standard was developed.

BASIS OF THE PROJECT 24-11(01) STANDARD FOR STAND-ALONE EMBANKMENTS

When we began our research for NCHRP Project 24-11(01) for stand-alone embankments at the very end of the 20th century, there was no geofoam-specific standard in the U.S. for any EPS product or functional application, even though EPS had existed as a construction material for approximately 50 years. This technological vacuum presented the challenge of developing a zero-based geofoam-specific standard as one of the significant work tasks and eventual outcomes of the Project 24-11(01) project. In retrospect a decade later, this challenge was actually much easier to deal with than our Project 24-11(02) requirements. Our challenge here involves reassessing, revising, and updating the Project 24-11(01) standard for the current Project 24-11(02) study for slopes. In many ways, the current situation, with its proliferation of conflicting standards, and generic state DOT specifications derived from them, presents an ever-changing, conflicting technological landscape. These conflicts may be arguably more confusing to design professionals, EPS molders, and construction contractors involved in a project using EPS-block geofoam for road construction than when there were no standards for guidance. Ultimately, this confusion and conflict adds costs in both time and funding for the project owner, stakeholders, and all too often, taxpayers. To understand how this current situation came about requires a detailed presentation of developments related to the Project 24-11(01) study.

The current, relatively widespread knowledge and use of EPS as a geofoam material in the U.S., primarily in its generic block-molded product form for the geosynthetic functional application of lightweight fill, began in the early 1990s. There is documented geotechnical engineering use of EPS going back to the early 1960s (EPS was invented circa 1950), but this early use can be considered experimental and not widespread, and certainly not widely disseminated in terms of technology transfer as has occurred over the last two decades.

The standard produced as part of the Project 24-11(01) study was developed during the 1999-2000 timeframe. The de facto 'stand-in' standard used then for most EPS-block geofoam projects in the U.S. was ASTM Standard C 578 titled *"Standard Specification for Rigid, Cellular Polystyrene Thermal*

Insulation," referred to hereinafter as "C 578." This standard, which had developed and evolved over many years, was and still is called *rigid cellular polystyrene* (RCPS). RCPS is the collective technical term for both EPS and XPS (*extruded polystyrene*) used in relatively thin (no more than a few tens of millimetres/inches thick) planks or panels for thermal insulation of building envelopes, e.g., within stud walls, or flat, membrane-type roofs. The common element of such applications is that RCPS is subjected to little or no load-bearing once installed. As such, the *thermal* properties of RCPS are the material properties of primary interest, especially the coefficient of thermal conductivity.

As was well known by the early 1990s, the primary shortcoming of C 578 for broader use in lightweight fill, small-strain geofoam applications that are *load-bearing* in nature is that this standard did not, and still does not, contain any criteria for small-strain stiffness material properties. Such critical criteria include the *elastic-limit stress* and *initial secant Young's modulus* (originally and formerly called *initial tangent Young's modulus*). In an EPS-block geofoam context, "small strain" is generally defined as being less than or equal to 1 percent compressive normal strain in unconfined compression under service loads. As discussed in detail in our Project 24-11(01) report, current design practice for all small-strain EPS-block geofoam applications such as lightweight fill is serviceability, not strength, based. This is important to note, given the growing trend toward using LRFD as the design methodology in geotechnical engineering. Consequently, small-strain stiffness material properties of block-molded EPS are essential parameters in any standard and project specification covering small-strain geofoam applications.

This need for supplementing material property requirements contained in C 578 with additional properties necessary to design for load-bearing was recognized by the U.S. EPS industry (referred to hereinafter as "Industry") as early as 1992 when they began development of product-specific technical literature. This material first appeared in print in 1994. Note that this was well before Project 24-11(01) commenced, and approximately five years before we began development of the standard eventually incorporated into the Project 24-11(01) report. A copy of this product literature is in Appendix H, originally developed and issued by AFM® Corporation. This company was, and still is, a marketing cooperative or consortium of individually owned and otherwise independent EPS block molders in North America. Significantly, in the mid-1990s when this product literature first appeared, AFM members collectively dominated the U.S. block-molded EPS business. This is now no longer the case. Consequently, their product literature and influence on the marketplace throughout the 1990s has significance and relevance to this discussion.

There are several important items in the AFM® Corporation document in Appendix H. First, the Roman-numeral material *types* (sometimes referred to as material *grades*) in C 578 were used for product reference purposes. Although it is possible in theory to make block-molded EPS at any density, for simplicity, this term will be used synonymously for *unit weight* as well, even though this is not strictly correct. Within a certain density range governed by material and production factors, it has long been standard practice—not only in the U.S. but around the world—to routinely manufacture EPS blocks only to certain specific standardized densities for overall manufacturing, inventory, and sales efficiency. In C 578, these standard material types and their corresponding densities are defined using Roman numerals. Note also that the material property requirements for compressive and flexural strength defined in C 578 were used in the AFM 1994 document as well.

Additional material properties for small-strain stiffness were included in the AFM 1994 document, specifically, the aforementioned elastic-limit stress (called "stress @ 1 percent strain" in that document) and initial secant Young's modulus (called "Modulus of Elasticity" in that document). The numerical values of these additional material properties were based on laboratory compression tests commissioned by AFM, specifically for this document, using material produced by AFM member plant(s). As can be seen, representative plots of compressive normal stress versus compressive normal strain in unconfined axial compression are also presented.

Another important but subtle point in the AFM 1994 product literature is that the various C 578 material types are clearly indicated to have two densities, "nominal" and "minimum." In consonance with Industry practice at the time, nominal density of block-molded EPS was intended to reflect the **average** density of an entire block (as would typically be used in a lightweight fill application), whereas

the minimum density was that which might exist for a relatively smaller piece, such as a *manufacturing quality control* (MQC) or *manufacturing quality assurance* (MQA) test specimen, cut from a block. To understand how and why use of two densities for a single block of EPS makes sense in this context requires an understanding of two important factors. The factors are related to the actual science of EPS block molding and historical use of block-molded EPS in decades prior to its use for geofoam applications such as lightweight fill.

First, it has long been known that when molding blocks of EPS, there will always be density variations (also called density *gradients*) within a block that are simply inherent and unavoidable in the EPS manufacturing process. This can be likened to cooking or baking food, wherein there will always be some varying degree of 'doneness' within the piece of food. The relative magnitude of these variations for a given block of EPS is variable. Such variables include numerous hardware and production factors, such as mold type and dimensions, as well as specific molding protocols used by an EPS molder. But the point remains—density variations have, and will *always* exist in every block of EPS.

The relevance of this is that historically, block-molded EPS was cut into relatively thin panels, sometimes as thin as 25 to 50mm (1 to 2 in.), for commercial sale for thermal insulation applications. This was, and still is, a significant market for EPS. This market is the reason C 578 was developed decades ago and is still a current standard. In addition, relatively small specimens for material property testing are also routinely cut from a block (cubes used for EPS compression testing are only 50mm (2 in.) wide). So even if a block were molded to C 578 Type I guidelines with a nominal, i.e., overall average, density of 16 kg/m^3 (1.0 lb./ft^3) for the entire block, it is virtually guaranteed that some particular thin panel and/or small test specimen cut from that block will have a slightly lower density. Thermal properties of EPS are relatively insensitive to its absolute density: lower-density material tends to have somewhat better thermal resistance properties simply because it contains more air volume; air volume provides the thermal resistivity of the material. As the original and historical use of block-molded EPS was for thermal insulation, it is no surprise that C 578 evolved. Hence, a particular piece of EPS cut from a block was allowed to have a slightly (approximately 10 percent) lower density than the average block as a whole, because the thermal insulation properties of that slightly-less-dense material were not compromised by relatively small deviations from the nominal or average density of the entire block. This lower allowable density is referred to as the "minimum" density in standards such as C 578, specifications, and product literature such as AFM 1994. So, for C 578 Type I material with the above-stated nominal or overall average block density of 16 kg/m^3 (1.0 lb./ft^3), an individual thin panel for commercial sale or a test specimen for quality-related testing would be considered acceptable from a mass/weight perspective if it had a density as low as 15 kg/m^3 (0.9 lb./ft^3), which is the minimum allowable for that material type. Note that this slightly-less-dense material would still have to meet all other material property minima for compressive and flexural strength, etc., stated in C 578. The importance and relevance to the current discussion of understanding the logical, defensible context in which this nominal versus minimum density for each C 578 material type evolved cannot be understated.

Unfortunately, by the 1990s, then-current versions of C 578 were only indicating the minimum required density for each of the material types covered by that document. One might argue that because the purpose of C 578 was to specify only ***minimum*** required values of material properties for quality-related purposes, specifying a nominal density was irrelevant, and even potentially misleading. For example, to produce C 578 Type I material with a minimum density of 15 kg/m^3 (0.9 lb./ft^3) at any point within a block one molder might, based on experience and manufacturing protocols, target a nominal density of 16 kg/m^3 (1.0 lb./ft^3) for the overall block. Another molder, using a different mold and protocol, might find that they have to target a slightly higher nominal value. However, as will be seen, this lack of formal mention of both nominal and minimum densities in ASTM standards has evolved to create problems in current practice. Nevertheless, as can be seen from the AFM 1994 product literature, there is no doubt that in that time frame, Industry was very aware of the difference between nominal and minimum densities, and was using the higher nominal values when marketing EPS for geofoam applications. In fact, because entire blocks and not thin panels are used in the vast majority of small-strain

geofoam applications, it is obvious that nominal or average block density and properties are more relevant than minimum properties that will only show up in small MQC and MQA test specimens.

When we developed the Project 24-11(01) standard, we were familiar on a first-hand basis with not only the history of C 578 and use of dual nominal/minimum densities in Industry, but the AFM 1994 product literature as well. Although we developed the Project 24-11(01) standard using a zero-based approach, from the outset we recognized the need for practicality and pragmatism for whatever standard we developed. Specifically, to produce a standard that would be immediately usable and attractive in practice, we appreciated the need to be consistent with routine manufacturing capabilities and practices of Industry at that time. Therefore, we adopted the same nominal or average block densities of 16 kg/m^3 (1.0 lb./ft^3), 20 kg/m^3 (1.25 lb./ft^3), 24 kg/m^3 (1.5 lb./ft^3), and 32 kg/m^3 (2.0 lb./ft^3) that Industry routinely produced and marketed at that time, as reflected in the AFM 1994 product literature and confirmed through direct discussions with appropriate Industry personnel. We elected not to use the lowest C 578 standard density of 12 kg/m^3 (0.75 lb./ft^3), then in routine production, as we felt such material would not have routine use in road applications involving stand-alone embankments due to its relatively small load-bearing capability.

However, though we wanted what eventually became the Project 24-11(01) standard to be in consonance with then-current Industry practice, after much consideration, we decided not to use C 578 Roman-numeral material type designations. We felt they were not intuitive and, in fact, were confusing by not being arranged in any logical order, e.g., monotonically increasing or decreasing in magnitude with density. Rather, we used a useful, intuitive material-type nomenclature such as "EPS100," wherein the number relates to *small-strain stiffness material properties* of that grade of EPS. Thus, with nomenclature closely related to its application, our Project 24-11(01) standard allows anyone to see the material-type designation label and immediately know the maximum allowable compressive stress that could be applied to that material (100 kPa in this example) relative to their specific project needs.

In the Project 24-11(01) standard, we used the same compressive and flexural strengths as in C 578 to be in consonance with Industry routine production material properties. Finally, we adopted the nominal/ minimum density philosophy of Industry as reflected in C 578 and AFM 1994 product literature. While typically entire blocks of EPS are used in small-strain geofoam applications, as opposed to thin panels used in thermal insulation for buildings, we recognized that the small specimens used for MQC/MQA testing in geofoam applications might have a slightly lower density than the overall block average, depending on where, from within the block, the test specimen sample was taken. So while we required in the Project 24-11(01) standard that an entire block must have a density that equals or exceeds the nominal, overall average value corresponding to its material type, we did allow for a slightly lower minimum density to be applicable to an individual MQC/MQA test specimen. This philosophy, which was completely consistent with Industry practice at the time, is shown in Table 6.1, which was taken from our Project 24-11(01) report.

Table 6.1. Minimum allowable density values from (01) standard for stand-alone embankments.

Material Type Designation		**Minimum Allowable Density/Unit Weight, kg/m^3 (lb./ft^3)**	
NCHRP 24-11(01)	**ASTM C 578**	**Block as a Whole**	**Any MQC/MQA Test Specimen**
EPS40	I	16 (1.0)	15 (0.90)
EPS50	VIII	20 (1.25)	18 (1.15)
EPS70	II	24 (1.5)	22 (1.35)
EPS100	IX	32 (2.0)	29 (1.80)

In summary, when developing the Project 24-11(01) standard, the interests of both end user and EPS molder were primary factors in producing a document of immediate practical value and use to all concerned. Thus, this standard incorporated all of the standard EPS densities, material properties for

density and compressive and flexural strengths, and Industry practices related to the use of dual nominal and minimum densities in routine production and use in the U.S. at the time.

The most challenging part of developing the Project 24-11(01) standard was selection of small-strain stiffness material properties of elastic-limit stress and initial tangent Young's modulus (as it was called). This required considerably more thought than other issues; there was no U.S. standard for these properties in effect at that time. The AFM 1994 product literature reflected guaranteed minimum small-strain stiffness material properties, but only from molders who were members of the AFM consortium. While these molders dominated the U.S. market, they were not the only molders capable of producing EPS blocks for the geofoam market. Consequently, we broadened our considerations as the intent of the Project 24-11(01) standard to be generic, rather than reflecting business interests or practices of any one group. Therefore, producing minimum required small-strain stiffness material properties as a function of EPS density for the standard, we considered not only values in AFM 1994 product literature, but also values from published literature worldwide known at the time. Details of the database we used can be found referenced in the Project 24-11(01) report.

All values considered in developing small-strain stiffness material properties for Project 24-11(01) standard are shown plotted in Figure 6.1. Note that only the elastic-limit stress is shown as the initial tangent Young's modulus (as it was called at the time) is linearly proportional to that parameter. Also shown is the density-stress relationship we eventually assumed for the Project 24-11(01) standard. Clearly, values chosen for the Project 24-11(01) standard were somewhat conservative relative to the above-referenced database. Given that our Project 24-11(01) standard was the first of its kind in the U.S., it was incumbent upon us to take a reasonably conservative posture toward material requirements. Note that values in the Project 24-11(01) standard are noticeably less than those given in the AFM 1994 product literature.

In summary, we must emphasize that the crucial and critical small-strain EPS stiffness material properties reflected in the Project 24-11(01) standard were well below material properties that the majority of U.S. EPS molders at that time promised customers in their own product literature. Further, values in the Project 24-11(01) standard were consistent with the broader body of available international knowledge. Therefore, we conclude that the overall material requirements reflected in the Project 24-11(01) standard, summarized in Table 6.2, were more than reasonable, as they were overall no more stringent than properties required or stated in either C 578 standards or manufacturer's existing product literature.

Table 6.2. Summary of minimum allowable material properties of MQC/MQA test specimens from (01) standard.

Material Designation	**Dry Density/ Unit Weight, kg/m^3 (lb./ft^3)**	**Compressive Strength, kPa (lbs./in^2)**	**Flexural Strength, kPa (lbs./in^2)**	**Elastic-Limit Stress, kPa (lbs./in^2)**	**Initial Tangent Young's Modulus, MN/m^2 (lbs./in^2)**
EPS40	15 (0.90)	69 (10)	173 (25)	40 (5.8)	4 (580)
EPS50	18 (1.15)	90 (13)	208 (30)	50 (7.2)	5 (725)
EPS70	22 (1.35)	104 (15)	276 (40)	70 (10.1)	7 (1015)
EPS100	29 (1.80)	173 (25)	345 (50)	100 (14.5)	10 (1450)

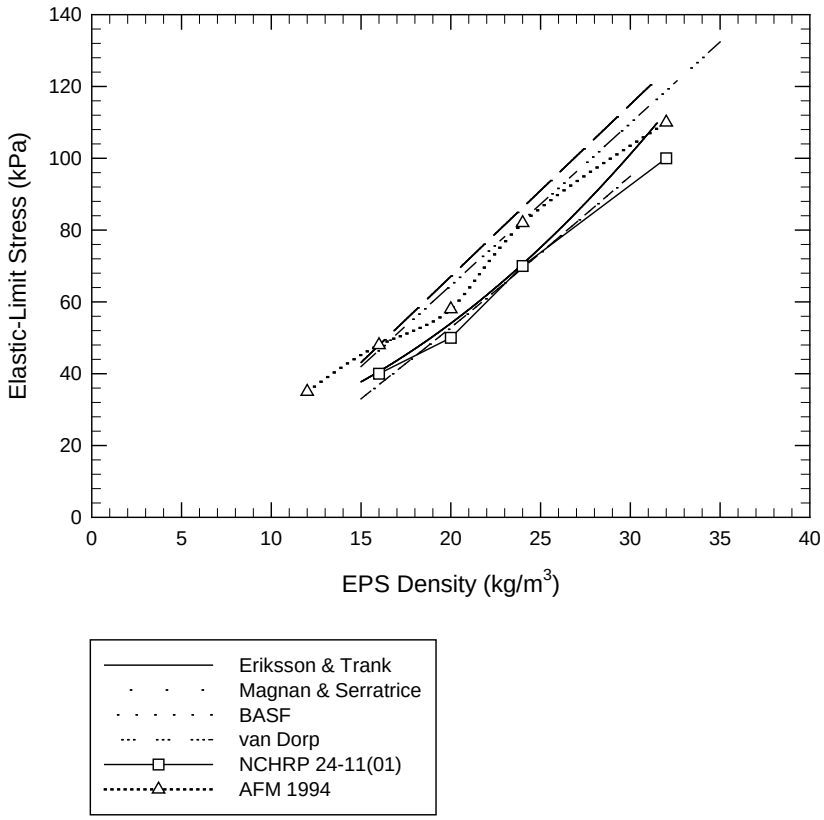

Figure 6.1. Elastic-limit stress and EPS density data considered in development of NCHRP Project 24-11(01) values.

HISTORICAL OVERVIEW OF EPS-BLOCK GEOFOAM STANDARDS SINCE NCHRP PROJECT 24-11(01)

Having explained and justified development of the Project 24-11(01) standard approximately a decade ago, we now address developments in the past decade since that standard was created. The first decade of the 21st century saw significant activity in development of standards for use of block-molded EPS as a geosynthetic product for small-strain geofoam functional applications. In addition, several DOTs developed their own generic specifications for routine project use of EPS-block geofoam for lightweight fill in road construction. Each specification tends to be a unique combination of selected verbiage from different standards, as well as other inputs; as a result, no two states have exactly the same specification.

However, the single most significant event in the issue of EPS-block geofoam standards for small-strain functional applications since development of the Project 24-11(01) standard has been the introduction, promotion, and use of a new ASTM standard, D 6817, titled *"Standard Specification for Rigid Cellular Polystyrene Geofoam,"* referred to hereinafter as "D 6817." This standard includes criteria for both EPS and XPS, and in many aspects is an expanded or extended version of C 578, and not zero-based, as was the Project 24-11(01) standard. Only the aspects of D 6817 relevant to the current EPS Project 24-11(02) are discussed here. It is important to note that material properties for the EPS portions of D 6817 were developed *solely by Industry* based on a new suite of tests performed specifically to generate data to use for this new standard. There are significant consequences to using this testing approach that will be discussed in some detail.

Several aspects of D 6817 immediately stand out. First, use of Roman numerals for designating material types was not carried over from C 578. In their place is a new material-type designation nomenclature consisting of the letters "EPS," followed by a two-digit number that approximates the density of the material in kg/m^3. However, instead of using the nominal density for that type of EPS, the minimum allowable density is used. This use of *minimum*, as opposed to *nominal*, or average block density, was acceptable in the C 578 standard—when thin EPS panels were predominant and non-load-bearing thermal insulation the intended product use. But it is questionable and not highly relevant for geofoam applications when an entire block is typically used, and load-bearing governs design.

This density usage, which is the cornerstone of D 6817, appears to reflect a significant shift in U.S. EPS molding practice. In the past, as typified by AFM 1994 product literature, a molder would promote the targeted nominal or average block density of the material, with the understanding that portions of a block might be somewhat less than that nominal value and other portions somewhat greater. It now appears the concept of using dual nominal/minimum densities in marketing block-molded EPS has been abandoned in favor of mentioning only the minimum density. Presumably, this means the minimum, not the higher nominal density, is used as a molding target. However, even newer EPS block molds will have some density variability in a given block. So a block molded to the minimum density, on average, has portions that fall below that minimum. Thus, they would be unacceptable under the terms of D 6817, as the requirements of that standard presumably apply to any test specimen cut from a block. This, of course, presumes that MQC/MQA testing is sufficient to detect portions of a block where the density may be sub-standard. However, the larger problem with this shift in Industry practice is that the standard material types in D 6817 now have densities that are out-of-sync with the original Project 24-11(01) standard, based on former *Industry practices* of using only nominal density as a molding target. As will be seen, this is only one of many conflicts between the Project 24-11(01) and D 6817 standards.

In an actual project, an EPS molder should, of necessity, mold to a target density that is comfortably higher than the minimum specified for the desired D 6817 material grade. This ensures that any sampled/tested block portion during project MQC/MQA does not fall below minimum allowable density, despite inevitable EPS block density variations that occur with modern molding equipment. In fact, use of modern molds may even ***increase*** potential for EPS-block density variations because modern molds tend to produce substantially larger blocks than were routinely used in U.S. practice even a decade ago. For example, in the 1990s a very common size U.S.-made EPS block was typically 600 x 1200 x 2400mm (2 x 4 x 8 ft.). Now, molds in routine commercial use mold blocks in excess of 1200 x 1200 x 7200mm (4 x 4 x 24 ft.). With larger molds comes the attendant problem of achieving material uniformity throughout, especially at higher densities typically required for small-strain geofoam applications. Therefore, why Industry abandoned use of nominal/minimum densities, which served both Industry and end user well for decades, and are essentially crucial for a geofoam-specific standard, is perplexing.

However, the most serious issue concerning D 6817 is minimum allowable values of small-strain stiffness material properties incorporated into the standard. The original version of D 6817, circa 2002, was based solely on a series of 28 unconfined-compression laboratory tests commissioned and funded solely by Industry. Testing, done on a round-robin basis in which EPS produced using raw material from different suppliers was molded, sampled, and tested by several different participants, was performed on a reported double-blind basis. Thus, a resin supplier or molder would not know who tested their material, and a tester would not know whose material they were testing. The work was coordinated by an Industry organization, the EPS Molders Association (EPSMA).

Summarized results from these Industry tests were provided during the current Project 24-11(02) study, and appear contradictory. On one hand, compressive strengths obtained were considerably higher than values used historically in C 578, as shown in Figure 6.2. Compressive strength of EPS is **not** a small-strain stiffness material property (although arbitrarily defined as such in ASTM standards) as the compressive stress at 10% compressive strain is well beyond the serviceability range of small-strain EPS-block geofoam applications. Nevertheless, compressive strength is a useful index property of overall EPS load-bearing quality. Individual data points obtained from testing performed to create the D 6817 standard are shown, together with relationships defining minimum allowable density-versus-strength requirements

incorporated into D 6817, and the similar relationship in C 578. Although we do not have definitive reasons for the significant increase in compressive strength reflected in D 6817 compared to C 578, it is possible it reflects improvement in mold and molding technology in the 60 years EPS has been in commercial production.

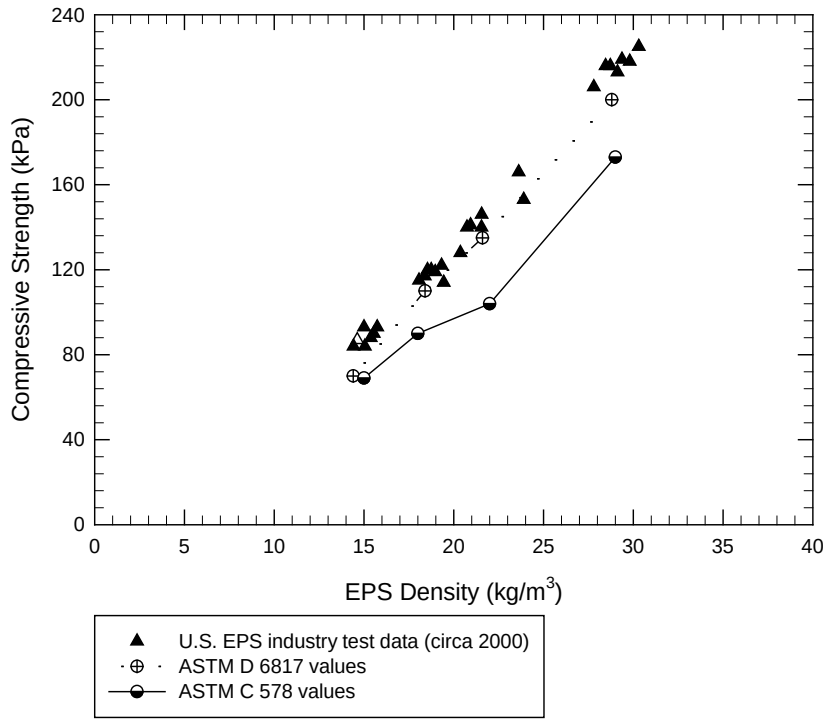

Figure 6.2. ASTM D 6817 round-robin compressive strength versus density test results.

Figure 6.3 shows minimum allowable flexural strengths adopted for D 6817, compared to corresponding C 578 values at the time D 6817 was first developed. Although flexural strength, like compressive strength, is not a small-strain stiffness material property, it is also a useful index parameter of EPS quality. In this case, flexural strength is a metric for the quality of bead fusion achieved during final block molding. Flexural strength of EPS is governed by its tensile strength which, in turn, depends on how the individual prepuff particles were thermally fused during final molding. We are not aware of any flexural testing performed specifically for development of D 6817, and it appears that the density-flexural strength relationship in C 578 was used for the original D 6817 version.

The contradiction occurs in D 6817 when examining small-strain stiffness material properties both for the 28 individual compression tests performed, as well as correlations of density with those properties eventually incorporated into D 6817. Results interpreted by Industry from the 28 compression tests are difficult to understand in view of results for compressive strength shown in Figure 6.2. Figure 6.4 shows elastic-limit stress as interpreted by Industry as a function of specimen density. Also shown are the best-fit line through these data, and the best-fit line minus one standard deviation. This latter line, for all practical purposes, forms a lower bound that encompasses all test data.

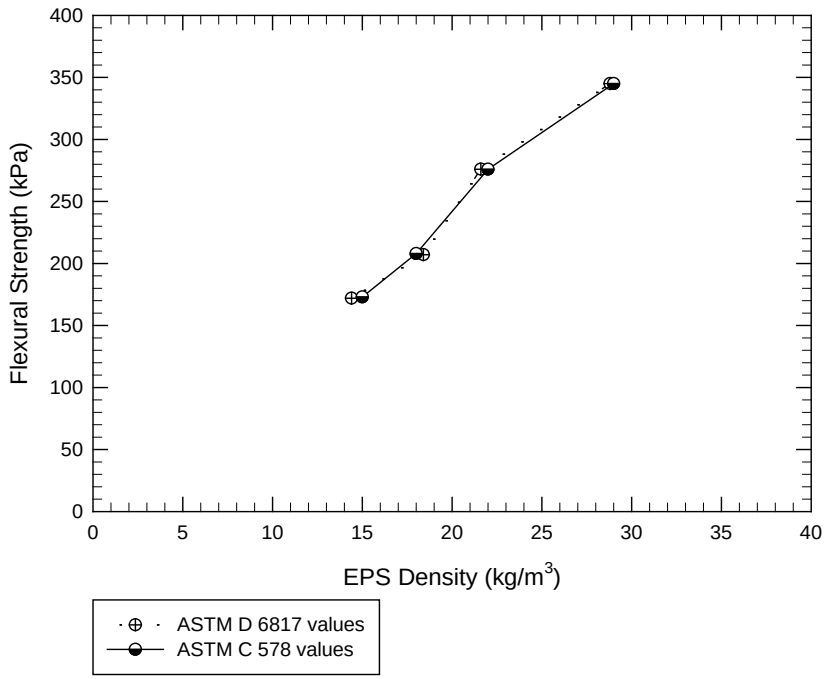

Figure 6.3. Comparison between ASTM D 6817 and ASTM C 578 flexural strength versus density values.

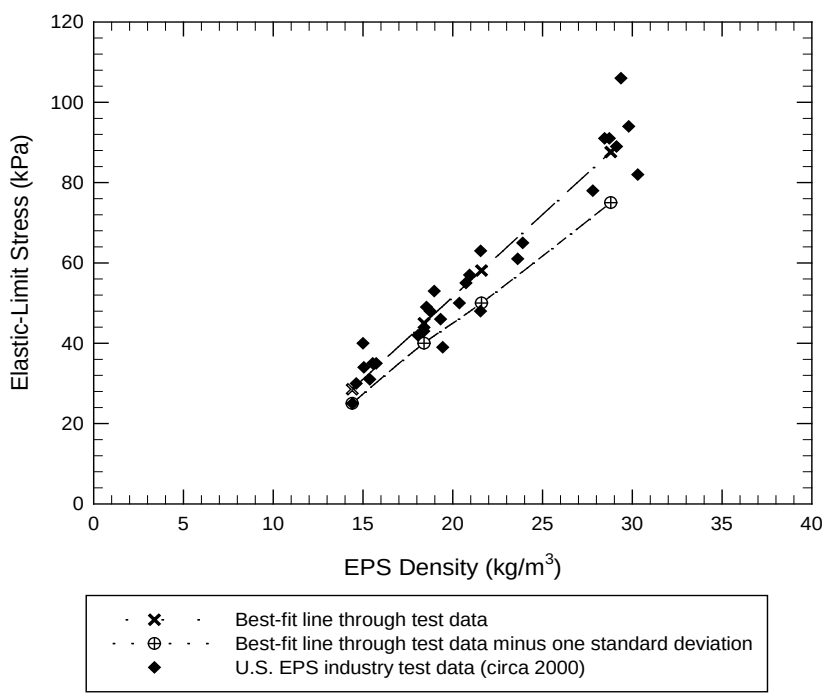

Figure 6.4. ASTM D 6817 round-robin elastic-limit stress versus density test results.

Industry used results shown in Figure 6.4 as the sole basis to develop the relationship between density and small-strain stiffness material properties in both the original and current version of D 6817 of this standard. Essentially, Industry ignored all other data available, such as in Figure 6.1, including data Industry itself generated in the 1990s. Thus, crucial small-strain stiffness material properties contained in the original and current D 6817 standards are based on only 28 pieces of data, obtained and interpreted solely by Industry. Using these data, Industry made several intentionally conservative assumptions that are cumulative in their net outcome.

First, using the best-fit-minus-one-standard-deviation (i.e. lower bound) line shown in Figure 6.4 reflects the most conservative interpretation of their test data. Next, when correlating between small-strain stiffness material properties and EPS density to develop minimum guaranteed values of material properties, density corresponding to the ***minimum*** allowable for a material type was used. As discussed previously, for any actual geofoam project, an EPS molder must mold to a target nominal or average block density likely to be at least 10 percent higher than the minimum specified in D 6817 for a given material type. This ensures that every point within that block equals or exceeds the minimum density allowed for that type. Therefore, because the small-strain stiffness properties are targeted and correlated to the minimum density, they present the most conservative, pessimistic assessment of both stiffness properties and load-bearing capability of the block as a whole, which will have a higher average density.

Further insight into implications of assumptions made in developing D 6817 small-strain stiffness material properties can be seen in Figure 6.5. This shows the same information in Figure 6.4, but with the relationship used in the Project 24-11(01) study superimposed on it. The Project 24-11(01) relationship has been retained for the revised, updated Project 24-11(02) standard, and is essentially identical to the best-fit line for the industry data used to create D 6817.

Even more enlightening is Figure 6.6, which shows individual test data generated by Industry circa 2000, as well as the relationship between density and small-strain stiffness material properties eventually adopted for D 6817, superimposed on all relationships previously shown in Figure 6.1 considered in development of the original Project 24-11(01) standard. The (01)/(02) standard is reflected in this figure as well. Most industry test data falls below (or well below) all the various relationships shown. Because of conservatisms reflected in D 6817, density-material property relationship in that standard is well below everything. Perhaps most significant and interesting is virtually all Industry test data, as well as the relationship adopted for use in D 6817, fall well below the relationship reflected in AFM 1994 product literature. This is shown in clearer detail in Figure 6.7. Figure 6.8 provides another, similar comparison, showing relationships between EPS density and small-strain stiffness material properties given in AFM 1994 product literature, the (01)/(02) standards, and D 6817.

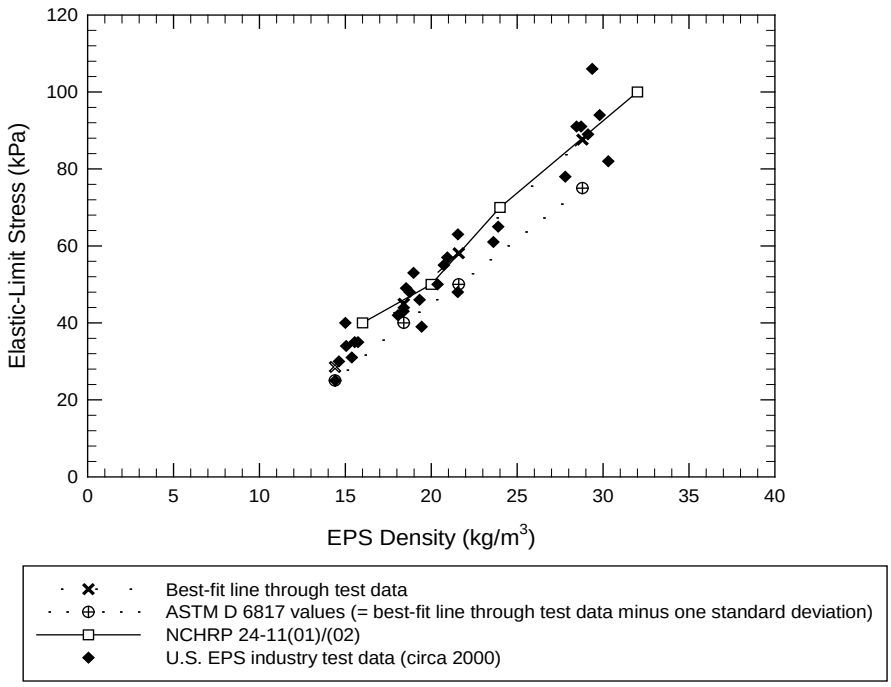

Figure 6.5. NCHRP Project 24-11(01)/(02) values superimposed on the ASTM D 6817 round-robin data values from Figure 6.4.

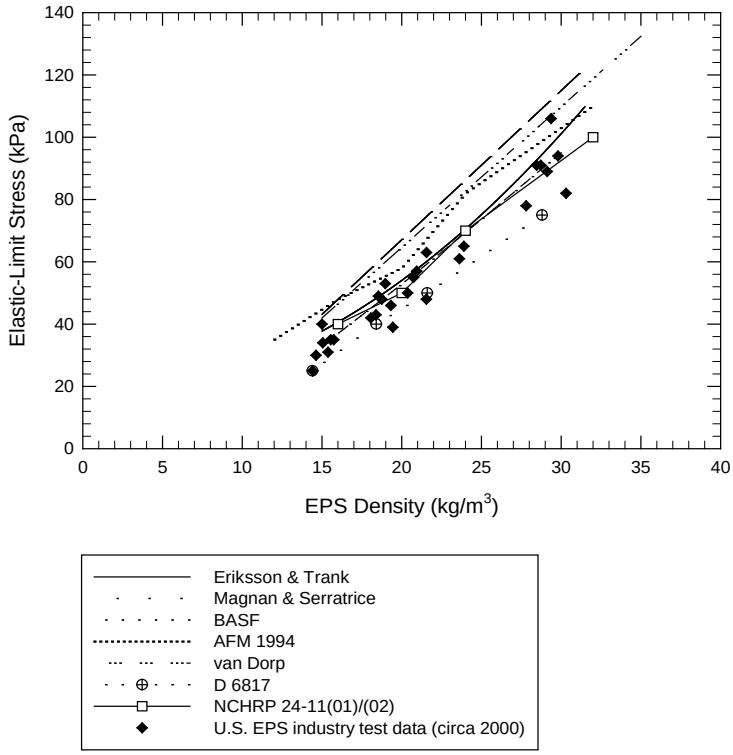

Figure 6.6. ASTM D 6817 values compared to data considered in development of NCHRP Project 24-11(01)/(02) values shown in Figure 6.1.

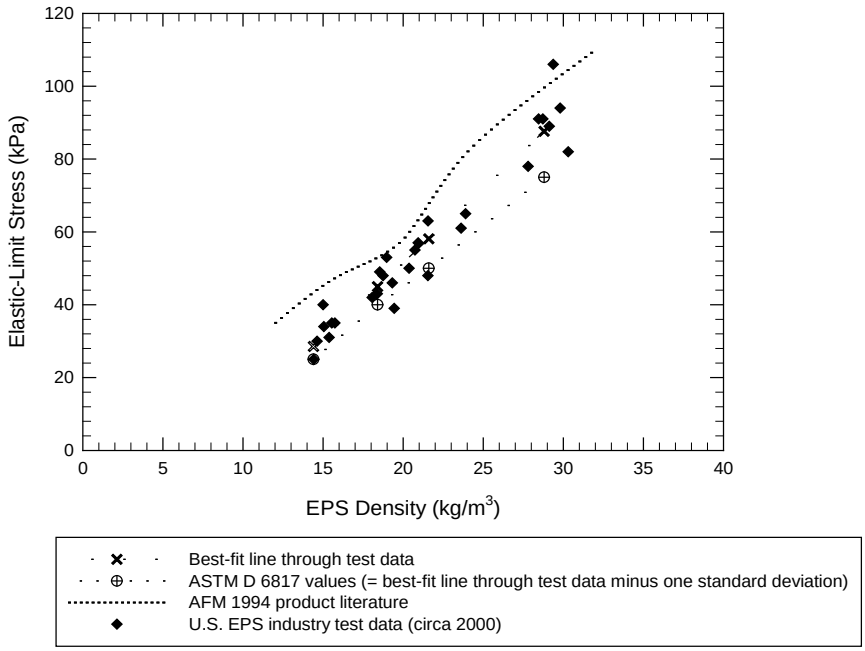

Figure 6.7. Comparison of ASTM D 6817 values and AFM 1994 product literature values.

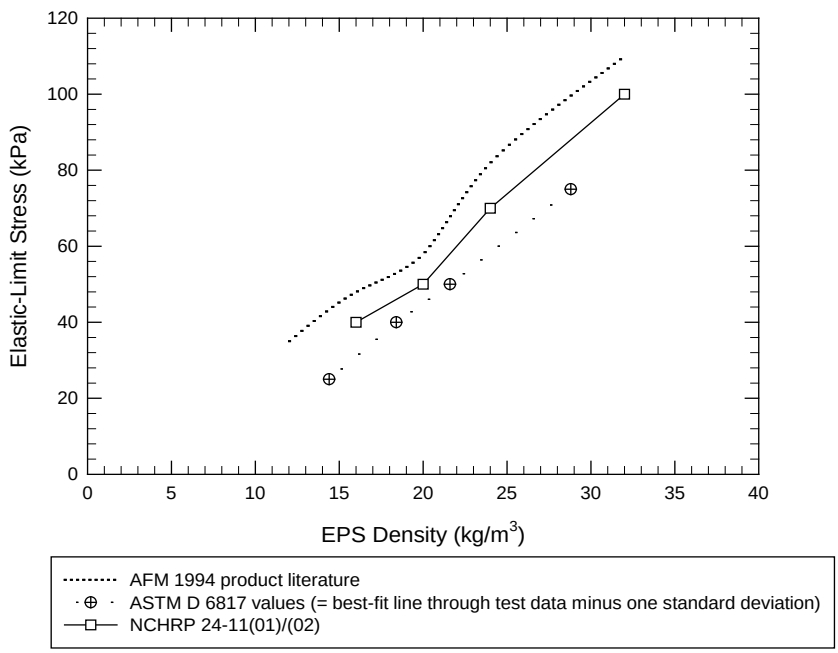

Figure 6.8. Comparison of NCHRP Project 24-11(01)/(02) values with ASTM D 6817 and AFM 1994 product literature values.

When looking collectively at information in Figure 6.6, 6.7, and 6.8, it is difficult to understand why Industry, using modern molding equipment and the D 6817 standard they developed, is essentially guaranteeing small-strain stiffness material properties for block-molded EPS significantly *lower* than in the 1990s. Furthermore, these properties are well below those found historically by others around the world decades ago, using older molding equipment. While outcomes reflected in D 6817 are the result of the doubly conservative data interpretation discussed previously, clearly the raw test data generated by Industry circa 2000 appears to be inherently conservative. This is seen in Figure 6.6, where these data are consistently, noticeably lower in general than collective test results generated worldwide by diverse sources, going back to the 1970s. This is especially difficult to understand when the same tests performed to develop D 6817 showed marked increases in compressive strength compared to historical values reflected in C 578, as shown in Figure 6.2.

In the early stages of the Project 24-11(02) study, anomalies of small-strain stiffness material properties incorporated in D 6817 became apparent. When brought to the attention of Industry, copies of the interpreted raw test results presented in Figures 6.2, 6.4, 6.5, 6.6, and 6.7 were provided to us. Our challenge was to learn the underlying cause for unexpectedly low values of small-strain stiffness material properties interpreted from industry tests performed circa 2000 and form the sole basis for D 6817. We concluded a number of possibilities that might explain observed results. Because of the significant technical and economic impact D 6817 has on current and future EPS-block geofoam practice in the U.S., the initial meeting with representatives of EPSMA was scheduled with the goal of understanding and resolving issues surrounding the Industry round-robin test data of circa 2000. Although EPSMA represents a fraction of U.S., EPS block molders, it is the only significant representative organization. Furthermore, because EPSMA oversaw and coordinated the round-robin testing program which generated the test data used to develop the original version of D 6817, they were a logical point-of-contact for outreach to Industry.

The first meeting with EPSMA representatives, via conference telephone call, was primarily of a general informational nature, with no technical outcomes. The second meeting, a Webcast, was held to understand the basis and genesis of small-strain stiffness material properties that were incorporated in the Project 24-11(01) and D 6817 standards. Unfortunately, most of the large number of participants from Industry did not have enough specific technical background to contribute to the discussion. Also, the inherent difficulty of presenting and discussing highly technical issues in a meaningful way via Webcast, and as only the Industry moderator could present visual information, desired outcomes were not reached. The Webcast was inconclusive in terms of meaningful technical outcomes. Therefore, no progress has been made in having a meaningful, detailed technical dialog with Industry; we do not have the required factual information. This leaves only conjecture as to reason(s) for stated results from the D 6817 round-robin testing.

After looking at all of the information made available to us by industry during the course of the present Project 24-11(02) study, for us to make a more definitive assessment of the Industry data generated circa 2000 we need access to original compression test stress-strain curves, as well as relevant peripheral information concerning testing hardware and protocols. There are three broad reasons for this request:

First, were the environmental conditioning requirements for EPS test specimens specified in ASTM standards utilized for all testing? Explicit standard protocols exist for conditioning and stabilizing EPS test specimens for temperature, humidity, and atmospheric pressure in a laboratory environment after specimens are prepared, and before they are tested. In our experience, these requirements, including a required 40-hour minimum waiting period, are unknown, forgotten, or simply ignored for most laboratory testing of EPS specimens and doing so can influence test results.

Two, what protocal was used to interpret the initial linear portion of the test curve? An analysis of the qualitative nature of the test curves is needed. Theoretically, compression tests on EPS should closely reflect those presented in AFM 1994 product literature in Appendix H. Specifically, there should be a nominally linear portion beginning immediately at the origin. However, it is not uncommon for the initial portion of the test curve to exhibit a slight upward concavity before linearity occurs. This can be due to a

variety of reasons related to how the test specimen is cut and prepared from a sample; seating of the test platens on the specimen, and other factors. In fact, concavities are so common in routine practice that ASTM standard test protocols explicitly address how this concavity is to be corrected graphically to produce a pseudo-linear initial test portion using an offset origin. However, these ASTM standards for performing and interpreting compression tests were developed for *non-geofoam thermal insulation applications*, where interpreted behavior under small compressive strains (i.e. ≤ 1 percent) is irrelevant to the overall test.

Experience in using EPS as a geofoam material has indicated that interpreted small-strain stiffness material properties can be extremely sensitive to the overall quality of the compression test. In particular, if testing protocols are poor, the initial concavity of the test curve can be relatively significant. As a result, after the test curve is empirically corrected for linearity, there may be an insufficient linear portion remaining for the full 1 percent compressive strain required to define the small-strain stiffness material properties. As a result, a portion of the post-yield, non-linear portion of the test curve is used for this purpose and values of small-strain stiffness material properties reported are too low. Thus, they do not reflect the actual small-strain stiffness of the material. On projects this can actually result in false-failure reports for MQC/MQA test specimens.

Three, was a correction made to the test curves for machine compression, i.e. mechanical 'slop', within the testing hardware? Experience has shown this is a very important issue, as slight compression within mechanical components of a load-test machine is unavoidable even with high-quality equipment in good working order. Extensive experience on the Boston Big Dig with hundreds of MQA tests on EPS specimens over several years indicated that this machine compression significantly affects the interpreted outcome of test results on the relatively small cube specimens of EPS tested routinely in practice. This is because compressive strain within a test specimen, whether it is EPS or soil or a Portland-cement concrete cylinder, is typically calculated based on what are called *global* displacements obtained by measuring displacement of the load-application cross-head of the testing machine. It is then assumed this displacement is the same as that experienced by the test specimen. In reality, the test specimen ***always*** undergoes less displacement and, therefore, less strain than is calculated using global measurements. For many materials this difference in strain is negligibly small. However, this has proven not to be the case for EPS (or for soil either, as has been known in geotechnical engineering practice for some time now).

It is possible to address this issue of machine compression in either of two ways. One is to make what are called *local* displacement measurements on the test specimen directly and base all strain calculations on these displacements. While this appears possible to do in concept for EPS specimens, and was proposed as far back as at least 1995, we are not aware of any commercially available test hardware for this purpose.

The other alternative is actually one that can and should be done routinely in practice. This requires performing a 'dummy' compression test using a piece of very stiff material (almost any metal would suffice, although steel would be preferred). Assuming that the stiff material does not compress significantly during the test (compression of the metal blank could be calculated from theory) so that any recorded compression represents machine compression. This test curve is then used to correct every 'raw' test curve obtained when testing actual EPS specimens to produce a corrected curve devoid of effects of machine compression. The small-strain stiffness material properties of EPS are then interpreted using the *corrected* stress-strain curve (which may itself have to be graphically corrected for initial non-linearity). Note that this dummy compression test must be repeated periodically during the life of the testing machine so that any cumulative wear-and-tear on machine components is taken into account.

The importance of this third factor whenever MQC/MQA testing of EPS is concerned cannot be emphasized too strongly. As extensive project experience on the Boston Big Dig indicated, correction for machine compression often meant the difference between an EPS test specimen failing or passing the minimum required small-strain stiffness material properties required by the project specification. The project specification properties were based on the Project 24-11(01) standard and, in fact, this was the first major project to take advantage of the standard. In view of this, our inability to ascertain whether or not correction for machine compression was made universally and consistently for all 28 tests that

comprised the U.S. EPS industry round-robin tests circa 2000 is unfortunate. As more than one laboratory was involved in the testing component, and in our experience, commercial laboratories doing general materials testing may find that correcting for machine correction is not important for the bulk of their work, there is a good probability it was not done.

In conclusion, with respect to the broad issue of changes in geofoam-specific standards over the past decade, it is clear that for a given EPS density, there are significantly different small-strain stiffness material property requirements between the NCHRP Project 24-11(01)/(02) and D 6817 standards. These differences are due to the fact that each standard was developed using an entirely different database. The Project 24-11(01)/(02) standard drew on all published or otherwise available data from around the world until the circa 2000 timeframe. The original D 6817 version was based on exactly 28 pieces of data, developed and interpreted solely by Industry. Furthermore, D 6817 has rejected the long-standing Industry practice of acknowledging relevance of both nominal and minimum densities of EPS blocks and used minimum density as the benchmark of overall block performance.

To date, implications of these differences between standards has proven to be significant, both technically and economically. Current indications are that this dual impact will not only continue, but grow in the future, unless there is a meaningful effort to reconcile differences between these standards. Unfortunately, Industry has never been supportive of the Project 24-11(01) standard, even though they are designed specifically for applications involving road construction. From the beginning, Industry has maintained a position that the Project 24-11(01) standard has "unreasonable" requirements for critical small-strain stiffness material properties, and only their D 6817 was reasonable. In recent years, Industry has become increasingly aggressive and proactive in promoting use of D 6817 via both print media and websites. Without comparable sustained financial resources to promote the Project 24-11(01) standard project experience, in recent years it has been clear that end users, including DOTs, are increasingly using D 6817 and not the more relevant Project 24-11(01) standard as the basis for their project specifications for use of EPS-block geofoam as lightweight fill in road construction. Increasingly, even if a project is specified based on the Project 24-11(01), standard some molders are refusing to make EPS blocks to that standard (they had in the past when a project specification required it). Rather, Industry informs end users that they will only supply blocks made to D 6817 criteria. Thus, end users must accept a D 6817 material type that will satisfy the Project 24-11(01) standard requirements.

This take-it-or-leave-it industry posture is clearly evident in recent product literature from FOAM CONTROL® EPS contained in Appendix H, which is unambiguous: any time a project specification calls for material according to a Project 24-11(01) standard type, ***only*** a D 6817 type will be supplied as a substitute. It is interesting that this literature was prepared by AFM, the organization that was offering material with ***more stringent*** requirements than the Project 24-11(01) standard as recently as a decade ago. This latest Industry position is particularly troubling, as they are basically promoting an all-or-nothing use of D 6817 which, as we have shown, was developed solely by Industry using a relatively modest dataset only they generated.

Cost implications of the AFM® Corporation (2010) document are significant; the final cost of EPS blocks is proportional to and strongly dependent on their density as raw material (sold commercially on a mass or weight basis) comprises the majority of the EPS unit cost. Implications of the AFM® Corporation (2010) document are that an end user is forced to accept material that may be 20 percent denser than that required by the Project 24-11(01) standard, presumably at a higher commensurate cost, because more mass of EPS must be purchased. Given that the vast majority of road projects are publically funded, potential cost implications and impact on construction budgets of the AFM® Corporation (2010) document is clear.

Cost implications of the FOAM CONTROL® EPS document shown in Appendix H are significant; the final cost of EPS blocks is proportional to and strongly dependent on their density as raw material (sold commercially on a mass or weight basis) comprises the majority of the EPS unit cost. Implications of the FOAM CONTROL® EPS document in Appendix H are that an end user is forced to accept material that may be 20 percent denser than that required by the Project 24-11(01) standard, presumably at a higher commensurate cost, because more mass of EPS must be purchased. Given that the

vast majority of road projects are publically funded, potential cost implications and impact on construction budgets of the Industry document (FOAM CONTROL® EPS document) contained in the Appendix H is clear.

Understandably, the concern is that government agencies will continue to be forced to spend more money than necessary because of the take-it-or-leave position of some portion of Industry regarding supply of material manufactured only to requirements of D 6817. That only 28 pieces of lab data and the conservative interpretation of those data would have such implications in practice is astounding. Thus, it is imperative that anomalously low small-strain stiffness material properties reflected in D 6817 be critically and objectively re-examined as the first step toward reconciliation and unification of standards to reflect the best interest of all concerned. The current state of EPS-block geofoam practice in the U.S. is poorly served by having two very different standards in simultaneous existence.

Based on our careful review of all available information during Project 24-11(02) study, we see no reason to modify small-strain stiffness material property requirements in our standard, in view of significant, potentially serious technical questions and issues concerning the database and interpretation of that database for development of D 6817. Our opinion that the original Project 24-11(01) standard, which remains fundamentally unchanged in the Project 24-11(02) standard, is sound and reasonable, as has been borne out by both past and current project use. Perhaps the most significant example is the well-known Boston Big Dig, which made extensive use of EPS-block geofoam as lightweight fill as a cost-saving initiative to replace elevated roadway structures supported on deep foundations with EPS fills supported directly on the ground. Had the designers on that project used the original version of D 6817 in effect at the time as the basis of their design, they would have concluded it was impossible to mold EPS blocks with sufficient load-bearing capacity for project needs. However, their project specification was more realistically based on a draft version of the Project 24-11(01) standard (the first known project to do so). Local molders easily supplied EPS blocks with necessary small-strain stiffness material properties (using relatively old molds), and the project was constructed successfully, at a more reasonable cost.

We believe there are other flaws and shortcomings inherent in D 6817 that require correction, if there is to be any reconciliation of standards. This includes the failure to explicitly identify both nominal (average) and minimum densities for a given material type, as well as the need to eliminate use of density as part of the nomenclature to designate material types. As has been discussed, density, per se, is a poor primary index property for small-strain stiffness material properties of block-molded EPS. To use density as part of a material-type nomenclature simply perpetuates and supports bad habits from the past, and is clearly not related to requirements for lightweight fill applications.

BASIS OF THE RECOMMENDED STANDARD INCLUDED IN APPENDIX F

Now, we will address changes made in the Project 24-11(02) standard compared to the Project 24-11(01) standard. It is clear from the preceding discussion that subsequent to our developing the (01) standard in the 1999-2000 timeframe, there have been both extensive alternative standards development, as evidenced by several versions of D 6817, as well as substantial major project experience where the (01) standard formed the primary basis for the project specification. With due consideration of this background of knowledge developed in the past decade, we approached a reassessment of the (01) standard as part of the (02) project tasks.

Our NCHRP Project 24-11(02) standard included in Appendix F contains six key revisions from the Project 24-11(01) standard. First, a commentary section is included so that users have some insight into the logic behind various aspects of the standard. Second, we eliminated the use of different minimum allowable density values for individual MQC/MQA test specimens versus a higher nominal or average density of the block as a whole, as shown in Table 6.1. The revised requirements in the preliminary (02) standard now dictate that both the block as a whole, and any test specimen from within that block, meet the same criteria are shown in Table 6.3.

Based on projects such as the CA/T project experience, as well as other project experience, it was found that there was frequently a misreading or misinterpretation (real or intentional) of the standard

because of the dual (nominal/minimum) density values for a given material type. The previous lower allowable densities for a specimen have been misinterpreted by molders to mean that minimum values of material properties included in the Project 24-11(01) minimum allowable values of MQC/MQA parameters for individual test specimens shown in Table 6.1 are "guaranteed" at the lower allowable specimen density. Therefore, the new minimum recommended material densities incorporated in the Project 24-11(02) recommended standard in Appendix F are the same for both specimens, and each block as a whole, as shown in Table 6.3.

Table 6.3 Minimum allowable density values for slopes included in (02) interim report.

Material Designation	**Minimum Allowable Density/Unit Weight, (kg/m^3 lb./ft^3)**	
	Each Block as a Whole	**Any Test MQC/MQA Specimen**
EPS40	16 (1.0)	**16 (1.0)**
EPS50	20 (1.25)	**20 (1.25)**
EPS70	24 (1.5)	**24 (1.5)**
EPS100	32 (2.0)	**32 (2.0)**

Note: Changes made from (01) embankment standard indicated in bold.

As discussed previously, the original Project 24-11(01) standard was crafted with consideration of the then-common Industry practice of using both a nominal (overall average) and minimum (for any given small piece or test specimen) density for EPS block, as evidenced by the AFM 1994 product literature in Appendix H. However, with the Industry trend in recent years to only referring to the minimum density of a test specimen, as is done in D 6817, it was clear that the language in the original Project 24-11(01) standard was causing inordinate contractual problems and claims despite the fact that it made (and still makes) technical sense. So we made changes reflected in Table 6.3 solely for administrative and not technical reasons. In addition to numerical changes shown in Table 6.3, more emphatic language was added to the revised Project 24-11(02) standard that emphasizes that there *is no correlation expressed, implied, or suggested* that the minimum required block density for a given grade of EPS will result in EPS that will meet the required minimum values of material properties. This commentary cautions users about improper interpretation that has, in the past, led to contractual issues and claims for extras on actual projects which drive up costs.

The third key change incorporated into the revised standard in Appendix F consists of increased minimum allowable values for compressive strength to reflect the increase in these values included in D 6817. Compressive strengths included in the Project 24-11(01) standard, as summarized in Table 6.2, were based on values in C 578, which was the only available standard for block-molded EPS available for our use at the time. The fourth key revision consists of increased requirements for flexural strength to be consistent with the change in unifying block and test-specimen densities. The fifth change consists changed wording related to the small-strain modulus from "Initial Tangent Young's Modulus" to "Initial Secant Young's Modulus" simply to correct semantics. There is no change in what this parameter represents (the average slope between 0 percent and 1 percent compressive strain in uniaxial compression) or how it is determined experimentally in practice. Table 6.2 provides a summary of the original material property values for stand-alone embankments contained in the Project 24-11(01) standard, and Table 6.4 provides the revised values included in the Appendix F standard for slopes.

Table 6.4. Minimum allowable values of MQC/MQA parameters for individual test specimens in (02) standard for slopes.

Material Designation	Dry Density/ Unit Weight, kg/m^3 (lb./ft^3)	Compressive Strength, kPa (lb./in^2)	Flexural Strength, kPa (lb./in^2)	Elastic-Limit Stress, kPa (lb./in^2)	Initial Secant Young's Modulus, MN/m^2 (lb./in^2)
EPS40	**16 (1.0)**	**85 (12.5)**	**185 (27)**	40 (5.8)	4 (580)
EPS50	**20 (1.25)**	**120 (17.5)**	**240 (35)**	50 (7.2)	5 (725)
EPS70	**24 (1.5)**	**155 (22.5)**	**300 (43)**	70 (10.1)	7 (1015)
EPS100	**32 (2.0)**	**230 (33.5)**	**380 (55)**	100 (14.5)	10 (1450)

Note: Changes made from (01) stand-alone embankment standard indicated in **bold**.

In Table 6.4, values of small-strain stiffness material properties of elastic-limit stress and initial secant Young's modulus remain unchanged from the Project 24-11(01) standard, for reasons already discussed.

In addition, we chose not to adopt the D 6817 material-type nomenclature built around material density for reasons already discussed. Realistically, load-bearing based nomenclature adopted for the Project 24-11(01) standard and continued with the Project 24-11(02) standard is a more logical and appropriate system to use for load-bearing geofoam applications.

Finally, we chose not to adopt the D 6817 practice of referring only to the density of small test specimens cut from a block. It is useful to maintain the distinction with the nominal or average density of the block as a whole, as this is how the material is used in actual applications, even though these values are now the same. This leaves open the possibility of returning to different values for these densities as was used in the Project 24-11(01) standard at some future date.

One of the significant issues that quickly became apparent in routine practical use of D 6817 after its initial introduction was that the upper-limit type (grade) of material for which this standard provided, called EPS29 in the material-type nomenclature used in this standard, had a maximum allowable load-bearing capacity that was only 75 percent of that for the upper-limit material type, EPS100, provided for in the Project 24-11(01) standard for embankments. This can be seen in Figure 6.9 by comparing the solid lines depicting the relationship between EPS density and small-strain stiffness material property of elastic-limit stress implied by these two standards.

Experience in practice quickly indicated that when D 6817 was chosen as the standard on which to base a project-specific specification, the conclusion was sometimes reached that EPS-block geofoam simply could not be used as part of a design alternative; there was no material type with sufficient load-bearing ability for project needs. As previously discussed, the well-known Boston Big Dig project, which is one of the larger EPS-block geofoam projects for road construction in the U.S. to date, could not have been designed and ultimately constructed successfully using EPS-block geofoam had the Project designers used D 6817. That such conclusions that conflicted with reality (e.g. there was no problem providing the necessary material for the Boston Big Dig despite the use of an older mold for most of the EPS block production) could be reached routinely using the original version of D 6817 was the result of the multiple conservatisms used to create the guaranteed minimum values of the critical small-strain stiffness material properties (elastic-limit stress and initial tangent or secant Young's modulus) contained in D 6817. The nature of these conservatisms was previously presented and discussed in extensive detail. How many potential EPS-block geofoam projects were lost in the U.S. due to this is not and will never be known, nor will those cost-savings be realized.

Subsequent to the initial release of D 6817, Industry performed 25 additional laboratory compression tests for the explicit purpose of developing D 6817 material types with maximum load-bearing capacities greater than EPS29. Interpreted results from these additional test data were provided by Industry during the Project 24-11(02) study and are shown in Figure 6.9, together with the original interpreted data

used to develop the original version of D 6817. Also shown in this figure, with a dashed line, are the two higher-capacity material types added to D 6817 during the 2000s. They are referred to as EPS39 and EPS46, with minimum allowable densities of 38.4 kg/m^3 (2.40 lb./ft^3) and 45.7 kg/m^3 (2.85 lb./ft^3), respectively. As can readily be seen, Industry applied the same conservative interpretation to the later, higher-density test data that they applied to initial data obtained circa 2000 used to develop material properties for the original version of D 6817. Specifically, the relationship between EPS density and the small-strain stiffness material properties was chosen to essentially encompass all the test data.

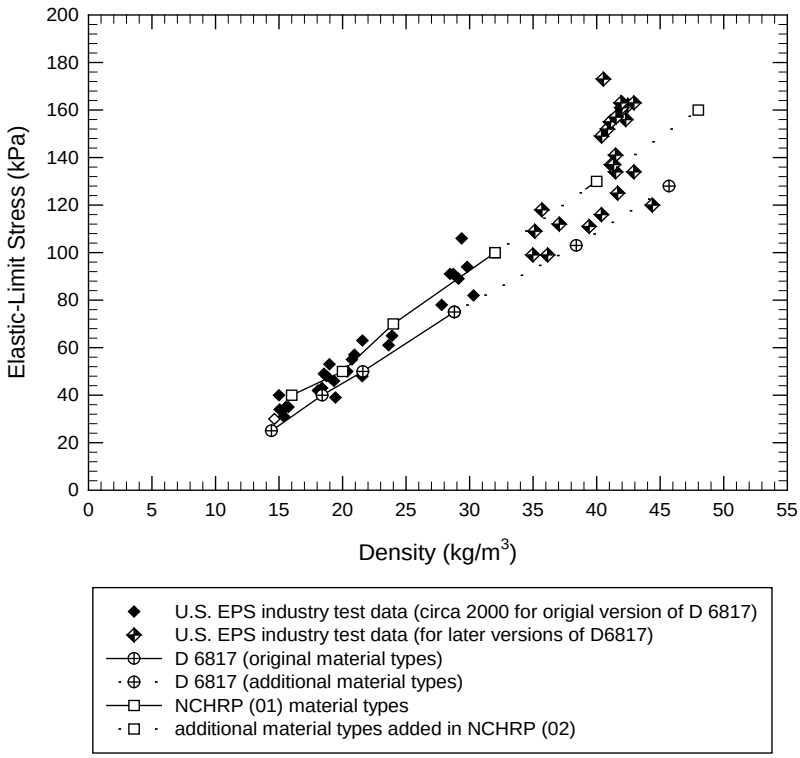

Figure 6.9. Additional ASTM D 6817 test data for higher EPS densities and additional EPS types added to the standard in Appendix F.

We were not provided with actual stress-strain curves for these additional 25 compression tests, nor with copies of stress-strain curves for the original 28 compression tests. Thus, the same questions and uncertainties concerning laboratory and data-interpretation protocols for the later compression tests exist as for the original tests. These questions were presented previously, so will not be repeated here. Suffice it to say, unanswered questions concerning these protocols could affect the values of elastic-limit stress for all the test data plotted in Figure 6.9.

That Industry was able to mold and test higher-density specimens of EPS to extend the utility of D 6817 to a greater load-bearing range reflects the increasing availability and use of newer EPS molds in the U.S. that can routinely mold EPS blocks to higher densities than in the past. Historically, a density of approximately 32 kg/m^3 (2 lb./ft^3) was considered a reasonably conservative upper-bound of EPS densities that the typical U.S. block molder could achieve routinely without taxing mold capabilities and performance. Most U.S. molders are used to routinely making material with a density of approximately 16 kg/m^3 (1 lb./ft^3) or less for thermal insulation and packaging applications. This 32 kg/m^3 (2 lb./ft^3) limit was one of several reasons why the original Project 24-11(01) standard, developed during 1999-2000, was intentionally capped at that density. However, with changes in molding technology that have occurred during the past decade, and considering that Industry felt comfortable to extend material types in D 6817

well beyond the 32 kg/m^3 (2 lb./ft^3) threshold, we felt that it is appropriate to add one or more higher-density material types to the NCHRP standard for the version contained in the Project 24-11 (02) study.

We decided on two new, additional types, designated EPS130 and EPS160. These two new EPS types represent the sixth key revsion from the Project 24-11(01) standard. The assumed material properties for these are shown in Tables 6.5 and 6.6. In addition, we have shown the relevant properties of these two new material types in Figure 6.9 as a dashed-line extension of the properties contained in the Project 24-11(01) standard.

Table 6.5. AASHTO Material-Type Designations for EPS-Block Geofoam.

Material Designation	**Minimum Allowable Density (Unit Weight), kg/m^3 (lbf/ft^3)**	
AASHTO (provisional)	**Each Block as a Whole**	**Any Test MQC/MQA Specimen**
EPS40	16 (1.0)	16 (1.0)
EPS50	20 (1.25)	20 (1.25)
EPS70	24 (1.5)	24 (1.5)
EPS100	32 (2.0)	32 (2.0)
EPS130	**40 (2.5)**	**40 (2.5)**
EPS160	**48 (3.0)**	**48 (3.0)**

Note: Additions made to (01) embankment standard and contained in the (02) standard indicated in **bold**.

Table 6.6. Minimum Allowable Values of MQC/MQA Parameters for Individual Test Specimens.

Material Designation	**Dry Density (Dry Unit Weight), kg/m^3 (lbs./ft^3)**	**Compressive Strength, kPa (lbs./in^2)**	**Flexural Strength, kPa (lbs./in^2)**	**Elastic-Limit Stress, kPa (lbs./in^2)**	**Initial Secant Young's Modulus, MN/m^2 (lbs./in^2)**
EPS40	16 (1.0)	85 (12.5)	185 (27)	40 (5.8)	4 (580)
EPS50	20 (1.25)	120 (17.5)	240 (35)	50 (7.2)	5 (725)
EPS70	24 (1.5)	155 (22.5)	300 (43)	70 (10.1)	7 (1015)
EPS100	32 (2.0)	230 (33.5)	380 (55)	100 (14.5)	10 (1450)
EPS130	**40 (2.5)**	**275 (40)**	**415 (60)**	**130 (18.8)**	**13 (1885)**
EPS160	**48 (3.0)**	**345 (50)**	**520 (75)**	**160 (23.2)**	**16 (2320)**

Note: Additions made to (01) embankment standard and contained in the (02) standard indicated in **bold**.

Note that only the EPS130 material type is likely to be commercially feasible at the present time for most U.S. EPS block molders to produce routinely on a large production-scale basis, as is required for most geofoam projects. Also, the maximum allowable compressive load-bearing for this material type (130 kPa) is essentially the same as the densest EPS material type (EPS46) currently specified in D 6817 (128 kPa). Therefore, end users should consider EPS130 to be a conservative upper-bound of material for which they should design and specify on a routine basis.

However, we believe it is reasonable to include the EPS160 material type in the Project 24-11(02) standard, as there may be projects with particularly severe load-bearing requirements that justify considering a material at the edge of the technological envelope for block-molded EPS in the U.S.; and this would require finding a molder capable of producing it in the quantities necessary for a given project. The data in Figure 6.9 clearly indicate that elastic-limit stresses in excess of 160 kPa are achievable at

present (although Industry data in Figure 6.9 are several years old). Furthermore, EPS block molds capable of producing material with a density of the order of 48 kg/m^3 (3 lb./ft^3) are known to exist in the U.S. at present. In addition, there is an alternative to the traditional block-molding EPS manufacturing process which uses manufacturing process called *pulfusion*. Pulfused EPS is produced as a continuous strip or ribbon of material, as opposed to discrete blocks. EPS densities of up to 80 kg/m^3 (5 lb./ft^3) are routinely achievable using the pulfusion process. Although pulfused EPS is not widely known among design professionals and contractors at the present time, it is nonetheless a material that is in routine commercial production and availability in the U.S.

SUMMARY

The NCHRP 24-11(02) standard in Appendix F contains six key revisions from the NCHRP 24-11(01) standard. First, included for the first time is a commentary section. Second, we eliminated use of different minimum allowable density values for individual MQC/MQA test specimens versus a higher nominal or average density of the block as a whole, so that both the block as a whole and any test specimen from within that block meet the same criteria. Third, we increased minimum allowable values for compressive strength to reflect the increase in these values included in D 6817. Fourth, we increased requirements for flexural strength to be consistent with the change in unifying block and test-specimen densities. Fifth, we changed the wording related to the small-strain modulus from “Initial Tangent Young's Modulus” to “Initial Secant Young's Modulus” simply to correct semantics. Sixth, we added two new, additional types, designated EPS130 and EPS160.

In conclusion, we fully appreciate that D 6817 has small-strain stiffness material properties that differ significantly from those in Project 24-11(01)/(02) standards. However, we feel that the extensive discussion presented in this section of the report demonstrates conclusively that Project 24-11(01)/(02) standards are based on sound logic and consideration of all knowledge acquired over the approximately 60 years that EPS has existed as a construction material. In addition and significantly, we have considered a decade of actual project use and experience using Project 24-11(01) standards. That experience indicates that standards developed for past and current NCHRP studies are reasonable when implemented properly in practice, which includes MQC/MQA laboratory testing performed in accordance with well-established ASTM protocols for test-specimen conditioning prior to testing, numerical correction of all stress-strain curves for machine compression, and graphical correction of stress-strains for initial concavity as necessary. Conversely, we feel that the original version of D 6817 was based on relatively limited test data performed in one closed, private study. Those data and their interpretation as incorporated into D 6817 is at odds with not only decades of worldwide experience, but even prior testing and practice by Industry itself. Therefore we categorically reject the Industry position that that the Project 24-11(01)/(02) standards are unreasonable in their material requirements. We continue to suggest that Industry provide additional specific technical details about their circa 2000 round-robin testing program as outlined earlier in this section so that we can attempt to resolve the conflicts between the NCHRP and ASTM standards.

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CHAPTER 7

CASE HISTORIES

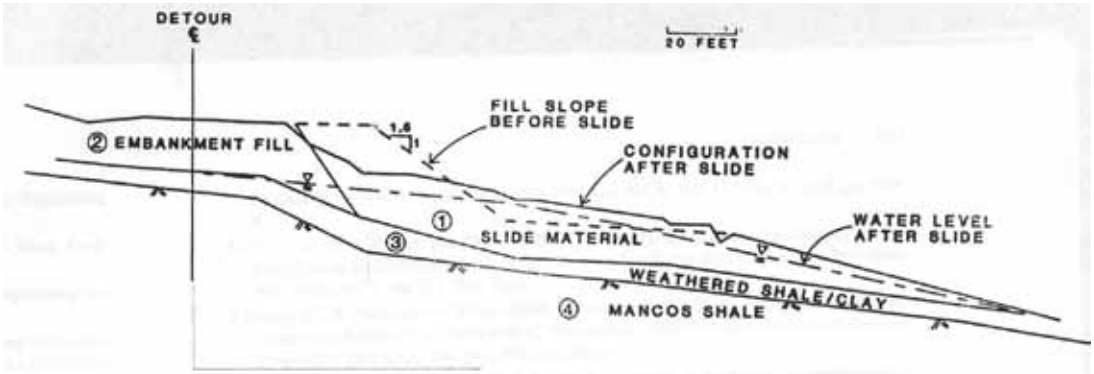
INTRODUCTION

The following four case histories are presented to provide examples of cost-effective and successful EPS-block geofoam slope stabilization projects completed in the U.S. The case histories presented in this chapter are intentionally limited to reflect only U.S. construction pricing and practices. Of course, there are many international geofoam case histories referred to throughout the report.

COLORADO: HIGHWAY 160

The Colorado Highway 160 (CO Hwy. 160) slope stabilization project between Mesa Verde National park and the City of Durango in southwestern Colorado is perhaps the earliest use of EPS-block geofoam for slope stabilization and repair. The information from this case history was obtained from Yeh and Gilmore (1992). The slide, which occurred in the spring of 1987, covered an area of approximately 0.4 ha (1 acre) and involved about 8,410 cubic meters (11,000 cubic yards) of slide material.

The initial analysis the Colorado DOT (CDOT) performed of the CO Hwy. 160 slope failure indicated that the landslide occurred as a result of high ground water in the slope due to an extended period of wet weather in the region. It was determined that the slip surface was non-circular in shape, running primarily along the interface between the roadway fill and a relatively thin layer composed of weathered shale and clay. In addition to the active failure surface, two additional and older slip surfaces were identified underlying the entire area, beneath the active slip surface. A cross-section view of the failed slope, including the soil properties used in the CDOT analysis, is provided below in Figure 7.1.



SOIL/ROCK PROPERTIES

SOIL/ROCK TYPE	TOTAL UNIT WEIGHT, PCF	MOISTURE %	L.L.	P.I.	C PSF	Φ
①	115	WET	-	-	50	15
②	115	16	35	14	500	20
③	120	14.1	36	15	1000	0
④	115	-	-	-	9000	0

Figure 7.1. Cross-section and soil properties of Colorado Hwy. 160 slide remediation (Yeh and Gilmore 1992).

An interceptor drain was initially installed on the uphill side of the slide mass along the highway to collect and discharge water away from the slide area, because the primary cause of the landslide was a high ground water level in the slope. The CDOT considered three further stabilization alternatives. The

first alternative, removing the slide debris and underlying overburden soils and placing the entire new embankment on the underlying shale, was not considered practical because of the presence of two older slides underlying the area, right-of-way limitations, and the amount of slide material that would need to be removed. The second alternative consisted of installing an earth retention system. The CDOT considered various types of earth retention systems, including a reinforced concrete wall, a geotextile mechanically stabilized earth wall, and a drilled shaft, as well as a pile wall. The CDOT did not consider the use of an earth retention system feasible because estimated costs of one million dollars was too high.

The third alternative consisted of stabilizing the slope by placing a counterweight consisting of on-site material at the toe of the slide mass, and placing lightweight material at the head of the slide mass in conjunction with installation of two tiers of toe trench drains. Additionally, a second interceptor drain, adjacent to the first interceptor drain previously installed on the uphill side of the road, could be installed. The advantages of this alternative included minimal site disturbance because the slide debris would not need to be fully removed, and a much lower cost of $160,000 compared to Alternatives 1 and 2.

A total of 648 cubic meters (848 cubic yards) of EPS blocks with a density of 20 kg/m^3 (1.25 lb./ft^3) were used. As shown in Figure 7.2, a drainage blanket consisting of filter drain material wrapped in a non-woven geotextile and 15cm (6-in.) diameter pipes spaced at 3m (10 ft) was installed below the EPS blocks. The purpose of the drainage blanket was to prevent hydrostatic uplift of EPS blocks.

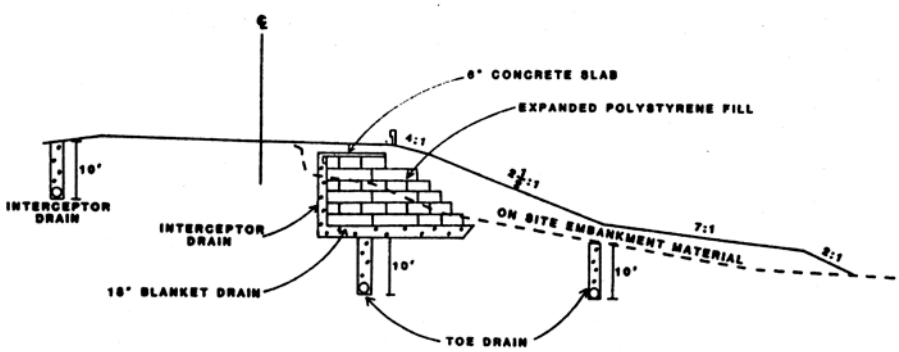


Figure 7.2. Geofoam and drainage system included in the Colorado Hwy. 160 slide remediation (Yeh and Gilmore, 1992; used with permission from ASCE).

Construction occurred during December 1988 and January 1989 because construction during winter months would provide a lesser need for temporary ground water control, since ground water levels are typically at the lowest level during the winter, and minimum slide material disturbance due to the frozen soil near the surface of the slide mass that would minimize opportunity for further movement of the slide mass. Placement of EPS block took about two weeks. One construction issue directly related to EPS block that had to be considered during winter construction was the need to secure the blocks at night, especially during winter snow storms, to prevent the blocks from moving because of high winds.

Settlement plates were installed to monitor the post construction performance of the EPS-block slope repair. Movements recorded between 1989 and 1990 were insignificant and averaged 0.2cm (0.1 in.). CDOT did not observe any associated apparent distress of the slope surface associated with these ground movements.

NEW YORK: STATE ROUTE 23A, TOWN OF JEWETT, GREENE COUNTY

This case history involves use of EPS-block geofoam to stabilize a roadway embankment on an unstable slope. Information for this project was obtained from Jutkofsky (1998) and Jutkofsky et al. (2000). The site is located in a mountain valley and slopes downward from north to south. Based on results of two borings performed on both sides of the roadway, the subsurface soils at the centerline of the roadway consist of about 1.5m (5 ft.) of gravelly silt fill. The underlying native soils consist of approximately 4.3m (14 ft.) of layered clayey silt and silty clay overlying 10.7m (35 ft.) of clayey silt. The water table was located at a depth of 2.4m (8 ft.) in the clayey silt and silty clay, or approximately 4.0m (13 ft.) below the pavement surface. Figure 7.3 presents a profile of all soil embankment and subsurface soils.

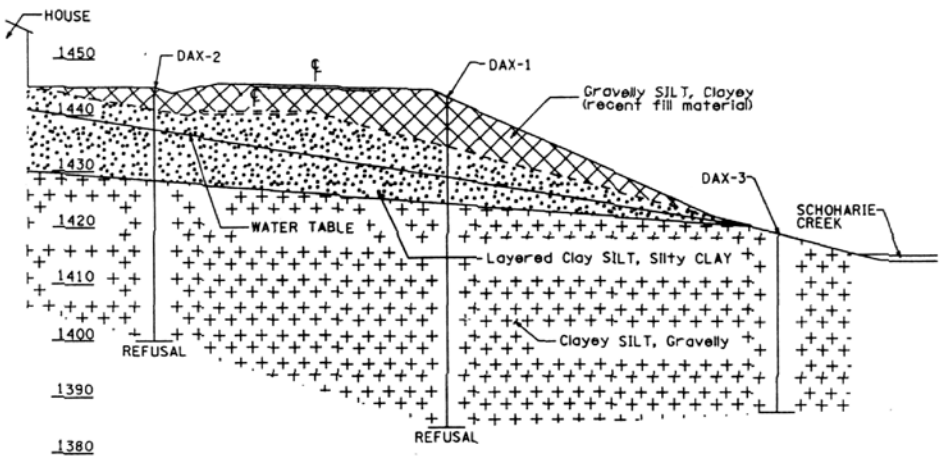


Figure 7.3. Profile of all soil embankment and subsurface soils (Jutkofsky 1998).

A 91m (300 ft.) section of Route 23A became unstable after the roadway was reconstructed in 1966. These movements resulted in a continuous maintenance problem and traffic hazard. In 1979, horizontal drains were installed to lower the ground water table. However, slope movements continued. Lateral movements measured over a period of 14 years after the drains were installed totaled 203mm (8 in.). Inclinometer data indicated that the failure surface was about 11 to 12m (36 to 40 ft.) below the roadway surface, which corresponds to the clayey silt layer. Figures 7.4 and 7.5 present a plan and profile view of the scarp, respectively. Consequently, in 1994, the New York State DOT (NYSDOT) evaluated the following remedial measures: soil removal and replacement with EPS blocks (weight reduction) and installing drainage, placement of a berm at the toe of the slope, use of a shear key, relocation of the roadway uphill away from the failure zone, lowering the grade and installing stone columns, and soil nailing. The NYSDOT selected the weight reduction and drainage option because it considered the other alternatives impractical due to limitations imposed by the site and its environment, and/or considered the other alternatives too costly.

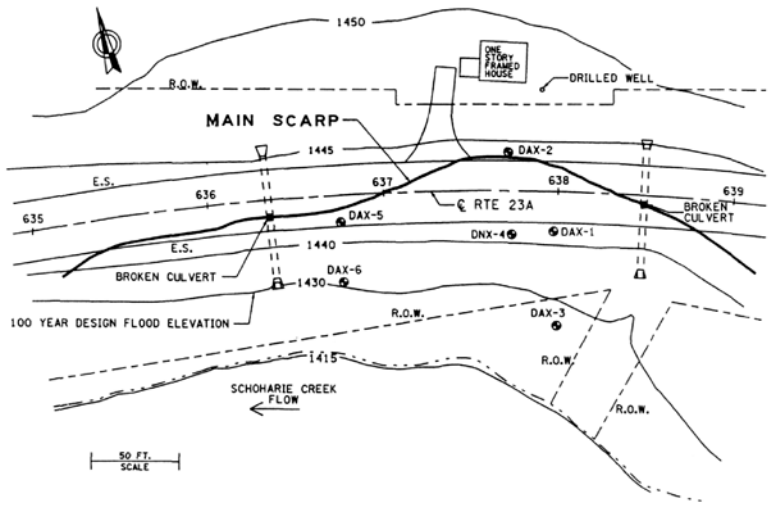


Figure 7.4. Plan view of scarp (Jutkofsky 1998).

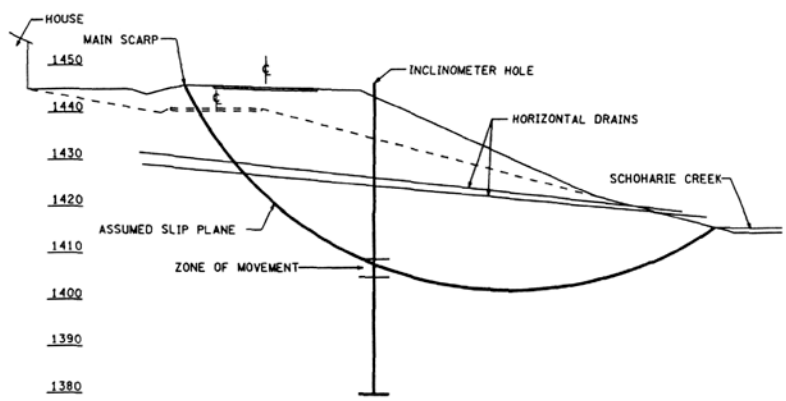


Figure 7.5. Profile of failure surface (Jutkofsky 1998).

Figure 7.6 shows a cross-section of the EPS-block geofoam embankment. Sheeting was required to support the excavation during soil removal and replacement due to the depth of soil removal required and the need to maintain one lane of traffic opened at all times adjacent to the excavation. The EPS-block geofoam fill system was designed against hydrostatic uplift because Schoharie Creek is located on the south side of the slope. Based on the 100-year flood, the depth of geofoam that could be used was limited to 4.6m (15 ft.). A subsurface drainage system was placed below the geofoam to lower the ground water table and to maintain a positive drainage path. The drainage system consisted of a 0.6m (2 ft.) thick layer of graded crushed stone with a network of 15cm (6 in.) diameter perforated polyethylene drainage pipes embedded in the stone. Both the stone and pipes are exposed on the embankment-slope face. The crushed stone also provided a working platform and a level surface for placement of geofoam blocks.

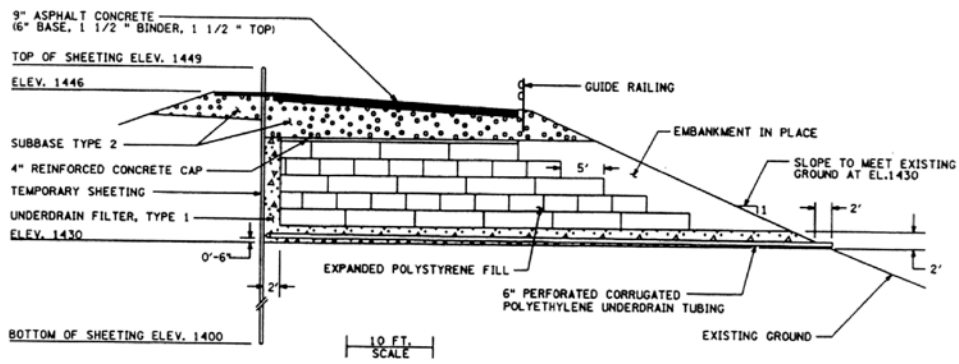


Figure 7.6. Profile of EPS-block geofoam embankment (Jutkofsky 1998).

The EPS block was placed in mid-November 1995 and all construction was completed in January 1996. The blocks, which had dimensions of 0.6 by 1.2 by 2.4m (2 by 4 by 8 ft.), were delivered to the site on flatbed trailers. About 76m^3 (100 yd^3) of blocks arrived per truckload. Blocks were unloaded by two laborers and carried and placed by four persons. Average placement rate was 1 hour to unload and place a trailer load of 40 blocks. This is a placement rate of 76m^3 (100 yd^3) per hour, or about 382m^3 (500 yd^3) per day. Two metal barbed inter-block connector plates were placed on each block. One plate was placed in the center and the second plate was placed near an edge to approximate a 1.2m (4 ft.) grid pattern. A 102mm (4 in.) thick reinforced concrete slab was placed over the geofoam. A minimum of 0.6m (2 ft.) of subbase material of graded crushed stone was placed over the concrete cap. This minimum thickness of subbase material was based on the Norwegian experience to minimize potential problems of differential pavement icing. A supplemental measure utilized by the NYSDOT to minimize differential icing included use of a subbase material with 25 to 60 percent passing the 6.35mm (1/4-in.) sieve to provide a high heat-sink capacity. The pavement consisted of 229mm (9 in.) of asphalt concrete.

The quantity of EPS fill initially estimated was 3,116m^3 (4,075 yd^3). The bid price for the EPS block was \$85.01 per m^3 (\$65 per yd^3). However, only 2,818m^3 (3,685 yd^3) of geofoam was used because the sheeting was driven 0.6m (2 ft.) off-line toward the excavation. To compensate for the reduced amount of soil removed, additional soil fill was placed along the toe of the slope. Removal of 2,818m^3 (3,685 yd^3) of soil and replacement with geofoam resulted in a net reduction of driving weight of about 5,352Mg (5,900 tons) and an increase in factor of safety from 1.0 to over 1.5.

No significant movements have been recorded in slope inclinometers between the end of construction and December 1998. Piezometers installed within the crushed stone drainage blanket below the geofoam have indicated no pore pressure buildup since installation in November 1995. The NYSDOT is obtaining readings twice a year during wet periods to monitor pore pressure buildup that may indicate that the drainage blanket is clogged, serving as an early warning of rising water table which may cause uplift of the geofoam. No differential icing during the winter, or pavement deterioration due to slight temperature increases recorded by thermistors in the subbase during the summers, have been observed between the end of construction and December 1998.

WISCONSIN: BAYFIELD COUNTY TRUNK HIGHWAY A

This case history involves use of EPS-block geofoam as a hillside fill to repair a slow-moving landslide that had persisted for over 20 years (Reuter and Rutz 2000). Project information for this case history was obtained from Reuter and Rutz (2000) and Reuter (2001). The Bayfield County Trunk Highway A in northern Wisconsin was 45m (148 ft.) wide and had a slope of approximately 14 degrees in the landslide area. The height of the embankment was 5m (16 ft.). The glaciolacustrine soils below the embankment consist of very soft, highly plastic clays and silts. The failure surface identified by an inclinometer is 6m (20 ft.) below grade and sliding was occurring in soft, highly plastic clays and silts. The roadway was frequently patched due to tension and lateral shear cracks that developed within the asphalt pavement.

In addition to a lightweight fill alternative, excavation of soils within the slide mass and replacement with granular fill was also considered. However, the total excavation alternative was not selected because it required excavation below ground water level and temporarily closing the highway. Soil from the head of the slide was removed and replaced with three layers of EPS-block geofoam. The geofoam had a density of 24 kg/m^3 (1.5 lbf/ft^3) and dimensions of 0.8 by 1.2 by 2.4m (2.7 by 4 by 8 ft.).

A drainage blanket consisting of 0.3m (1 ft.) of free-draining sand conforming to Wisconsin Department of Transportation (WDOT) Section 209, Grade 1 and a system of 200mm (8 in.) diameter slotted plastic pipe, was placed below the EPS block. Drain pipes were placed parallel to the road at the back of the excavation and transverse to the road at 15m (50 ft.) intervals. The transverse pipes extended from the parallel pipe at the back of the excavation to the embankment face. An impermeable membrane was placed on top of the geofoam as protection against petroleum spills.

The top of the geofoam was kept at a depth of 1.5m (4.9 ft.) below the final pavement surface to minimize potential for differential icing conditions. The fill placed over the EPS block and sides of the embankment consists of free-draining sand. The installed cost of EPS block was $61.50 per m^3 ($47.00 per yd^3). EPS-block geofoam was also used to remediate two other landslides along Bayfield County Trunk Highway A, but exact cost information is not available for these applications. Figures 7.7 and 7.8 show construction photos of the project.

Figure 7.7. Geofoam block placement on the embankment at Bayfield County Trunk Highway A (G. Reuter).

Figure 7.8. Geofoam block placement on the embankment at Bayfield County Trunk Highway A (G. Reuter).

ALABAMA: SLOPE CORRECTION ON SR 44 IN GUIN

A section of AL State Route 44 in Guin, Alabama had been intermittently sliding for several years. By spring of 1996, after a very wet season contributed to severe pavement distress as shown in Figure 7.9, the Alabama Department of Transportation (ALDOT) decided to explore different slope stabilization alternatives. Figure 5.2 shows the resulting scarp due to slope movement.

The ALDOT investigated excavation and replacement of the slide material using lightweight expanded clay and expanded shale, sawdust, and wood chips. However, ALDOT ruled out these lightweight fill materials because they did not decrease the overall weight of the slide mass sufficiently to stabilize the slope. They also considered chemical stabilization, but also ruled it out because it to did not decrease the overall weight of the slide mass sufficiently to stabilize the slope, and because of its high cost. Based on the successful use of EPS-block geofoam by the CDOT as part of the Hwy. 160 slide repair and by the NYSDOT as part of the State Route 23A slope stabilization, the ALDOT evaluated the EPS block alternative.

Figure 7.9. Pavement cracking on SR 44 in Guin, AL (ALDOT).

Figure 7.10 shows a cross-section of the slope prior to remediation, and Figure 7.11 shows a cross-section of the stabilized slide with EPS-block geofoam fill. Table 7.1 provides a summary of the material properties and Table 7.2 provides a summary of results of slope stability analyses performed for both the primary and secondary slip surfaces.

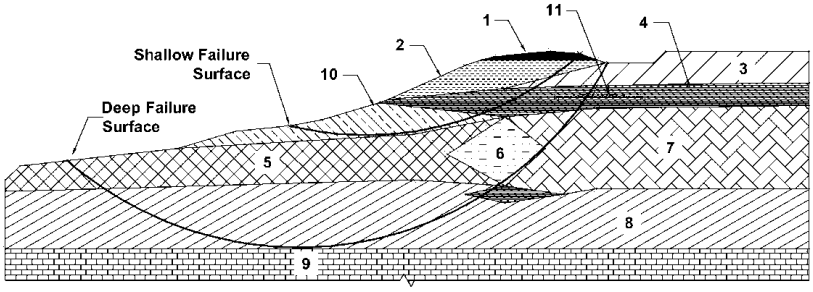


Figure 7.10. Subsurface profile of landslide from Guin Co. AL case history prior to placement of EPS geofoam.

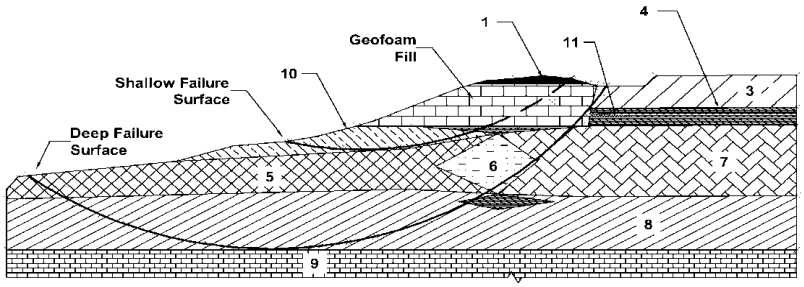


Figure 7.11. Subsurface profile of landslide from Guin Co., AL, case history following placement of EPS geofoam.

Table 7.1. Material properties used in Guin Co., AL, case history.

Material #	Material Description	γ_{total} (lb/ft^3)	γ_{sat} lb/ft^3)	c (lb/ft^2)	φ (deg)
	Geofoam Fill		*- See Table 7.2 -*		
1	Fill: Sandy Clay	120	130	2100	17
2	Fill: Sand-Clay-Gravel	111	125	0	30
3	Medium Moist-Wet Sandy Gravel	120	130	2900	17
4	Very Loose Wet Sand	94	115	0	29
5	Stiff Damp Silty Clay	120	130	1300	0
6	Medium Wet Silty Clay(ey Sand)	125	135	0	32
7	Hard Wet Silty Clay	133	140	4050	0
8	Very Stiff Damp Silty Clay	125	135	2000	0
9	Hard Moist Silty Clay	133	143	4000	0
10	Loose Wet Silty Gravelly Sand	111	125	0	29
11	"Hard Pan" Lens	133	143	6250	0

Table 7.2 Factors of safety for primary & secondary landslides using various grades of EPS-block geofoam (Alabama Department of Transportation, 2004).

EPS Type	–	*EPS 40*	*EPS 50*	*EPS 70*	*EPS 100*
Unit Weight of EPS	0.75 lb/ft^3	1.0 lb/ft^3	1.25 lb/ft^3	1.5 lb/ft^3	2.0 lb/ft^3
Strength Parameters	c = 360 lb/ft^2	c = 720 lb/ft^2	c = 936 lb/ft^2	c = 1080 lb/ft^2	c = 1800 lb/ft^2
F for Primary Landslide (Deep)	1.16	1.16	1.16	1.17	1.17
F for Secondary Landslide (Shallow)	1.33	1.73	1.97	2.12	2.64

The report section on Modeling EPS Block Shear Strength in Chapter 3 provides the significance of slope stability results shown in Table 7.2. The ALDOT utilized *EPS100* blocks for this project because it yielded a higher factor of safety for the shallower secondary landslide. Figure 7.12 provides a typical cross-section of the EPS block slope repair.

Figures 5.8 and 5.9 show the presence of free water due to ground water seepage during the AL 44 project excavation. Therefore, a permanent drainage system consisting of perforated underdrains within a granular layer overlying a geotextile was utilized, as shown by Figures 5.10 and 5.11 and as noted in the design cross-section in Figure 7.12. Figures 5.28 and 5.29 show the completed concrete drainage channel constructed as part of the AL 44 project drainage system that diverts the water collected in the underdrains away from the completed EPS-block geofoam stabilized slope.

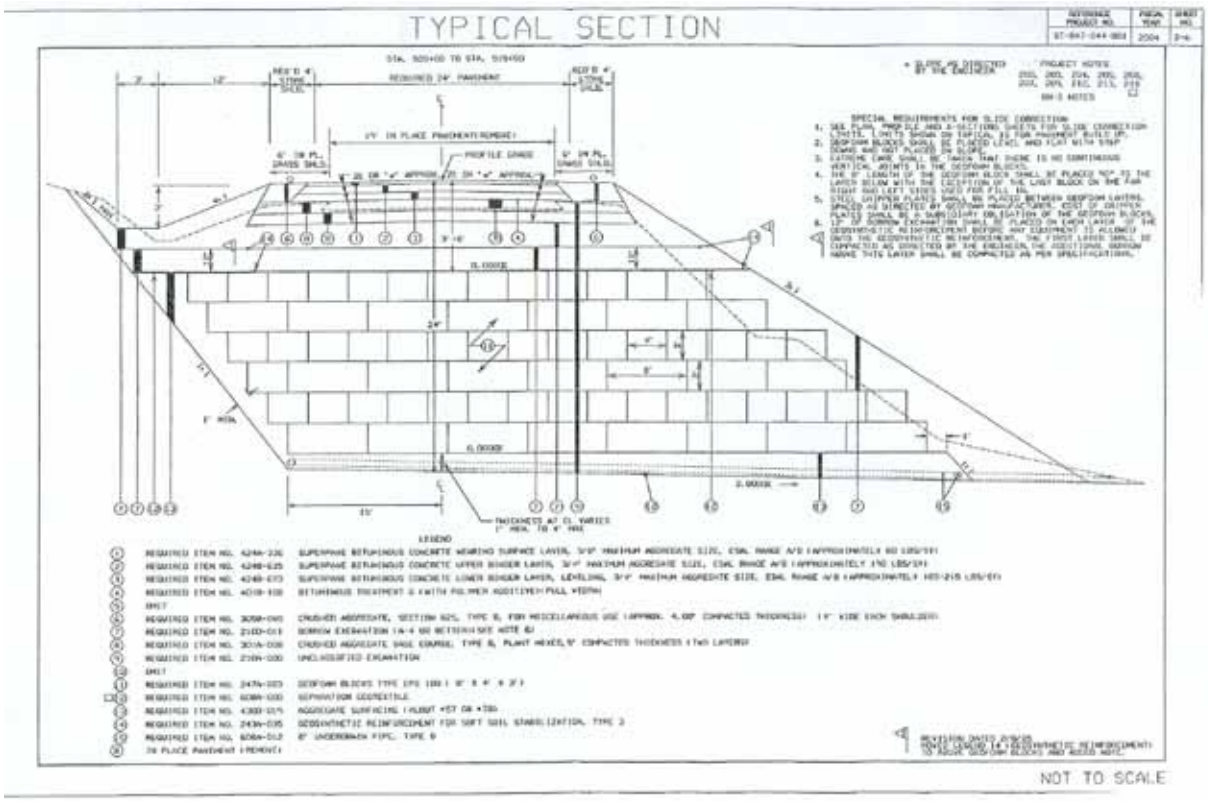


Figure 7.12. Typical cross-section of the Guin, AL, slope stabilized with EPS blocks (ALDOT).

Figures 5.3 through 5.21, 5.23 through 5.26, 5.28, and 5.29 consist of photos of the construction process. For this construction project, 12,947m^3 (16,934 yd^3) of geofoam blocks were used. With the cost of *EPS 100* block geofoam at \$131/ m^3 (\$100/yd^3), the total cost of geofoam was \$1,693,400. The cost for the entire project was \$5,143,035.

SUMMARY

This chapter presents four case histories of geofoam usage in slope stabilization and repair. These case histories demonstrate clearly that EPS-block geofoam can contribute to cost-effective and successful slope stabilization and repair. For example, EPS-block geofoam was selected by state DOT representatives or their representatives over a partial or total slide material removal and replacement with another earth material during the CDOT Highway 160, NYSDOT State Route 23A, and Wisconsin Bayfield County Trunk Highway A projects because the removal and replacement procedure was too costly, right-of-way limitations, concerns with impacting adjacent environmentally sensitive areas, concerns with the need to implement an extensive temporary dewatering system during the removal and replacement procedure, and the need to close the road during the removal and replacement procedure. The CDOT Highway 160 project

also demonstrated that stabilizing a slope with EPS blocks can be especially cost-effective in comparison with traditional earth retention systems.

The ALDOT State Route 44 project showed that the lower density EPS block, compared to other types of lightweight fills such as expanded shale, sawdust or wood chips, can yield a slope with the desired stability where alternative lightweight fill materials cannot. The CDOT Highway 160 project also demonstrated that EPS block can be placed during the winter in cold weather climates when the water level may be the lowest, thus minimizing the need for an extensive temporary dewatering system during construction.

All four case histories included use of a drainage system below EPS block to prevent water from accumulating above the bottom of the EPS block, and in some cases, incorporated a drainage system between the adjacent upper slope material and EPS block to collect and divert seepage water and thereby alleviate seepage pressures. Therefore, these case histories substantiate the recommendation included in the proposed design procedure of EPS-block geofoam slope systems presented in Chapter 4 and included in the design guideline in Appendix B that all EPS-block geofoam slope systems incorporate drainage systems to alleviate the need to consider and design for hydrostatic uplift (floatation) and translation due to water. Therefore, hydrostatic uplift and translation due to water failure mechanisms are not included in the recommended design procedure shown in Figure 4.25.

The literature search performed as part of this study revealed that, unlike use of EPS-block geofoam for stand-alone embankments over soft ground, the U.S. case history experience with EPS-block geofoam in slope stabilization is limited. However, it is anticipated that results of this report will facilitate use of EPS-block geofoam for slope stabilization and repair in the U.S. and, consequently, designers involved with slope stabilization and repair will consider EPS-block geofoam as an alternative to slope stabilization more in the future than in the past.

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CHAPTER 8

ECONOMIC ANALYSIS

INTRODUCTION

To develop an optimal design, designers need EPS-block geofoam cost data of slope stability applications to perform a cost comparison with other slope stabilization alternatives. Therefore, an important aspect of this research project was to quantify the economic advantages of using EPS-block geofoam as a design alternative compared to other lightweight fill materials and traditional alternatives to stabilizing slopes.

Various strategies were employed to obtain cost comparison data. For example, cost information was solicited via a geofoam usage questionnaire during Task 1. Industry representatives who replied to the project survey that had or were planning to utilize geofoam in slope stabilization were contacted. Additionally, DOTs, companies, and individuals referenced in relevant geofoam technical literature that have been involved in slope stability projects using lightweight fills were also contacted. Despite these various strategies, availability of cost information and economic assessments is limited because the U.S. case history experience with EPS-block geofoam in slope stabilization is limited.

An economic analysis was performed as part of the NCHRP Project 24-11(01) study for stand-alone embankments over soft ground, with results included in a comprehensive report (Stark et al., 2004a). The stand-alone embankment economic analysis summary includes a cost comparison between various lightweight fill materials, a comparison of lightweight fill material properties, and a cost summary of various ground improvement techniques. The cost data on various lightweight fills and ground improvement techniques was included in Project 24-11(01) documents because it provides a convenient means of performing a general cost comparison between use of EPS-block geofoam and various soft ground treatment alternatives. The purpose of this section is to provide a summary of new cost information related to use of EPS-block geofoam in slope applications obtained during the Project 24-11(02) study.

Comparisons between various design alternatives require an economic analysis, such as a cost-benefit analysis, be performed. A cost-benefit analysis considers intangible consequences or impacts of an alternative in addition to tangible costs and benefits. Therefore, any cost comparison involving EPS-block geofoam should include not only basic capital construction costs, but should also include potential intangible benefits such as accelerated construction, ability to easily implement phased construction, and minimal long-term maintenance. A summary of intangible benefits is presented later in this report section. Unfortunately, many intangible benefits of EPS-block geofoam as a construction material can be difficult to quantify, which has historically led to EPS-block geofoam being evaluated primarily on a material cost basis, without accounting for many of its additional advantages. While this approach is simple and easy to implement, it may not contribute effectively to a realistic cost comparison analysis.

When considering EPS-block geofoam as an alternative for a slope stability project, design professionals should be careful to consider the full variety of EPS-block geofoam options. For example, a traditional EPS-block geofoam fill with a 150mm (6 in.) thick load distribution slab and a vertical precast PCC facing wall might appear uneconomical when initially compared to alternative slope stabilization procedures. However, an alternative design using multiple densities of EPS block to support vertical loads with an exterior insulation facing system (EIFS, or synthetic stucco) to protect EPS block might prove economical. Thus, when conducting a cost evaluation involving EPS-block geofoam, the impact of various system components (such as metal connectors, load distribution slab, and facing system) on the overall cost should be considered. In reality, the cost of these components can have a more significant impact on overall cost of the EPS-block slope system than the actual cost of EPS blocks themselves. Therefore, by considering the full range of options for these system components, overall project cost can often be significantly reduced.

SUMMARY OF COST DATA

As a part of the NCHRP Project 24-11(02) project, additional cost data has been collected and compiled below in Table 8.1. Table 8.1 includes new cost data obtained during this study, as well as cost data collected for the Project 24-11(01) study, and is intended to complement and update information given in the first report. The cost summary in Table 8.1 reveals two trends. First, EPS-block geofoam prices vary widely. Second, the price of EPS-blocks has increased from the late 1990s to the mid 2000s.

The first trend is in agreement with the results of the Geofoam Usage Survey (see Appendix A) conducted as part of this project, which indicates one concern in considering EPS-block geofoam use in potential lightweight fill applications is lack of reliable cost information. This concern also supports the general consensus of the FHWA that bid prices highway projects have received nationally are extremely variable and difficult to estimate (Nichols, 2008). The wide variance in price is perhaps one of the greatest hindrances to further adoption of EPS-block geofoam in the U.S.

The potential factors that influence the cost of EPS-block geofoam in stand-alone embankment projects over soft ground may also be applicable in slope stabilization projects. Therefore, Table 8.2, which was included in the Project 24-11(01) report, was updated and included herein. This list is not comprehensive, and may not include all items that may be required on a slope stabilization project. However, the number of factors indicated in Table 8.2 that may impact the cost of EPS-block geofoam may help explain the large cost variance shown in Table 8.1.

Costs for EPS blocks included in Table 8.1 may not be necessarily based on actual cost of the EPS block only, but may include items such as mechanical connectors. Additionally, the distance from the molding facility to project site will vary between various projects listed in Table 8.1 and may contribute to some of the cost disparities if shipping is included in the costs. Nonetheless, cost comparisons using data from Table 8.1 are useful if these cost disparities are considered.

The second trend of increasing EPS block prices in the last decade, especially in the mid to late 2000s, can be partially attributed to the cost of petroleum/oil. Polystyrene is a derivative of petroleum. Therefore, fluctuations in the price of oil can have a significant impact on the cost of EPS-block geofoam. Table 8.3 shows the average cost of oil and EPS-block geofoam for the years 2000, 2002, and 2007. The cost of oil from 2000 to 2002 decreased slightly, about $4 per barrel, and the cost of EPS-block geofoam remained steady. Between the years 2000 and 2007, the cost of oil increased by about $42 per barrel and the cost of EPS-block geofoam increased by about $18 per cubic meter. The cost of oil shown in Table 8.3 was obtained from the Energy Information Administration (2008).

Table 8.1. Summary of EPS-block geofoam costs.

Date	Location of Project	Project Type	EPS Density kg/m^3 (lb./ft^3)	Quantity of EPS-Block m^3 (yd^3)	Unit Cost of EPS-Block \$/m^3 (\$/yd^3) (1)	Unit Cost of EPS-Block Adjusted to 2008 Dollars \$/m^3 (\$/yd^3) (4)	Normalized Cost of EPS-Block Geofoam Adjusted to 2008 Dollars \$/kg (\$/lb.) (4)	Approximate Placement Rate m^3/day (yd^3/day)	Contract Value
1993	Wyoming	Bridge Approach	24 (1.5)	377 (493)	39.00-72.00 (30.00-55.00) (2)	58.00-107.50 (44.75-82.00) (2)	2.42-4.48 (1.10-2.02) (2)	----	\$79,732
1993-1994	Hawaii	Embankment	22 (1.35)	13,470 (17,600)	----	----	----	560 (735)	----
1995	Indiana	Embankment	24 (1.5)	4,707 (6,156)	86.59 (66.20)	122.49 (93.65)	5.10 (2.31)	428 (560)	\$607,207
1995	New York	Slope	20 (1.25)	2,819 (3,585)	85.01 (65.00)	120.26 (91.96)	6.01 (2.72)	382 (500)	----
1995	Washington	Bridge Approach	----	1,835 (2,400)	72.00 (55.00)	101.86 (77.81)	----	----	----
1995±	Washington	Embankment	18 (1.13)	411 (537)	88.25 (67.50)	124.85 (95.49)	6.94 (3.13)	----	----
1997 1999±	Wyoming	Bridge Approach	24 (1.5)	146 (191)	104.00 (79.51)	137.56 (105.17)	5.73 (2.60)	----	\$30,326
1999±	Connecticut	Embankment	----	321 (420)	98.00 (75.00)	126.82 (97.06)	----	----	----
1999±	Maine	Embankment	----	----	57.21 (43.74) FOB Site	74.04 (56.60)	----	----	----
1999±	Michigan	Embankment	----	1,052 (1,376)	52.50/43.00 (40.14/32.88) (3)	67.94/55.65 (51.95/42.55) (3)	----	----	\$1,960,245/ \$2,202,667
1999±	Michigan	Embankment	----	4,919 (6,434)	58.50/50.00 (44.73/38.23) (3)	75.71/64.71 (57.89/49.47) (3)	----	----	\$5,696,732/ \$5,970,269
1999±	Utah	Vertical Embankment	18 (1.13)	100,000 (130,800)	65.00 (50.00) (w/o facing wall) 75.00 (57.00) (w/ facing wall)	84.12 (64.71) (*w/o facing wall*) 97.06 (73.76) (w/ *facing wall*)	4.67 (2.12) (*w/o facing wall*) 5.39 (2.42) (*w/ facing wall*)	362 (474)	\$1.5 Billion (Entire Project)
1999	Illinois	Embankment	24 (1.5)	15,291 (20,000)	----	----	----	313 (409)	----
1999	Wisconsin	Slope	24 (1.5)	----	61.50 (47.00)	79.59 (60.82)	----	----	----
2004	Alabama	Slope	29 (1.8)	12,947 (16,934)	130.80 (100.00)	149.29 (114.13)	5.15 (2.35)	----	\$5,143,000
2004-2005	Virginia	Bridge Abutment	24 (1.5)	15,475 (20,240)	204.92 (156.76)	226.22 (173.05)	9.43 (4.27)	100 16ft length blocks/day	----
2006-2007	Maine	Bridge Approach	29 (1.8)	3,326 (4,350)	190.00 (145.00)	197.61 (150.81)	6.81 (3.10)	----	\$600,000

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.
(4) Dollar value conversions based on Consumer Price Index (CPI) using calculator provided by The Federal Reserve Bank of Minneapolis. This calculator may be accessed online via: http://www.minneapolisfed.org/index.cfm

Table 8.2 Potential costs associated with an EPS-block geofoam project.

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 vs. 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. Manufacturer's facilities
   - 4.1 Number, size, and age of molds
   - 4.2 Rate of block production
   - 4.3 Temporary storage facilities
5. Shop drawings
6. Complexity of factory cut of blocks
7. Insecticide
8. Transportation from molder to job site
   - 8.1 Rate of delivery
9. Overall project volume
10. Project schedule

DESIGN DETAIL COSTS:

1. Use of connector plates
2. Geometric 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

Table 8.3 Comparison of EPS-block geofoam price per unit volume with average fuel cost (Energy Information Administration, 2008).

		EPS-Block Geofoam Unit Price ($/m^3)		
NCHRP Density (kg/m^3)	*ASTM Density (kg/m^3)*	2000 (Stark et al. 2004)	2002 (Negussey, 2002)	2007 (Molder A)
----	**11.2**	----	$32.00	----
----	**14.4**	----	$38.00	----
16	**----**	$42.51	----	$59.33
----	**18.4**	----	$48.00	----
20	**----**	$52.97	----	$68.22
----	**21.6**	----	$58.00	----
24	**----**	$67.36	----	$77.12
----	**28.8**	----	$76.00	----
32	**----**	$83.06	----	$97.89
Avg. Fuel Cost (EIA) ($ per barrel)		**$30.26**	**$26.15**	**$72.52**

Note: ---- Data not available

DRAFT EPS-BLOCK GEOFOAM PRICE ADJUSTMENT SPECIAL PROVISION

Short-term oil price fluctuations can have a significant impact on the cost of EPS-block geofoam during multi-phased projects. Therefore, there is a need to develop a price adjustment contract provision similar to special provisions that DOTs have developed for other construction materials, such as bituminous asphalt binder. A suggested example of a price adjustment contract provision for EPS-block geofoam is included in Appendix I. This draft provision is based on the TN Department of Transportation special provision for price adjustment for bituminous material (TDOT, 2000). This draft special provision should be reviewed by appropriate contracting officials before it is used on any project. One issue that may require further investigation involves determining whether the price adjustment should be based on unexpanded polystyrene solid resin beads or expanded polystyrene beads, also known as “pre-puff.”

In addition to influencing the cost of raw materials such as polystyrene resin beads, fluctuations in the price of oil can also have a significant influence on the cost of EPS-block geofoam by impacting the cost of energy needed to expand the unexpanded resin bead to form pre-puff, as well as to mold the pre-puff into blocks. This influence may be significant and it represents another link between the price of crude oil and the price of EPS-block geofoam that should be considered when formulating a special provision for EPS-block geofoam price adjustment.

PRELIMINARY SELECTION OF EPS BLOCK TYPE FOR DEVELOPING A PRELIMINARY COST ESTIMATE

Step 10 (Load-Bearing) of the design procedure presented herein consists of selecting an EPS type with an adequate elastic limit stress to support the overlying pavement system, if required, and overlying dead-load and traffic stresses without excessive EPS compression that could lead to excessive settlement of the pavement or final embankment surface. In general, the higher the required elastic limit stress, the higher the EPS block density. As shown in Table 8.3, the cost of EPS block increases as density increases.

A preliminary EPS block type is typically required prior to completion of the slope stabilization design to develop a preliminary cost estimate to determine the feasibility of stabilizing the slope, or to

compare various stabilization methods and select a specific stabilization method. Therefore, the availability of guidance to select a preliminary EPS block type for preparation of a preliminary cost estimate for EPS blocks would be useful in practice. Consequently, a wheel load stress distribution chart, similar to the chart included in the Vencel Resil's Civils Handbook of EPS Civil Engineering Products (Vencel Resil, Undated), was developed. The purpose of this stress distribution chart is to aid the designer in estimating stresses with depth within the EPS blocks for preliminary selection of EPS block type and preparation of a preliminary estimate of EPS block costs. The chart is not intended to be used for design of the EPS-block fill mass.

AASHTO LRFD Bridge Design Specifications indicate that vehicular live loadings on bridges or incidental structures shall consist of a combination of the following two loads: (1) design truck or design tandem, and (2) design lane load (AASHTO, 2007, 2010). Only the vehicular live load loading of the design truck is included in this analysis. Characteristics of the design truck are shown in Figure 8.1. As shown, design truck loadings consists of one 35 kN axle load and two 145 kN axle loads. Spacing between the two 145 kN axles should ideally be varied between 4,300mm to 9,000mm to produce the extreme force effect, e.g., shear and/or moment. For selection of EPS blocks, the key force effect is normal stress. Therefore, typically the lower 4,300 mm spacing will produce the greatest normal stresses with depth. Dead loads and dynamic load allowances are not included in this simplistic analysis, but must be considered in the design; the purpose of this analysis is to develop a stress distribution chart a designer can use to perform a preliminary estimate of EPS blocks costs, and not for design.

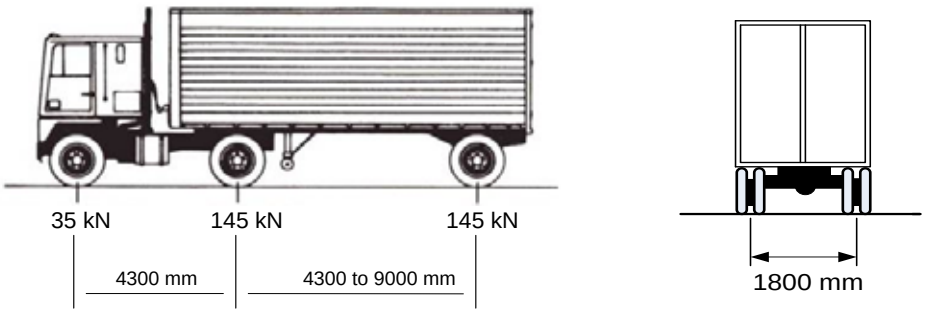

Figure 8.1. Characteristics of the AASHTO design truck (From AASHTO LRFD Bridge Design Specifications, 2007, by the Association of State Highway and Transportation Officials, Washington, D.C., used by permission).

Figure 8.2 shows the AASHTO wheel contact area, which AASHTO indicates can consist of one or two tires. As shown in this figure, the contact area of a wheel is assumed to be a single rectangle with a width of 510mm and length of 250mm. The smaller 250mm length dimension is parallel to traffic direction and the larger 510mm width dimension is perpendicular to traffic direction. The tire pressure is assumed to be uniformly distributed over the contact area.

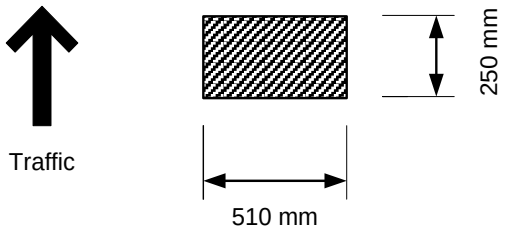

Figure 8.2. Wheel contact area and corresponding traffic direction.

Figure 8.3 shows the design truck wheel contact area designations included in the stress distribution analysis.

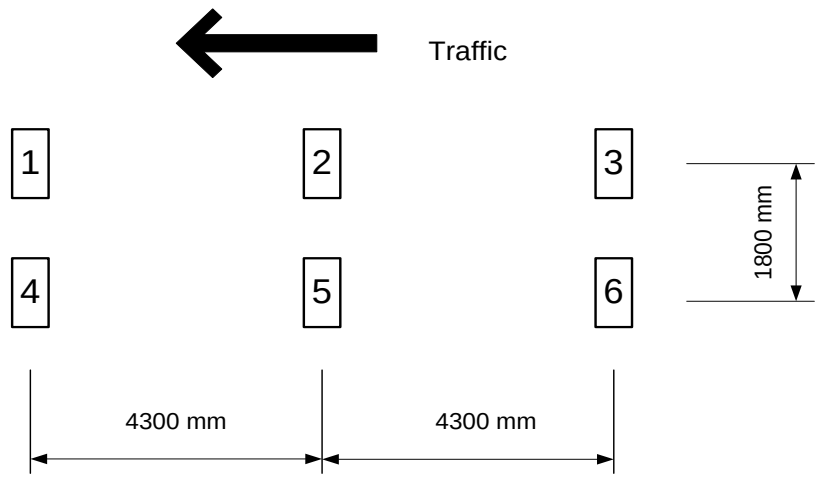

Figure 8.3. Design truck wheel contact area designations and corresponding traffic direction.

The distribution of stress with depth, *p*, is assumed to occur at a ratio of 1 (horizontal): 2 (vertical) and is based on Equation 8.1:

$$p = \frac{Q}{(B+Z)(L+Z)} \tag{8.1}$$

where

Q = traffic load on one tire or dual tires, kN

B = width of the contact area, m

L = length of the contact area, m

Z = depth below the surface, m

Figure 8.4 illustrates the stress distribution of the front 35 kN axle shown in Figure 8.1, and the resulting stress distribution corresponds to stresses of contact areas 1 and 4, as shown in Figure 8.3. As indicated in Figure 8.4, stresses of the two 17.5 kN contact areas overlap at a depth of approximately 1.29m. Therefore, from the surface of the load application to a depth of 1.29m, stresses between the two contact areas are independent of each other, while at depths below 1.29m, the total stress with depth is the sum of the stresses underlying each of the two contact areas. For example, at a depth of 3m, total stress is 3.06 kPa, which is the sum of the stresses from each of the two 17.5 kN contact areas of 1.53 kPa per contact area.

Figure 8.5 shows the stress distribution of one of the 145 kN axles shown in Figure 8.1, and is representative of contact areas 2 and 5 or 3 and 6 depicted in Figure 8.3. As indicated by Figure 8.5, the stresses of the two 72.5 kN contact areas also overlap at a depth of approximately 1.29m.

Figure 8.6 shows the stress distribution parallel to traffic direction and represents the stresses from contact areas 1, 2, and 3 or 4, 5, and 6 shown in Figure 8.3. As indicated in Figure 8.6, stresses in

the traffic direction overlap at a depth of 4.05m, and the total stress is 4.59 kPa between the 17.5 kN and 72.5 kN contact areas, and 7.40 kPa between the two 72.5 kN contact areas.

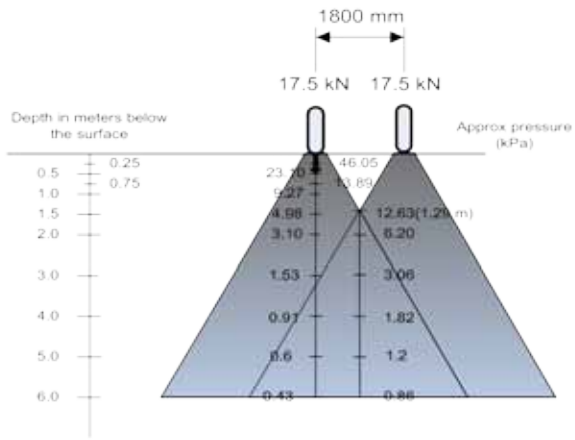

Figure 8.4. Stress distribution of design truck 35 kN axle corresponding to contact areas 1 and 4 shown in Figure 8.3.

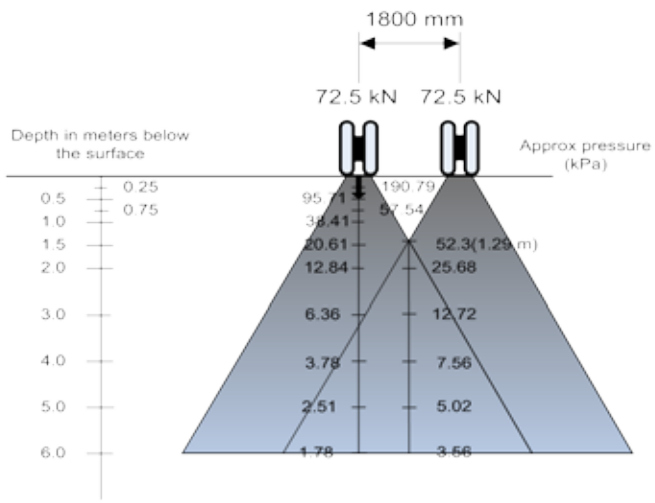

Figure 8.5. Stress distribution of design truck single 145 kN axle corresponding to contact areas 2 and 5 or 3 and 6 shown in Figure 8.3.

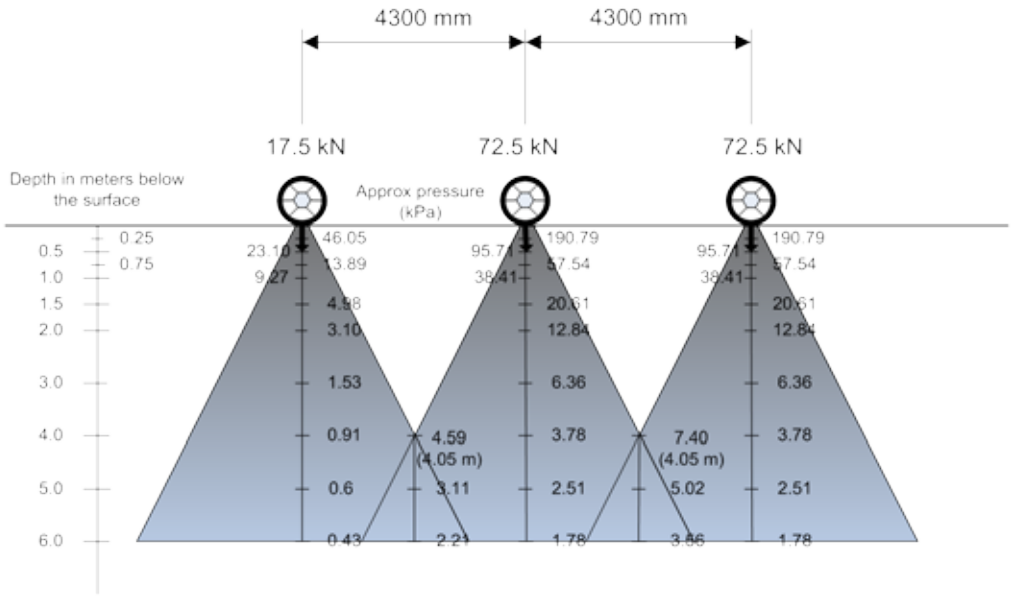

Figure 8.6. Stress distribution parallel to traffic direction corresponding to contact areas 1, 2, and 3 or 4, 5, and 6 shown in Figure 8.3.

Figure 8.7 shows the resultant of all the contact areas of the design truck shown in Figure 8.3. Figure 8.7 includes the sum of stresses depicted in Figures 8.4 through 8.6. For example, the stresses shown below the 35 kN axle are based on the total stresses shown in Figure 8.4, and stresses shown below the 145 kN axles are based on the total stresses shown in Figure 8.5. Results of Figure 8.6 indicate that the stress overlap in traffic direction occurs at a depth of 4.05m. Therefore, as shown in Figure 8.7, the total stress at depths of 4.05m and greater is the sum of each of the two adjacent axles. For example, at a depth of 5m below the surface, total stress between the 35 kN and 145 kN axle is 6.22 kPa, which is the sum of the stresses directly below the 35 kN axle of 1.2 kPa and below the 145 kN axle of 5.02 kPa. Also, at a depth of 5m, total stress between the two 145 kN axles is 10.04 kPa, which is the sum of the stresses directly below the two 145 kN axles of 5.02 kPa per axle.

Figure 8.7 can be used to obtain a preliminary cost estimate. For example, if the proposed height of the EPS fill mass is 5m, Figure 8.7 can be used to obtain the estimated largest stress, with depth as shown in Table 8.4. After the largest stress at various depths is determined, an EPS type with an elastic limit stress that exceeds the expected maximum stress is selected, as shown in Table 8.4. Once the preliminary EPS type is determined, preliminary EPS prices can be obtained from local molders and an estimated preliminary cost estimate can be prepared. The purpose of Figure 8.7 is to aid the designer in estimating stresses with depth within EPS blocks for preliminary selection of EPS block type and preparation of a preliminary cost estimate for EPS block. The chart is not intended to be used for design of EPS-block fill mass.

The final EPS block types and costs should be based on the complete recommended design procedure, because the procedure considers dead loads and dynamic loads and the expected stress distribution through the pavement system for projects that include a pavement system. The pavement system will typically dissipate stresses more effectively than the assumed 1(horizontal): 2 (vertical) stress distributions included in the Figure 8.7 analysis.

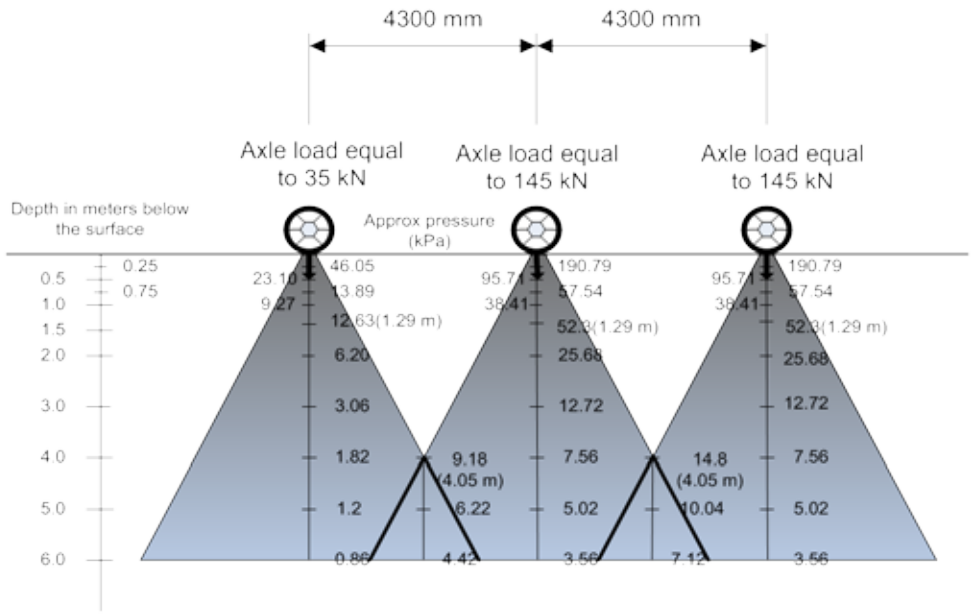

Figure 8.7. Stress distribution from all contact areas of the design truck shown in Figure 8.3.

Table 8.4. Summary of example problem showing EPS block selection based on Figure 8.7.

Depth (m)	**Estimated Largest Preliminary Stress (kPa)**	**Preliminary EPS Type**
0.25	191	Standard EPS type not available. However, for preliminary costing assume *EPS 100*
0.5	96	*EPS100*
0.75	58	*EPS70*
1.0	38	*EPS70*
1.29	52	*EPS70*
2	26	*EPS40*
3	13	*EPS40*
4	15	*EPS40*
5	10	*EPS40*

COST CONSIDERATION IN RECOMMENDED DESIGN GUIDELINE

Table 8.3 shows the cost of EPS blocks increases as density increases. The cost of EPS-block geofoam is dependent on the density of the block because a higher density block requires more polystyrene. Therefore, there is a cost incentive to select EPS blocks with the lowest possible density that will provide adequate support for the proposed loads.

The recommended design procedure for use of EPS-block geofoam in slope applications shown in Figure 4.25 is based on use of EPS blocks with the lowest possible density that provide adequate load-bearing capacity within tolerable settlements. Because the applied vertical stress within an EPS-block fill mass decreases with depth, it is possible to use multiple densities of EPS blocks in the slope cross-section.

Therefore, an advantage of the recommended deformation-based design procedure is the calculation of stresses and strains within the EPS mass allows selection of the type of EPS blocks to be optimized by selecting blocks with the lowest density that will yield a minimum elastic limit stress that exceed the anticipated applied stresses. For example, lower density blocks can be used at greater depths than higher density blocks that must used directly under the pavement system. Therefore, the density of EPS blocks can be optimized and thus specified for various portions of the slope. Selection of EPS blocks with the lowest possible density yields a cost-efficient EPS-block geofoam slope, as shown in Table 8.3.

In summary, the recommended design procedure for use of EPS blocks in slopes shown in Figure 4.25 considers a pavement system with the minimum required thickness, a fill mass with the minimum thickness of EPS-block geofoam, and use of an EPS block with the lowest possible density to achieve the most cost- effective design. In addition to these cost aspects, the overall benefits that use of EPS-block geofoam contributes to a project, e.g., accelerated construction and no removal of the entire slide mass, as discussed below, should be considered as part of the slope stabilization method decision-making process.

CONSIDERATION OF OVERALL BENEFITS

As indicated in the NCHRP Project 24-11(01) report, the cost of EPS-block geofoam is higher than other types of lightweight fills based only on cost of material. However, use of EPS blocks can be more economical compared to the use of other types of lightweight fills and slope stabilization procedures if intangible benefits of using EPS blocks are reflected in the overall cost. These include:

- Ease of construction,
- Can contribute to accelerated construction,
- Ability to easily implement phased construction,
- Entire slide surface does not have to be removed because of the low driving stresses,
- Can be readily stored for use in emergency slope stabilization repairs,
- Ability to reuse EPS blocks utilized in temporary fills,
- Ability to be placed in adverse weather conditions,
- Possible elimination of the need for surcharging and staged construction,
- Decreased maintenance costs as a result of less settlement from the low density of EPS-block geofoam,
- Alleviation of a need to acquire additional right-of-way for traditional slope stabilization methods due to the ease with which EPS-block geofoam can be used to construct vertical-sided fills,
- Reduction of lateral stress on bridge approach abutments,
- Excellent durability,
- Potential construction without utility relocation, and
- Excellent seismic behavior.

A more in-depth discussion of these benefits, as well as other issues related to the costs associated with EPS-block geofoam construction, is provided in the Project 24-11(01) report (Stark et al., 2004a). When performing an analysis to compare EPS-block geofoam to other potential alternatives, overall benefits of utilizing EPS-block geofoam should be considered when evaluating it as a potential alternative for a slope construction project. The benefit of accelerated construction that use of EPS-block geofoam can provide was a key factor in the decision to use EPS-block geofoam in projects such as the I-15 reconstruction project in Salt Lake City, UT; the Central Artery/Tunnel Project in Boston, MA; and the I-95/Route 1 Interchange (Woodrow Wilson Bridge Replacement) in Alexandria, VA (Nichols, 2008).

POTENTIAL IMPACT OF EXISTING MATERIAL STANDARDS ON COST

The material designation system included in the American Society of Materials and Testing (ASTM) standard related to geofoam, ASTM D 6817, is based on density (American Society for Testing and Materials, 2007). Although ASTM D 6817 can also be used to specify EPS blocks for lightweight fill applications, designers and specifiers should be aware that the ASTM specification utilizes a density-based designation system, and the corresponding ASTM elastic limit stress versus density relationship is different than the relationship incorporated in the recommended standard included in the NCHRP reports (Stark et al., 2004a).

Figure 8.8 shows that for a required elastic limit stress, a higher density is required based on the current ASTM material properties compared to the NCHRP relationship. A higher block density requires more polystyrene, which may result in higher block costs, as shown in Table 8.3. For example, if an elastic limit stress of 70kPa is required for a project from Figure 8.8, the minimum standard specimen density available that will provide this elastic limit stress is 24 kg/m^3 based on the NCHRP relationship, and the minimum standard specimen density available is 28.8 kg/m^3 based on the ASTM material properties. From Table 8.3, the 2007 cost is approximately $77 per cubic meter if the NCHRP standard is used and $89 per cubic meter if the ASTM standard is used. This is a difference of $12 per cubic meter.

For a slope stabilization project such as the Alabama project that utilized 12,947 cubic meters of EPS block, the cost increase of using ASTM versus NCHRP would be approximately $155,400. The resulting price differential between NCHRP and ASTM material properties could make use of EPS-block geofoam too costly as a stabilization alternative if specifications are based on the ASTM standard, and/or if bid prices are based on the ASTM standard. However, it is anticipated that if owners, designers, molders, and construction contractors are aware of the differences between the two standards, current confusion regarding availability of these two standards can be minimized. Also, it is anticipated that molders will become flexible in their molding process to supply blocks with a minimum specified elastic limit stress at the lowest possible density and develop prices that are more attractive to facilitate use of EPS blocks in slope stabilization projects. Use of performance-based specifications may also contribute to more economical EPS-block geofoam projects.

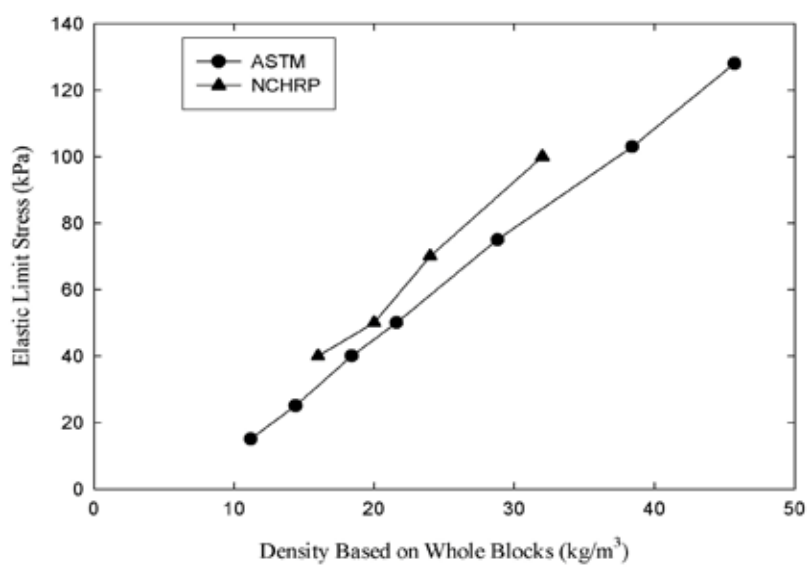

Figure 8.8. Comparison of density and elastic limit values between the interim NCHRP standard (Stark et al., 2004(a) and (b)) and ASTM D 681.

SUMMARY

A review of existing available EPS-block geofoam cost data indicates that EPS-block geofoam prices vary widely, and that the price of EPS blocks has increased recently due to the substantial increase in the price of oil. Therefore, a draft price adjustment contract provision similar to the special provisions that DOTs have utilized for other construction materials, such as bituminous asphalt binder, was developed as part of this project, and is included in Appendix I. The purpose of the price adjustment contract provision is to minimize the impact of short-term oil price fluctuations on the cost of EPS-block geofoam during multi-phased projects.

Figure 8.7 is a wheel load stress distribution chart developed to aid designers in estimating stresses with depth within the EPS blocks for preliminary selection of EPS block type and preparation of a preliminary cost estimate of EPS block to determine the feasibility of stabilizing the slope with EPS blocks, or to compare various stabilization methods during selection of a specific stabilization method. The chart is not intended to be used for design of the EPS-block fill mass.

To assist designers in designing a cost efficient EPS-block geofoam slope, the recommended design procedure for use of EPS blocks in slopes shown in Figure 4.25 considers a pavement system with the minimum required thickness, a minimum thickness of EPS-block geofoam, and use of an EPS block with the lowest possible density. Therefore, the design procedure will produce a technically adequate and cost-efficient design. However, in addition to the cost of the EPS blocks, overall intangible benefits that use of EPS-block geofoam contribute to a project should also be considered as part of the slope stabilization decision-making process. A summary of these intangible benefits are included in this economic analysis. In particular, the benefit of accelerated construction that use of EPS-block geofoam can provide should be evaluated, because it has been a key factor in the decision to use EPS-block geofoam in recent projects in the U.S.

The wide variance in price of EPS-block geofoam is perhaps one of the greatest hindrances to further adoption of EPS-block geofoam in the U.S. This wide variance in price may be attributed to a number of potential factors that can impact the cost of EPS-block geofoam. These potential factors are summarized in Table 8.2 and include factors related to manufacturing, design, and construction.

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CHAPTER 9

CONCLUSIONS, RECOMMENDATIONS, AND SUGGESTED RESEARCH

INTRODUCTION

A major transportation problem in the U.S. is that current highway capacity is insufficient to meet growing demand. Therefore, new roadway alignments and/or widening of existing roadway embankments will be required to solve current and future highway capacity problems. It is anticipated that the potential for landslides, which currently poses a major geologic hazard in the U.S., will increase as new roadway alignments are constructed and/or existing roadway embankments are widened.

EPS-block geofoam is a unique lightweight fill material that can provide a safe and economical solution to slope stabilization and repair. Benefits of utilizing EPS-block geofoam as a lightweight fill material include:

- Ease of construction,
- Can contribute to accelerated construction,
- Ability to easily implement phased construction,
- Entire slide surface does not have to be removed because of the low driving stresses,
- Can be readily stored for use in emergency slope stabilization repairs,
- Ability to reuse EPS blocks utilized in temporary fills,
- Ability to be placed in adverse weather conditions,
- Possible elimination of the need for surcharging and staged construction,
- Decreased maintenance costs as a result of less settlement from low density of EPS-block geofoam,
- Alleviation of a need to acquire additional right-of-way for traditional slope stabilization methods due to the ease with which EPS-block geofoam can be used to construct vertical-sided fills,
- Reduction of lateral stress on bridge approach abutments,
- Excellent durability,
- Potential construction without utility relocation, and
- Excellent seismic behavior.

The benefit of accelerated construction that use of EPS-block geofoam can provide was a key factor in the decision to use EPS-block geofoam in projects such as the I-15 reconstruction project in Salt Lake City, UT; the Central Artery/Tunnel Project (CA/T) in Boston, MA; and the I-95/Route 1 Interchange (Woodrow Wilson Bridge Replacement) in Alexandria, VA (Nichols, 2008). EPS blocks utilized in slope stabilization and repair may not support a pavement system or heavy structural loads. Therefore, the potential to utilize EPS block with recycled EPS exists. Use of recycled EPS block would be an attractive “green” product that reduces waste by recycling polystyrene scrap, and would also reduce raw material costs in EPS production (Horvath, 2008).

Although use of EPS-block geofoam for lightweight fill in stand-alone embankments and bridge approaches over soft ground has increased since completion of NCHRP Project 24-11(01) deliverables, an additional application of EPS-block geofoam for the function of lightweight fill that has been commonly used in Japan, but not extensively utilized in the U.S., is in slope stabilization applications. Therefore, a need existed in the U.S. to develop formal and detailed design documents, a design guideline, and an appropriate material and construction standard for use of EPS-block geofoam for slope stabilization projects. Slope stabilization projects include new roadways, as well as repair of existing roadways damaged by slope instability or movement. This need resulted in the current NCHRP Project 24-11(02), the results of which are described in this report. The overall objective of this research was to develop a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design

guidance for engineers to facilitate use of EPS-block geofoam for the function of lightweight fill in slope stability applications. This document includes the design guideline as well as an appropriate material and construction standard.

The general consensus reached at the first *International Workshop on Lightweight Geo-Materials* held on March 26-27, 2002, in Tokyo, Japan, is that although new weight-reduction techniques for decreasing applied loads have recently been developed, standardization of design and construction methods is required (A Report on the *International Workshop on Lightweight Geo-Materials,* 2002). Research results from Project 24-11(01), in conjunction with the results of this project, standardize design and construction standards for use of EPS-block geofoam in various U.S. highway applications.

The completed research consists of the following five primary research products: (1) summary of relevant engineering properties, (2) a comprehensive design guideline, (3) a material and construction standard, (4) economic data, and (5) a detailed numerical example. A summary of engineering properties of EPS-block geofoam that are relevant in the design of slopes is included in Chapter 3. A recommended design guideline is included in Appendix B. Chapter 4 provides background to the design guideline and is the commentary to the design guideline. A recommended combined material and construction standard covering block-molded EPS for use as lightweight fill in slope stabilization and repair is included in Appendix F. Chapter 6 provides the basis of the recommended standard. Cost information related to use of EPS-block geofoam in slope applications is included in Chapter 8. A detailed numerical example that demonstrates the recommended design guideline included in Appendix B and summarized in Chapter 4 is included in Appendix E.

In addition to the five primary research products listed above, an overview of construction tasks that are frequently encountered during EPS-block geofoam slope projects is included in Chapter 5. Four case histories are presented in Chapter 7 that provide examples of cost-effective and successful EPS-block geofoam slope stabilization projects completed in the U.S.

The purpose of this report is to provide those who have primary involvement with roadway embankment projects with design guidance for use of EPS-block geofoam in slope stability applications, to include design professionals, manufacturers/suppliers, contractors, regulators, and owners. End users of the research include engineers, who perform the design and develop specifications, and owners, including the FHWA, state DOTs, and local county and city transportation departments that own, operate, and maintain the roadway.

This chapter provides a summary of the primary conclusions, and specific areas where further research contributes to further acceptance and deployment of EPS-block geofoam in slope stabilization and repair.

MAJOR CONCLUSIONS

A summary of major conclusions is provided below, separated into categories of design, material and construction standard, construction practices, and economic issues.

Design

A review of current slope stability and landslide remediation textbooks (Abramson et al. 2002; Cornforth 2005; Duncan and Wright 2005; Transportation Research Board, 1996) revealed a lack of formal design guidelines to design slopes or remediate slides by reducing the weight of the slide mass using lightweight fill. Although a comprehensive design procedure was not available, a recommended design guideline was developed during this project based on general design guidance for use of geofoam in slope stability applications included in various literature (Horvath, 1995; Negussey, 2002; Tsukamoto, 1996), as well as on the NCHRP Project 24-11(01) recommended design guideline for stand-alone EPS-block geofoam embankments over soft soil.

Chapter 4 includes background information on design methodology incorporated in abbreviated form in the recommended design guideline included in Appendix B. Design of an EPS-block geofoam

slope system considers the interaction of the following three major components, which are shown in Figure 4.4: existing slope material, the fill mass, and the pavement system. The three potential failure modes that can occur due to interaction of those major components of an EPS slope system that must be considered during stability evaluation of an EPS-block geofoam slope system include: external instability of the overall EPS-block geofoam slope system configuration, internal instability of the fill mass, and pavement system failure.

Design for external stability of the overall EPS-block geofoam slope system considers failure mechanisms that involve the existing slope material only, as shown in Figure 4.5, as well as failure mechanisms that involve both the fill mass and existing slope material, as shown in Figure 4.6. The external stability failure mechanisms included in the proposed design procedure consist of static slope stability, settlement, and bearing capacity. Additional failure mechanisms associated with external seismic stability include seismic slope instability, seismic induced settlement, seismic bearing capacity failure, seismic sliding, and seismic overturning.

Design for internal stability considers failure mechanisms within the EPS-block geofoam fill mass. The three internal instability failure mechanisms evaluated in the design guideline are seismic horizontal sliding, seismic load-bearing and static load-bearing of EPS block.

The objective of pavement system design is to select the most economical arrangement and thickness of pavement materials for the subgrade provided by the underlying EPS block. The design criteria are to prevent premature failure of the pavement system, as well as to minimize the potential for differential icing (a potential safety hazard) and solar heating (which can lead to premature pavement failure) in those areas where climatic conditions make these potential problems. Also, when designing the pavement cross-section overall, consideration must be given to providing sufficient support, either by direct embedment or structural anchorage, for any road hardware (guardrails, barriers, median dividers, lighting, signage and utilities).

Figure 4.25 shows the recommended design procedure for EPS-block geofoam slope fills. All steps are required if the existing or proposed roadway is located *within* the limits of the existing or anticipated slide mass <u>and/or</u> the existing or anticipated slide mass is located *below* the roadway as shown in Figure 4.17(b) and 4.18(b). If the existing or proposed roadway is located *outside* the limits of the existing or anticipated slide mass <u>and/or</u> the existing or anticipated slide mass is located *above* the roadway as shown in Figure 4.17(a) and 4.18(a), the design procedure does not include Steps 8 and 9, which are directly related to the design of the pavement system, because the EPS-block geofoam slope system may not include a pavement system. For EPS block utilized in slope stabilization and repair that do not support a pavement system or heavy structural loads, the potential to utilize EPS blocks with recycled EPS exists. As previously mentioned, the use of recycled EPS blocks would be an attractive "green" product that reduces waste by recycling polystyrene scrap, and would also reduce the raw materials costs in the production of EPS.

The recommended design procedure is applicable for both slope-sided fills and vertical-sided fills as depicted in Figures 4.2(a) and 4.2(b), respectively, except that overturning of the entire fill mass at the interface between the bottom of the assemblage of EPS blocks and underlying foundation material as a result of horizontal forces that is part of external seismic stability, Step 6, is applicable primarily for only vertical-sided fills.

One challenge of slope stabilization design with lightweight fill is to determine the volume and location of EPS block within the slope that will yield the required level of stability or factor of safety at the least cost. Because EPS-block geofoam is typically more expensive than soil on a cost-per-unit-volume basis for the material alone, it is desirable to optimize the volume of EPS used, yet still satisfy design criteria concerning stability. Therefore, to achieve the most cost-effective design, a design goal for most projects is to use the minimum amount of EPS blocks possible that will satisfy the requirements for external and internal stability.

The determination of optimal volume and location of EPS block will typically require iterative analysis based on various locations and thicknesses until a cross section that yields the minimum volume

of lightweight fill is obtained. However, other factors will also impact the final design volume and location of EPS block, such as:

- Construction equipment access to perform excavation work,
- Ease of accessibility for EPS block delivery and placement,
- Impact on traffic if lightweight fill will be incorporated below an existing roadway, and
- Right-of-way constraints and/or constraints due to nearby structures.

It should be noted that although minimization of EPS volume is the goal on most projects, for some projects it may be desirable to maximize use of EPS. For example, economization of EPS volume may not be a concern in some emergency slope repair projects or projects with an accelerated construction schedule.

Preliminary width and location of the EPS-block geofoam fill mass within the slope will be dependent on results of evaluation of the preliminary geometric requirements of the proposed EPS-block fill mass performed as part of Step 1. The most effective location of the lightweight fill mass will be near the head (upper portion) of the existing slide mass or proposed slope, because reducing the load at the head by removing existing earth material and replacing it with a lighter fill material contributes the most to reducing destabilizing forces that tend to cause slope instability. Location of the fill mass within the slope selected in Steps 3 and 4 are only preliminary, because the location of the fill mass, as well as the thickness, may change as various iterations of the fill mass arrangement are evaluated to obtain a fill mass arrangement that will satisfy the design criteria of various failure mechanisms analyzed in each supplemental design step shown in Figure 4.25.

In some projects, volume and location of EPS blocks within the slope will be constrained by previously indicated factors. For example, for the case of the existing road located within the existing slide mass and existing slide mass located below the roadway, as shown in Figures 4.17 b) and 4.18 b), i.e., the roadway is near the head of the slide mass, location of the EPS fill mass will typically be limited within the existing roadway location because of right-of-way constraints. However, in some projects the volume and location of EPS within the slope may not be obvious, and may require that various iterations of the fill mass arrangement be evaluated to obtain a fill mass arrangement that will satisfy the design requirements of various failure mechanisms that are analyzed in each design step shown in Figure 4.25. Therefore, as part of this Project 24-11(02), a study was performed to develop a procedure for optimizing the volume and location of EPS blocks within the slope to minimize the number of iterations that may be required to satisfy the design criterion.

Appendix C presents two procedures for optimizing the volume and location of EPS block within the slope. One procedure is for slides involving rotational slip surfaces, and the other for translational slides. The purpose of the optimization methods is only to obtain an approximate location within the slope where placement of EPS block will have the greatest impact in stabilizing the slope while requiring the minimum volume of EPS blocks. A separate static slope stability analysis must be performed as part of Step 5 of the design procedure, as shown in Figure 4.25, with a better slope stability analysis method that preferably satisfies full equilibrium, such as Spencer's method. Step 5 should be relied on to verify that the overall slope configuration meets the desired factor of safety.

The design procedure is based on a self-stable adjacent upper slope to prevent earth pressures on the EPS-block fill mass that can result in horizontal sliding between blocks. If the adjacent slope material cannot be cut to a long-term stable slope angle, an earth-retention system must be used in conjunction with the ESP-block fill mass to resist the applied earth force.

Many EPS-block geofoam slope case histories evaluated as part of this Project 24-11(02) research included use of underdrain systems below the EPS block to prevent water from accumulating above the bottom of EPS block, and in some cases, incorporated a drainage system between the adjacent upper slope material and EPS block to collect and divert seepage water, and thereby alleviate seepage pressures. Thus, based on current design precedent, it is recommended that all EPS-block geofoam slope systems

incorporate drainage systems. It should be noted that in addition to a permanent drainage system, temporary dewatering and drainage systems need to be considered during construction.

In addition to technical aspects of the design, cost must also be considered. Because EPS-block geofoam is typically a more expensive material than soil on a cost-per-unit-volume basis for the material alone, it is desirable to optimize the design to minimize the volume of EPS block used, yet still satisfy the technical design aspects of various failure mechanisms. It is possible, in concept, to optimize the final design of both the pavement system and overall EPS block slope system considering both performance and cost so that a technically effective and cost-efficient geofoam slope system is obtained. However, because of the inherent interaction between the three major components of a geofoam slope system shown in Figure 4.4, overall design optimization of a slope incorporating EPS-block geofoam requires iterative analyses to achieve a technically acceptable design at the lowest overall cost. In order to minimize the iterative analysis, the design algorithm shown in Figure 4.25 was developed. The design procedure depicted in this figure considers a pavement system with the minimum required thickness, a fill mass with the minimum thickness of EPS-block geofoam, and use of an EPS block with the lowest possible density. Therefore, the design procedure will produce a cost-efficient design.

Based on a review of a case history that involved widening of an existing roadway embankment (Tsukamoto, 1996), the design procedure shown in Figure 4.25 may also be applicable to design of embankment widening sections that incorporate EPS-block geofoam. However, further analyses and sensitivity studies are required to determine the applicability of the design procedure shown in Figure 4.25 for widening of slopes.

Currently, no formal design guidelines to use any type of lightweight fill for slope stabilization by reducing the driving forces are available. Therefore, the proposed recommended design guideline that was developed herein for EPS-block geofoam can also serve as a blueprint for use of other types of lightweight fills in slope stability applications.

Research has revealed important analysis and design differences between use of EPS-block geofoam for the lightweight fill function in slope applications versus stand-alone applications over soft ground. The primary differences between slope applications versus stand-alone embankments over soft ground are summarized below:

- Site characterization is usually much more complex and difficult because it typically involves explorations made on an existing slope and concomitant access difficulties; the slope cross-section often consists of multiple soil and rock layers that vary in geometry both parallel and perpendicular to the road alignment; and piezometric conditions may be very complex and even seasonal in variation.
- The governing design issue is usually based on a ULS failure involving the analysis of shear surfaces using material strength and limit-equilibrium techniques. SLS considerations involving material compressibility and global settlement of the fill are rarely a concern.
- There is always an unbalanced earth load, often relatively significant in magnitude, acting on the EPS mass that must be addressed as part of the design process.
- Piezometric conditions are often a significant factor to be addressed in design. In fact, if the use of EPS geofoam is being considered to reconstruct a failed or failing area, piezometric issues typically contribute to the cause of the failure in the first place.
- The volume of EPS placed within the overall slope cross-section may be relatively limited. Furthermore, optimal location of the EPS mass within the overall slope cross-section is not intuitively obvious.
- The road pavement may not overlie the portion of the slope where the EPS is placed. Therefore load conditions on EPS block may be such that blocks of relatively low density can be used, which can achieve significant economies in the overall design.

Material and Construction Standard

A recommended standard for use of EPS-block geofoam for lightweight fill in slope stabilization is included in Appendix F. The objective during this Project 24-11(02) study was to modify the Project 24-11(01) standard that is applicable to stand-alone embankments over soft ground to make it specific to geofoam usage in slope stability applications. The Project 24-11(02) standard included in Appendix F contains six key revisions from the Project 24-11(01) standard. First, we included for the first time a commentary section. Second, we eliminated use of different minimum allowable density values for individual MQC/MQA test specimens versus a higher nominal or average density of the block as a whole, so that both the block as a whole and any test specimen from within that block meet the same criteria. Third, we increased minimum allowable values for compressive strength to reflect the increase in these values included in D 6817. Fourth, we increased requirements for flexural strength to be consistent with the change in unifying block and test-specimen densities. Fifth, we changed the wording related to the small-strain modulus from "Initial Tangent Young's Modulus" to "Initial Secant Young's Modulus" simply to correct semantics. Sixth, we added two new, additional types, designated *EPS130* and *EPS160*.

The primary issue related to the recommended material and construction standard included in Project 24-11(01) reports, and in the recommended standard for slope applications included in Appendix F that was evident from replies to the project questionnaire included in Appendix A is the current confusion between the recommended NCHRP standard and ASTM D 6817 material properties. However, based on the consideration of knowledge acquired over the approximately 60 years that EPS has existed as a construction material, and the decade of actual project use and experience using the standard for stand-alone embankments included in Project 24-11(01) reports, standards developed for the past and current NCHRP studies are reasonable when implemented properly in practice.

Proper implementation includes MQC/MQA laboratory testing performed in accordance with well-established ASTM protocols for test-specimen conditioning prior to testing, numerical correction of all stress-strain curves for machine compression, and graphical correction of stress-strains for initial concavity as necessary. Conversely, the original version of D 6817 was based on relatively limited test data performed in one closed, private study. Those data and their interpretation as incorporated into D 6817 is at odds with not only decades of worldwide experience, but even prior testing and protocols by Industry itself. Therefore, the Industry position that NCHRP Project 24-11(01)/(02) standards are unreasonable in their material requirements is not justified. It is recommended that Industry provide additional specific technical details about their circa 2000 round-robin testing program as outlined in Chapter 6 so that conflicts between the NCHRP and ASTM standards can be resolved.

We concur with a recent article that appeared in Geo-Strata magazine which indicates although alignment of the two standards is preferred, the immediate need consists of better educating stakeholders on the basis, benefits, and limitations of both standards for structural and non-structural applications (Nichols, 2008).

Construction Practices

An overview of construction tasks frequently encountered during EPS-block geofoam slope projects is included in Chapter 5. Construction topics include site preparation; drainage; EPS block shipment, handling, and storage; CQA/CQC of EPS blocks; block placement; backfill placement between EPS block and adjacent earth slopes; phased construction; accommodation of utilities and road hardware; facing wall; earth retention system; pavement construction; and post-construction monitoring. Chapter 5 is a supplement to the material and construction standard included in Appendix F and the construction practices discussion included in the Project 24-11(01) report (Stark et al., 2004a).

Figures and photographs that may aid in preparation of bid and construction documents are included in Chapter 5. Additionally, Appendix G includes various design details; Appendix H includes example specifications utilized in geofoam projects. Construction details included in Appendix G, which were obtained from actual geofoam construction drawings used in projects throughout the U.S., can be

used as a guide for developing site-specific drawings or details. The details presented relate to a variety of geofoam issues, such as configuration of EPS blocks, inclusion of utilities and roadway hardware, construction of a load distribution slabs over the EPS, and construction of facing walls.

In addition to ensuring that the correct EPS block type is placed, it is also important to ensure that methods used by the contractor to construct the overall EPS-block geofoam slope produce an acceptable slope system that complies with assumptions inherent in the recommended design procedure. For example, the design procedure assumes that the adjacent slope is self-stable to prevent earth loads from developing on the EPS-block fill mass, and that an adequate drainage system is provided to prevent hydrostatic and seepage forces from developing within the EPS-block fill mass. Therefore, it is necessary to monitor the construction process to ensure that indeed the adjacent slope is stable and the drainage system is constructed properly.

In addition to a permanent drainage system, a temporary dewatering and drainage system may be required during construction to prevent flotation of EPS block due to water collecting in and around the area where EPS blocks are being placed. Additionally, adequate overburden such as use of “soft” weights should be applied to the top of blocks to prevent blocks from being picked up or displaced by high winds.

One issue raised as part of a slide correction project involved the payment quantity of EPS block versus backfill material at the interface between the EPS block and adjacent cut slope. To alleviate this potential pay quantity discrepancy, it is recommended that the drawings specifically show the limits of EPS block placement along the EPS block and adjacent earth slope.

When necessary, an EPS-block geofoam fill can be constructed in phases, allowing one portion of the fill to be completed before beginning construction on the next portion. The advantage of this approach is that it can eliminate the need to completely close down an existing roadway in order to repair the unstable portion of a slope.

Economic Issues

A review of existing available EPS-block geofoam cost data indicates that EPS-block geofoam prices vary widely, and prices of EPS blocks have substantially increased recently due to the substantial increase in the price of oil. Therefore, a draft price adjustment contract special provision, similar to the special provisions that DOTs have utilized for other construction materials such as bituminous asphalt binder, was developed as part of this project and is included in Appendix I. The purpose of the adjustment contract special provision is to minimize the impact of short-term oil price fluctuations on the cost of EPS-block geofoam during multi-phased projects.

In an effort to assist designers with designing a cost-efficient EPS-block geofoam slope, the recommended design procedure for use of EPS block in slopes considers a pavement system with the minimum required thickness, a fill mass with the minimum thickness of EPS-block geofoam, and use of an EPS block with the lowest possible density. Therefore, the design procedure will produce a technically cost-efficient design. However, in addition to the cost of EPS blocks, overall intangible benefits that the use of EPS-block geofoam can contribute should also be considered as part of the slope stabilization decision-making process. These intangible benefits are listed in the introduction of this chapter. A more in-depth discussion of these benefits, as well as other issues related to the costs associated with EPS-block geofoam construction, is provided in Chapter 8 and in the Project 24-11(01) report (Stark et al., 2004a).

When attempting to evaluate the feasibility of using EPS-block geofoam for a slope stabilization project, it is important to consider the unique characteristics of EPS-block geofoam as a construction material. For example, experience has demonstrated that EPS-block geofoam can be placed quickly. Once the site is prepared, the actual process of moving and positioning EPS blocks requires minimal equipment and labor. EPS-block geofoam blocks are transported and placed easily, even at many project sites inaccessible to heavy equipment. Although some specific safety measures may have to be implemented, placement of EPS block can be continued in almost any kind of weather, whereas many other slope stabilization methods may be delayed by rain or snow. Use of EPS-block geofoam may also facilitate

phased construction and may minimize disruption to traffic by eliminating the need to close down an existing roadway in order to repair the unstable portion of a slope or widen an existing embankment.

Another important consideration is the fact that EPS-block geofoam is a manufactured construction material that can be produced by the molder and then stockpiled at a designated site until needed. Therefore, a state DOT agency could potentially store a supply of EPS blocks that could be used for emergency landslide mitigation or repair. Also, EPS blocks can be molded in advance of the actual placement date and either transported immediately when needed, or stockpiled at the site for immediate use. Thus, use of EPS block in slope application projects can easily contribute to an accelerated construction schedule.

When performing an analysis to compare EPS-block geofoam to other potential slope stabilization alternatives, both tangible and intangible benefits of utilizing EPS-block geofoam should be considered when evaluating it as a potential alternative for a slope construction project. The benefit of accelerated construction that use of EPS-block geofoam can provide has been a key contribution to the decision to use EPS-block geofoam in projects such as the I-15 reconstruction project in Salt Lake City, UT; the CA/T Project in Boston, MA; and the I-95/Route 1 Interchange (Woodrow Wilson Bridge Replacement) in Alexandria, VA (Nichols, 2008). Therefore, the benefit of accelerated construction that use of EPS-block geofoam can provide should be evaluated, since it has been a key factor in the decision to use EPS-block geofoam in recent projects in the U.S.

The wide variance in price of EPS-block geofoam is perhaps one of the greatest hindrances to further adoption of EPS-block geofoam in the U.S. This wide variance in price may be attributed to the number of potential factors that can impact the cost of EPS-block geofoam. These potential factors are summarized in Chapter 8 and includes factors related to manufacturing, design, and construction.

SUGGESTED FUTURE RESEARCH

Overview

Research did identify issues where further research would enhance the current state of knowledge of EPS-block geofoam in slope stabilization and repair applications. These issues are categorized into design issues, material properties and construction issues, and general issues.

Design Issues

The two primary design issues where further research would enhance design procedure include development of a rational method that considers strength of EPS blocks in external static stability analysis, and full-scale evaluation of the interaction of the EPS-block fill mass with adjacent slope material during a seismic event. These two primary research issues are summarized below.

Rational Method that Considers Strength of EPS Blocks in External Static Stability Analysis.

Although limit equilibrium methods have been used to evaluate the external static stability of EPS-block geofoam slope fills, the literature search revealed some uncertainties in modeling of shear strength of EPS blocks in slope stability analysis. For example, the literature search performed for this study revealed that five alternatives are available to model the shear strength of EPS blocks in limit equilibrium analysis as summarized below:

1. Applying a surcharge to the surface of the foundation material that approximates the dead weight of EPS blocks and any loads on top of EPS blocks, such as those due to weight of the pavement system, so the shear strength of EPS blocks does not have to be considered.

2. Modeling EPS blocks with a friction angle of one degree and a cohesion of zero so EPS blocks do not contribute significantly to the factor of safety, because of uncertainties in estimating how much shear resistance EPS blocks actually contribute in the field.
3. Assuming failure occurs between EPS blocks, i.e., along EPS/EPS interfaces, and using an appropriate interface friction angle.
4. Assuming that failure occurs through the individual EPS blocks and using a cohesion value to represent the internal shear strength of a geofoam block.
5. Assuming that failure occurs through individual blocks as well as between EPS blocks, and using an appropriate cohesion and interface friction angle.

With the exception of the first alternative that models the EPS-block geofoam fill mass as a surcharge, the primary difference between various scenarios involves the assumption about the sliding mechanism through the EPS-block fill mass.

Much of the uncertainty surrounding modeling of EPS-block geofoam shear strength using limit equilibrium methods of analysis stems from the unusual material properties of EPS compared to earth materials. For example, the Mohr-Coulomb shear strength model is typically used to model shear strength of materials in limit equilibrium methods of slope stability analysis. However, an EPS block is a solid material, with a stress-strain behavior that is strain-hardening as well as time-dependent based on relative stress levels within the material. This strain-hardening behavior cannot be modeled with traditional limit equilibrium methods of slope stability analysis that typically model earth materials using the Mohr-Coulomb shear strength model. Therefore, a strain-hardening material model for use in slope stability analysis is needed.

The issue of stress-strain incompatibility between EPS-block fill mass and underlying and adjacent natural material that can lead to progressive failure requires study. The main progressive failure issue is determination of shear strength of the geofoam and adjacent and underlying natural slope material that can be relied on in a typical limit equilibrium method of analysis, because the stress-strain behavior between EPS block and natural slope material may not be compatible. Use of EPS-block geofoam for slope stabilization has involved both soil and rock slope materials. Therefore, the progressive failure study should consider both soil and rock slope materials.

An EPS-block fill mass consists of discrete blocks. Overall interaction of these discrete blocks cannot be modeled with traditional limit equilibrium methods of slope stability analysis. Therefore, an analytical issue that requires further research using numerical modeling, physical testing, and/or observation of full-scale structures includes determining whether an external slope stability failure induces failure through individual EPS blocks, or whether blocks remain intact and displace as individual elements as a result of slope instability.

An appropriate shear strength model may also be required to analyze stability of embankments that may be widened with EPS block because use of EPS block for lightweight fill in embankment widening cases is similar to incorporating EPS block in slope applications. Thus, an accurate model for expressing shear strength of EPS blocks is needed to ensure a safe and economical design, regardless of the condition of the surrounding natural materials.

Full-Scale Evaluation of the Interaction of the EPS-Block Fill Mass with Adjacent Slope Material during a Seismic Event.

Existing seismic design procedures for use of EPS-block geofoam are based on work performed predominantly on stand-alone embankments over soft soils, and does not consider the influence of adjacent slope material. The recommended procedure for considering the effect of natural slope material on external and internal seismic stability of the EPS-block geofoam slope system proposed in the design guideline in Appendix B consists of determining the magnitude of seismic earth pressure based on the Mononobe-Okabe (M-O) method. However, results from full-scale tests are needed to validate the M-O

method for considering the impact of seismic inertia forces from the adjacent slope material on the EPS-block geofoam fill mass slope system.

Material Properties and Construction Issues

Issues related to geofoam material properties and the recommended material and construction standard where further research would enhance the current state of knowledge of EPS-block geofoam in slope stabilization and repair applications are summarized below:

- **Education:** The primary issue related to the recommended material and construction standard included in Project 24-11(01) reports and recommended standard for slope applications included in Appendix F that was evident from replies to the project questionnaire included in Appendix A is the current confusion between the NCHRP recommended standard and ASTM D 6817 standard. A recent article that appeared in Geo-Strata magazine indicates that although alignment of the two standards is preferred, the immediate need consists of better educating stakeholders on the basis, benefits, and limitations of both standards for structural and non-structural applications (Nichols, 2008).
- **Seasoning time:** Re-evaluating minimum time required for seasoning EPS blocks to outgas the blowing agent to an acceptable level.
- **Comparison of stress-strain behavior of full-size EPS blocks versus small test specimens:** Comparison of stress-strain behavior of full-size EPS blocks versus small test specimens routinely used in practice for engineering property and quality control/assurance testing. This study can include development of reliable correlations between Young's modulus as measured in small laboratory test specimens and behavior of full-size EPS blocks in situ.
- **Small-strain creep model:** Development of an accurate small-strain creep model so creep strains can be reliably estimated for lightweight fills. This should include correlations between laboratory and in-situ creep data.
- **Non-invasive testing device:** Development of a non-invasive testing device such as a sonic-wave device for routine on-site evaluation of the average density, initial tangent Young's modulus of an EPS block, and, if possible, average elastic-limit stress.
- **CQA manual:** Development of a standardized construction quality assurance (CQA) procedure and manual for EPS block to provide greater guidance to end users.

General Issues

General issues where further research would enhance the current state of knowledge of EPS-block geofoam in slope stabilization and repair applications are summarized below:

- **Mechanical connectors, shear keys, and adhesives:** The primary function of current use of mechanical connectors appears to be to keep EPS blocks in place when subjected to wet, icy, or windy working conditions during construction (Horvath, 2001), and to prevent shifting under traffic loads when only a few layers of blocks are used (Duskov, 1994). A second and more significant function of mechanical connectors is to supply additional resistance between EPS blocks to support lateral loads. However, a rational methodology for determining the number, spacing, and placement location of mechanical connectors to support lateral loads is needed. It is anticipated that a full scale load test will be required to develop the lateral resistance design procedure. Effectiveness of barbed metal plate mechanical connectors, especially under reverse loading conditions has been recently disputed (Bartlett et al., 2000; Sanders and Seedhouse, 1994; Sheeley, 2000). Therefore, the effectiveness of barbed metal plate mechanical connectors during seismic loading conditions requires further investigation. An alternative to mechanical connectors

is the use of shear keys (Bartlett and Lawton, 2008). Shear keys consist of half-height EPS blocks that are periodically installed within the fill mass to interrupt the horizontal joints that are typically present in EPS fills. However, a method of analyzing the number and location of shear keys is needed. The long-term durability and feasibility of adhesives should also be evaluated.

- **Failure case histories:** During this study, information was obtained from several diverse and unconnected sources that indicates there have been a surprising number of failures, primarily of a serviceability nature, of EPS-block geofoam 'earthworks' for roads since the start of the new millennium. In at least two cases, vertical displacement of the road surface due to unsatisfactory performance of EPS block was apparently so severe that the owning agency had to remove the EPS block and replace them with alternative materials. One of these failures was referenced by the New York State DOT as part of their response to Question A5ii of the project questionnaire included in Appendix A. While these failures, which number approximately ten known to date, may not be an epidemic, they are nonetheless extremely troubling, given the historically problem-free performance of EPS-block geofoam roadway earthworks in earlier years. A general overview on the emerging trends in failures involving EPS-block geofoam fills is provided by Horvath (2010).

  Although most of these known failures do not involve slopes per se, these case histories are extremely relevant nonetheless for at least two reasons. Broad issues of design, materials, and construction that alone or in combination may have resulted in the failures have direct relevance to the current research that focused on slopes because of the broad similarities in overall design of embankments and slopes. Secondly, regardless of the specific type of transportation earthwork incorporating EPS-block geofoam that suffers a failure, mere occurrence of failures with any type of application involving EPS-block geofoam only serves to destroy the confidence of owners and design professionals to use or continue using this geotechnology.

  Therefore, it is imperative for the overall widespread use of EPS-block geofoam geotechnology in general, and the success of current research in particular, to learn to the maximum extent practicable from these failures and to address them by incorporating lessons learned into future practice.
- **Wind loads:** Although wind loading is considered in the design of stand-alone embankments incorporating EPS blocks, wind loading does not appear to be a potential failure mechanism for EPS-block geofoam slopes in slope applications because EPS blocks will typically be horizontally confined by existing slope material on one side of the slope. Therefore, the proposed design procedure does not include wind loading as a potential failure mechanism. However, it is recommended that additional research be performed of available wind pressure results on structures located on the sides of slopes to further evaluate the need to consider wind as a potential failure mechanism.
- **Applicability of the design procedure for slope stabilization and repair to roadway embankment widening:** Based on a review of a case history that involved widening of an existing roadway embankment presented by Tsukamoto (1996), the design procedure for use of EPS-block geofoam in slope stabilization and repair may also be applicable to the design of embankment widening sections that incorporate EPS-block geofoam. However, further study is required to determine the applicability of the design procedure for slope stabilization and repair to the design of roadway embankment widened sections that incorporate EPS block.
- **LRFD:** The current state-of-practice of slope stability analysis is based on SLD because of problems associated with use of LRFD in slope stability analysis. Therefore, the recommended design guideline included in this report is based on the SLD approach. When inconsistencies with applying the LRFD methodology to slope stability analysis are resolved, an LRFD based design procedure for EPS-block geofoam slopes will be required.

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Appendix A

Geofoam Usage Survey/Questionnaire with Replies

A.1 INTRODUCTION

A geofoam usage survey was conducted via a questionnaire developed by the project team to obtain case history information, cost data, design details and other geofoam related information.

The questionnaire was posted on-line on March 9, 2007 at
http://www.ce.memphis.edu/arellano/geofoam_usage_survey.htm.
The official deadline date for responses was May 25, 2007.

Notification of the questionnaire was sent via e-mail to the designated TRB representative in each state, as well as the Commonwealth of Puerto Rico. Notification of the questionnaire was also sent to the EPS Molders Association (EPSMA) in Crofton, Maryland, via e-mail.

A copy of the questionnaire is included in this Appendix. A summary of the survey replies follows the copy of the questionnaire.

A.2 COPY OF SURVEY

CHAPTER 1 EPS-BLOCK GEOFOAM USAGE QUESTIONNAIRE

National Cooperative Highway Research Program (NCHRP)

Contract No. HR 24-11(02)

"Guidelines for Geofoam Applications in Slope Stability Projects"

Questionnaire Background (optional)

Agency or Company Information:

First Name

Last Name *

Title *

Organization *

Street Address *

Address (cont.)

Address (cont.)

City *

State/Province *

Zip/Postal Code *

Country

Work Phone *

FAX

E-mail *

* Required Field

Note: Only replies that are submitted with the required fields above will appear in project reports

Part A: Expanded Polystyrene (EPS)-Block Geofoam Usage

The purpose of this part of the survey is to help us identify general opinions of EPS-block geofoam technology from the viewpoint of those in practice.

A1. Which category best describes the agency or company you represent?

- ○ government agency
- ○ design engineer

- ☐ expanded polystyrene manufacturer/supplier
- ☐ EPS block molder
- ☐ EPS-geofoam distributor
- ☐ construction contractor/builder
- ☐ Other (Explain briefly below)

A2. Have you **considered** or **utilized** the NCHRP Project 24-11 recommended design guideline or recommended material and construction standard (check all that apply)?

- ☐ Yes, considered design guideline
- ☐ Yes, utilized design guideline
- ☐ Yes, considered construction standard
- ☐ Yes, utilized construction standard
- ☐ I **am not** familiar with the Project 24-11 recommended design guideline or material and construction standard.
- ☐ I **am** familiar with the Project 24-11 reports but have not had the opportunity to utilize the recommended design guideline or construction standard.

☐ I **am** familiar with the Project 24-11 reports but have chosen not to utilize the recommended design guideline or construction standard.

If you checked the last box, please briefly explain your reason(s) for not using these standards.

A3. Has the agency or company that you represent ever specified, supplied or installed EPS-block geofoam in a lightweight fill for any type of road?

○ No **(Please proceed to Question A4)**

○ Yes **(Please proceed to Question A5)**

A4. If the answer to Question A3 was "No", what is the primary reason why not?

If the answer to Question A3 was "Yes", please proceed to Question A5i. If the answer to Question A3 was "No", please proceed to Part B.

A5i. To help us understand what is the primary benefit of using EPS-block geofoam so that we can develop this aspect to the fullest, what is the primary positive reason for using this material in road applications?

A5ii. To help us understand what aspect(s) of EPS-block geofoam most need improvement, what is the primary negative aspect or issue that you can state about this material in road applications? (Note: Exclude cost of the EPS blocks.)

A5iii. For which of the following application(s) have you designed, specified, supplied, or installed EPS-block geofoam for the function of lightweight fill in road construction (check all that apply)?

- ☐ Embankment over soft ground
- ☐ Bridge approach over soft ground
- ☐ Design of planned soil slopes involving new roadway construction
- ☐ Remedial repair or remediation of existing unstable soil slopes involving existing roadways

☐ Other (Explain briefly below.)

A5iv. Concerning the EPS-block geofoam specification used on your most recent project**,** what was the source (check all that apply)?

☐ NCHRP 24-11 Recommended EPS-Block Geofoam Standard for Lightweight Fill in Road Embankments and Bridge Approach Fills on Soft Ground

☐ State or County Department of Transportation Standard/Provisional Specification

☐ Other in-house-developed specification

☐ Provided by EPS molder or distributor

☐ ASTM D6817-02 Standard Specification for Rigid Cellular Polystyrene Geofoam

A5v. What one item in the specification you used would you recommend be revised?

Part B: Design of Planned Soil Slopes Involving New Roadway Construction

The purpose of this part is to provide us with information that is useful to some of the specific goals of this project related to design of planned soil slopes involving new roadway construction. For the purpose of this questionnaire, planned soil slopes are existing slopes that are in a stable state prior to construction and new engineered slopes to include cut slopes.

B1. Who is primarily involved with design associated with planned soil slopes involving new roadway construction?

- ○ Government-agency personnel
- ○ Private consultants

B2. Who is primarily involved with construction associated with planned soil slopes involving new roadway construction?

- ○ Government-agency personnel
- ○ Private contractors

B3. Which analysis methods* do you utilize to evaluate planned soil slopes involving new roadway construction projects

(check all that apply)?

*The methods below were obtained from the book titled Soil Strength and Slope Stability by J. Michael Duncan and Stephen G. Wright.

- ☐ Simple methods of analysis that involve a single equation to compute the factor of safety
- ☐ Slope stability charts
- ☐ Spreadsheet software
- ☐ Commercial slope stability computer programs Which one(s)?
- ☐ Other (Describe briefly.)

B4. Have you utilized other types of lightweight fill materials in slope stability projects associated with planned soil slopes involving new roadway construction?

- ○ No
- ○ Yes (Which lightweight fill types?)

B5. Are you currently utilizing load resistance factor design (LRFD) procedures in design involving soil slopes associated with new roadway construction?

- ○ No
- ○ Yes

B6. Do you perform seismic slope stability analysis in slope designs involving planned soil slopes associated with new roadway construction?

- ○ No, seismic loading is not a design consideration in our area.
- ○ No, seismic loading is a design consideration in our geographic area, but we do not perform seismic slope stability analysis.
- ○ Yes

Part C: Remedial Repair or Remediation of Existing Unstable Soil Slopes Involving Existing Roadways

The purpose of this part is to provide us with information that is useful to some of the specific goals of this project related to remedial repair or remediation of existing unstable soil slopes involving existing roadways.

C1. Who is primarily involved with design associated with remedial repair or remediation of existing unstable soil slopes involving existing roadways?

- ○ Government-agency personnel
- ○ Private consultants

C2. Who is primarily involved with construction associated with remedial repair or remediation of existing unstable soil slopes involving existing roadways?

- ○ Government-agency personnel
- ○ Private contractors

C3. Which analysis methods* do you utilize to evaluate remedial repair or remediation procedures involving unstable soil slopes in existing roadway construction projects (check all that apply)?

*The methods below were obtained from the book titled Soil Strength and Slope Stability by J. Michael Duncan and Stephen G. Wright.

- ☐ Simple methods of analysis that involve a single equation to compute the factor of safety
- ☐ Slope stability charts
- ☐ Spreadsheet software
- ☐ Commercial slope stability computer programs Which one(s)?

- ☐ Other (Describe briefly)

C4. Have you utilized other types of lightweight fill materials in slope stability projects associated with remedial repair or remediation of existing unstable soil slopes involving existing roadways?

- ☐ No
- ☐ Yes (Which lightweight fill types?)

C5. Are you currently utilizing load resistance factor design (LRFD) procedures in slope designs involving remedial repair or remediation of existing soil slopes associated with existing roadways?

- ☐ No
- ☐ Yes

C6. Do you perform seismic slope stability analysis in slope designs involving remedial repair or remediation of existing unstable soil slopes associated with existing roadways?

- ☐ No, seismic loading not a design consideration in our geographic area.

☐ No, seismic loading is a design consideration in our geographic area, but we do not perform seismic slope stability analysis.

☐ Yes

Part D: General

D1. Which slope stability procedures* have you utilized for design of planned soil slopes associated with new roadway construction and/or for remedial repair or remediation of existing unstable soil slopes associated with existing roadways (check all that apply)?

*The procedures outlined below were obtained from TRB Special Report 247, Landslides: Investigation and Mitigation.

Category	Procedure	Design of Planned Slopes	Remedial Repair or Remediation
Avoid Problem	Relocate facility	☐	☐
	Completely or partially remove unstable materials	☐	☐
	Install bridge	☐	☐
Reduce driving forces	Change line or grade	☐	☐
	Drain surface	☐	☐
	Drain subsurface	☐	☐
	Reduce weight	☐	☐

Increase resistance forces by			
Applying external force	Use buttress and counterweight fills; toe berms	☐	☐
	Use structural systems	☐	☐
	Install Anchors	☐	☐
Increasing internal strength	Drain subsurface	☐	☐
	Use reinforced backfill	☐	☐
	Install in-situ reinforcement	☐	☐
	Use biotechnical stabilization	☐	☐
	Treat chemically	☐	☐
	Use electro osmosis	☐	☐
	Treat thermally	☐	☐

D2. What other slope stability procedures have you utilized for

i. design of planned soil slopes associated with new road construction?

ii. remedial repair or remediation of existing unstable soil slopes associated with existing roads?

D3. Overall, what one item would you like us to consider or include in the NCHRP 24-11(02) project documents that would be of greatest use to you in designing, supplying or installing EPS-block geofoam for road construction?

D4. Would you prefer that that the NCHRP 24-11(02) results for slope applications be integrated with the Project 24-11 reports for stand-alone embankments over soft ground or that the two project reports be separate?

- ○ Integrate the two
- ○ Keep separate

D5. Do you have any of the following supporting documentation that you think may be helpful to us in achieving the research objectives and that you would be willing to provide (check all that apply-we will contact you for follow up)?

	EPS-block geofoam in design of planned soil slopes involving new road construction	**EPS-block geofoam associated with remedial repair or remediation of existing unstable soil slopes involving existing roads**	**EPS-block geofoam in stand-alone embankments on soft ground**	**Bridge Approaches**	**Other types of light-weight fills**
Plans or design details	☐	☐	☐	☐	☐
Specifications	☐	☐	☐	☐	☐
Design Reports	☐	☐	☐	☐	☐
Cost estimates and comparisons	☐	☐	☐	☐	☐
Field instrumentation/ performance data	☐	☐	☐	☐	☐
Photographs	☐	☐	☐	☐	☐
Other case history information	☐	☐	☐	☐	☐

Please feel free to provide any additional comments below.

Thank you for participating. Your input is invaluable for the success of this project.

Please press submit button to complete survey.

Submit Form | Reset Form

For comments or questions regarding this web site, please contact David Arellano at darellan@memphis.edu.

Revised:

A.3 SUMMARY OF RESULTS

The questionnaire is divided into four parts: Parts A through D. The purpose of Part A was to help us identify general opinions of EPS-block geofoam technology from the viewpoint of those in practice.

PART A: EXPANDED POLYSTYREN (EPS)-BLOCK GEOFOAM USGE

Question A1: Which category best describes the agency or company you represent?
[**17**] government agency
[**3**] design engineer
[**3**] expanded polystyrene manufacturer/supplier
[**6**] EPS block molder
[**0**] EPS-geofoam distributor
[**0**] construction contractor/builder
[**0**] Other (Explain)

Twenty-nine questionnaire responses were received. Seventeen responses were received from state DOTs, six replies were obtained from EPS molders, three replies were received from design engineers, and three from manufacturers/suppliers of expandable polystyrene. Table A.1 provides a summary of state DOTs, (or "state agency") and others that responded to the questionnaire.

Table A.1. Summary of respondents to the Questionnaire

DOT	EPS BLOCK MOLDER	DESIGN ENGINEER	EXPANDABLE POLYSTYRENE MFG/SUPPLIER
AK	AR	IL	KS
CO	CT	MN	PA
CT	GA	WA	UT
IA	MI		
ID	MN		
IL	SD		
GA			
MD			
ME			
MI			
MN			
MT			
NE			
NY			
VA			
VT			
WV			

<u>Question A2</u>: Have you **considered** or **utilized** the NCHRP Project 24-11 recommended design guideline or recommended material and construction standard? (check all that apply)

[**9**] Yes, considered design guideline

AR (molder)	IL (DOT)
WA (design engineer)	ME (DOT)
CT (DOT)	MI (DOT)
CO (DOT)	MT (DOT)
ID (DOT)	

[**6**] Yes, utilized design guideline

IL (design engineer)	ME (DOT)
IL (DOT)	VT (DOT)
MN (design engineer)	WA (design engineer)

[**7**] Yes, considered construction standard

WA (design engineer)	
CT (DOT)	MI (DOT
ID (DOT)	VT (DOT)
ME (DOT)	WV (DOT)

[**3**] Yes, utilized construction standard

ME (DOT)	VT (DOT)
IL (DOT)	

[**11**] I **am not** familiar with the Project 24-11 recommended design guideline or material and construction standard.

GA (molder)	PA (mfg/supplier)	MD (DOT)
MI (molder)	UT (mfg/supplier)	MN (DOT)
MN (molder)	GA (DOT)	NE (DOT)
KS (mfg/supplier)	IA (DOT)	

[**5**] I **am** familiar with the Project 24-11 reports but have not had the opportunity to utilize the recommended design guideline or construction standard.

AR (molder)	AK (DOT)
CT (molder)	NY (DOT)
SD (molder)	

[**0**] I **am** familiar with the Project 24-11 reports but have chosen not to utilize the recommended design guideline or construction standard. If you checked the last box, please briefly explain your reason(s) for not using these standards.

<u>Question A3:</u> Has the agency or company that you represent ever specified, supplied or installed EPS-block geofoam in a lightweight fill for any type of road?

[**8**] No **(Please proceed to <u>Question A4</u>)**

CT (molder)	MN (DOT)
AK (DOT)	MT (DOT)
CT (DOT)	NE (DOT)
ID (DOT)	WV (DOT)

[**21**] Yes **(Please proceed to <u>Question A5</u>)**

AR (molder)	UT (mfg/supplier)	MD (DOT)
GA (molder)	IL (design engineer)	ME (DOT)
MI (molder)	MN (design engineer)	MI (DOT)
MN (molder)	WA (design engineer)	NY (DOT)
SD (molder)	CO (DOT)	VA (DOT)
KS (mfg/supplier)	GA (DOT)	VT (DOT)
PA (mfg/supplier)	IA (DOT)	IL (DOT)

<u>Question A4:</u> If the answer to Question A3 was "No," what is the primary reason why not?

CT (EPS block molder) we quoted an embankment stabilization project in MA, but lost the bid.
AK (DOT) Familiarity, cost.
CT (DOT) Have not had a project where it would be applicable in the last 5 to 10 years. We do, however, intend to use geofoam in a project currently in the preliminary design phase.
ID (DOT) Cost.
MN (DOT) Previous County Engineers were not familiar with the product. I have used it in several road embankment applications while employed with St. Louis County, Minnesota.
MT (DOT) Cost.
NE (DOT) No response.
WV (DOT) Lack of agency experience with product.

If the answer to Question A3 was "Yes", please proceed to Question A5i. **If the answer to Question A3 was "No", please proceed to** Part B.

<u>Question A5i:</u> To help us understand what is the primary benefit of using EPS-block geofoam so that we can develop this aspect to the fullest, <u>what is the primary positive reason for using this material</u> in road applications?

AR (EPS block molder) Benefits of EPS Geofoam include: Load supporting, non-biodegrading, non-water absorbing, easy transport and installation. Availability, quick delivery of product that can be cut in the field. Proven over a 20 year period with FHWA designation as "material of choice" for soil stabilization projects.
GA (EPS block molder) Low density, for stress and deformation-related construction problems. Thermal insulation for frost-heave problems.
MI (EPS block molder) creating a base capable of supporting the road and traffic above.
MN (EPS block molder) Lightweight fill, expedite construction timeline.
SD (EPS block molder) Quick install, Easy to handle, Extreme lightweight.
KS (Mfg/supplier) Speed of construction as no preloading required. Ability to quickly place blocks. Ability to keep existing utilities in place as well as run new utilities more easily. Existing poor soil quality, lack of access to traditional fill, weight of Geofoam is about 1/80th of traditional fill. Zero slope capacity of Geofoam.
PA (Mfg/supplier) It is an economical substitute to using natural fill/aggregate at locations where they are in scarcity.
UT (Mfg/supplier) The primary reason for using Geofoam in road applications is weight reduction on utilities, structures (such as culverts) and soft unstable sub-soils. Using Geofoam helps solves problems such as stability, settlement and time restraints.
IL (Design engineer) Reduces lateral loading on vertical wall components, reduces vertical loadings on soft subgrades (i.e., reduces settlements and increases bearing capacity), and reduces vertical loadings adjacent to slopes (i.e., *reduces driving forces* in slope stability calculations).

MN (Design engineer) Very lightweight with relatively high strength.
WA (Design engineer) Reduced weight for fills on soft or compressible foundation soils. Reduced driving loads for fills (not necessarily roads) constructed on steep slopes.
CO (DOT) Benefit of a light-weight fill.
GA (DOT) product was specified to provide reduced loading over an existing culvert on a roadway widening project
IA (DOT) To reduce loads on old RCB culvert that we did not want to replace (one instance).
MD (DOT) The major benefit is weight of material and speedy construction benefit. The use of light-weight material will allow rapid construction and minimize long term maintenance program.
ME (DOT) Settlement mitigation.
MI (DOT) Lighten up overburden pressure of added fill.
MN (DOT) I used it primarily as a light-weight fill over deep organic deposits in road embankment construction. Another application was as insulation to reduce the effects of frost heave.
NY (DOT) The extremely light weight of EPS solves many geotechnical problems that would otherwise be very difficult or expensive to solve.
VA (DOT) Light weight
VT (DOT) We used EPS Geofoam in an abutment backfill situation over soft soils to expedite construction, eliminate the need for surcharging and the possible installation of wick drains.

Question A5ii: To help us understand what aspect(s) of EPS-block geofoam most need improvement, what is the primary negative aspect or issue that you can state about this material in road applications? (Note: Exclude cost of the EPS blocks.)

AR (EPS block molder) Education and product knowledge are the key factors that need improvement in the industry. EPS material supplied needs uniform quality and consistency. Standards and quality need to be the focus point between vendors and users before purchasing. Some effort being made to develop a national standard for geofoam materials which would eliminate the variability.
GA (EPS block molder) Incorrect installation.
MI (EPS block molder) the problem of overcoming the buoyancy properties of geofoam in the instance of elevated water conditions.
SD (EPS block molder) Pre-staging needs to be addressed with the owner/contractor. Difficulty in transporting the blocks to the jobsite. Design/Engineering problems (limited engineers with geofoam knowledge). No marketing plan as an industry.
KS (Mfg/supplier) High winds at jobsite can be a challenge, protection of material prior to placement, flammable, buoyancy. However, easy solved with proper design/specification.
PA (Mfg/supplier) To ensure that QA/QC program can be implemented in the field so the quality of each block molded meet or comply with its specification.
IL (Design engineer) Flotation under ponded water conditions and limited resistance to traffic loadings.
MN (Design engineer) Must be careful with differential icing effects.
WA (Design engineer) There appears to be an under-appreciation for the effect of sample size on interpreted properties. This likely results in EPS being found to be unsuitable for some applications because the tests on small dimension specimens suggest too great compression under the loads. Testing of larger dimension specimens or correlations to allow adjustment of test results for tests on small dimension specimens would be helpful.
CO (DOT) There is a lack of product understanding among some project and senior maintenance personnel. Also, per most recent experience involves use of EPS for landslide stabilization. The confidence in the slope stability analysis of a slope with EPS is limited due the assumptions that are made and the lack of long term performance data. An additional concern related to

long term performance in slide projects relates to the consequences if a portion of the slide were to move and potentially remove support from the EPS mass.

GA (DOT) 1. Need for geomembrane to protect geofoam from petroleum contamination
2. Constructability/staging issues building around new fills.

IA (DOT) None except cost.

IL (DOT) Possible floatation problems during construction. Recently we've been informed that, due to rising oil prices, manufacturers are producing substandard blocks not providing consistent correlates between density and strength/stiffness that are currently assumed in design. As a result, NYDOT is having problems with excessive creep (as they believe). We are very concerned about this issue and are considering to eliminate a proposed EPS wall from the design on one project.

MD (DOT) Major issue is rapid increase in cost of material. Other minor issue which is addressed in many publications is the fire and rodent attack.

ME (DOT) Contractors are not familiar with the material.

MI (DOT) High water table areas because EPS floats.

MN (DOT) Differential frosting, petroleum damage, and application of a concrete slab stress distribution layer above the foam.

NY (DOT) Inconsistent quality of the supplied material. We are currently removing about 2500 cubic yards of EPS from an Interstate highway and replacing it with another type of lightweight material because the EPS as supplied is compressing at a very high rate.

VA (DOT) Manufacturing quality control/quality assurance (i.e. acceptance of the material)

VT (DOT) The lack of tech transfer efforts and the perception that the technology hasn't been mainstreamed in the highway construction community are the main deterrents to use. This lack of familiarity with the product by both contractors and designers has limited the number of projects where the EPS has been specified and used. Overcoming buoyant forces and durability are also concerns expressed by designers.

Question A5iii: For which of the following application(s) have you designed, specified, supplied, or installed EPS-block geofoam for the function of lightweight fill in road construction? (check all that apply)

	DOT	DESIGN ENGINEER	EXPANDABLE POLYSTYRENE MFG/SUPPLIER	EPS BLOCK MOLDER
A5iii-1 Embankment over soft ground	MD, ME, MI, MN, NY, VA, IL	MN, WA	UT	MN, MI, AR, GA, SD
A5iii-2 Bridge approach over soft ground	CO, ME, MI, MN, NY, VA, VT	WA	UT, KS	MN, MI, AR, SD
A5iii-3 Preventable Stabilization of Soil Slopes (new road construction)	GA, NY, VA		UT	AR
A5iii-4 Remedial repair or remediation of existing unstable slopes (existing roads)	CO, MN, NY	MN		MN, AR, GA,SD

A5 Other (Explain briefly below)

AR (EPS block molder): parking lots, stadiums, shopping centers, airport runways, water treatment plants.

MI (EPS block molder): reduced weight in fill material over existing utilities.

MN (EPS block molder): sloped ramps on plaza, tapered infill for building rehab. Extension of walkways over existing construction.

SD (EPS block molder): Lateral load reduction on noise walls.

PA (Mfg/supplier): We are looking into a potential new IP application using EPS Geofoam.

UT (Mfg/supplier): Geofoam has been used to eliminate differential settlement on buried utilities. Geofoam has also been used to eliminate settlement damage to nearby structures close to embankments. Geofoam has been used to reduce weight on culverts and other underground structures when embankments have been elevated.

KS (Mfg/supplier): broadening roadway over existing culvert. Culvert would not have handled traditional fill weight.

IL (DOT): Ease and speed of construction, and being superlight weight.

IL (Design engineer): Bikeway support immediately adjacent to vertical sheet piling that had been weakened by lateral deflection during the original roadway construction due to excessive ponding of water behind the sheetpiling during a rainfall event.

WA (Design engineer): Repair of existing unstable slope without roadway (homeowner property on steep slope, where slope failure resulted in loss of yard. EPS fill to be placed to restore yard. EPS behind retaining structures on steep slopes to reduce lateral loads on wall and driving forces on steep slope.
GA (DOT): use of geofoam over existing culvert for reduced loading.
IA (DOT): See A5i above.
MI (DOT): Have designed and specified EPS for reducing pressure behind existing bridge abutments.
MN (DOT): Insulate subsoils against freezing in isolated areas of severe frost heave.
IL (DOT) Temporary EPS block wall for stage construction, but the wall will eventually be backfill with soil and remain under roadway.

Question A5iv: Concerning the EPS-block geofoam specification used on your most recent project**,** what was the source? (check all that apply)

[ ] NCHRP 24-11 Recommended EPS-Block Geofoam Standard for Lightweight Fill in Road Embankments and Bridge Approach Fills on Soft Ground
[ ] State or County Department of Transportation Standard/Provisional Specification
[ ] Other in-house-developed specification
[ ] Provided by EPS molder or distributor
[ ] ASTM D6817-02 Standard Specification for Rigid Cellular Polystyrene Geofoam

	DOT	**DESIGN ENGINEER**	**EXPANDABLE POLYSTYRENE MFG/SUPPLIER**	**EPS BLOCK MOLDER**
NCHRP 24-11	ME, VA, VT, IL	IL, WA		
State or County Department of Transportation Standard/Provisional Specification	CO, GA, ME, NY, VA, IL	MN	KS	AR, SD, MN
Other in-house-developed specification	GA, IA, MD, ME, MI, VT			AR
Provided by EPS molder or distributor	MN, IL	IL	KS	AR, MI
ASTM D6817-02 Standard Specification for Rigid Cellular Polystyrene Geofoam	GA, ME, MI, MN, IL	IL, WA	UT, KS	AR, GA, MI

Question A5v: What one item in the specification you used would you recommend be revised?

GA (EPS block molder) Standardize Federal, State and ASTM Spec.
SD (EPS block molder) N/A
MN (EPS block molder) That they utilize the ASTM D6817 standard for specifying the Geofoam in future projects.
MI (EPS block molder) The elimination of the "termite treatment" called out in a specific EPS molders specification.
KS (Mfg/supplier) I would advise a single source specification. There is much confusion because of the number of specifications. When confirming compression ratings of EPS, some Geofoam suppliers are using larger cubes than the 2" specified in D-1621. This results in higher compression numbers which means they can supply lower densities and win the job. Ok if all are on a level playing field, but that is not happening.
IL (Design engineer) No specific item stands out. NOTE: I wished to check both selections in B1 and C1. It would only let me select one choice.
IL (DOT) The wall is not constructed yet and we're not sure if any of the specs items will actually work best. However, based on previous experience with blocks floatation due to contractor's negligence, it's important to emphasize contractor's responsibility to ensure dry construction by any means.
MN (Design engineer) The need for a concrete load transfer slab.
IA (DOT) Not used EPS enough to know.
VT (DOT) We should have included a tolerance for the allowable density for the EPS blocks. We also needed to provide better specifications on allowable environmental exposure, duration and methods protection.
CO (DOT) Not enough familiarity with specification used to comment.
NY (DOT) Provide for more pre-testing of the material by the owner and less reliance upon third-party certification.
MN (DOT) That foam be placed in a minimum of two layers. No single layer applications should be permitted.
VA (DOT) Clearly specify properties of EPS - currently using "AASHTO EPS 70" which industry says is misleading according to ASTM D 6817.

PART B: DESIGN OF PLANNED SOIL SLOPES INVOLVING NEW ROADWAY CONSTRUCTION

The purpose of this part is to provide us with information that is useful to some of the specific goals of this project related to design of planned soil slopes involving new roadway construction. For the purpose of this questionnaire, planned soil slopes are existing slopes that are in a stable state prior to construction and new engineered slopes to include cut slopes.

Question B1: Who is primarily involved with design associated with planned soil slopes involving new roadway construction?

[**13**] Government-agency personnel
Molder (2); Design Engineer (1); Mfg/supplier (~~0~~); DOT (9)
[**13**] Private consultants
Molder (1); Design Engineer (2); Mfg/supplier (3); DOT (7)

Question B2: Who is primarily involved with construction associated with planned soil slopes involving new roadway construction?

[**5**] Government-agency personnel

Molder (0); Design Engineer (0); Mfg/supplier (0); DOT (5)

[**21**] Private contractors

Molder (3); Design Engineer (3); Mfg/supplier (3); DOT (11)

Question B3: Which analysis methods* do you utilize to evaluate planned soil slopes involving new roadway construction projects? (check all that apply)

*The methods below were obtained from the book titled *Soil Strength and Slope Stability* by J. Michael Duncan and Stephen G. Wright.

[2] Simple methods of analysis that involve a single equation to compute the factor of safety

[**1**] Slope stability charts

[**2**] Spreadsheet software

	DOT	**DESIGN ENGINEER**	**EXPANDABLE POLYSTYRENE MFG/SUPPLIER**	**EPS BLOCK MOLDER**
Simple methods of analysis/single equation to compute safety factor	CT, MD			
Slope stability charts		IL		
Spreadsheet software	CT, WV			
Commercial slope stability computer program (Which ones?) (SEE BELOW)	AK, CT, CO, GA, IA, ID, IL, ME, MI, MT, NE, NY, VA, VT, WV	IL, MN, WA		

B-4, Commercial slope stability computer programs. Which one(s)?:

IL (Design engineer): **PC-STABL**
MN (Design engineer): **GSLOPE**, **UTEXAS3**
WA (Design engineer): **Slope/WPCStabl (StedWin)UTexas**
AK (DOT): **GSTABL7**, **STABLPRO**
CO (DOT): **Slope/W**
CT (DOT): **STAB**L, **ReSSA**
GA (DOT): **winstabl**
IA (DOT): **PCSTABL**
ID (DOT): **SLIDE**, **XSTABL**
IL (DOT): **SLIDE**
MD (DOT): **Stedwin**, Sted, and **Ressa** program
MT (DOT): **GSTAB**L
VT (DOT): **Slide 5.0 From Rocscience**
NE (DOT): **STEDwin** and **GSTABL7**, **ReSSA(1.0)**
WV (DOT): **WinStable**
NY (DOT): **xstabl**
MI (DOT): **RSSA**
ME (DOT): **Slope W**

<u>Question B3:</u> Other (Describe briefly):

GA (EPS block molder) B3_text_5: Outside engineering firms.
SD (EPS block molder) B3_text_5: n/a we don't design.
IA (DOT) B3_text_5: Local experience and techniques.

<u>Question B4:</u> Have you utilized other types of lightweight fill materials in slope stability projects associated with planned soil slopes involving new roadway construction?

[**19**] No
[10] Yes (Which lightweight fill types?):

IL (Design engineer) CLSM (controlled low strength material).
MN (Design engineer) Shredded tires, wood chips.
WA (Design engineer) Lightweight foamed cement fill was used for railroad embankment construction.
CT (DOT) Shale Aggregate.
GA (DOT) lightweight foamed concrete.
ID (DOT) Saw dust, wood fiber, light weight rock (volcanic).
ME (DOT) Tire Derived Aggregate, Expanded Shale.
MI (DOT) Lightweight aggregate.
NY (DOT) Tire shreds, expanded shale, blast furnace slag, pumice, cellular foamed concrete.
VA (DOT) Low density cementitious fills (Elastiizell, AJ Voton).
IL (DOT) Light weight aggregates and cellular concrete.

Question B5: Are you currently utilizing load resistance factor design (LRFD) procedures in design involving soil slopes associated with new roadway construction?

[**18**] No
[**3**] Yes

	DOT	DESIGN ENGINEER	EXPANDABLE POLYSTYRENE MFG/SUPPLIER	EPS BLOCK MOLDER
NO	AK, CO, GA, IA, ID, MD, MI, MN, MT, NE, NY, VA, VT, IL	MN, WA	PA	GA
YES	CT, ME, WV			

Question B6: Do you perform seismic slope stability analysis in slope designs involving planned soil slopes associated with new roadway construction?

[**16**] No, seismic loading is not a design consideration in our area.
[**2**] No, seismic loading is a design consideration in our geographic area, but we do not perform seismic slope stability analysis.
[**5**] Yes.

	DOT	DESIGN ENGINEER	EXPANDABLE POLYSTYRENE MFG/SUPPLIER	EPS BLOCK MOLDER
No, seismic loading is not a design consideration in our area	CO, GA, IA, MD, MI, MN, NE, NY, VA, VT, WV	MN	PA	GA, SD
No, seismic loading is a design consideration in our geographic area, but we do not perform seismic slope stability analysis.	AK, CT			
Yes	ID, ME, MT	IL, WA		

PART C: REMEDIAL REPAIR OR REMEDIATION OF EXISTING UNSTABLE SOIL SLOPES INVOLVING EXISTING ROADWAYS

The purpose of this part is to provide us with information that is useful to some of the specific goals of this project related to remedial repair or remediation of existing unstable soil slopes involving existing roadways.

Question C1: Who is primarily involved with design associated with remedial repair or remediation of existing unstable soil slopes involving existing roadways?

[**18**] Government-agency personnel
[**6**] Private consultants

Question C2: Who is primarily involved with construction associated with remedial repair or remediation of existing unstable soil slopes involving existing roadways?

[**10**] Government-agency personnel
[**14**] Private contractors

Question C3: Which analysis methods* do you utilize to evaluate remedial repair or remediation procedures involving unstable soil slopes in existing roadway construction projects (check all that apply)?

*The methods below were obtained from the book titled *Soil Strength and Slope Stability* by J. Michael Duncan and Stephen G. Wright.

[**1**]C3-1 Simple methods of analysis that involve a single equation to compute the factor of safety
[**0**]C3-2 Slope stability charts
[**2**]C3-3 Spreadsheet software

C3: Commercial slope stability computer programs:

IL (Design engineer): PC-STABL
MN (Design engineer): GSLOPE, UTEXAS3
WA (Design engineer): see above
AK (DOT): GSTABL7, STABLPRO
CO (DOT): Slope/W
CT (DOT): ResSSA
GA (DOT): winstabl
IA (DOT): PCSTABL
ID (DOT): SLIDE, XSTABL
IL (DOT) XSTABL and recently we've used SLIDE
MD (DOT): Sted win, Sted , and Ressa program
ME (DOT): Slope W
MI (DOT): RSSA
MT (DOT): GSTABL
NY (DOT): xstabl
VT (DOT): Slide 5.0 from Rocscience
WV (DOT) WinStable

C3: Other (Describe briefly.)

GA (EPS block molder): Outside engineering forms.
SD (EPS block molder): n/a We don't design fills.
IA (DOT): Local experience and techniques.

Question C4: Have you utilized other types of lightweight fill materials in slope stability projects associated with remedial repair or remediation of existing unstable soil slopes involving existing roadways?

[**17**] No
[**12**] Yes (Which lightweight fill types?)

MN (Design engineer) Shredded tires
WA (Design engineer) See above
AK (DOT) wood chips
CO (DOT) We haven't used but have considered use of expanded shales and tire bales
ID (DOT) Saw dust, wood fiber, light weight rock
MD (DOT) Majority of time we use Riprap (105 pcf) which is lighter weight than regular soil.
ME (DOT) Expanded shale
MI (DOT) Lightweight aggregate, geotextiles, geogrid reinforcement.
MN (DOT) Shredded tire chips
NY (DOT) See list above
VA (DOT) Low Density cementitious fill (Elastizell)
VT (DOT) Shredded Tires

Question C5: Are you currently utilizing load resistance factor design (LRFD) procedures in slope designs involving remedial repair or remediation of existing soil slopes associated with existing roadways?

[**19**] No: DOT (14); Molder (2); Design Engineer (3)
[**2**] Yes: DOT (2); Molder (0); Design Engineer (0)

Question C6: Do you perform seismic slope stability analysis in slope designs involving remedial repair or remediation of existing unstable soil slopes associated with existing roadways?

[**14**] No, seismic loading not a design consideration in our geographic area.

[**2**] No, seismic loading is a design consideration in our geographic area, but we do not perform seismic slope stability analysis.

[**6**] Yes

	DOT	**DESIGN ENGINEER**	**EXPANDABLE POLYSTYRENE MFG/SUPPLIER**	**EPS BLOCK MOLDER**
No, seismic loading not a design consideration in our geographic area	CO, GA, IA, MD, MI, MN, NE, NY, VA, VT, WV	MN		GA, SD
No, seismic loading is a design consideration in our geographic area, but we do not perform seismic slope stability analysis	AK, CT,			
Yes	ID, ME, MT, IL	IL, WA		

PART D: GENERAL

Question D1: Which slope stability procedures* have you utilized for design of planned soil slopes associated with new roadway construction and/or for remedial repair or remediation of existing unstable soil slopes associated with existing roadways (check all that apply)?

*The procedures outlined below were obtained from TRB Special Report 247, Landslides: Investigation and Mitigation.

Category	Procedure	Design of Planned Slopes	Remedial Repair or Remediation
Avoid Problem	Relocate facility	WA, IL, AK, CT, CO, GA, IA, ID, MD, ME, MN, MT, NE, NY, VA, VT	WA, IL, AK, ID, MD, ME, MN, MT, NE, NY, VA, VT, WV
	Completely or partially remove unstable materials	WA, MN, AK, CT,CO, GA, IA, MD, ME, MI, MN, MT, NE, NY, VA, VT	WA, MN, AK, CT, CO, GA, IA, ID, MD, ME, MI, MN, MT, NE, NY, VA, VT, WV
	Install bridge	WA, IL, MN, AK, GA, MD, MT, NY, WV	WA, MT, NY
Reduce driving forces	Change line or grade	WA, AK, CT,CO, GA, IA, ID, MD, ME, MN, MT, NE, NY, VA, VT	AK, CT, GA, IA, ID, ME, MN, MT, NE, NY, VA, VT
	Drain surface	WA, IL, AK, CT, CO, GA, IA, ID, MD, ME, MI, MT, NE, NY, VA, VT, IL	WA, IL, AK, CT, CO, IA, ID, ME, MT, NE, NY, VA, VT, WV, IL
	Drain subsurface	WA, IL, MN, AK, CT, CO, IA, ID, ME, MI, MN, MT, NE, NY, VA, VT,IL	PA, WA, IL, MN, AK, CT, CO, GA, IA, ID, ME, MI, MN, MT, NE, NY, VA, VT, WV, IL
	Reduce weight	WA, IL, MN, AK, CT, CO, GA, IA, ID, MD, ME, MI, MN, MT, NY, VA, VT	WA, IL, MN, AK, CO, IA, ID, ME, MI, MN, MT, NY, VA, VT, WV
Increase resistance forces by			
Apply external force	Use buttress and counterweight fills; toe berms	WA, IL, MN, AK, IA, ID, MD, ME, MT, NY, VA, VT, IL	WA, IL, MN, AK, CO, GA, IA, ID, MD, ME, MN, MT, NY, VA, VT, WV, IL
	Use structural systems	WA, IL, AK, CT, CO, GA, IA, MN, MT, NY, VA, IL	WA, IL, MN, AK, CO, GA, MN, MT, NY, VA, VT, WV, IL
	Install Anchors	PA, WA, IL, MN, AK, CT, CO, GA, IA, MD, MI, MT, NY, VT, IL	WA, IL, MN, AK, CO, GA, IA, MD, ME, MI, MT, NY, WV, IL
Increase internal strength	Drain subsurface	WA, IL, MN, AK, CT, CO, IA, ID, MD, ME, MN, MT, NY, VA, VT, IL	PA, WA, IL, MN, AK, CT, CO, GA, IA, ID, MD, ME, MN, MT, NY, VA, VT, WV, IL
	Use reinforced backfill	WA, IL, MN, AK, CT, CO, GA, IA, ID, MD, ME, MI, MN, MT, NE, NY, VA, VT, IL	WA, MN, AK, CT, CO, IA, ID, MD, ME, MI, MN, MT, NE, NY, VA, VT, WV, IL
	Install in-situ reinforcement	WA, IL, MN, AK, CO, GA, IA, ID, MD, ME, NY, VA, VT, IL	WA, MN, AK, CO, GA, IA, MD, ME, NY, VA, VT, IL
	Use biotechnical stabilization	MN, MD	MD
	Treat chemically	MN, IA, MD, VA	MD, VA
	Use electro osmosis		
	Treat thermally	AK	AK

(**DOT**; **DESIGN ENGINEER**; **MFG/SUPPLIER**)

Question D2: What other slope stability procedures have you utilized for:

i. design of planned soil slopes associated with new road construction?

CT (DOT) Riprap (for shallow slope failures)
IA (DOT) Wick drains, stone columns, etc where applicable.
MD (DOT) Shear key, use toe walls
ME (DOT) hand calcs, Stable 6
NY (DOT) Pile-supported embankment, reinforced slab at grade on piles.
VA (DOT) Deep soil mixing; PV drains and surcharge; pile supported embankments; geosynthetics

ii. remedial repair or remediation of existing unstable soil slopes associated with existing roads?

MN (Design engineer) Use of rock filled shear trenches.
CT (DOT) Riprap (for shallow slope failures)
GA (DOT) deep injection grouting
IA (DOT) Potentially same as above.
ME (DOT) hand calcs, Stable 6
NY (DOT) Launched soil nails, chimney drain buttresses.
VA (DOT) Compaction grouting; structural systems; buttresses; regrading; subsurface drainage

Question D3: Overall, what one item would you like us to consider or include in the NCHRP 24-11(02) project documents that would be of greatest use to you in designing, supplying or installing EPS-block geofoam for road construction?

IL (Design engineer) Case histories of successful projects and "lessons learned" for unsuccessful projects.
MN (Design engineer) Applicable methodology for analyzing potential failure surface extending through the geofoam material itself.
AK (DOT) Design guidelines
CO (DOT) A reliable design method for EPS blocks in a slope that incorporates the interaction between soil and EPS.
CT (DOT) Guide Special Provision
GA (DOT) examples of design drawings for different applications
IA (DOT) Reduction in cost of EPS materials.
IL (DOT) Revisit wheel loads and stresses recommended in 24-11(1). They seem to be overly conservative.
ME (DOT) training for contractor and resident
MI (DOT) More information on preparation of base material and top surface treatment (concrete slab, liner, etc.)
MN (DOT) Alternatives to use of a concrete capping slab over the geofoam.
MT (DOT) Detailed cost data on supplying and installing geofoam
NY (DOT) Some general knowledge of the process used to create the EPS blocks so that we can better judge the process needed to monitor and control the material as it is supplied to us. Geotechnical design with EPS is simple and easy, and construction is quite straightforward. Making sure that the supplied material is correct and adequate is the real concern.
VA (DOT) Typical design examples; more details on protection of EPS from hydrocarbons and design procedure for load distribution slab
VT (DOT) A concise user's guide that could be used for effective technology transfer and training would be of most benefit.

Question D4: Would you prefer that that the NCHRP 24-11(02) results for slope applications be integrated with the Project 24-11 reports for stand-alone embankments over soft ground or that the two project reports be separate?

[**11**] Integrate the two

DOT (8); Molder (1); Design Engineer (2)

[**6**] Keep separate

DOT (5); Design Engineer (1)

Question D5: Do you have any of the following supporting documentation that you think may be helpful to us in achieving the research objectives and that you would be willing to provide (check all that apply- we will contact you for follow up)?

EPS-block geofoam supporting documentation in/with:

	Design of planned soil slopes (new road construction)	**Remedial repair/remediation of existing unstable soil slops (existing roads)**	**In stand-alone embankments on soft ground**	**Bridge Approaches**	**Other types of light-weight fills**
Plans	IL, GA, ME, MT, NY	CO, MT, NY	MN, MT, NY,UT	ME, MT, NY, VA, VT	GA, MT, NY
Specifications	IL, GA, ME, MI, MT, NY	CO, MT, NY	MD, MN, MT, NY, UT	ME, MT, NY, VA	MT, NY
Design Reports		CO			GA
Cost Estimates and Comparisons	ME, MT, NY	MT, NY	MD, MT, NY	ME, MT, NY, VA	MT, NY
Field Instrumentation/ Performance Data	NY	NY	NY, UT	NY, VT	NY
Photographs	IL, ME, NY, UT	CO, NY	MN, NY, UT	ME, NY, UT, VA, VT	NY
Other case history info	ME	NY	MN, UT	ME, NY, VT	NY

(**DOT**; **DESIGN ENGINEER**; **MFG/SUPPLIER**; **MFG/SUPPLIER**)

Please feel free to provide any additional comments below.

CT (EPS block molder) As an EPS block molder we quote on and manufacture EPS blocks to the density and dimensions that are specified on the RFQ.

GA (EPS block molder) Due to the fact that we utilize outside engineering (when required), we can not answer many of the survey questions.

SD (EPS block molder) Much of the questions doesn't pertain to me as a manufacturer. I have not gotten involved in designing a geofoam project to date.

KS (Mfg/supplier) As a manufacture of EPS Geofoam, several questions above are outside of my scope of expertise. Good luck with your research! Please let me know if you have any questions about my answers or would like to tour an EPS manufacturing facility. Bob Nickloy 800-638-3626 ext. 218.

IL (Design engineer) I noted this above, but will repeat here. I wished to check both selections in B1 and C1. It would only let me select one choice.

IL (DOT) Unfortunately, this project does not seem to promise many improvements over the previous projects. To make a real difference, it should move on to other fields of applications, such as temp/permanent walls, rather than re-inventing the wheel for embankments and slopes. Also, a greater emphasis should be placed on the new manufacturing processes to see if the correlations that have previously been established between density and strength/stiffness are still applicable to the new geofoam products.

MD (DOT) We appreciate if we may get a copy of NCHRP 24-11(02) report.

VT (DOT) We currently have a research project with U-Mass in which we instrumented two pile supported abutments backfilled with geofoam. The results of this research will be available in December 2007.

WV (DOT) Personally, I have given EPS much thought, and have recommended it for landslide repair as a consultant. I would like to suggest using alignment dimples to aid in installation and Velcro strips to keep it from blowing. Also, can use the "peanut" sized foam contained in sacks to fill in around edges.

VA (DOT) Variability and difficulty in predicting cost is a significant factor in decision to use ESP geofoam.

End of Survey Summary

Appendix B

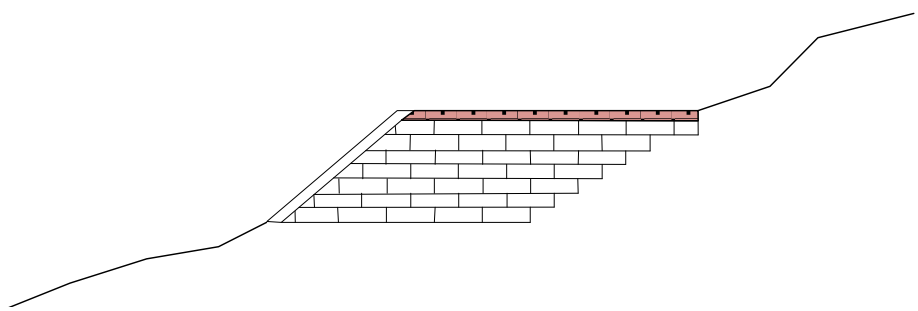
Recommended Design Guideline

B.1 INTRODUCTION

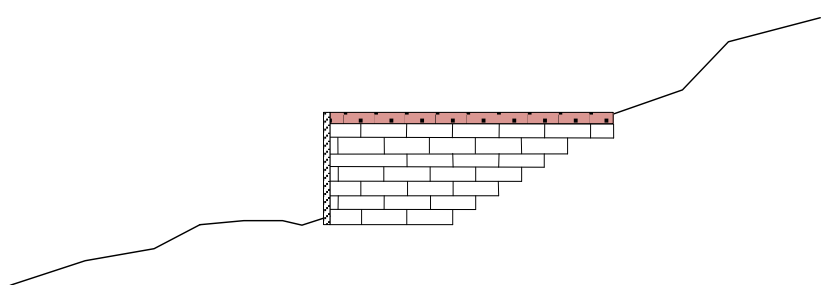
This recommended design guideline is intended to provide design guidance to civil engineers experienced in geotechnical engineering when designing slopes that incorporate expanded polystyrene (EPS)-block geofoam as a lightweight fill. Use of lightweight fill is an expedient slope-stabilization procedure used to reduce the weight of the sliding mass, thereby reducing the driving forces of the sliding mass.

The design guideline is limited to slope stability applications (sometimes referred to as side-hill fills), as shown in Figure B.1. The use of EPS-block geofoam in slope applications can involve a slope-sided fill (Figure B.1(a)) or a vertical-sided fill (Figure B.1(b)). The latter application is sometimes referred to as a geofoam wall, and is unique to EPS-block geofoam. Use of a vertical-sided fill reduces the amount of right-of-way needed and minimizes the impact of fill loads on nearby structures. For vertical-sided embankment walls, the exposed sides should be covered with a facing. The facing does not need to provide any structural capacity to retain the blocks, because the blocks are self-stable, so the primary function is to protect the blocks from environmental factors.

Applications for use of EPS-block geofoam as lightweight fill in stand-alone embankments on soft ground that have a transverse (cross-sectional) geometry where the two sides are more or less of equal height, as shown in Figure B.2 are excluded from this study because they are the subject of a separate study (Stark et al., 2004a, Stark et al., 2004b).

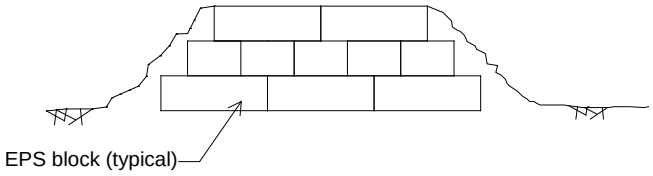


a) Slope-sided fill.

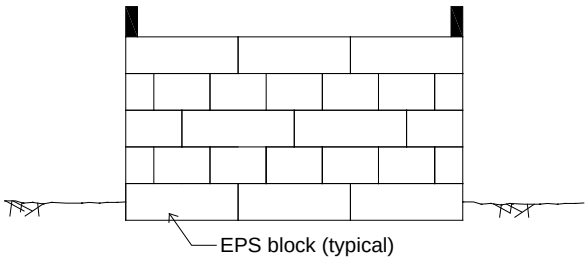


b) Vertical-sided fill (Geofoam wall).

Figure B.1. Typical EPS-block geofoam applications involving side-hill fills.



a) Slope-sided fill.



b) Vertical-sided fill (Geofoam wall).

Figure B.2. Typical EPS-block geofoam applications involving stand-alone embankments (Horvath, 1995, Stark et al., 2004a).

This guideline was prepared as part of the National Cooperative Highway Research Program (NCHRP) Project 24-11(02) titled *"Guidelines for Geofoam Applications in Slope Stability Projects"* administered by the Transportation Research Board (TRB). Chapter 4 of this project report is provided as essential commentary accompanying this guideline. It is suggested that users of this guideline review the accompanying report for necessary technical background. This guideline is intended for use in conjunction with the recommended material and construction standard presented in Appendix F of the report.

This recommended design guideline is based on the traditional Service Load Design (SLD) approach because current state-of-practice of slope stability analysis is based on SLD. Until inconsistencies with applying the Load and Resistance Factor Design (LRFD) methodology to slope stability analysis are resolved, an LRFD based design procedure for EPS-block geofoam slopes cannot be developed. A discussion of both SLD and LRFD design approaches is provided in the NCHRP Project 24-11(02) report.

B.2 MAJOR COMPONENTS OF AN EPS-BLOCK GEOFOAM SLOPE SYSTEM

As indicated in Figure B.3, an EPS-block geofoam slope system consists of three major components:

- The **existing slope material**, which can be divided into the upper and lower slope. The slope material directly below the fill mass may also be referred to as the foundation material.
- The proposed **fill mass**, which consists primarily of EPS-block geofoam. In addition, depending on whether the fill mass has sloped (slope-sided fill) or vertical (vertical-sided fill) sides, there will be either soil or a protective structural cover over the sides of EPS blocks.
- The proposed **pavement system**, which is defined as including all material layers, bound and unbound, placed above the EPS blocks.

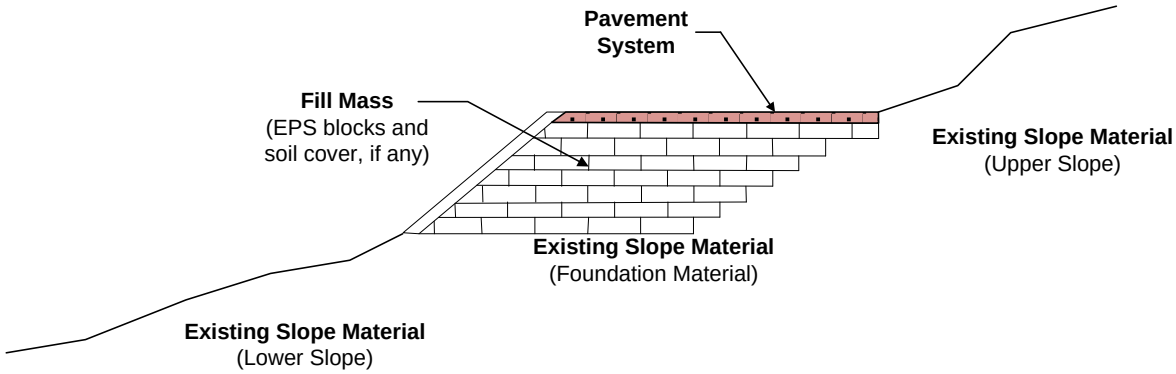


Figure B.3. Major components of an EPS-block geofoam slope system.

B.3 FAILURE MODES

B.3.1 Overview

To design against failure, the overall design process of an EPS-block geofoam slope system includes evaluation of three failure modes and must include the following design considerations:

- Design for **external stability** of the overall EPS-block geofoam slope system configuration.
- Design for **internal stability** of the fill mass.
- Design of an appropriate **pavement system** for the subgrade provided by the underlying EPS blocks. Analysis of this failure mode is only required if a pavement system will be included above the EPS-fill mass.

Table B.1 provides a summary of the three failure modes and various failure mechanisms that need to be considered for each failure mode. Each failure mechanism has also been categorized into either an ultimate limit state (ULS) or serviceability limit state (SLS) failure. The three failure modes are subsequently described in more detail.

Table B.1. Summary of failure modes and mechanisms incorporated in the proposed design procedure for EPS-block geofoam as a lightweight fill in slope stability applications.

FAILURE MODE	LIMIT STATE	FAILURE MECHANISM	ACCOUNTS FOR
External Instability	ULS	Static slope stability	Global stability involving a deep-seated slip surface and slip surfaces involving the existing slope material only (Figure B.4). Also considers slip surfaces that involve both the fill mass and existing slope material (Figure B.5).
	ULS	Seismic slope stability	Same as for static slope stability but considers seismic induced loads.
	SLS	Seismic settlement	Earthquake-induced settlement due to compression of the existing foundation material (Figure B.12) such as those resulting from liquefaction, seismic-induced slope movement, regional tectonic surface effects, foundation soil compression due to cyclic soil densification, and increase due to dynamic loads caused by rocking of the fill mass (Day, 2002).
	ULS	Seismic bearing capacity	Bearing capacity failure of the existing foundation earth material (Figure B.11) due to seismic loading and, potentially, a decrease in the shear strength of the foundation material.
	ULS	Seismic sliding	Sliding of the entire EPS-block geofoam fill mass (Figure B.9) due to seismic induced loads.
	ULS	Seismic overturning	Overturning of the entire embankment at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation material as a result of seismic forces (Figure B.10).
	SLS	Settlement	Excessive and/or differential settlement from vertical and lateral deformations of the underlying foundation soil (Figure B.12).
	ULS	Bearing capacity	Bearing capacity failure of the existing foundation earth material (Figure B.11) resulting in downward vertical movement of the entire fill mass into the foundation soil.
Internal Instability	ULS	Seismic sliding	Horizontal sliding between layers of blocks and/or between the pavement system and the upper layer of blocks (Figure B.17) due to seismic induced loads.
	SLS	Seismic load bearing (seismic rocking)	Excessive vertical deformation of EPS blocks (Figure B.18) due increase in the vertical normal stress within the EPS-block fill mass due to the moment produced by the seismic induced inertia force.
	SLS	Load bearing	Excessive vertical deformation of EPS blocks (Figure B.18) due excessive initial (immediate) deformations under dead or gravity loads from the overlying pavement system, excessive long-term (for the design life of the fill) creep deformations under the same gravity loads, and/or excessive non-elastic or irreversible deformations under repetitive traffic loads.
Pavement System Failure	SLS	Flexible or rigid pavement	Premature failure of the pavement system (Figure B.22), as well as to minimize the potential for differential icing (a potential safety hazard). Providing sufficient support, either by direct embedment or structural anchorage, for any road hardware (guardrails, barriers, median dividers, lighting, signage and utilities).

SLS = serviceability limit state
ULS = ultimate limit state

B.3.2 External Instability Failure Mode

The primary failure mechanism that needs to be evaluated for EPS-block geofoam in slope stabilization applications is overall (external) slope stability, because this is typically the primary reason for considering the use of EPS block in slope applications. Based on experience with mechanically stabilized earth walls (MSEWs), problems related with MSEWs have primarily involved global instability (Leshchinsky, 2002). Therefore, global stability of an EPS-block geofoam slope system may also prove to be a critical failure mechanism.

Design for external stability of the overall EPS-block geofoam slope system considers failure mechanisms that involve the existing slope material only, as shown in Figure B.4, as well as failure mechanisms that involve both the fill mass and the existing slope material, as shown in Figure B.5.

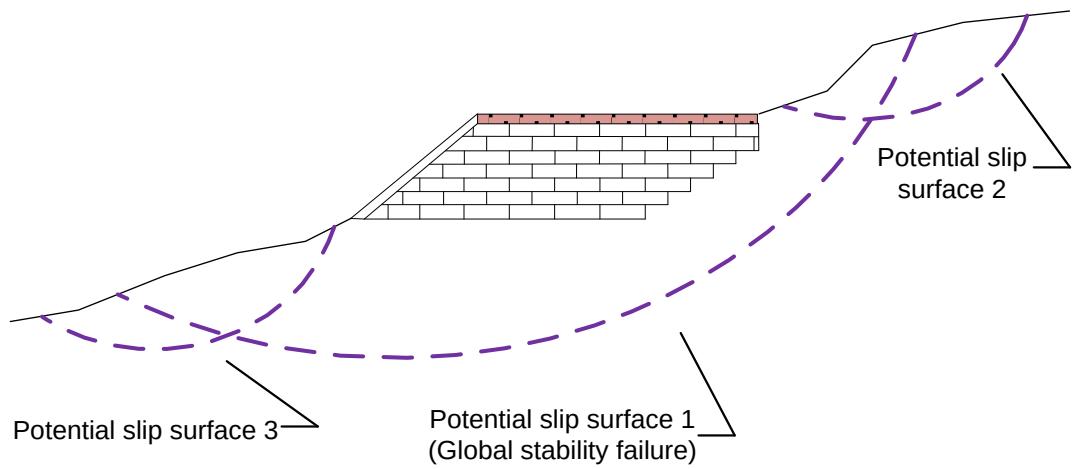


Figure B.4. Static and seismic slope stability involving existing soil slope material only.

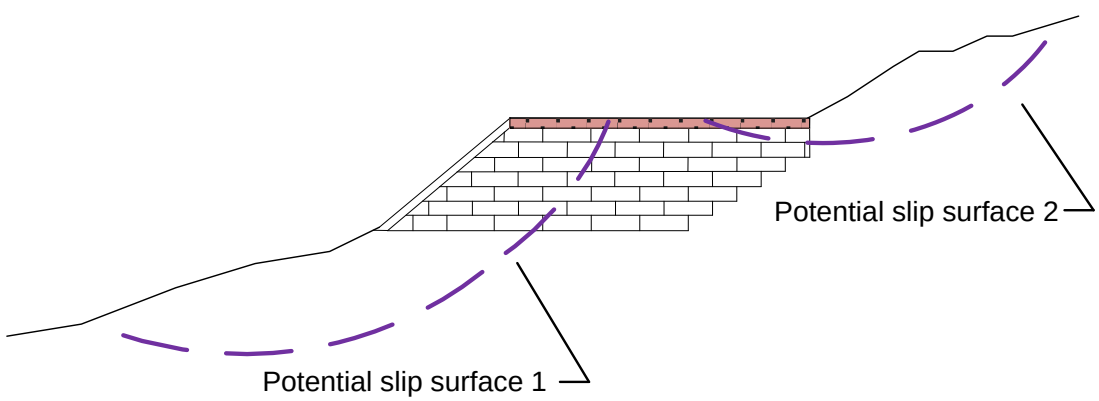


Figure B.5. Static and seismic slope stability involving both the fill mass and existing soil slope material.

As shown in Table B.1, external stability failure mechanisms included in the proposed design procedure consist of static slope stability, settlement, and bearing capacity. Additional failure mechanisms associated with external seismic stability include seismic slope instability, seismic induced settlement, seismic bearing capacity failure, seismic sliding, and seismic overturning. These failure considerations, together with other project-specific design inputs, such as right-of-way constraints, limiting impact on

underlying and/or adjacent structures, and construction time, usually govern the overall cross-sectional geometry of the fill. Because EPS-block geofoam is typically a more expensive material than soil on a cost-per-unit-volume basis for the material alone, it is desirable to optimize the design to minimize the volume of EPS used, yet still satisfy external instability design criteria concerning settlement, bearing capacity, and static and seismic slope stability.

Based on current design precedent, it is recommended that all EPS-block geofoam slope systems incorporate drainage systems below EPS block to prevent water from accumulating above the bottom of EPS blocks, and a drainage system between the adjacent upper slope material and EPS block to collect and divert seepage water, and thereby alleviate seepage pressures. This design guideline is based on installation of an appropriate drainage system to prevent development of hydrostatic and seepage pressures below the EPS-block fill mass that may cause uplift of the fill mass and adjacent to the fill mass that may cause sliding and instability of the fill mass.

The design procedure is based on a self-stable adjacent upper slope to prevent earth pressures on the EPS fill mass that can result in horizontal sliding of the overall fill mass. If the adjacent slope material cannot be cut to a long-term stable slope angle, an earth-retention system must be used to resist the applied earth force. Various types of earth-retention systems are included in Chapter 4 of the NCHRP Project 24-11(02) report.

B.3.3 Internal Instability Failure Mode

Design for internal stability considers failure mechanisms within the EPS-block geofoam fill mass. As shown in Table B.1, three internal instability failure mechanisms evaluated during design are seismic horizontal sliding, seismic load-bearing of EPS blocks, and static load-bearing of EPS blocks.

The design procedure is based on a self-stable adjacent upper slope to prevent earth pressures on the EPS fill mass that can result in horizontal sliding between blocks. If the adjacent slope material cannot be cut to a long-term stable slope angle, an earth-retention system must be used to resist the applied earth force. Various types of earth-retention systems are included in Chapter 4 of the Project 24-11(02) report.

B.3.4 Pavement System Failure Mode

The pavement system is defined as including all materials, bound and unbound, placed above EPS block. Design of an appropriate pavement system considers the subgrade provided by underlying EPS block. The design criterion is to prevent premature failure of the pavement system such as rutting or cracking. Also, when designing the pavement cross-section, some consideration should be given to providing sufficient support, either by direct embedment or structural anchorage, for any road hardware such as guardrails, barriers, median dividers, lighting, signage and utilities. Analysis of this failure mode is only required if a pavement system will be included above the EPS-fill mass.

B.4 DESIGN LOADS

B.4.1 Overview

The term “failure” as used in this recommended design guideline is a *loss of function*. This is the same definition incorporated in the Project 24-11(01) design guideline for stand-alone embankments over soft ground. Failure or loss of function of an EPS-block geofoam slope system may occur as either a *collapse failure* (the *ultimate* or *strength limit state*, ULS) or a s*erviceability failure* (the *service limit state*, SLS). Therefore the analysis and design of geofoam slope systems must consider these two limiting conditions.

A geofoam slope system may undergo an ULS failure if the applied loads produce stresses that exceed the resistances provided by the whole geofoam slope system or any of its individual components. As shown in Table B.1, ULS failure can occur as an external and internal failure mode. A geofoam slope

system may undergo an external collapse failure as part of slope instability due to either static and/or seismic loads, and as part of a bearing capacity failure of the foundation material. An internal ULS failure can occur as part of seismic slope instability.

A geofoam slope system may undergo an external serviceability failure if excessive total or differential deformation develops over time. An internal serviceability failure may occur if the load-bearing capacity of EPS blocks is exceeded. The geofoam slope system may also undergo a serviceability failure if premature failure of the pavement system occurs. Premature failure of the pavement system may include an uneven and often cracked pavement surface that may require frequent repaving and possibly other maintenance.

The overall design objective for minimizing the potential against a collapse failure is to ensure that the resistance of the EPS-block slope system against failure exceeds the loads producing failure. Therefore, the ULS analysis must satisfy the following equation:

ULS: resistance of EPS-block geofoam slope system to failure >
EPS-block geofoam slope system loads producing failure **(B.1)**

The overall design objective for minimizing the potential against a serviceability failure is to ensure that the estimated deformation of the EPS-block geofoam slope system does not exceed the maximum acceptable deformation. Therefore, the SLS analysis must satisfy the following equation:

SLS: estimated deformation of EPS-block geofoam slope system ≤
maximum acceptable deformation **(B.2)**

Two primary approaches are available to evaluate Equations B.1 and B.2; Service Load Design (SLD) and Load and Resistance Factor Design (LRFD). A summary of these two design approaches is provided in the Project 24-11(02) report. However, because the current state-of-practice of slope stability analysis is based on SLD, the recommended design guideline is also based on the SLD approach. Until inconsistencies with applying the LRFD methodology to slope stability analysis are resolved, an LRFD-based design procedure for EPS-block geofoam slopes cannot be developed.

The three primary loads applicable for design of EPS-block geofoam slopes are dead loads (D), live loads (L), and seismic loads (EQ). In addition to these three primary load types, centrifugal loads (CF) may also be significant for EPS-block geofoam structures. Additionally, loads associated with design of barriers and railings may need to be considered. For ultimate limit state calculations, the worst expected loadings are typically used, while for serviceability limit state calculations, typical or average expected loadings are used.

B.4.2 Dead (Gravity) Loads

Components of the slope fill system, depicted in Figure B.3, that contribute to gravity loading and need to be considered in design include:

- The weight of the overlying pavement system, which includes any reinforced PCC slab that might be used at the pavement system and geofoam interface.
- The weight of soil cover placed on the sides of a slope-sided embankment, or weight of the protective facing wall elements of a vertical-sided embankment.
- The net effective weight of any earth material placed between the existing ground surface (foundation material) and bottom of EPS blocks.

Gravity loads can be calculated based on a preliminary assumed cross-section, including the pavement system and any cover material over the sides of the embankment. To establish this preliminary cross-section of the embankment, and to begin the design procedure, it is recommended that the

preliminary pavement system be assumed to be 1m (3 ft.) thick, and the various component layers of the pavement system be assumed to have a total unit weight of 20 kN/m^3 (130 lbf/ft^3) for initial design purposes. This pavement system thickness is used only to establish a preliminary cross-section of the embankment and to begin the design procedure. The design procedure also includes a separate pavement design step that includes a structural pavement analysis to finalize the pavement system design.

Table B.2 provides estimated EPS block densities based on long-term water content test results performed in Norway for the standard material designations incorporated in the material and construction standard. The recommended design procedure is based on use of a permanent drainage system. Therefore, the most applicable densities for use in determining dead loads are either the dry densities per the Japanese design recommendations, or densities based on a water content of 1 percent by volume per the Norwegian test results. Although EPS-block geofoam can be manufactured to various densities, the preliminary design can be based on a density of 20 kg/m^3 (1.25 lbf./ft^3). Therefore, the dry unit weight of the EPS can be taken to be 200 N/m^3 (1.25 lbf/ft^3), or a unit weight at 1 percent water content by volume of 300 N/m^3 (1.9 lbf/ft^3) for preliminary design.

Table B.2. Estimated EPS block densities for various water contents.

Material Designation	**Density kg/m^3 (lbf./ft^3)**			
	Minimum Allowable (*Dry*)	Permanently Submerged (*10%*)*	Periodically Submerged (*4%*)*	Above highest ground-water level (*1%*)*
EPS40	16 (1.0)	116 (7.2)	56 (3.5)	26 (1.6)
EPS50	20 (1.25)	120 (7.5)	60 (3.7)	30 (1.9)
EPS70	24 (1.5)	124 (7.7)	64 (4.0)	34 (2.1)
EPS100	32 (2.0)	132 (8.2)	72 (4.5)	42 (2.6)

* Water content by volume basis.

Use of the dry unit weight versus the unit weight at 1 percent water content by volume in design will depend on the failure mechanism being evaluated. For example, when evaluating certain failure mechanisms, such as internal load bearing of the EPS blocks; settlement; and bearing capacity failure of the foundation material, use of a higher unit weight for the EPS-block geofoam would be conservative; however, it should be noted that this is not the case for all failure mechanisms. For example, when evaluating external slope stability, the higher unit weight would result in increased driving forces, which would make using the higher unit weight more conservative, and therefore more appropriate for design. But, by using a higher unit weight for EPS, normal stresses along the slip surface are also increased, which, in turn, results in an increase in shear strength. So, use of the unit weight at 1 percent water content by volume increases both the driving force and resisting forces in the slope analysis, making it difficult to discern which unit weight value is truly more conservative. For cases such as this, the best approach is to perform the analysis using both the dry unit weight and unit weight at 1 percent water absorption by volume, and compare the results. The value for the unit weight that results in the lower factor of safety is the unit weight which should be used for that particular failure mechanism.

B.4.3 Live (Traffic) Loads

A live-load surcharge pressure equal to 610mm (2 ft.) of an 18.9 kN/m^3 (120 lbf./ft^3) surcharge material can be used to model traffic stresses at the top of the embankment. The exception to use of a surcharge pressure to represent traffic loads is in the evaluation of load-bearing capacity of EPS block. The basic procedure for designing against load bearing failure is to calculate the maximum vertical stress due to the overlying pavement system and traffic loads at various levels within the EPS mass, and select

the EPS that exhibits an elastic limit stress that is greater than the calculated or required elastic limit stress at the depth being considered. Traffic loads are a major consideration in the load-bearing capacity calculations. Therefore, the effects of traffic loading and traffic configuration are critical to the load-bearing analysis and are explicitly estimated as a part of it.

B.4.4 Seismic Loads

Seismic loading is a short-term event that is considered in geotechnical problems including road embankments and slopes. Seismic loading can affect both external and internal stability of an EPS-block geofoam slope system. Considerations for seismic external stability analyses are similar for embankments constructed of EPS-block geofoam or earth materials. These considerations include various SLS and ULS mechanisms, such as seismic settlement and liquefaction, that are primarily independent of the nature of the embankment or fill mass material because they depend on the seismic risk at a particular site and nature and thickness of the natural soil overlying the bedrock.

The required seismic design policies vary between state DOTs. For example, the Washington State Department of Transportation (WSDOT) policy on cut slopes in soil and rock, fill slopes, and embankments is that instability due to seismic events should be evaluated. However, mitigation of instability is not always required, due to the high cost of requiring mitigation of cut and fill slopes and embankments statewide. However, stabilization is required for slopes that impact an adjacent structure if failure due to seismic loading occurs (Washington State Department of Transportation, 2006).

The failure mechanisms considered for external seismic stability analysis include slope instability, horizontal sliding of the entire EPS-block geofoam fill mass, overturning of a vertical sided embankment, bearing capacity failure of existing foundation earth material, and settlement of existing foundation material. The general external seismic analysis procedure consists of performing a pseudo-static analysis to evaluate slope instability. The procedure to incorporate the effect of the natural slope material on external horizontal sliding of the entire EPS-block fill mass and overturning of a vertical-sided embankment consists of determining the magnitude of seismic earth pressure based on the Mononobe-Okabe (M-O) method (Okabe, 1926, Mononobe, 1929).

Failure mechanisms considered for internal seismic stability analysis include horizontal sliding between layers of block and/or between the pavement system and upper layer of block, and load-bearing failure of EPS blocks. The general internal seismic analysis procedure consists of decoupling the determination of the overall seismic response acceleration of the EPS-block geofoam embankment into the determination of the seismic response of the natural slope material followed by the seismic response of the EPS-block fill mass. The seismic response results of adjacent natural slope material and the EPS-block fill mass can then be used to evaluate each potential seismic failure mechanism separately.

B.4.5 Centrifugal Loads

Although loads due to centrifugal forces are not typically considered in design of earth structures, they may prove significant in the design of slopes incorporating EPS-block geofoam. According to AASHTO specifications (AASHTO, 1996), the centrifugal load (CF) category is intended to account for the reaction forces exerted on a curved highway bridge as vehicles go around the curve.

For most earth structures, the sum of the reaction forces developed at the roadway is so small compared to the mass of the underlying subgrade that centrifugal loading can be safely ignored; however, because EPS-block geofoam has such an extremely low density, the inertia of the fill mass may not be large enough to justify neglecting the centrifugal forces developed at a curved roadway surface. As with seismic loading, any lateral loads applied to EPS-block geofoam fill must be given special consideration to prevent shifting and shearing at the interfaces between layers of blocks.

Because the centrifugal loads on the roadway are directed away from the center of the curve, the centrifugal forces acting on the roadway may not always tend to act as a destabilizing force. This is a key difference between use of EPS-block geofoam in slopes as opposed to stand-alone embankments. If the

roadway curve is oriented in such a way that the center of the curve lies on the side of the roadway away from the slope, centrifugal forces generated by curving vehicles may actually push EPS fill back into the slope. This is, in essence, a stabilizing force acting on the slope. In view of this fact, it is recommended that centrifugal loads be considered only in the case of a curved roadway for which the center of the curve lies on the side of the roadway toward the slope. For instances where the roadway curve has its center on the side of the road opposite the slope, effects of centrifugal forces should not be taken into account. This practice will ensure a safe, conservative design for the slope and EPS fill.

Two potential failure mechanisms relating to centrifugal force loads were include horizontal sliding at some critical interface within the EPS fill system, and overturning of the entire fill about its toe. A sensitivity study based on AASHTO (2002) recommendations regarding highway geometry was performed to evaluate the failure mechanisms of horizontal sliding. This study, described in detail in Appendix D of the Project 24-11(02) report, indicated that for most EPS-block geofoam slope projects with roadways of two or more lanes that conform to AASHTO design standards for highway geometry, it is very unlikely that sliding due to centrifugal force loads would be critical. However, it is recommended that any projects involving a geosynthetic layer in the upper portion of the fill system, such as a geomembrane separation layer between EPS-block geofoam fill and the pavement system, should implement a testing program to determine the interface friction angle between EPS-block geofoam and the particular geosynthetic that is to be used.

It may also be advisable to perform a centrifugal force sliding analysis for projects involving very narrow roadways with a short radius curve. The analysis in Appendix D assumed the shear force that resisted sliding resulted from the weight of a pavement block that was roughly 20 ft. (6m) wide. Thus, any roadway that is narrower than 20 ft. (6m) should be checked to ensure that the weight of the pavement block beneath the applicable AASHTO design truck has sufficient weight to develop the necessary shear resistance at the critical interface.

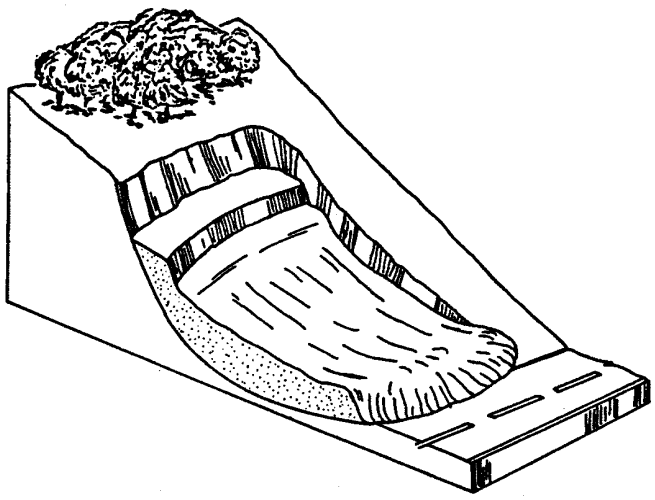
The second failure mechanism, overturning about the toe of EPS-block geofoam fill, could not be analyzed in any generalized way because the analysis is so heavily dependent on the specific geometry of the EPS-block geofoam fill system. However, because of the similarities between this failure mechanism and the failure mechanism of seismic overturning, it was concluded that projects which include an analysis of seismic overturning in the design process may neglect the consideration of overturning due to vehicle centrifugal forces. This conclusion is based on the fact that the analysis for seismic overturning considers essentially the same failure mechanism using different loads, which, in almost every instance, will be greater in magnitude than centrifugal force loads. Thus, if the factor of safety against seismic overturning is found to be acceptable, it can be inferred that the factor of safety against overturning due to vehicle centrifugal forces will be acceptable as well. However, it is recommended that projects that do not include an analysis of seismic overturning in the design process take into account the possibility of overturning due to vehicle centrifugal forces. This is especially true for those projects involving vertical-sided EPS fill.

B.4.6 Barrier & Railing Loads

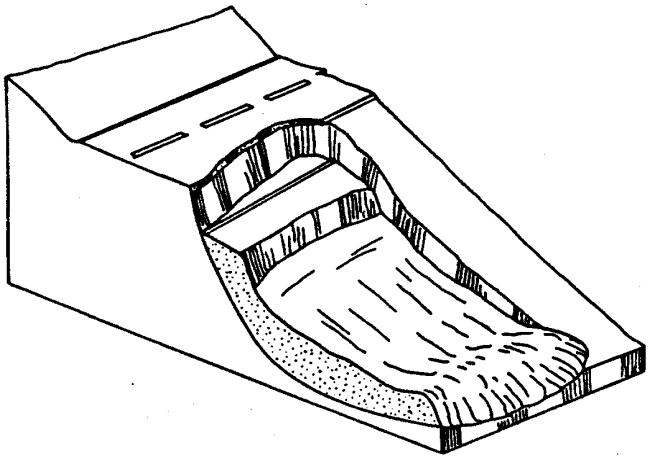
Road hardware such as guardrails, barriers, median dividers, lighting, signage and utilities can be incorporated in the EPS-block geofoam slope system by direct embedment or structural anchorage. The alternative for accommodating shallow utilities and road hardware (barriers and dividers, light poles, signage) is to provide a sufficient thickness of the pavement system to allow conventional burial or embedment within soil or, in the case of appurtenant elements, provide for anchorage to a PCC slab or footing constructed within the pavement section. Barriers or guardrails are typically required with vertical-sided embankments. Design of traffic railings is addressed in Section 13 of the Bridge Design Specifications (AASHTO, 2007) and in the AASHTO Road Design Guide (2002).

B.5 OVERVIEW OF DESIGN PROCEDURE

EPS-block geofoam has been widely used as a lightweight fill material to improve the stability of both soil and rock slopes. Both rotational (See Figure B.6) and translational (See Figure B.7) modes of sliding can occur in both soil and rock slopes. Based on a review of available case histories, it appears that a circular (rotational) failure surface is more common for lightweight fills applied to soil slopes, while a noncircular (translational) failure surface is more frequently applied to rock slopes.

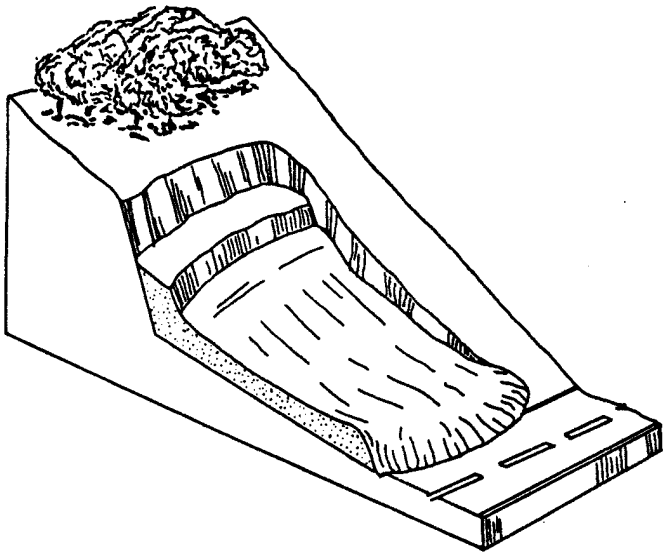


a) Rotational slide above roadway

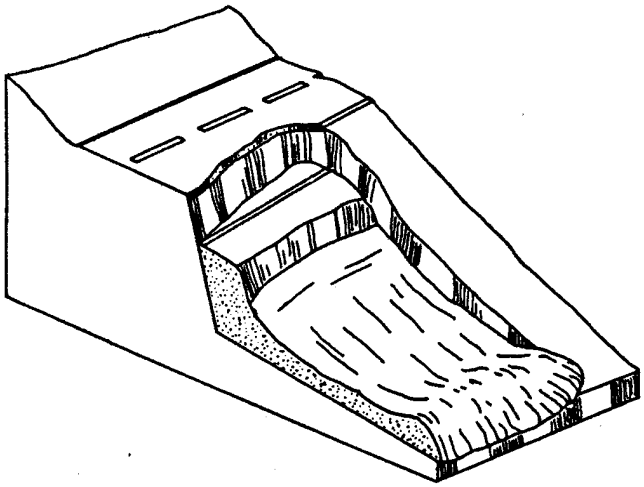


b) Rotational slide below roadway

Figure B.6. Rotational slides (Hopkins et al., 1988).



a) Translational slide above roadway



b) Translational slide below roadway

Figure B.7. Translational slides (Hopkins, et al., 1988).

Slope instability may involve engineered fill and cut slopes. If slope instability is anticipated due to proposed placement of a fill in the upper slope, use of EPS-block geofoam as a lightweight fill may be feasible. Use of lightweight fill in the upper portion of the cut slope prior to construction of the cut slope may contribute to stability of an otherwise unstable cut slope. Use of EPS-block geofoam in a proposed cut slope may be especially beneficial if a steep cut slope is required because of right-of-way constraints.

Figure B.8 shows the complete recommended design procedure for EPS-block geofoam slope fills.

Figure B.8. Complete design procedure for EPS-block geofoam slope fills.

All steps are required if the existing or proposed roadway is located *within* the limits of the existing or anticipated slide mass and/or the existing or anticipated slide mass is located *below* the roadway as shown in Figure B.6(b) and B.7(b). If the existing or proposed roadway is located *outside* the limits of the existing or anticipated slide mass and/or the existing or anticipated slide mass is located *above* the roadway as shown in Figure B.6(a) and B.7(a), the design procedure does not include Steps 8 and 9, which are directly related to design of the pavement system, because the EPS-block geofoam slope system may not include a pavement system. The pavement system failure mode may not be an applicable failure mode, because if the roadway is near the toe of the slide mass, stabilization of the slide mass with EPS-block geofoam will occur primarily at the head of the slide and, consequently, the EPS-block geofoam slope system may not include the pavement system. Therefore, Steps 8 and 9, which involve the pavement system, may not be required.

Design of an EPS-block geofoam slope system requires consideration of interaction between the three major components of an EPS-block slope system shown in Figure B.3, i.e., existing slope material, fill mass, and pavement system. Because of this interaction, the design procedure involves interconnected analyses between the three components. For example, some issues of pavement system design act in opposition to some design issues involving external and internal stability of an EPS-block geofoam slope system, because a robust pavement system is a benefit for long-term durability of the pavement system, but the larger dead load from a thicker pavement system may decrease the factor of safety of the failure mechanisms involving external and internal stability of the geofoam slope system. Therefore, some compromise between failure mechanisms is required during design to obtain a technically acceptable design.

However, in addition to technical aspects of the design, cost must also be considered. Because EPS-block geofoam is typically a more expensive material than soil on a cost-per-unit-volume basis for material alone, it is desirable to optimize the design to minimize the volume of EPS used, yet still satisfy technical design aspects of various failure mechanisms. It is possible, in concept, to optimize the final design of both the pavement system and overall EPS block slope system considering both performance and cost so a technically effective and cost-efficient geofoam slope system is obtained. However, because of the inherent interaction between components, overall design optimization of a slope incorporating EPS-block geofoam requires iterative analyses to achieve a technically acceptable design at the lowest overall cost. To minimize the iterative analysis, the design algorithm shown in Figure B.8 was developed. The design procedure depicted in this figure considers a pavement system with the minimum required thickness, a fill mass with the minimum thickness of EPS-block geofoam, and use of an EPS block with the lowest possible density. Therefore, the design procedure will produce a cost-efficient design.

The recommended design procedure is applicable for both slope-sided fills and vertical-sided fills as depicted in Figures B.1(a) and B.1(b), respectively, except that overturning of the entire fill mass at the interface between the bottom of the assemblage of EPS block and underlying foundation material as a result of horizontal forces that is part of external seismic stability, Step 6, is applicable primarily for only vertical-sided fills.

B.6 DESIGN PROCEDURE

B.6.1 Step 1: Background Investigation

The purpose of background investigation is to obtain and gather information required to determine the feasibility of using EPS-block geofoam as a lightweight fill alternative in the proposed slope, as well as to design and construct the EPS-block geofoam slope system. This step consists of project site evaluation and criteria selection.

The extent of site evaluation required on a project will be dependent on the type of project, i.e. new or existing structure. Turner and McGuffy (1996) provide general guidance for performing field investigations of landslides based on these two types of projects. Details about the site investigation process for landslides can be found in various landslide texts (Abramson et al., 2002, Cornforth, 2005,

Duncan and Wright, 2005, Transportation Research Board, 1996). Duncan and Wright (2005) provide recommendations for selection of a method of slope stabilization and repair. Additionally, the process of soil and rock property selection can be found in the FHWA Geotechnical Engineering Circular No. 5 (Sabatini et al., 2002). If the purpose of lightweight fill is to remediate an existing slide, the average shear strength along the existing slip surface can be back-calculated using back-analysis methods.

When attempting to evaluate the feasibility of using EPS-block geofoam for a slope stabilization project, it is important to consider the unique characteristics of EPS-block geofoam as a construction material. For example, experience has demonstrated that EPS-block geofoam can be placed extremely quickly. Once the site is prepared, the actual process of moving and positioning EPS blocks requires minimal equipment and labor. EPS-block geofoam blocks can be transported and placed easily, even at many project sites inaccessible to heavy equipment. Although some specific safety measures may have to be implemented, placement of EPS blocks can be continued in almost any kind of weather, whereas many other slope stabilization methods may be delayed by rain or snow.

Another important consideration is that EPS-block geofoam is a manufactured construction material which can be produced by the molder, and then stockpiled at a designated site until needed. Therefore, a state Department of Transportation (DOT) agency could potentially store a supply of EPS blocks that could be used for emergency landslide mitigation or repair. Also, EPS blocks can be molded in advance of the actual placement date and can be either transported immediately when needed, or stockpiled at the site for immediate use. Thus, use of EPS blocks in slope application projects can easily contribute to an accelerated construction schedule.

In addition to site evaluation, Step 1 includes criteria selection. Establishment of project criteria involves selecting a desired design life for the proposed EPS-block geofoam slope system, estimating design loads, selecting an appropriate factor of safety with respect to the various failure mechanisms that must be considered in the design, selecting desired settlement tolerances, and evaluating geometric requirements of the proposed EPS-block fill mass.

It is recommended that similar design life ranges recommended for reinforced soil slopes also be adopted for EPS-block geofoam slope systems. Permanent EPS-block geofoam slope systems can be designed for a minimum service life of 75 years, and temporary systems can be designed for a service life of 36 months or less (Elias et al., 2001). However, it should be noted that based on the current state of knowledge, the actual design life may be greater than 75 years because EPS is inherently non-biodegradable and will not dissolve, deteriorate, or change chemically in the ground or ground water. Also, based on the current state of knowledge, no maintenance of EPS blocks is required.

Guidelines for selection of design loads were presented in Section B.4. Recommended minimum factors of safety for use in analyzing the various failure mechanisms of the design procedure shown in Figure B.8 are included in the appropriate design step section.

An evaluation of geometric requirements of the proposed EPS-block fill mass considers requirements of the local transportation agency and limitations due to site-specific restrictions. These requirements and restrictions will be used in Step 3 to select a preliminary fill mass arrangement. They will also be considered throughout the remaining steps of the design process as various iterations of the fill mass arrangement are evaluated to obtain a fill mass arrangement that will satisfy the design criteria for various failure mechanisms analyzed in each design step. Project-specific design inputs, such as right-of-way constraints, limiting impact on underlying and/or adjacent structures, and construction time usually govern the overall cross-sectional geometry of the fill. For example, use of a vertical-sided fill will minimize the impact to nearby structures, including underground utilities. An assessment should be made of any adjacent structures, utilities and transportation facilities (roads, railroads), both existing and proposed, that may be affected by loads imposed on the ground by the proposed EPS-block geofoam slope system.

B.6.2 Step 2: Select a preliminary type of EPS and assume preliminary pavement system design

The second step of the design procedure is to select a preliminary type of EPS-block geofoam and to design a preliminary pavement system. As indicated in Section B.4.2, although EPS-block geofoam can be manufactured to various densities, the preliminary design can be based on a density of 20 kg/m^3 (1.25 lbf./ft^3). Therefore, the dry unit weight of EPS can be taken to be 200 N/m^3 (1.25 lbf./ft^3), or a unit weight at 1 percent water content by volume of 300 N/m^3 (1.9 lbf./ft^3) for preliminary design. However, for some failure mechanisms, unit weight based on long-term water absorption, such as the unit weight at 1 percent water content may be more appropriate. It should be noted that these water contents are expressed on a volumetric basis rather than a gravimetric basis. That is, water content of EPS is given as the ratio of the volume of air contained in the sample to its total volume. This volumetric expression of water content, although rarely used in geotechnical engineering, is common in the foam manufacturing industry.

Although the pavement system has not been designed at this point, the preliminary pavement system can be assumed to be 1m (3 ft.) thick, and the various component layers of the pavement system can be assumed to have a total unit weight of 20 kN/m^3 (130 lbf./ft^3) for initial design purposes. The final pavement system will be based on a pavement structural analysis as part of Step 8.

B.6.3 Step 3: Optimize Volume & Location of EPS Fill

The third step of the design procedure is to determine a preliminary fill mass arrangement. Because EPS-block geofoam is typically more expensive than soil on a cost-per-unit-volume basis for material alone, it is usually desirable to optimize the volume of EPS used, yet still satisfy design criteria concerning stability. Therefore, to achieve the most cost-effective design, a design goal for most projects is to use the minimum amount of EPS blocks possible that will satisfy requirements for external and internal stability. The analyses of all external and internal stability failure mechanisms shown in Table B.1 are based on verifying that the initial depth and extent of existing slope material removal and the resulting EPS-block geofoam fill configuration will provide an overall stable slope.

Determination of optimal volume and location of EPS block will typically require iterative analysis based on various locations and thicknesses until a cross-section that yields the minimum volume of lightweight fill is obtained. However, other factors will also impact the final design volume and location of EPS block such as:

- Construction equipment access to perform excavation work,
- Ease of accessibility for EPS block delivery and placement,
- Impact on traffic if lightweight fill will be incorporated below an existing roadway, and
- Right-of-way constraints and/or constraints due to nearby structures.

It should be noted that although minimization of EPS volume is the goal on most projects, for some projects it may be desirable to maximize use of EPS. For example, economization of EPS volume may not be a concern in some emergency slope repair projects or projects with an accelerated construction schedule.

A minimum of two layers of block should be used beneath roads, because a single layer of blocks can shift under traffic loads and lead to premature failure (Horvath, 1999). Block thicknesses typically range between 610mm (24 in.) to 1000mm (39 in.). Therefore, it is recommended that a minimum of two EPS blocks with a thickness of 610mm (24 in.) each, or a total initial height of 1.2m (4 ft.) be considered for EPS block height to determine the preliminary fill mass arrangement. Therefore, the preliminary fill mass arrangement can consist of the preliminary pavement system thickness of 1m (3 ft.) and the thickness of two EPS blocks of 1.2m (4 ft). The thickness of EPS-block geofoam may change as various iterations of the fill mass arrangement are evaluated to obtain a fill mass arrangement that will satisfy the

design criteria of various failure mechanisms that are analyzed in each supplemental design step shown in Figure B.8.

For engineered fill embankments constructed on slopes, if the previously suggested initial preliminary fill mass arrangement will not yield the required finished grade and additional fill material is required, the preliminary fill mass arrangement can consist of the pavement system, EPS-block geofoam, and an underlying layer of natural fill. The preliminary height of natural fill is the total embankment height required based on the background investigation, less the preliminary pavement system thickness of 1m (3 ft.), and less the thickness of two EPS blocks of 1.2m (4 ft.).

The preliminary width and location of the EPS-block geofoam fill mass within the slope will be dependent on the results of evaluation of the preliminary geometric requirements of the proposed EPS-block fill mass performed as part of Step 1. The most effective location of the lightweight fill mass will be near the head (upper portion) of the existing slide mass or proposed slope, because reducing the load at the head by removing existing earth material and replacing it with a lighter fill material contributes the most to reducing destabilizing forces that tend to cause slope instability. Location of the fill mass within the slope selected in this step is only preliminary, because the location of the fill mass, as well as the thickness, may change as various iterations of the fill mass arrangement are evaluated to obtain a fill mass arrangement to satisfy the design criteria of various failure mechanisms analyzed in each supplemental design step shown in Figure B.8.

In some projects the volume and location of EPS blocks within the slope will be constrained by site- and/or project-specific factors. For example, for the case of the existing road located within the existing slide mass and existing slide mass located below the roadway as shown in Figures B.6(b) and B.7(b), i.e., the roadway is near the head of the slide mass, location of the EPS fill mass will typically be limited within the existing roadway location because of right-of-way constraints. However, in some projects the volume and location of EPS within the slope may not be obvious, and may require that various iterations of the fill mass arrangement be evaluated to obtain a fill mass arrangement that will satisfy the design requirements of various failure mechanisms that are analyzed in each design step shown in Figure B.8. Therefore, as part of this Project 24-11(02), a study was performed to develop a procedure for optimizing the volume and location of EPS blocks within the slope to minimize the number of iterations that may be required to satisfy the design criterion.

Appendix C of the Project 24-11(02) report presents two methods for optimizing the volume and location of EPS block within the slope, one for slides involving rotational, and the other for translational slides. The purpose of the optimization methods is only to obtain an approximate location within the slope where placement of EPS block will have the greatest impact in stabilizing the slope, while requiring the minimum volume of EPS block. A separate static slope stability analysis must be performed as part of Step 5 of the design procedure, as shown in Figure B.8, with a better slope stability analysis method that preferably satisfies full equilibrium, such as Spencer's method. Step 5 should be relied on to verify that the overall slope configuration meets the desired factor of safety.

The optimization procedures presented in Appendix C are optional within the proposed design procedure shown in Figure B.8. In lieu of performing one of the optimization procedures, the designer can select a preliminary volume and location of EPS blocks within the slope and proceed with Step 5.

B.6.4 Step 4: Modify Optimized EPS Fill as Needed for Constructability

This step consists of evaluating the EPS block configuration obtained from the optimization procedures described in Appendix C and performing minor alterations to the fill mass configuration to ensure that the fill mass is constructible. As will be apparent upon examination of the optimization procedures described in Appendix C, it is unlikely the results of the optimization procedure can be used to obtain the final EPS block configuration because the configuration obtained from the optimization procedures described in Appendix C may be impractical to replicate in the field. In some cases, the geometry of the optimized EPS fill may simply be too complicated to be manufactured or constructed with a reasonable amount of time and effort. However, the optimized EPS-block geofoam fill geometry

obtained will still provide a useful starting point from which to design an EPS geofoam fill that is both efficient and constructible. Minor alterations may be required to adapt the optimized EPS fill design to the specific project requirements and site restrictions.

B.6.5 Step 5: Static Slope Stability (External)

Once a preliminary fill mass arrangement is determined, then the primary failure mechanism that needs to be evaluated is overall (external) slope stability, because this is typically the primary reason for considering use of EPS block in slope applications. Thus, as shown in Figure B.8, overall external static slope stability is Step 5 of the proposed design procedure.

The purpose of this step is to analyze the EPS-block geofoam fill configuration that was developed in Steps 2 through 4 to ensure that the proposed slope system will have an acceptable factor of safety. In most cases, this required factor of safety will be specified by the policy of the supervising agency, such as the state DOT. Typical values for the required factor of safety range from as low as 1.15 for slide remediation projects to as high as 1.5 for new slope construction.

The procedure for evaluation of external static slope stability will typically consist of the following two phases:

- Evaluate existing slope conditions.
- Evaluate the proposed stability of the slope for various lightweight fill configurations and determine the optimum quantity and location of EPS-block geofoam that will yield the desired stability. The design objective in this phase is to determine an optimum EPS block configuration and location within the slope that will result in the lowest cost. Thus, it is desirable to minimize the volume of EPS used, yet still satisfy technical design aspects of external static slope stability. If the desired stability is not obtained using lightweight fill alone, consider additional remediation alternatives in conjunction with lightweight fill.

Design for external static stability considers potential slip surfaces involving the existing soil slope material only, as shown in Figure B.4, as well as potential slip surfaces that involve both the fill mass and existing slope material, as shown in Figure B.5. Of course, if the use of lightweight fill is to remediate an existing slide, the existing slip surface identified as part of Step 1 will be the critical slip surface that should be evaluated. However, slip surfaces such as those shown in Figures B.4 and B.5 should also be considered to evaluate the impact of the lightweight fill mass configuration on the short- as well as long-term stability of the slope.

Conventional limit equilibrium methods can be used to evaluate the stability of potential slip surfaces involving existing soil slope material only, as shown in Figure B.4. Slip surfaces such as slip surface 1 and 3 evaluate the impact of the EPS block system on the overall stability of the slope and stability of the lower portion of the slope, respectively. Slip surface 2 evaluates both the short-term and long-term behavior of the upper slope immediately adjacent to the EPS-block fill system to ensure that no applied earth forces from the adjacent earth are applied to the EPS fill. The design procedure is based on a self-stable adjacent upper slope. If the adjacent upper slope material cannot be cut to a long-term stable slope angle, an earth-retention system must be used to resist the applied earth force. Various types of earth-retention systems are included in Chapter 4 of the Project 24-11(02) report.

Limit equilibrium methods can also be used to evaluate the external static stability of slip surfaces involving both the fill mass and existing slope material, as shown in Figure B.5. However, one current disadvantage of using limit equilibrium methods of analysis to evaluate slip surfaces that extend through the fill mass and existing slope material is the uncertainty in modeling the strength of the EPS blocks. Chapter 3 of the Project 24-11(02) report presents a summary of various methods available for modeling shear strength of EPS block for external static slope stability.

As shown in Figure B.5, potential slip surfaces that extend through both the fill mass and existing slope material to be evaluated include slip surfaces through the upper slope and EPS block (slip surface 2)

as well as slip surfaces through the EPS block and lower slope material (slip surface 1). However, if the upper slope material is designed to be stable for both short-term and long-term conditions, an analysis of a potential slip surface, as shown by slip surface 2 in Figure B.5, should yield a stable condition.

Additionally, note that a slip surface entirely within the EPS blocks need not be considered, and is not shown in Figure B.5 because there is little or no static driving force applied along the horizontal portion of the internal failure surfaces, since the horizontal joints are assumed to be completely horizontal and typical static loads are vertical. The fact that embankments with vertical sides can be constructed demonstrates the validity of this conclusion. Therefore, the focus of external static stability analyses involving both the fill mass and existing slope material will be on slip surfaces involving EPS block and the lower slope material, as depicted by slip surface 1 in Figure B.5.

Only circular slip surfaces are shown in Figures B.4 and B.5. Based on a review of available lightweight fill case histories, it appears that circular (rotational) slip surfaces are more common for lightweight fills applied to soil slopes, while noncircular (translational) slip surfaces are more frequently associated with rock slopes. This holds true for EPS-block geofoam as well as other types of lightweight fill material.

B.6.6 Step 6: Seismic Slope Stability (External)

B.6.6.1 Overview

Potential failure mechanisms associated with the external seismic instability failure mode include slope instability involving slip surfaces that include existing slope material only, as shown in Figure B.4, and/or both the fill mass and existing slope material as shown in Figure B.5, horizontal sliding of the entire EPS-block geofoam fill mass as shown by Figure B.9, overturning of a vertical-sided embankment as shown by Figure B.10, bearing capacity failure of the existing foundation earth material as shown in Figure B.11, and settlement of the existing foundation material as shown by Figure B.12.

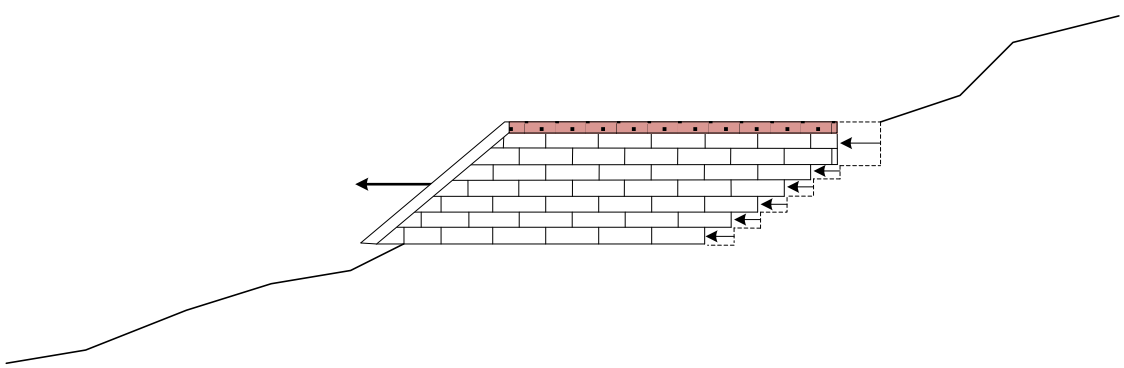


Figure B.9. External seismic stability failure involving horizontal sliding of the entire embankment.

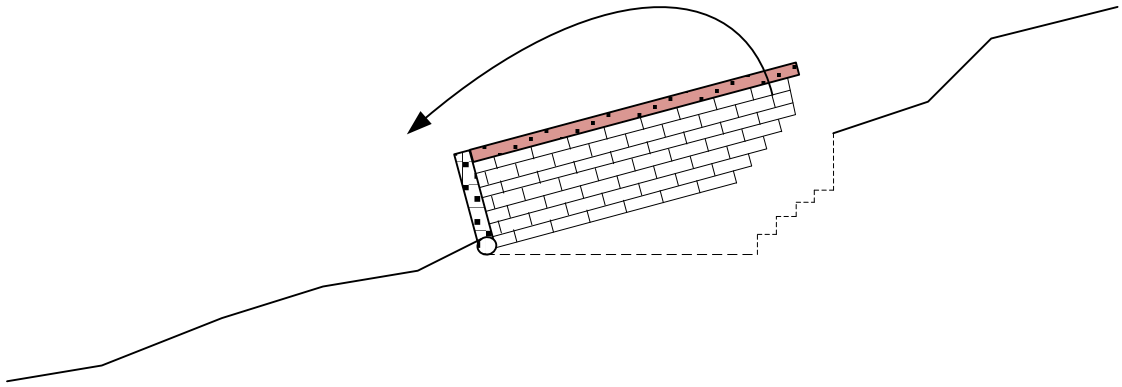


Figure B.10. External seismic stability failure involving overturning of an entire vertical embankment about the toe of the embankment.

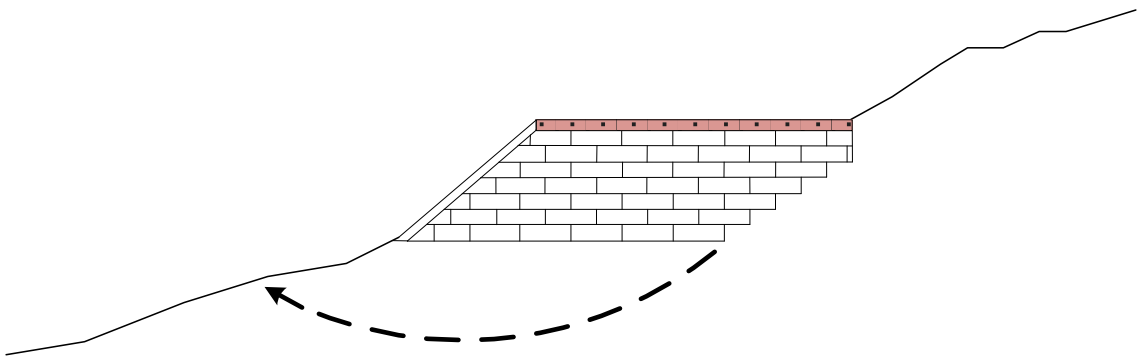


Figure B.11. Bearing capacity failure of the embankment due to general shear failure or local shear failure.

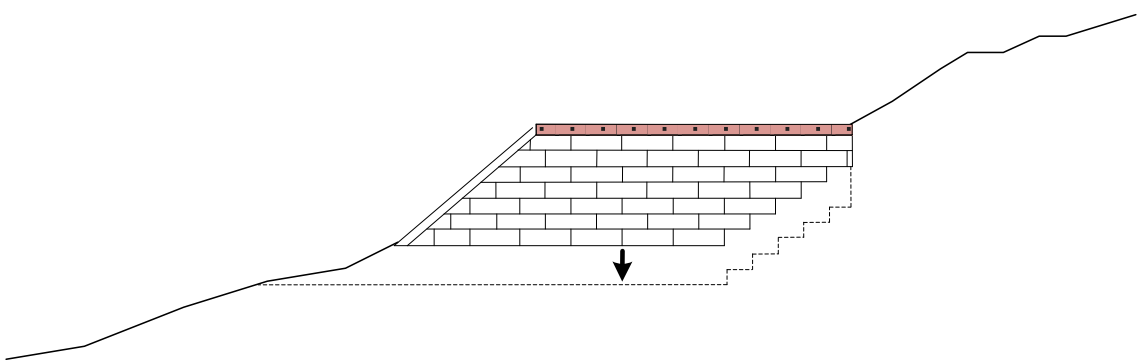


Figure B.12. Excessive settlement.

The pseudo-static stability analysis procedure that is typically used to evaluate seismic stability of soil slopes and embankments can also be used to evaluate external seismic stability of EPS-block geofoam slopes. This method involves modeling the earthquake shaking with an equivalent static horizontal and/or vertical force (F_h and F_v, respectively) that acts permanently, not temporarily, on the slope. The horizontal and vertical forces (F_h and F_v, respectively) equal the slide mass or the mass of the vertical slice (m) multiplied by the appropriate seismic acceleration (a_h or a_v), i.e., $F = m \cdot a$, as shown

by Equations B.3 and B.4. These equations also show the relation between the seismic accelerations and seismic coefficients, which are sometimes referenced in seismic stability literature, and may also be provided by local seismic design codes and guidelines.

$$F_h = \frac{a_h W}{g} = k_h W \quad \textbf{(B.3)}$$

$$F_v = \frac{a_v W}{g} = k_v W \quad \textbf{(B.4)}$$

where

F_h, F_v	=	equivalent static force in the horizontal or vertical direction, respectively
a_h, a_v	=	selected ground acceleration, usually some fraction of PGA
W	=	weight of EPS geofoam fill + weight of pavement system
g	=	acceleration due to gravity
k_h, k_v	=	horizontal and vertical seismic coefficients, respectively

The selected ground acceleration can be obtained by performing a seismic ground shaking hazard analysis. As noted by the WSDOT, the four types of seismic ground shaking hazard analysis include: (1) use of a specification/code based hazard with specification/code based ground motion response, (2) use of a specification/code based hazard with site-specific ground motion response, (3) use of site-specific hazard with specification/code based ground motion response, and (4) use of site-specific hazard with site-specific ground motion response (Washington State Department of Transportation, 2010). The AASHTO specifications (2010) provide the conditions when a site-specific hazard analysis and/or a site-specific ground motion response analysis should be considered. Additionally, the AASHTO 2010 specifications also provide a procedure for performing a seismic ground shaking hazard analysis based on a specification/code based hazard with specification/code based ground motion response analysis.

The primary assumption of the pseudo-static analysis procedure is that all effects of the ground motion produced by the earthquake can be accounted for by incorporating static forces determined from Equations B.3 and B.4 into the analysis of the various potential failure mechanisms involving seismic stability of the EPS-block geofoam fill mass.

As shown in Figures B.4 and B.5, potential slip surfaces involving the existing slope material only, as well as slip surfaces involving both the fill mass and existing slope material, should be considered as part of external seismic slope stability. A pseudo-static seismic slope stability analysis involves application of a horizontal and/or vertical force to the center of gravity of the critical slide mass and in the direction of the exposed slope. If a stability method is used that involves dividing the slide mass into vertical slices, the horizontal and vertical forces are applied to the center of gravity of each vertical slice that simulates inertial forces generated by the ground motion. The pseudo-static horizontal and vertical force must be applied to the slide mass that is delineated by the critical static failure surface.

In addition to evaluation of seismic slope stability of the overall EPS-block fill mass, i.e., external seismic stability, failure mechanisms of horizontal sliding of the entire EPS-block fill mass, overturning of vertical embankments, bearing capacity failure of the existing foundation, and settlement of existing foundation material due to seismic loading should also be evaluated.

B.6.6.2 Evaluation of Seismic Slope Stability of the Overall EPS Block Fill Mass

The steps in a pseudo-static slope stability analysis are summarized below.

1. Locate the critical static failure surface(s), i.e., the static failure surface with the lowest factor of safety that passes through the existing slope material only, as well as a slip surface involving both the fill mass and existing slope material, using a slope stability method that satisfies all conditions of equilibrium, e.g., Spencer's stability method (1967). This value of factor of safety should satisfy the required value of static factor of safety before initiating the pseudo-static analysis.
2. Modify static shear strength values for cohesive or liquefiable soils situated along the critical static failure surface to reflect a strength loss due to earthquake shaking.
3. Determine the equivalent horizontal and vertical force using Equations B.3 and B.4 that will be applied to the center of gravity of the critical static failure surface. Some slope stability software programs will calculate these static forces directly from the seismic accelerations. If a stability method is used that involves dividing the slide mass into vertical slices, the horizontal and vertical forces are applied to the center of gravity of each vertical slice.
4. Calculate the pseudo-static factor of safety, *F'*, for the critical static failure surface and ensure it meets the required value. The Naval Facilities Engineering Command (1986) indicates that for transient loads, such as earthquakes, safety factors as low as 1.2 or 1.15 may be tolerated. Day (2002) indicates that in southern California, a minimum factor of safety of 1.1 to 1.15 is considered acceptable for a pseudo-static slope stability analysis. A factor of safety between 1.0 and 1.2 is indicated by Kavazanjian et al. (1997). WSDOT recommends a minimum factor of safety of 1.1 for slopes involving or adjacent to walls and structure foundations, and a minimum factor of safety of 1.05 for other slopes (cuts, fills, and landslide repairs) (WSDOT, 2010). The safety of factor required will most likely vary from state to state. Therefore, local DOT factor of safety requirements for seismic stability should be used.

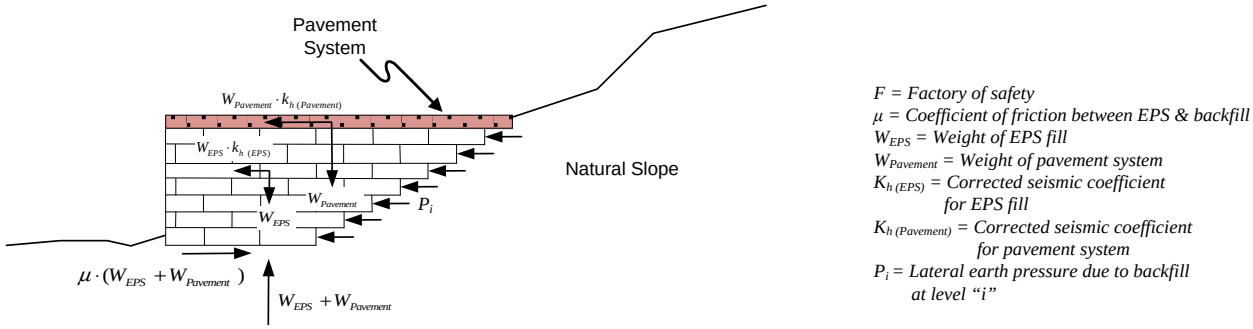
The vertical pseudo-static force is typically neglected in pseudo-static analysis of earth slopes because the vertical force generally alternates between reducing and increasing both the driving and resisting forces in the slope. Consequently, it generally has a significantly smaller influence on the resulting factor of safety than the horizontal force (Kramer, 1996).

B.6.6.3 Horizontal Sliding

Horizontal sliding analysis considers potential sliding of the entire EPS-block geofoam fill mass, as shown by Figure B.9. Figure B.13 shows the recommended model for evaluating external sliding stability.

The recommended procedure for considering the effect of natural slope material on external and internal seismic stability of the EPS-block geofoam slope system consists of determining the magnitude of the seismic earth pressure based on the Mononobe-Okabe (M-O) method. Under seismic conditions, the EPS-block fill mass will be subjected to both static forces and inertia forces from both the backfill and natural slope material based on the M-O method.

Figure B.14 shows the assumed back of wall of the EPS-block geofoam fill mass used in the M-O method. As shown, the back of wall is assumed to be along the bottom exterior edges of EPS blocks. Figure B.14 also shows the wall geometry, wedge force diagram, and the forces associated with the M-O method.



$$F = \frac{\mu (W_{EPS} + W_{Pavement})}{W_{EPS} \cdot k_{h\,(EPS)} + W_{Pavement} \cdot k_{h\,(Pavement)} + \Sigma P_i} \geq Allowable\ \ value$$

Figure B.13. External seismic stability for sliding (Public Works Research Institute, 1992).

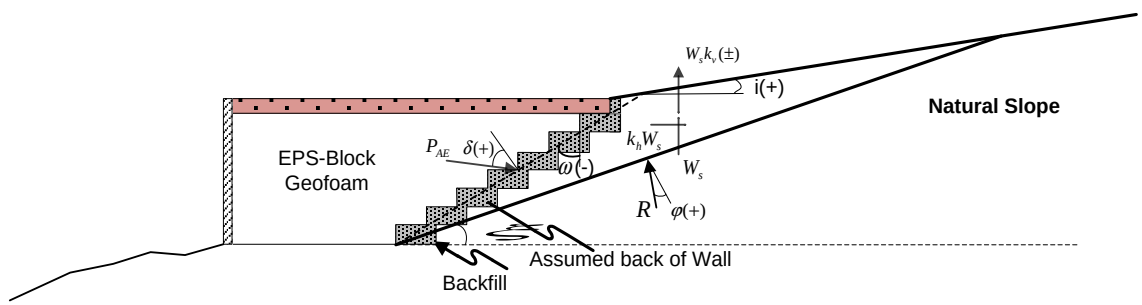


Figure B.14. Forces behind the EPS-block geofoam mass in the Mononobe-Okabe Method.

Equation B.5 provides the total (static and dynamic) earthquake active earth pressure coefficient, K_{AE} and Equation B.6 provides the M-O relationship for the total active earth pressure force.

$$K_{AE} = \frac{\cos^2(\phi - \psi - \omega)}{\cos\psi \cos^2\omega \cos(\delta + \omega + \psi)} \times \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \psi - i)}{\cos(\delta + \psi + \omega)\cos(i - \omega)}} \right]^{-2} \qquad \textbf{(B.5)}$$

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 \left(1 - k_v\right), \qquad \textbf{(B.6)}$$

where

ϕ = friction angle of soil (deg),

δ = friction angle between soil and assumed wall (deg),

$\psi = \arctan(\frac{k_h}{1 - k_v})$,

ω = slope of the asummed back of wall to the vertical (deg) (Note that ω is negative in sign in this case.),

i = back of wall slope angle (deg),

k_h = horizontal acceleration coefficient,

k_v = vertical acceleration coefficient,

P_{AE}=total (static and dynamic) active earth pressure force,

R=resultant of the shear and normal forces on the surface of failure,

W_s=effective weight of the soil wedge,

H = Height of the soil face, and

γ = Unit weight of soil.

As shown in Figure B.14, the seismic inertia force, represented by $W_s k_v (\pm)$, can act in an upward or downward direction, and both cases should ideally be considered. As noted in the NCHRP Report 611(Anderson et al., 2008), the effect of vertical seismic loading is traditionally neglected. The rationale for neglecting vertical loading is generally attributed to the fact that higher frequency vertical accelerations will be out of phase with the horizontal accelerations, and will have positive and negative contributions to wall pressures, which on average, can reasonably be neglected for design.

The M-O method provides the magnitude of the total (static and dynamic) active seismic earth pressure force, but not a specific force location nor an equivalent pressure distribution behind the wall. The location of the resultant force and an equivalent earth pressure distribution are needed for external and internal seismic stability analysis of EPS-block geofoam slopes. The M-O total (static and dynamic) earthquake active earth pressure can be separated into a static and a seismic component. Richards and Elms (1979) indicate that Seed and Whitman (1970) suggested that the resultant of the static component of active earth pressure acts at H/3 from the bottom of the wall, and that the dynamic effect can be taken to act at a height of 0.6H above the base. The recent AASHTO manual also indicates that the resultant of the static component of the active earth pressure with no earthquake effects may be taken as H/3 (AASHTO, 2010). The typical pressure distribution that is assumed for the static active earth pressure is a triangular pressure distribution that is 0 at the top and $K_A \gamma H$ at the bottom.

The WSDOT geotechnical design manual (2010) also indicates that the resultant of the dynamic component is 0.6H from the bottom and that the pressure distribution for the dynamic effect is an inverted trapezoid with the pressure at the top of $0.8\Delta K_{ae}\gamma H$ and the pressure at the bottom of $0.2\Delta K_{ae}\gamma H$. Therefore, the locations of the static and seismic component of active earth pressure shown in Figure B.15 are recommended for analysis of geofoam slope systems. Equation B.7 provides the static component of the active earth pressure coefficient.

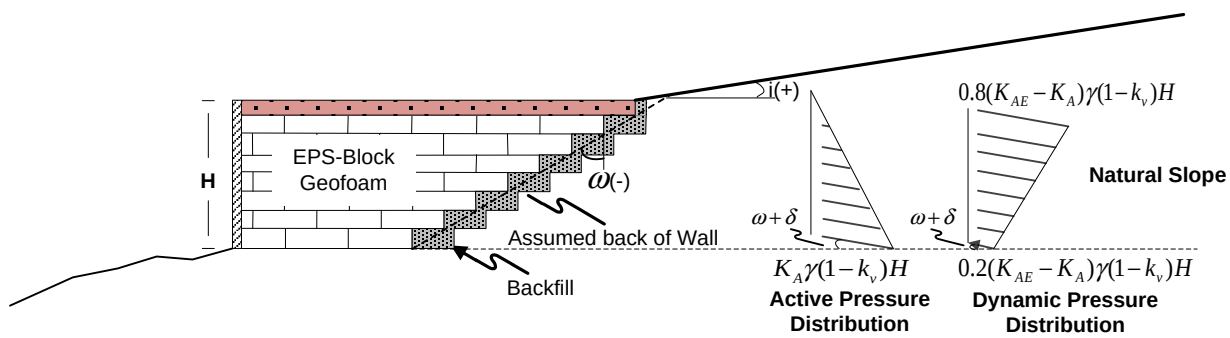


Figure B.15. Static and dynamic components of active earth pressure.

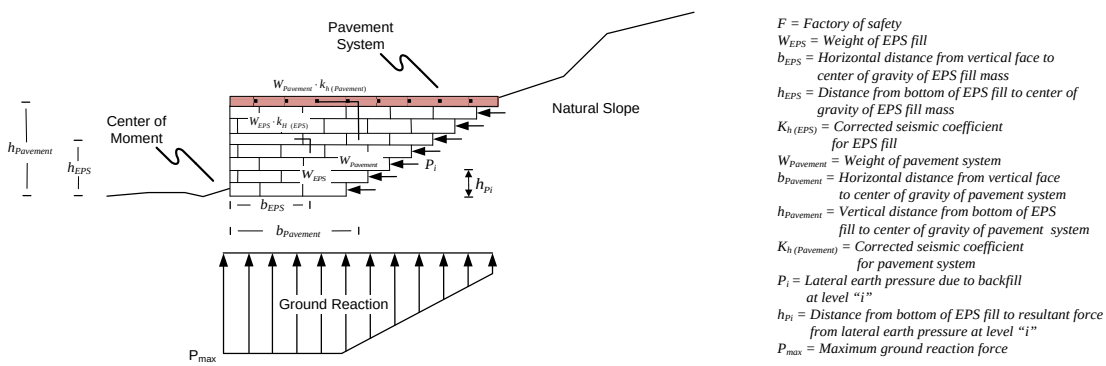
$$K_A = \frac{\cos^2(\phi-\omega)}{\cos^2\omega\cos(\delta+\omega)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-i)}{\cos(\delta+\omega)\cos(i-\omega)}}\right]^2} \qquad \textbf{(B.7)}$$

Note that in the M-O equation for calculating the total active force (see Equation B.6), the unit weight of the soil is multiplied by 1-K_v to account for seismic acceleration effects. Therefore, to maintain consistency with the M-O equation, the pressure diagrams shown in Figure B.15 include the 1-K_v correction for both the static as well as the dynamic components of earth pressure.

After the static and seismic (dynamic) components of active earth pressure are determined, the external seismic stability failure mechanism of horizontal sliding can be analyzed by using Figure B.13.

B.6.6.4 Overturning

For tall and narrow vertical-sided slope fills, overturning of the entire embankment at the interface between the bottom of the assemblage of EPS blocks and underlying foundation material as a result of seismic forces should be considered as depicted in Figure B.10. Figure B.16 shows the recommended model for evaluating external overturning.



$$F = \frac{W_{EPS} \cdot b_{EPS} + W_{Pavement} \cdot b_{Pavement}}{W_{EPS} \cdot k_{h\,(EPS)} \cdot h_{Pavement} + W_{Pavement} \cdot k_{h\,(Pavement)} \cdot h_{Pavement} + \Sigma P_i \cdot h_{Pi}} \geq Allowable\ value$$

Figure B.16. Seismic stability for overturning (tipping) and bearing capacity (Public Works Research Institute, 1992).

B.6.6.5 Bearing Capacity

Bearing capacity failure of the existing foundation earth material as shown in Figure B.11 due to seismic loading and, potentially, a decrease in shear strength of foundation material can be considered using the model shown in Figure B.16. The Japanese design manual indicates that if the width of the bottom of the EPS-block fill mass is small compared with the height, and if the area is susceptible to rocking, the required factor of safety against bearing capacity must be increased (Public Works Research Institute, 1992). However, no further guidance is provided about the magnitude of factor of safety increase to utilize.

B.6.6.6 Settlement

Potential settlement of the existing foundation material as shown by Figure B.12 should also be considered. The settlement that is performed as part of external seismic stability analysis considers earthquake-induced settlements, such as those resulting from liquefaction, seismic-induced slope movement, regional tectonic surface effects, foundation soil compression due to cyclic soil densification, and increase in vertical stress due to dynamic loads caused by rocking of the fill mass (Day, 2002). Methods for considering these earthquake-induced settlements can be found in various references (Day, 2002, Kavazanjian et al., 1997).

B.6.7 Step 7: Seismic Stability (Internal)

B.6.7.1 Overview

The main difference between internal and external seismic stability is that in internal seismic stability analysis, sliding is assumed to occur only within the EPS fill mass. Two failure mechanisms that involve internal stability of the EPS-block geofoam fill mass include horizontal sliding between blocks and/or between the pavement system and upper layer of blocks, as shown by Figure B.17, and load-bearing failure of the EPS blocks due to the inertia stresses, as shown in Figure B.18.

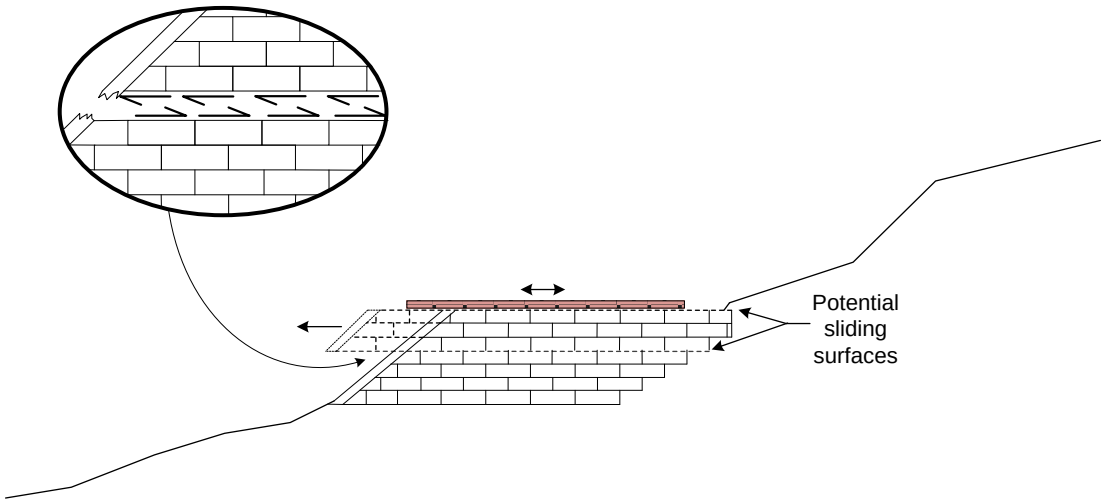


Figure B.17. Internal seismic stability failure involving horizontal sliding between blocks and/or between the pavement system and upper layer of blocks due to seismic loading.

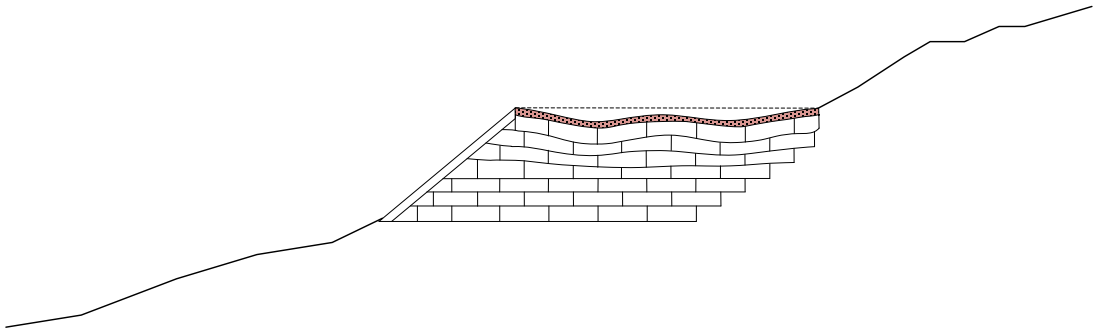


Figure B.18. Load-bearing failure of the blocks involving excessive vertical deformation.

Internal seismic stability design of EPS-block geofoam slopes consists of determining the seismic-response acceleration of the existing natural slope material and the EPS-block geofoam fill mass and evaluating the various potential failure mechanisms indicated above. The current state-of-practice of internal seismic stability analysis is to decouple the determination of the overall seismic response acceleration into the determination of the seismic response of natural slope material, followed by the seismic response of the EPS-block fill mass. Additionally, it is current state-of-practice to evaluate each potential seismic failure mechanism separately. Therefore, internal seismic stability analysis and design of EPS-bock geofoam slope systems can be separated into the following three primary steps: (7i) estimating the seismic-response acceleration at the existing ground surface or base (subgrade level) of the EPS fill

mass by performing a site-specific assessment, (7ii) estimating the seismic-response acceleration at the top of the EPS fill mass, (7iii) performing pseudo-static stability analyses of the various failure mechanisms.

B.6.7.2 Estimating the Seismic-Response Acceleration of Existing Ground Surface or Base (Subgrade Level) of the EPS fill mass (Step 7i)

As described in section B.6.6.1, the selected ground acceleration can be obtained by performing a seismic ground shaking hazard analysis. As noted by the WSDOT, the four types of seismic ground shaking hazard analysis include: (1) use of a specification/code based hazard with specification/code based ground motion response, (2) use of a specification/code based hazard with site-specific ground motion response, (3) use of site-specific hazard with specification/code based ground motion response, and (4) use of site-specific hazard with site-specific ground motion response (Washington State Department of Transportation, 2010). The AASHTO specifications (2010) provide the conditions when a site-specific hazard analysis and/or a site-specific ground motion response analysis should be considered. Additionally, AASHTO 2010 specifications also provide a procedure for performing a seismic ground shaking hazard analysis based on a specification/code based hazard with specification/code-based ground motion response analysis. Figure B.19 shows a typical site-modified acceleration response spectrum.

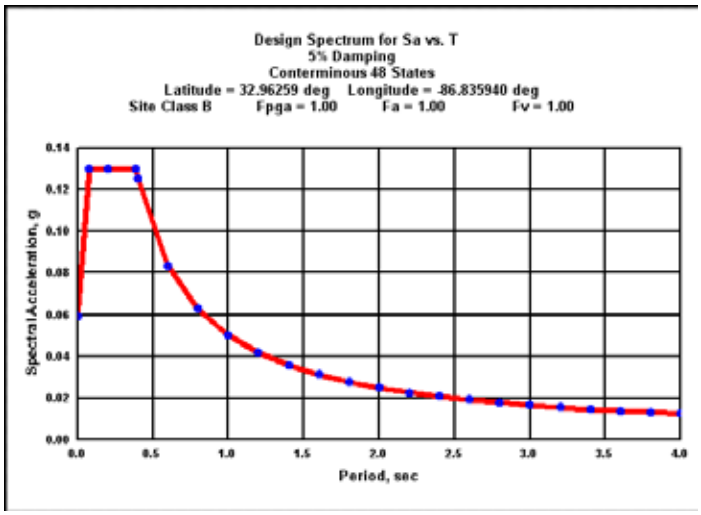


Figure B.19. Example of a site-modified acceleration response spectrum.

B.6.7.3 Estimating Seismic-Response Acceleration at the Top of the EPS Fill Mass (Step 7ii)

After estimating the free surface motion acceleration in Step 7i, seismic-response acceleration at the top of the EPS-block geofoam fill mass must be estimated. The two approaches available for determining the seismic-response acceleration of the EPS-block geofoam fill mass can be categorized into a simplified method and a detailed method (Horvath, 1995). Only the simplified seismic response method is included herein. For EPS-block geofoam slopes located such that failures have the potential to be especially catastrophic, a more rigorous analysis of seismic stability, e.g., numerical analysis, may be required.

The simplified method of determining the seismic-response acceleration of the EPS-block geofoam fill mass consists of the following steps: a) calculate the fundamental period of the EPS-block geofoam fill mass system, b) determine the site-response acceleration of the EPS-block geofoam fill mass system, c) determine the seismic coefficients, d) calculate the seismic-inertia force produced by the

horizontal acceleration acting on the lumped mass of the single-degree-of-freedom (SDOF) system (Horvath, 1995, Riad and Horvath, 2004).

B.6.7.3.1a) Calculate the Fundamental Period of the EPS-Block Geofoam Fill Mass System. Figure B.20 depicts the SDOF model. As shown, the EPS-block fill structure can be modeled as an equivalent fixed-end, cantilevered beam.

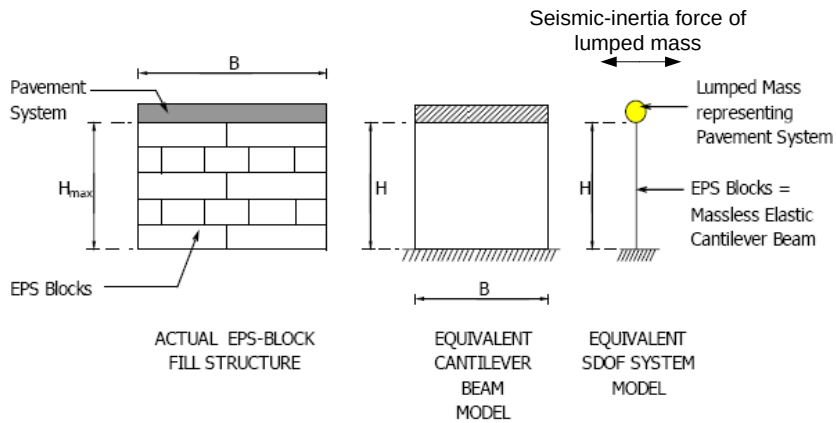


Figure B.20. SDOF model for EPS (Riad, 2005; used with permission from ASCE).

The fundamental period of the equivalent SDOF system model can be obtained from Equation B.8. Any consistent set of units may be used with this equation. As noted by Horvath (2004), both flexural and shear components of bending are considered in the derivation of the equation:

$$T_0 = 2\pi\left\{\frac{\sigma'_{v_0} H}{E_{t_i} g}\left[4\left(\frac{H}{B}\right)^2 + \left(\frac{12}{5}\right)(1+\upsilon)\right]\right\}^{1/2} \qquad \textbf{(B.8)}$$

where

T_0	=	resonant period of the SDOF system
H	=	height of embankment
E_{t_i}	=	initial tangent Young's modulus of the EPS
g	=	gravitational constant = 9.81 m/s^2 = 32.2 ft/s^2
B	=	embankment width
υ	=	Poisson's ratio for the EPS (typically taken to be ≈ 0.1 within the elastic range as is applicable for lightweight-fill applications

The resonant frequency is the reciprocal of the fundamental period and can be obtained from the equation below:

$$f_0 = \frac{1}{T_0} \qquad \textbf{(B.9)}$$

where

f_0 = resonant frequency of the SDOF system

Figure B.21 shows the two models that can be used to estimate seismic-response acceleration of the EPS fill mass in slope applications. The Japanese design manual suggests that the SDOF model can also be used to estimate seismic-response acceleration of the EPS fill mass in slope applications by converting the sloped EPS cross-section shown in Figure B.21(a) to an equivalent stand-alone embankment cross-sectional area (Horvath, 1995, Public Works Research Institute, 1992).

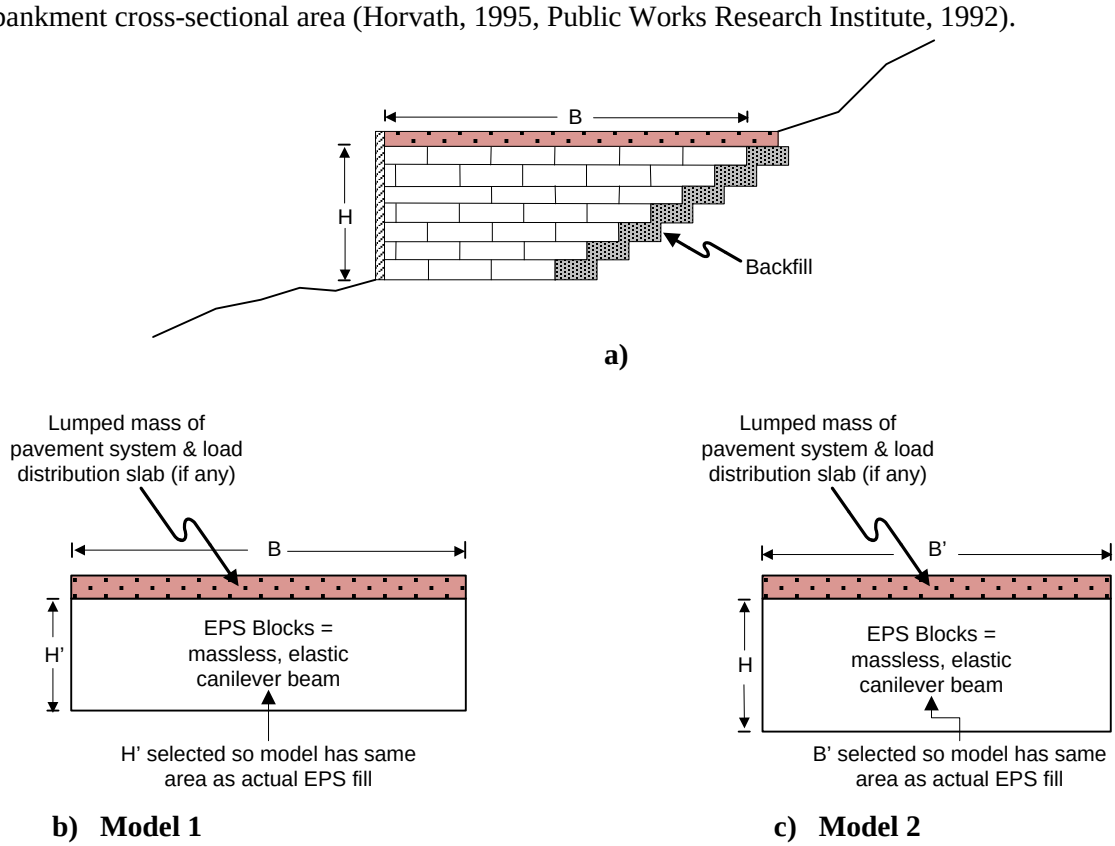


Figure B.21. Approximate (simplified) seismic modeling of an EPS fill.

In Model 1, which is shown in Figure B.21(b), the equivalent model cross-section is based on determining an equivalent height, H', that will yield the same sloped cross-sectional area using the actual top width, B, of the actual slope structure shown in Figure B.21(a). In Model 2, which is shown in Figure B.21(c), the equivalent model cross-section is based on determining an equivalent width, B', that will yield the same sloped cross-sectional area using the actual height, H, of the actual slope structure shown in Figure B.21(a).

After dimensions of the cross-section of each model are obtained, determine the resonant period of the SDOF system for both Model 1 and Model 2, T_1 and T_2, respectively, as shown by the equations below:

$$T_1 = 2\pi[(\frac{\sigma'_{V0}H'}{E_{t_i}g})(4(\frac{H'}{B})^2 + (\frac{12}{5})((1+\nu))]^{0.5} \quad \textbf{(B.10)}$$

$$T_2 = 2\pi[(\frac{\sigma'_{V0}H}{E_{t_i}g})(4(\frac{H}{B'})^2 + (\frac{12}{5})((1+\nu))]^{0.5} \quad \textbf{(B.11)}$$

B.6.7.3.2b) Determine the Site-Response Acceleration of the EPS-Block Geofoam Fill Mass System. Determine the spectral acceleration value, S_a, from the site-modified acceleration response spectrum for each resonant period determined in Section B.6.7.3.1a. Select the higher resulting spectral acceleration value, S_a, that is obtained from the site-modified acceleration response spectrum.

B.6.7.3.3c) Determine the Seismic Coefficients. Use the larger spectral acceleration value, S_a, from B.6.7.3.2b to determine the horizontal seismic coefficient, i.e., $k_v=S_a/g$.u. This larger seismic coefficient can be used in pseudo-static slope stability analysis. As previously noted, typically only the horizontal seismic coefficient is considered in pseudo-static analysis.

B.6.7.3.4d) Calculate the Seismic-Inertia Force Produced by Horizontal Acceleration Acting on the Lumped Mass of the Single-Degree-of-Freedom (SDOF) System. The seismic-inertia force can be determined using the seismic coefficient obtained in Section B.6.7.3.3c and from Equations B.3 and B.4.

B.6.7.4 Performing Pseudo-Static Limit Equilibrium Stability Analyses of Various Failure Mechanisms (Step 7iii)

B.6.7.4.1 Horizontal Sliding. Internal horizontal sliding analysis involves evaluating the potential for horizontal sliding between the pavement system and upper layer of blocks and/or between layers of block when subjected to external loads, as shown in Figure B.17. Stability against horizontal sliding is determined by the interface shear resistance between the pavement system and upper surface of the EPS mass and the interface friction between adjacent EPS blocks. Therefore, a discussion of methods used to ensure adequate block interlock is initially presented, prior to describing the seismic internal stability analysis procedure for horizontal sliding.

Internal stability of an EPS-block fill mass is maintained if it acts as a single, coherent mass when subjected to external loads. Since EPS block consists of individual blocks, the collection of blocks will behave as a coherent mass if individual EPS blocks exhibit adequate vertical and horizontal interlock. Sufficient interlock between blocks involves consideration of the overall block layout (which primarily controls vertical interlocking) and inter-block shear (which primarily controls horizontal interlocking). Guidelines for an appropriate layout of EPS blocks to obtain adequate interlocking in the vertical direction are included in the recommended standard included in Appendix F.

There are two modes of shear of interest in lightweight fill applications: (1) internal shear strength within a specimen of EPS and (2) external shear strength (interface sliding resistance) between EPS blocks, or between an EPS block and a dissimilar material (soil, other geosynthetic, etc.). The latter mode is the primary shear mode of interest in internal stability assessment under horizontal loads such as seismic shaking.

In general, shearing resistance at the interface between two materials, similar or dissimilar, can be defined by the classical Coulomb (dry) friction equation:

$$\tau = \sigma_n \cdot \mu = \sigma_n \cdot \tan(\delta) \qquad \textbf{(B.12)}$$

where

τ = shear strength
σ_n = normal stress
μ = coefficient of friction
δ = interface friction angle.

Because of variations in specimen dimensions, displacement rate, roughness of EPS surfaces, and other factors, a range in EPS/EPS interface friction angles have been reported. Reported peak shear strength values, δ_{Peak}, ranged from 32 degrees to 48 degrees, and residual shear strength values, $\delta_{Residual}$, ranged from 27 degrees to 35 degrees. These ranges are based on normal stresses ranging from 10 to 80 kPa. Unfortunately, the stress range corresponding to residual values is not included in the literature. Therefore, the value of $\delta = 30$ degrees, recommended as part of the Project 24-11(01) study, still appears reasonable for preliminary design.

The primary interface types other than EPS/EPS typically encountered in EPS-block geofoam embankments include EPS/soil, EPS/concrete, and EPS/geosynthetics. Such dissimilar materials may be used as a separation layer between EPS block and the pavement system, or between EPS block and the natural foundation layer. Materials sometimes utilized between the pavement system and EPS block include a geotextile, geomembrane, a PCC slab, geogrid, geocell with soil or PCC fill, soil cement, or pozzolanic stabilized materials. Materials sometimes utilized between EPS block and the natural foundation soil include granular material such as sand and geotextiles. A summary of EPS/dissimilar interface strength results from the literature—which may be used for *preliminary* design —is included in Chapter 3 of the Project 24-11(02) report. However, the final design interface strength should be based on interface strength tests performed with proposed interface materials.

If the calculated resistance forces along the horizontal planes between EPS blocks are insufficient to resist horizontal driving forces, additional resistance between EPS blocks is generally provided by adding mechanical inter-block connectors (typically prefabricated barbed metal plates) along the horizontal interfaces between EPS blocks, or by adding a shear key. The most common type of mechanical connector used in the U.S. is a prefabricated barbed metal plate. The use of mechanical connectors between layers of EPS blocks can be modeled by considering the horizontal interface between blocks according to the classical Mohr-Coulomb failure criterion:

$$\tau = c_a + \sigma'_n \tan\delta \qquad \textbf{(B.13)}$$

where

τ = shear strength

c_a = pseudo cohesion by connectors expressed as an average value per unit area

σ'_n =effective vertical normal stress at the interface

δ = interface friction angle of EPS/EPS.

At the present time, all mechanical connectors available in the U.S. are of proprietary designs. Therefore, data regarding resistance provided by such connectors and placement location must be obtained from the supplier or via independent testing. For example, it is reported that each 102mm by 102mm (4 in. by 4 in.) plate exhibits a design pseudo-cohesion of 267 N (60 lbs.). This resistance is based on tests performed on EPS block with a density of 16 kg/m³ (1 lbf./ft³) in accordance with ASTM C 578, and includes a factor of safety of two (AFM® Corporation, 1994).

An alternative to mechanical connectors is the use of shear keys (Bartlett and Lawton, 2008). Shear keys consist of half-height EPS blocks, periodically installed within the fill mass to interrupt the horizontal joints typically present in EPS fills. A method of analyzing the number and location of shear keys is currently not available. It is possible to perform a sliding stability analysis by incorporating the additional resistance provided by mechanical connectors or shear keys as part of a horizontal sliding analysis at various depths of the EPS fill.

The external horizontal sliding analysis procedure described in Section B.6.6.3 can also be used to perform a sliding analysis between the pavement system and uppermost layer of EPS block, as well as between the various vertical horizontal interfaces of EPS blocks.

B.6.7.4.2 Seismic Load-Bearing. The seismic-inertia forces associated with the earthquake will produce an increase in vertical normal stress within the EPS-block geofoam fill mass. These seismic-inertia forces are assumed to act at the center of gravity of the EPS-block fill mass, which, because the majority of the system's mass is concentrated in the pavement system, should be located near the top of the fill mass, at the horizontal center of the embankment cross-section. These dynamic stresses must be combined with the gravity normal stresses in Step 10.

If the vertical component of acceleration is significant enough to be considered in the design, additional stress from the vertical acceleration must be considered in both the load-bearing analysis and selection of geofoam type.

B.6.8 Step 8: Pavement System Design

The purpose of Step 8 is to perform a pavement structural analysis that considers the subgrade support provided by EPS block. The estimated pavement section determined in Step 2 is preliminary only, and facilitates estimation of dead loads for evaluation of Steps 3 through 7 of the design procedure shown in Figure B.8. Step 8 provides the pavement configuration based on anticipated loading conditions.

Steps 8 and 9 will typically be required if the existing or proposed roadway is located within the existing or anticipated slide mass, and the existing or anticipated slide mass is located below the roadway, as shown in Figures B.6(b) and B.7(b), i.e., the roadway is near the head of the slide mass. However, a pavement system design will typically not be required if the existing or proposed roadway is located outside the limits of the existing or anticipated slide mass, and/or the existing or anticipated slide mass is located above the roadway, as shown in Figures B.6(a) and B.7(a), i.e., the roadway is near the toe of the slide mass. It is anticipated that EPS-block geofoam used for this latter slope application will not support any structural loads other than possibly soil fill above the block. Therefore, as shown in Figure B.8, Steps 8 and 9 will not be required, and only failure mechanisms associated with external and internal instability failure modes, as shown in Table B.1, need to be considered. The pavement system failure mode may not be an applicable failure mode, because if the roadway is near the toe of the slide mass, stabilization of the slide mass with EPS-block geofoam will occur primarily at the head of the slide and, consequently, the EPS-block geofoam slope system may not include the pavement system.

For the case of an existing or proposed roadway located within the existing or anticipated slide mass, and the existing or anticipated slide mass located below the roadway, pavement system design (Step 8) follows seismic stability. The pavement system is defined as including all materials, bound and unbound, placed above EPS blocks.

The objective of pavement system design is to select the most economical arrangement and thickness of pavement materials for the subgrade provided by the underlying EPS block. The design criteria are to prevent premature failure of the pavement system (Figure B.22), as well as to minimize the potential for differential icing (a potential safety hazard) and solar heating (which can lead to premature pavement failure) in those areas where climatic conditions make these potential problems. Also, when designing the pavement cross-section overall, consideration must be given to providing sufficient support, either by direct embedment or structural anchorage, for any road hardware (guardrails, barriers, median dividers, lighting, signage and utilities).

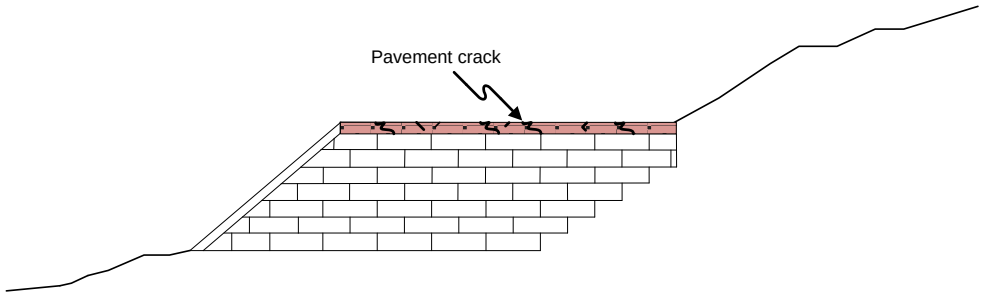


Figure B.22. Pavement failure due to cracking.

A unique aspect of pavement design over lightweight fill is that the design must also consider potential failure mechanisms associated with external and internal stability of the overall EPS-block geofoam slope system. Regarding use of a thicker pavement system:

- Benefits include increased pavement life and internal load-bearing capacity of the EPS-block fill mass, and reduced potential for differential icing and solar heating, as well as better accommodation of shallow utilities and road hardware.
- Drawbacks include increased weight, which decreases the factor of safety of the external stability failure mechanisms and the seismic internal stability of the EPS-block geofoam fill mass, and higher total project costs.

Thus, some compromise is required to optimize the final design of both the pavement system and overall fill mass. Benefits and drawbacks of utilizing a thicker pavement system, as well as procedures for design of pavement systems over EPS-block geofoam embankments, are further discussed in the Project 24-11(01) report (Stark et al., 2004a).

The literature search revealed a lack of current research results that focuses on the design of pavement systems overlying EPS block, especially design parameters for use in design of pavement systems based on the AASHTO Mechanistic-Empirical Pavement Design Guide (AASHTO, 2008). Therefore, the same pavement system design procedures described in the Project 24-11(01) reports can also be utilized for EPS block slope systems.

The literature search also revealed a lack of current research results that focus on the design of pavement systems that include a separation layer between the top of EPS block and overlying pavement system. A separation layer can have two functions: (1) to enhance the overall performance and life of the pavement system by providing reinforcement, separation, and/or filtration and (2) to enhance the durability of EPS block, both during and after construction.

The use of a 100 to 150mm (4 to 6 in.) thick reinforced PCC slab is currently the state of practice, primarily because it is considered a necessity for providing sufficient lateral confinement of unbound pavement layers and load distribution when using EPS-block geofoam, and because of historical usage of PCC slabs dating back to the earliest EPS-block geofoam lightweight fills in Norway in the 1970s. The original function of the PCC slab was for pavement reinforcement, and the intent was to allow use of a minimum pavement system thickness. In later designs, the PCC slab was also used to function as a barrier against potential petroleum spills. However, use of a PCC slab for this function is questionable, due to the usual long-term development of cracks in PCC slabs. PCC slabs generally represent a significant relative cost, so should only be used if specifically required as determined during design.

B.6.9 Step 9: Evaluation of pavement system design on previous failure mechanisms already analyzed

A unique aspect of pavement design over lightweight fill is that the design must also consider the affect of the final pavement system design on static and seismic slope stability, because the thickness and type of pavement system materials will affect static and seismic stability. Therefore, Step 9 consists of determining if the pavement system design obtained in Step 8 results in a change in the overburden stress compared to the preliminary pavement system initially utilized in Step 2. If the overburden stress between the pavement system obtained in Step 8 is the same as the overburden stress from the preliminary pavement system initially utilized in Step 2, Step 10 (load-bearing analysis) can be performed. If the overburden stress between the Step 8 and 2 pavement systems are different, static and seismic slope stability must be rechecked and, therefore, the design procedure reverts back to Step 5.

B.6.10 Step 10: Load-Bearing (Internal)

Step 10 involves load-bearing analysis. A load-bearing analysis consists of selecting an EPS type with suitable properties to be able to support the overlying pavement system and traffic loads without excessive EPS compression which could lead to excessive settlement of the pavement surface. To ensure adequate performance of EPS block, three design goals must be achieved. First, the initial (immediate) deformations under dead or gravity loads from the overlying pavement system must be within acceptable limits. Second, the long-term (for the design life of the fill) creep deformations under the same gravity loads must be within acceptable limits. Third, non-elastic or irreversible deformations under repetitive traffic loads must be within acceptable limits.

Elastic limit stress, σ_e, is the parameter used to evaluate the three deformation issues presented above. The elastic limit stress of EPS geofoam is defined as the compressive stress at 1 percent strain as measured in a standard rapid-loading compression test (Stark et al., 2004a). The basic procedure for designing against load-bearing failure is to calculate the maximum vertical stresses at various levels within the EPS mass (typically the pavement system/EPS interface is most critical) and select the EPS type that exhibits an elastic limit stress that is greater than the required elastic limit stress at the depth being considered. If EPS blocks with a higher elastic limit stress than what is currently available locally are required, consideration can be given to modifying the pavement system design, such as adding a separation layer to further distribute live loads and decrease the stresses at the top of the EPS-block fill mass. For example, a PCC load distribution slab can be included in the pavement system to decrease the stresses within EPS blocks. Table F.2 in Appendix F provides the minimum recommended values of elastic limit stress for various EPS densities.

The procedure for evaluating load-bearing capacity of EPS as part of internal stability is outlined in the following thirteen steps:

1. Estimate traffic loads.
2. Add impact allowance to traffic loads.
3. Estimate traffic stresses at top of EPS blocks.
4. Estimate gravity stresses at top of EPS blocks.
5. Calculate total stresses at top of EPS blocks.
6. Determine minimum required elastic limit stress for EPS under pavement system.
7. Select appropriate EPS block to satisfy the required EPS elastic limit stress for underneath the pavement system, e.g., *EPS50*, *EPS70*, *EPS100*, *EPS130*, or *EPS160*.
8. Select preliminary pavement system type and determine if a separation layer is required.
9. Estimate traffic stresses at various depths within the EPS blocks.
10. Estimate gravity stresses at various depths within the EPS blocks.
11. Calculate total stresses at various depths within the EPS blocks.
12. Determine minimum required elastic limit stress at various depths.

13. Select appropriate EPS block to satisfy the required EPS elastic limit stress at various depths in the embankment.

The details for performing each of the above load-bearing steps is included in the Project 24-11(01) report (Stark et al., 2004a) and is also provided by Arellano and Stark (2009).

B.6.11 Step 11: Settlement

Step 11 consists of estimating settlement of the proposed EPS-block geofoam slope system. Total settlement of an EPS-block geofoam fill mass embankment, S_{total}, consists of five components as indicated by Equation B.14.

$$S_{total} = S_{if} + S_i + S_p + S_s + S_{cf} \quad \textbf{(B.14)}$$

where

S_{total} = total settlement,
S_{if} = immediate or elastic settlement of the fill mass,
S_i = immediate or elastic settlement of the foundation soil,
S_p = end-of-primary (EOP) consolidation of the foundation soil,
S_s = secondary consolidation of the foundation soil, and
S_{cf} = long-term vertical deformation (creep) of the fill mass.

A summary of settlement analysis procedures to estimate settlement are described in the Project 24-11(01) report (Stark et al., 2004a). If the resulting settlement is not feasible, the fill mass arrangement can be revised by changing the thickness, width and/or location of EPS blocks within the slope such that the stresses tending to cause settlement are decreased to obtain the desired settlement. It may be beneficial to partially excavate a portion of the foundation material and replace excavated material with EPS-block geofoam to limit the final effective vertical stress to a tolerable level.

B.6.12 Step 12: Bearing Capacity (External)

Step 12 consists of evaluating bearing capacity failure of the foundation material as a potential external failure mode of an EPS-block geofoam embankment. Bearing capacity failure occurs if the applied stress exceeds the bearing capacity of the foundation material, which is related to its shear strength. Failure is only considered through the foundation material because Step 10 addresses internal stability or load-bearing failure through the EPS-block geofoam fill.

The general expression for the ultimate bearing capacity of soil, q_{ult}, is shown in Equation B.15 below (Kimmerling, 2002):

$$q_{ult} = c(N_c) + q(N_q) + 0.5(\gamma)(B_f)(N_\gamma) \quad \textbf{(B.15)}$$

where

q_{ult} = ultimate gross bearing capacity
c = cohesion of soil
N_c = bearing capacity factor for the cohesion term
q = surcharge at the base of the footing
N_q = bearing capacity factor for the surcharge term
B_f = footing width

γ = unit weight of soil beneath the footing

N_γ = bearing capacity factor for soil unit weight.

Values for bearing capacity factors N_c, N_q, and N_γ are based on the friction angle, φ', of the foundation soil, and may be obtained from various references, including the FHWA Geotechnical Engineering Circular No. 6 (Kimmerling, 2002).

The presence of ground water near the bottom of the footing may reduce shear strength of the foundation soil, thus reducing its ultimate bearing capacity. To account for these effects, correction factors, C_{W_q} and C_{W_γ}, can be calculated using Equations B.16a and B.16b, as shown below: (Kimmerling, 2002):

$$C_{W_q} = 0.5 + 0.5\left(\frac{D_W}{D_f}\right) \leq 1.0 \qquad \textbf{(B.16a)}$$

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{D_W}{1.5B_f + D_f}\right) \leq 1.0 \qquad \textbf{(B.16b)}$$

where

D_W = depth of groundwater

D_f = depth of embedment of footing

B_f = footing width.

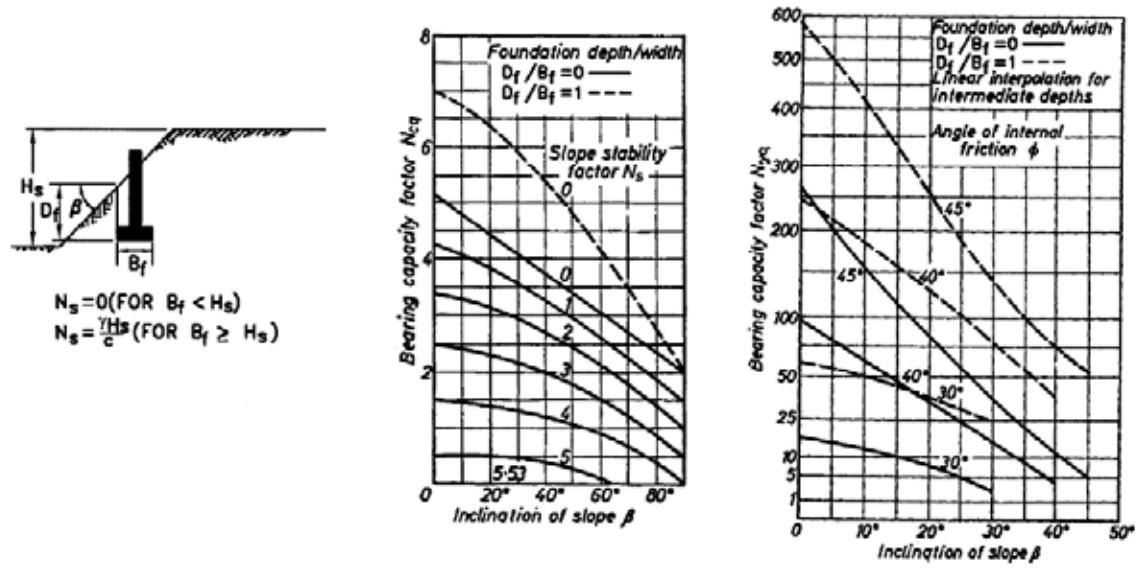
Incorporating the correction factors from Equations B.16a and B.16b into Equation B.15 yields Equation B.17, shown below, which can be used to calculate the ultimate bearing capacity of the soil, taking into account the effects of the ground water table:

$$q_{ult} = c(N_c) + q(N_q)(C_{W_q}) + 0.5(\gamma)(B_f)(N_\gamma)(C_{W_\gamma}) \qquad \textbf{(B.17)}$$

Placing a footing on or near a slope, instead of on level ground, can also have an effect on the bearing capacity. The effects of the sloped ground are accounted for by replacing the bearing capacity factors, N_c and N_γ, with corrected bearing capacity factors, N_{cq} and $N_{\gamma q}$. The resulting equation for bearing capacity of a shallow footing on sloped ground is shown below (Kimmerling, 2002):

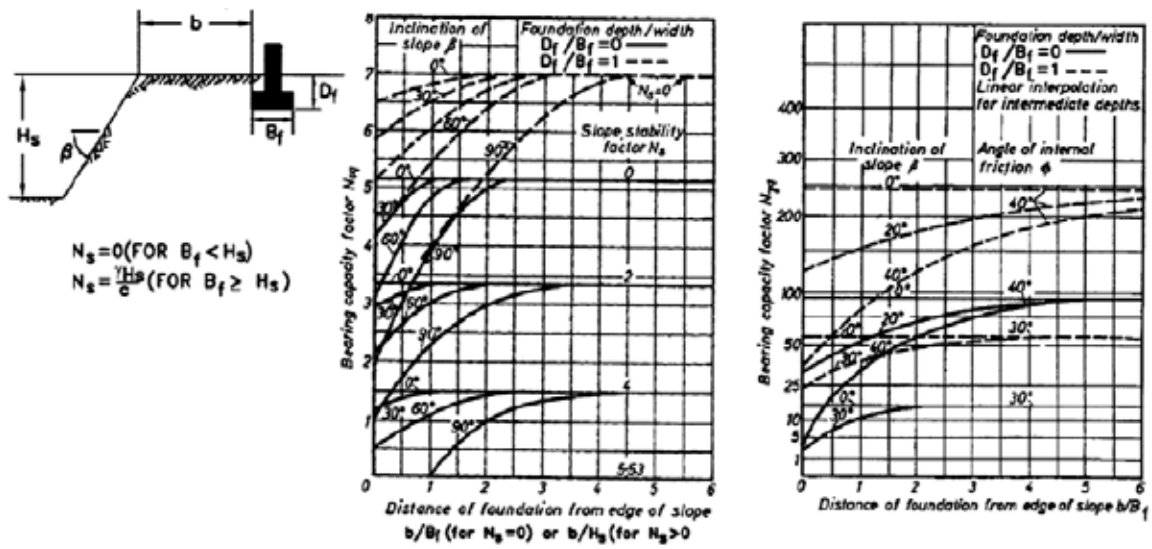
$$q_{ult} = c(N_{cq}) + 0.5(\gamma)(B_f)(N_{\gamma q}) \qquad \textbf{(B.18)}$$

Values for N_{cq} and $N_{\gamma q}$ must be obtained using Figures B.23 and B.24, respectively.



a) Geometry *b) Cohesive ($\phi = 0^0$) soils* *c) Cohesionless ($c = 0$) soils*

Figure B.23. Modified bearing capacity factor for surcharge, N_{cq}, for footings on sloping ground (AASHTO, 1996, Kimmerling, 2002, Meyerhof, 1957; used by permission of Elsevier Limited).



a) Geometry *b) Cohesive ($\phi = 0^0$) soils* *c) Cohesionless ($c = 0$) soils*

Figure B.24. Modified bearing capacity factor for surcharge, $N_{\gamma q}$, for footings on sloping ground (AASHTO, 1996, Kimmerling, 2002, Meyerhof, 1957; used by permission of Elsevier Limited).

In some cases the friction angle of the soil is not in the range of values provided by Figures B.23 and B.24. The general expression for the ultimate bearing capacity of soil, q_{ult}, as given by Coduto (2001) is shown below in Equation B.19, and can be used when Figures B.23 and B.24 are not applicable. For the vast majority of cases involving EPS-block geofoam slope fills, the *s*, *d*, *i*, and *b* factors will all be equal to 1.0. For special circumstances where this is not the case, Coduto (2001) provides guidance for calculating appropriate correction factors.

$$q_{ult} = c' N_c s_c d_c i_c b_{ci} g_c + \sigma'_{vD} N_q s_q d_q i_q b_q g_q + 0.5\gamma'_{soil} B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma \quad \textbf{(B.19)}$$

where

c' = effective cohesion of the foundation material
σ'_{vD} = vertical effective stress due to soil overburden at bottom of EPS fill
B = width (perpendicular to long axis) of the EPS fill
N_c, N_q, N_γ = bearing capacity factors
s_c, s_q, s_γ = shape factors = 1.0 for most EPS slope fills
d_c, d_q, d_γ = depth factors = 1.0 for most EPS slope fills
i_c, i_q, i_γ = load inclination factors $\leq$ 1.0 for most EPS slope fills
b_c, b_q, b_γ = base inclination factors = 1.0 for most EPS slope fills
g_c = ground inclination factor=$1 - \frac{\beta}{147^0}$ where β = slope inclination
g_q = g_γ = ground inclination factor= $(1 - \tan\beta)^2$
γ'_{soil} = effective unit weight of soil

To calculate the effective unit weight of the soil, γ'_{soil}, for use in Equation B.19, the location of the ground water table must be identified as falling under one of three different cases:

Case 1 applies when the depth from the ground surface to the ground water table, D_w, is located at or above the bottom of the EPS-block geofoam fill, i.e., $D_w \leq D$ where *D* is the depth of the EPS-block geofoam fill. Equation B.20 is applicable for Case 1.

Case 2 applies when the depth from the ground surface to the ground water table, D_w, falls between the bottom of the EPS-block geofoam fill and the lower limit of the zone of influence, which is defined as $D + B$, where *D* is the depth of the EPS-block geofoam fill and *B* is the width of the EPS-block geofoam fill. In other words, Case 2 is defined as $D < D_w < D + B$. Equation B.21 is applicable for Case 2.

Case 3 applies when the ground water table is below the zone of influence (i.e., $D + B \leq D_w$). For Case 3, no groundwater correction is needed, as shown in Equation B.22:

Case 1:
$$\gamma'_{soil} = \gamma_{soil} - \gamma_w \quad \textbf{(B.20)}$$

Case 2:
$$\gamma'_{soil} = \gamma_{soil} - \gamma_w \left(1 - \frac{D_w - D}{B}\right) \quad \textbf{(B.21)}$$

Case 3:
$$\gamma'_{soil} = \gamma_{soil} \quad \textbf{(B.22)}$$

The FHWA Geotechnical Engineering Circular No. 6 (Kimmerling, 2002) suggests that the bearing capacity of eccentric loaded footings be based on effective footing dimensions. Thus, the effective footing width, B', would be used in Equation B.19. Additionally, the shape factors should also be based on the effective footing dimensions.

If the desired factor of safety against bearing capacity failure is not feasible, the fill mass arrangement can be revised by changing the thickness, width and/or location of EPS block within the slope such that the stresses on the foundation material can be reduced.

B.6.13 Step 13: Final Design Details

The final step of the design procedure, Step 13, consists of preparing final design details of the EPS-block geofoam slope system. In addition to cross-sectional and longitudinal geometry details of the slope, additional details that may require consideration and further analysis include drainage system, road hardware (guardrails, barriers, median dividers, lighting, signage and utilities), and facing system.

The drainage system is a critical component of the EPS-block geofoam slope system because the recommended design procedure is based on inclusion of an effective permanent drainage system to prevent hydrostatic uplift (flotation) and translation of the EPS-block geofoam fill mass due to water. Many of the EPS-block geofoam slope case histories evaluated as part of this research included use of underdrain systems below EPS block to prevent water from accumulating above the bottom of the EPS block, and in some cases, incorporated a drainage system between the adjacent upper slope material and EPS block to collect and divert groundwater, and thereby alleviate seepage pressures.

Road hardware such as guardrails, barriers, median dividers, lighting, signage and utilities can be incorporated in the EPS-block geofoam slope system by direct embedment or structural anchorage. The alternatives for accommodating shallow utilities and road hardware (barriers and dividers, light poles, signage) is to provide a sufficient thickness of the pavement system to allow conventional burial or embedment within soil or, in the case of appurtenant elements, provide for anchorage to a PCC slab or footing that is constructed within the pavement section.

Barriers or guardrails are typically required with vertical-sided embankments. Design of traffic railings is addressed in Section 13 of the Bridge Design Specifications (AASHTO, 2007) and in the AASHTO Road Design Guide (AASHTO, 2002). Several road hardware details are provided in Appendix G of the Project 24-11(02) report. Additional loads such as vehicle impact loads may need to be considered.

If a vertical-sided fill is used, a facing wall system is required to protect EPS blocks. The facing does not have to provide any structural capacity to retain the blocks because the blocks are self-stable. The primary function of the facing wall is to protect blocks from damage caused by environmental factors. Selection of the type of facing system to use is based on three general criteria: (1) facing must be self-supporting or physically attached to EPS blocks, (2) architectural/aesthetic requirements, and (3) cost. The following materials have been successfully used for facing geofoam walls:

- prefabricated metal (steel or aluminum) panels,
- precast PCC panels, either full height or segmental (such as used in mechanically stabilized earth walls, MSEWs),
- segmental retaining wall (SRW) blocks which are typically precast PCC,
- shotcrete,
- geosynthetic vegetative mats, and
- exterior insulation finish systems (EIFS).

Other materials that might be suitable for facing geofoam walls include:

- wood panels or planks, and
- EPS-compatible paint for temporary fills.

Regardless of the type of facing system used, the resulting vertical stress on foundation soil must be considered in calculations for both settlement and global stability. The weight of the facing elements should be obtained from a supplier, or estimated, to ensure that the correct weight is used in calculations for that specific type of facing system.

Regarding who actually designs the block layout; traditionally this was done by the design engineer for the project. However, this is appropriate only if the designer knows the exact block dimensions beforehand. In current U.S. practice, there will generally be more than one EPS block molder who could potentially supply a given project. In most cases, block sizes will vary somewhat between molders due to different make, model and age of molds. Therefore, the trend in U.S. practice is to leave the exact block layout design to the molder. The design engineer simply:

- Shows the desired limits of the EPS mass on contract drawings, specifying zones of different EPS densities as desired;
- Includes the above conceptual guidelines in the contract specifications for use by the molder in developing shop drawings; and
- Reviews the submitted shop drawings during construction.

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Appendix C

Optimization Procedure

APPENDIX C

OPTIMIZATION PROCEDURE

INTRODUCTION

This appendix explains the concepts behind the optimization procedure used in Step 3 of the design procedure shown in Figure 4.25 and suggests an approach for implementing the optimization in practice. Additionally, two case histories are used to evaluate the effectiveness of the procedure, and illustrate potential benefits of using it, with special attention given to comparing the proposed optimization procedure with Negussey and Srirajan's (2001) Moment Reduction Method.

MOTIVATION

It may be asked why a designer would want to put forth the effort necessary to determine the optimum volume and location for an EPS-block geofoam fill when it is not particularly difficult to develop an estimate of required volume of EPS geofoam to stabilize a slope? However, this appendix will show that the optimization procedure proposed here requires very little time and effort to implement, especially if the suggested spreadsheet format is used to set up the optimization. Additionally, using this procedure can potentially result in significant cost savings for the project by identifying the most efficient configuration possible for EPS fill.

At the time of the literature search, the only known method to optimize the volume and location of EPS fill was the Moment Reduction Method presented by Negussey and Srirajan (2001), was considered too inefficient and time-consuming to be useful in practice. Therefore, one of the objectives of this report was to develop a method for optimizing the volume and location of EPS fill that would be simple enough to be used in practice.

The ability to ensure that EPS fill will be designed in the most efficient way possible could be a significant advantage to designers, especially on large projects, since EPS-block geofoam generally has a much higher material cost than conventional fill materials. Even using the traditional trial-and-error approach to sizing EPS fill, cost savings resulting from accelerated construction times made possible by EPS-block geofoam generally far outweigh the additional material costs associated with EPS block. However, significant additional cost savings could potentially result from using a more efficient design for EPS-block geofoam fill, thus making it even more attractive as a slope stabilization alternative. This conclusion is supported by evidence from two case histories involving EPS-block geofoam fills for slope stabilization, which will be presented later in this chapter.

The purpose of the optimization methods is only to obtain an approximate location within the slope where the placement of EPS blocks will have the greatest impact in stabilizing the slope, while requiring a minimum volume of EPS block. A separate static slope stability analysis must be performed as part of Step 5 of the design procedure, as shown in Figure 4.25, with a better method that preferably satisfies full equilibrium, such as Spencer's method. Step 5 should be relied on to verify that the overall slope configuration meets the desired factor of safety.

As will become apparent here, it is unlikely that the results of the optimization procedure can be used directly to obtain the final EPS block configuration because the configuration obtained from these optimization procedures may be impractical to replicate in the field. In some cases, the geometry of the optimized EPS fill may simply be too complicated to be manufactured or constructed with a reasonable amount of time and effort. However, the optimized EPS-block geofoam fill geometry obtained will still provide a useful starting point from which to design an EPS geofoam fill that is both efficient and constructible. Minor alterations may be required to adopt the optimized EPS fill design to specific project requirements and site restrictions as part of Step 4 of the design procedure, as shown in Figure 4.25.

The optimization procedures presented herein are optional within the proposed design procedure shown in Figure 4.25. In lieu of performing one of the optimization procedures, the designer can select a preliminary volume and location of EPS blocks within the slope and proceed with Step 5.

OPTIMIZING THE VOLUME AND LOCATION OF EPS FILL

Background

The stability of a given slope has traditionally been quantified using a *factor of safety*. This factor of safety may be generally defined as the ratio of a slope's capacity to resist movement (i.e., shear strength of the material at the slip surface) versus forces or moments that tend to contribute to that movement. It should be noted that this gives rise to two distinct ways of defining the factor of safety.

- The first defines the factor of safety in terms of moment equilibrium.
- The second defines it in terms of force equilibrium.

To determine the factor of safety of a slope, it is common to use an analysis method that divides the portion of the slope material that is in danger of failure into vertical slices. The surface along which the failure mass slides is known as the "slip surface" and is generally classified as being either circular, as shown in Figure C.1, or non-circular, as shown in Figure C.2:

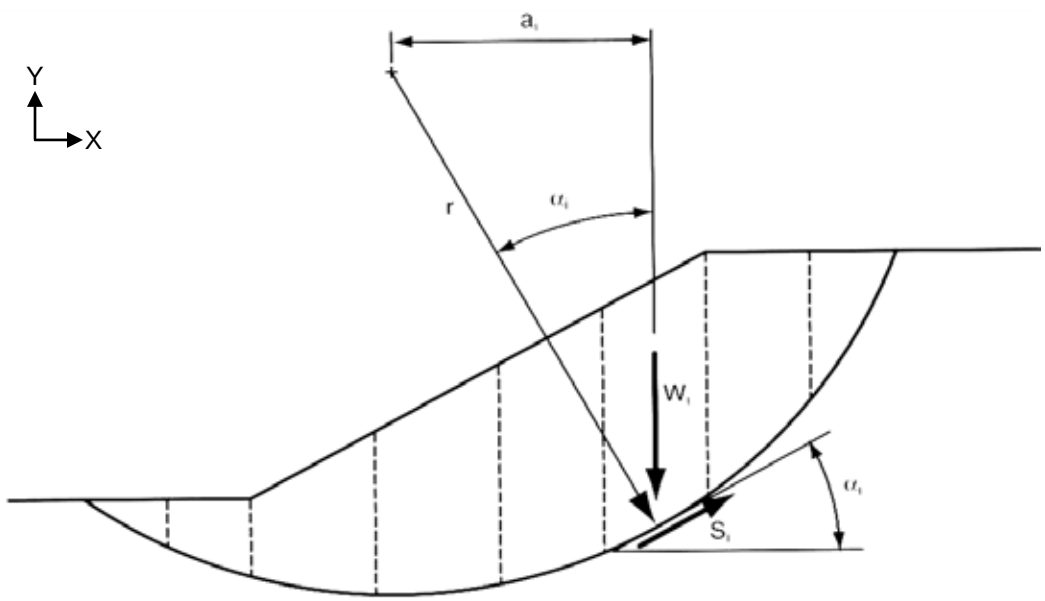


Figure C.1. Circular slip surface with overlying soil mass subdivided into vertical slices. (From Soil Strength and Slope Stability, by Duncan, J. M., and Wright, S. G., 2005; used by permission of John Wiley & Sons).

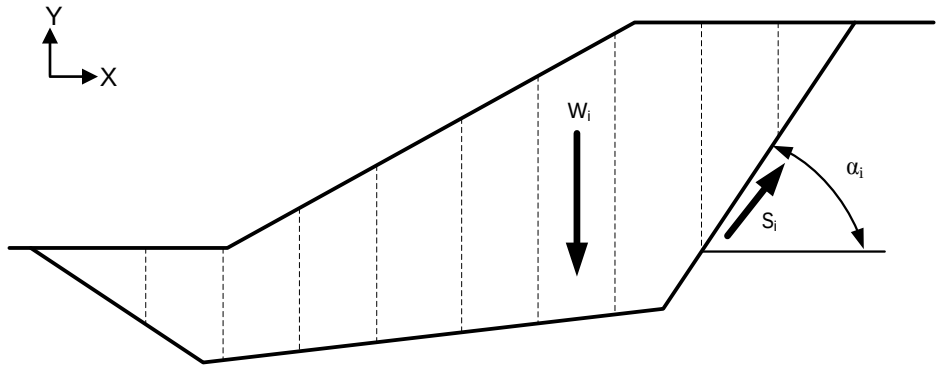


Figure C.2. Non-circular slip surface with overlying soil mass subdivided into vertical slices.

In both cases shown above, the failure mass has been divided into a series of vertical slices, each having some weight, W_i, which contributes to its tendency to slide downhill. This movement is opposed by the shear resistance along the slip surface, denoted by S_i. The angle of the bottom of each slice with respect to the horizontal is represented by α_i. By analyzing each slice in terms of either moment equilibrium or force equilibrium, and then determining the ratio of the sum of all resisting moments (or forces) in all of the slices, versus the sum of all the driving moments (or forces) in all of the slices, the factor of safety for the entire slope can be determined. Equation C.1(a) gives an expression defining the factor of safety in terms of moment equilibrium, while Equation C.1(b) gives an expression for the factor of safety in terms of force equilibrium. A more detailed explanation of the differences between force equilibrium analysis and moment equilibrium analysis can be found in Duncan and Wright's (2005) textbook on slope stability. It should be noted that the factor of safety is assumed to be the same at all points along the slip surface. Thus, the factor of safety represents an overall average value along the slip surface:

$$FS = \frac{\Sigma M_R}{\Sigma M_D} \qquad \textbf{C.1(a)}$$

$$FS = \frac{\Sigma F_R}{\Sigma F_D} \qquad \textbf{C.1(b)}$$

where

$\Sigma M_R =$ sum of the moments resisting movement in all of the slices

$\Sigma M_D =$ sum of the moments driving movement in all of the slices

$\Sigma F_R =$ sum of the forces resisting movement of failure mass

$\Sigma F_D =$ sum of all forces driving movement of failure mass

The practice of using EPS-block geofoam for slope stabilization is based on the idea that by replacing a portion of natural slope material, whether soil or rock, with EPS blocks, the forces or moments driving movement of the failure mass downhill can be reduced. Thus, certain slices within the failure mass will have some portion of the soil in them replaced by lighter weight EPS block, reducing the overall weight of the slice and contributing to stability of the entire failure mass.

It should be pointed out that by adding geofoam to reduce the weight of a slice, both the driving and the resisting forces and moments acting on that slice are reduced. As noted above, for any given slice, the primary driving force will generally be the weight of the soil that makes up that slice. The primary

resisting force will be the shear resistance along the slip surface at the bottom of the slice due to the shear strength of the soil. Because the shear strength of soil is dependent on the normal stress acting on the slip surface, which in this case is the result of the weight of the slice, adding geofoam to decrease the weight of a slice results in a decrease in both the driving and resisting forces or moments. However, in almost every case, reduction of the driving forces will be significantly greater than the reduction in the resisting forces. Otherwise, EPS-block geofoam would be of no use for slope stabilization.

The sum of the net changes in the driving forces or moments in all of the slices in a given failure mass may be denoted by M_{red} for moment equilibrium methods or by F_{red} for force equilibrium methods. For the sake of simplicity, the remainder of this explanation will be presented in terms of M_{red}. The reason for this is that the optimization method for circular failure surfaces is based on the Ordinary Method of Slices, also known as Fellenius' Method, which is a moment equilibrium procedure as shown below in Equation C.2. This method assumes that the slope will fail along a circular slip surface and that all inter-slice force may be neglected:

$$FS = \frac{\Sigma M_R}{\Sigma M_D} = \frac{\Sigma\left[c'L + (W\cos\alpha - uL\cos^2\alpha)\tan\phi'\right]}{W\sin\alpha} \qquad \textbf{(C.2)}$$

where

ΣM_R = sum of the resisting moments acting on all of the slices
ΣM_D = sum of the driving moments acting on all of the slices
c' = effective cohesion of the soil near the slip surface
L = length of the bottom of each slice (See Figure C.3)
W = total weight of each slice = $\gamma_{soil}(H + H')b$
γ_{soil} = unit weight of soil
H = height of each slice above the water Table (See Figure C.3)
H' = height of each slice below the water Table (See Figure C.3)
b = width of each slice
α = angle from the horizontal to the inclination of the bottom of each slice (See Figure C.3)
u = pore water pressure
ϕ' = effective friction angle of the soil near the slip surface

Note that this is a modified version of the original Ordinary Method of Slices. The original method, developed by Fellenius (1936), was shown to yield negative effective stresses along the slip surface under certain conditions, resulting in slight inaccuracies in the factor of safety values calculated under these conditions. This analysis, performed by Turnbull and Hvorslev (1967), led them to develop the modified version of the Ordinary Method of Slices, shown above in Equation C.2. The primary difference between Turnbull and Hvorslev's (1967) modified version of the Ordinary Method of Slices and the original method proposed by Fellenius (1936) is that Turnbull and Hvorslev's equation incorporates the $\cos^2\alpha$ term into the numerator, as shown in Equation C.2, to correct inaccuracies in the calculation of effective stress at the slip surface.

The ordinary method of slices is less accurate than other procedures of slices, especially for effective stress analyses whereby accuracy decreases as pore water pressures become larger (Duncan and Wright, 2005). However, accuracy can be increased for effective stress analysis by using the factor of safety equation that we recommended, Equation C.2. The reason the Ordinary Method of Slices was used is that it permits the factor of safety to be calculated directly, and does not require a trial-and-error solution for the factor of safety. Also, the purpose of the optimization procedure is to only optimize the

volume and location of EPS block. A separate static slope stability analysis must be performed as part of Step 5 of the design procedure, as shown in Figure 4.25, with a better method that preferably satisfies full equilibrium, such as Spencer's method. Step 5 should be relied on to verify that the overall slope configuration meets the desired factor of safety.

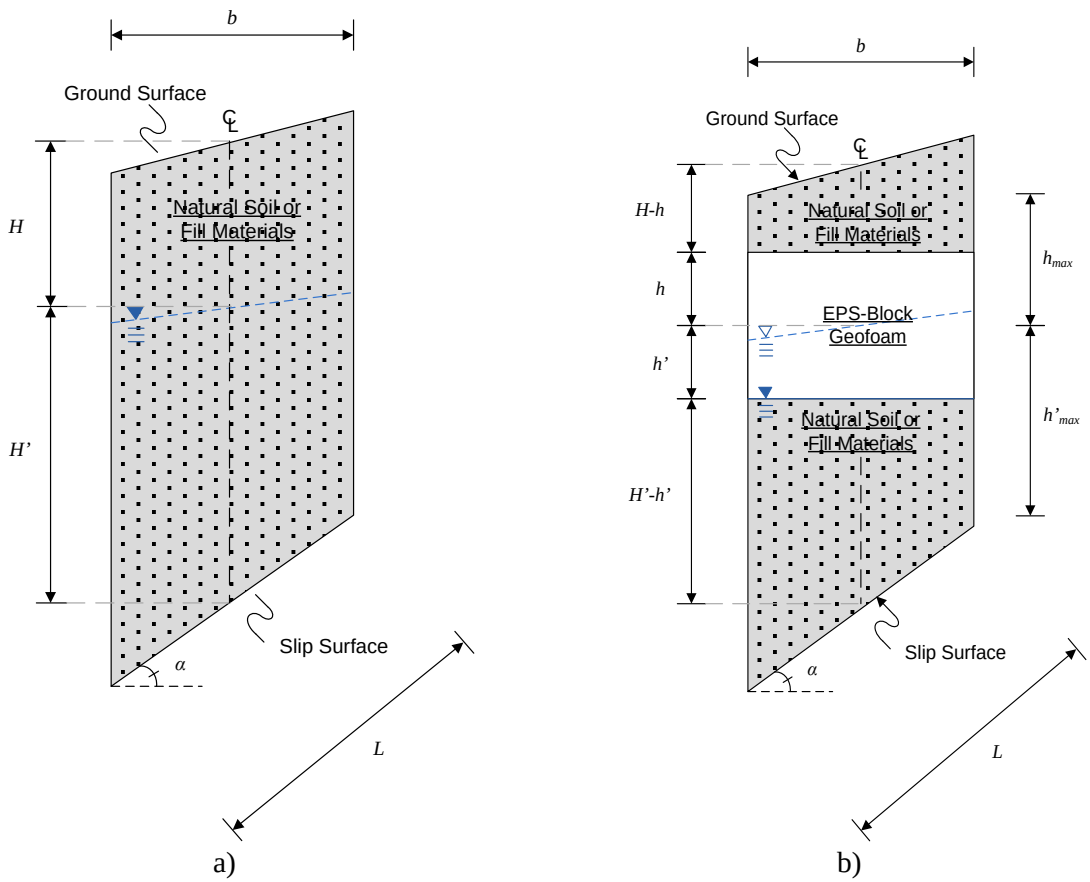


Figure C.3. Definitions of variables used for a) slice prior to installation of EPS-block geofoam and b) slice after installation of EPS-block geofoam.

The optimization method for circular slip surfaces is considerably simpler than the optimization method for non-circular failure surfaces, which is based on the Simplified Janbu Method, which is, in turn, based on principles of force equilibrium, as shown in Equation C.3. While the Simplified Janbu Method does not make any assumptions regarding the shape of the slip surface, it does assume that inter-slice shear forces may be neglected:

$$FS = \frac{\Sigma A}{Q + \Sigma B} = \frac{\sum \frac{[c' + (p-u)\tan\phi']b}{n_\alpha}}{Q + pb\tan\alpha} \qquad \textbf{(C.3)}$$

where

$p = \gamma_{soil}[H + H']$ (See Figure C.3)

$n_\alpha = \cos^2\alpha\left(1 + \frac{\tan\alpha\tan\phi'}{FS}\right)$

$Q =$ net horizontal inter-slice force imbalance = $-\Sigma(-S\cos\alpha + N\sin\alpha)$

$S =$ shear resistance developed at the bottom of each slice

$N =$ normal force acting at the bottom of each slice

Therefore, the simpler moment equilibrium procedure based on the Ordinary Method of Slices will be explained first, before moving on to the more complex force equilibrium procedure based on the Simplified Janbu Method.

As discussed above, M_{red} represents the sum of net changes in the driving moments, which are defined by the denominator of Equation C.2, in all of the slices in a given failure mass due to the addition of the geofoam. This net reduction in the driving moments, M_{red}, can be incorporated into the factor of safety expression from Equation C.1(a) as shown below in Equation C.4:

$$FS = \frac{\Sigma M_R}{\Sigma M_D - M_{red}} \qquad \textbf{(C.4)}$$

Again, it should be pointed out that this M_{red} represents not simply the reduction in the driving moment due to addition of EPS-block geofoam, rather it represents the *net* reduction in the driving moment. That is, it includes effects of the geofoam on both the driving moments and resisting moments. This relationship is stated in mathematical terms in Equation C.5 below. This concept is emphasized because it is the key to understanding the meaning of the optimization constraint equations presented later in this chapter:

$$M_{red} = \Sigma\Delta M_D - \Sigma\Delta M_R \qquad \textbf{(C.5)}$$

where

$M_{red} =$ the net reduction in the driving moments of all the slices in a given failure mass

$\Sigma\Delta M_D =$ the sum of the changes in the driving moments of all the slices in a given failure mass due to the addition of geofoam

$\Sigma\Delta M_R =$ the sum of the changes in the resisting moments of all the slices in a given failure mass due to the addition of geofoam

By examining Equations C.4 and C.5, it may become apparent that placing M_{red} in the denominator of Equation C.4 is essentially a way of considering the change in resisting moments due to the geofoam, $\Sigma\Delta M_R$, as a decrease in the change in driving moments due to the geofoam, $\Sigma\Delta M_D$. In other words, $\Sigma\Delta M_R$ is treated as a decrease in the magnitude of $\Sigma\Delta M_D$. While this approach to modeling the effects of EPS fill is not in strict mathematical agreement with the definition of the factor of safety as shown in Equations C.1(a) and C.1(b), it greatly simplifies derivations of the constraint equations and provides a good approximation of the true value. As will be shown in the case history analyses presented later in this chapter, this assumption regarding M_{red} has only a very small impact on the accuracy of the optimization procedure. Thus, in the

interest of keeping the procedure as simple and easy-to-use as possible, this assumption was considered acceptable.

Optimization Method for Circular Slip Surfaces

This approach to optimizing volume and location of an EPS-block geofoam fill within a slope is formulated as a very simple optimization problem with a simple objective function and a set of constraint equations that must be satisfied. These relationships are provided below in Equations C.6 through C.12. The objective function, given by Equation C.6, should be minimized subject to the four constraints given in Equations C.7 through C.10. Because there are so many different variables involved in these equations, it may be helpful to refer to Figure C.3 to help understand each equation.

Objective Function:

$$Z(h,h') = \sum_{i=1}^{n} (h_i + h'_i) \qquad \textbf{(C.6)}$$

Subject to constraints:

$$h_i, h'_i \geq 0 \qquad \textbf{(C.7)}$$

$$h_i \leq h_{i\,MAX} = H_i - d_{top} \qquad \textbf{(C.8)}$$

$$h'_i \leq h'_{i\,MAX} = H'_i - d_{bottom} \qquad \textbf{(C.9)}$$

$$M_{Red} = \sum M_{D0}\left(1 - \frac{FS_0}{FS_1}\right) = \Sigma\Delta M_D - \Sigma\Delta M_R \qquad \textbf{(C.10)}$$

where

$$\Delta M_D = (\gamma_{soil} - \gamma_{EPS})(h + h')b\sin\alpha \qquad \textbf{(C.11)}$$

$$\Delta M_R = \left[(\gamma_{soil} - \gamma_{EPS})(h + h')b - \gamma_w h' L\cos\alpha\right]\cos\alpha\tan\phi' \qquad \textbf{(C.12)}$$

h_i = height of EPS-block geofoam to be included in slice "*i*" above ground water table

h'_i = height of EPS-block geofoam to be included in slice "*i*" below ground water table

$h_{i\,MAX}$ = max allowable height of EPS-block geofoam in slice "i" above ground water table

$h'_{i\,MAX}$ = max allowable height of EPS-block geofoam in slice "i" below ground water table

d_{top} = buffer distance between top of EPS blocks and ground surface

d_{bottom} = buffer distance between bottom of EPS blocks and slip surface

γ_{soil} = unit weight of natural soil

γ_{EPS} = unit weight of EPS-block geofoam

γ_w = unit weight of water

b_i = Width of slice "i"

α_i = Angle from the horizontal to bottom of slice "i"

ϕ' = effective friction angle of soil

$\sum M_{D\,0}$ = sum of driving moments from stability analysis of natural slope prior to installation of EPS-block geofoam

FS_0 = factor of safety of existing slope prior to the construction of the EPS-block geofoam fill

FS_1 = target factor of safety for slope after the construction of the EPS-block geofoam fill

As previously noted, this optimization method assumes changes in both the driving and resisting moments due to construction of EPS-block geofoam fill can be accounted for in the M_{red} term used in Equation C.10. This method also assumes that EPS fill will be placed in a soil mass having relatively uniform unit weight and shear strength parameters. A detailed explanation of how to set up this optimization procedure in a spreadsheet is provided in a subsequent section.

The goal of the optimization is to obtain a minimum value for the objective function given by Equation C.6, subject to constraints given by Equations C.7 through C.10. The first constraint, Equation C.7, essentially states that there can be no negative heights of geofoam assigned to any slice. This constraint is necessary to ensure that the optimization stays true to the real-world situation it is intended to model. Trying to place a negative height of geofoam into a slice simply makes no sense.

The second and third constraints, given by Equations C.8 and C.9 respectively, are intended to prevent the height of geofoam specified for any slice from extending beyond the boundaries of the slice itself. As can be seen in Figure C.3, every slice within the failure mass is bounded by the ground surface above and the slip surface beneath. Thus, the maximum possible value for h_i, the height of EPS geofoam to be placed above the ground water table, is the difference in elevation between the ground surface and the ground water table. Likewise, the maximum possible value for h'_i, the height of EPS geofoam to be placed below the ground water table, is the difference in elevation between the ground water table and slip surface. The values of d_{top} and d_{bottom} are intended to define a sort of "buffer zone" between EPS fill and the ground surface, in the case of d_{top}, or the slip surface, in the case of d_{bottom}. More details about how to select appropriate values for these two parameters are provided later in the discussion.

The fourth and final constraint, Equation C.10, is the key to the entire procedure. It is this equation that relates the height of geofoam to be placed in each slice to the value of M_{red} and thus, to the target factor of safety, FS_1.

It should be noted that in Figure C.3(b), the ground water table depicted by a dashed line represents the location of the original ground water table that was present prior to construction of the EPS-block geofoam fill. The ground water table depicted by a solid line running along the bottom of the EPS block represents the maximum possible water level in that slice after construction of the EPS-block geofoam fill. The reason for the difference is that the entire design procedure is based on the assumption that an adequate subsurface drainage system will be installed behind and beneath the EPS-block geofoam fill. Thus, the ground water table will automatically be lowered when the EPS-block geofoam fill is constructed, because the fill system will include a subsurface drainage system. The effects of this lowering of the ground water table and resulting increase in effective stress at the bottom of each slice are accounted for in the derivation of both optimization methods. For slopes containing no ground water, both optimization procedures can be simplified by simply assuming that H' and h' are equal to zero. Additional guidance on implementing this procedure will be provided later in this chapter.

Optimization Method for Non-Circular Slip Surfaces

Just as with circular slip surfaces, optimization of an EPS-block geofoam fill for a slide involving a non-circular slip surface can be set up as a simple optimization problem, with an objective function and a set of constraint equations. The only difference between the method for circular slip surfaces described above and this method for non-circular slip surfaces is the slope stability analysis method upon which each method is based. The above method was derived from the Ordinary Method of Slices (Fellenius, 1936), which satisfies only moment equilibrium, and is applicable only to slides with circular slip surfaces. This optimization method is derived from the Simplified Janbu Method (Janbu, 1954), which

satisfies only force equilibrium and is applicable to slides with a slip surface of any shape, circular or non-circular. Therefore, technically, this second method could be used for both cases; however, because the constraint equations derived from the Ordinary Method of Slices are considerably simpler and slightly more accurate than those based on the Simplified Janbu Method, it was concluded that it might be helpful to include both methods. Reasons for the difference in accuracy between the two methods will be discussed in detail later in this chapter.

The Simplified Janbu Method is based on the concept of force equilibrium. Janbu derived his equation for the factor of safety of a slope directly from equations of static force equilibrium (Janbu, 1954). In other words, Janbu's equation for the factor of safety of a slope requires that the sum of all forces acting on the failure mass in both the X and Y directions, as shown in Figure C.2, be equal to zero in each direction (Janbu, 1954). The optimization procedure for non-circular failure surfaces is provided below in Equations C.13 through C.21.

Objective Function:

$$Z(h,h') = \sum_{i=1}^{n} (h_i + h'_i) \qquad \textbf{(C.13)}$$

Subject to constraints:

$$h_i \geq 0 \qquad \textbf{(C.14)}$$

$$h_i \leq h_{i\,MAX} = H_i - d_{top} \qquad \textbf{(C.15)}$$

$$h'_i \leq h'_{i\,MAX} = H'_i - d_{bottom} \qquad \textbf{(C.16)}$$

$$F_{red} = \left(\Sigma B_0 + Q_0\right)\left(1 - \frac{FS_0}{FS_1}\right) = (2 \times \Sigma\Delta B - \Sigma\Delta\overline{Q}) - \Sigma\,\Delta A \qquad \textbf{(C.17)}$$

where

$$\Delta B = (\gamma_{soil} - \gamma_{EPS})(h + h')b\tan\alpha \qquad \textbf{(C.18)}$$

$$\Delta A = \frac{\left[(\gamma_{soil} - \gamma_{EPS})(h + h') - \gamma_w h'\right]b\tan\phi'}{n_{\alpha 1}} \qquad \textbf{(C.19)}$$

$$\Delta\overline{Q} = \left(\frac{1}{FS_0 n_{\alpha 0}} - \frac{1}{FS_1 n_{\alpha 1}}\right)A_0 n_{\alpha 0} + \left(\frac{1}{FS_1 n_{\alpha 1}}\right)\left[(\gamma_{soil} - \gamma_{EPS})(h + h') - \gamma_w h'\right]b\tan\phi' \qquad \textbf{(C.20)}$$

$$n_{\alpha 1} = \cos^2\alpha\left(1 + \frac{\tan\alpha\tan\phi'}{FS_1}\right) \qquad \textbf{(C.21)}$$

H_i = height of natural soil within slice "*i*" above water table prior to construction of EPS-block geofoam fill

H'_i = height of natural soil within slice "*i*" below water table prior to construction of EPS-block geofoam fill

h_i = height of EPS-block geofoam to be included in slice "*i*" above ground water table

h'_i = height of EPS-block geofoam to be included in slice "*i*" below ground water table

$h_{i\,MAX}$ = max allowable height of EPS-block geofoam in slice "*i*" above ground water table

$h'_{i\,MAX}$ = max allowable height of EPS-block geofoam in slice "*i*" above ground water table

d_{top} = buffer distance between top of EPS blocks and ground surface

d_{bottom} = buffer distance between bottom of EPS blocks and slip surface

γ_{soil} = unit weight of natural soil

γ_{EPS} = unit weight of EPS-block geofoam

γ_w = unit weight of water

b_i = width of slice "i"

α_i = angle from horizontal to bottom of slice "i"

ϕ' = effective friction angle of soil

$\Sigma B_0 + Q_0$ = sum of driving forces from stability analysis of slope prior to construction of EPS block geofoam fill

$Q_0 = -\Sigma(-S_0 \cos\alpha + N_0 \sin\alpha)$

N_0 = normal force acting on base of slice in existing slope

S_0 = shear force acting on base of slice in existing slope

A_0 = value of *A* from Simplified Janbu analysis (Equation C.3) of the slope prior to construction of EPS-block geofoam fill

FS_0 = factor of safety of existing slope prior to construction of EPS-block geofoam fill

FS_1 = target factor of safety for slope after construction of EPS-block geofoam fill

While this set of equations and variables may look imposing, the concept behind them is essentially the same as that discussed in the previous section. Since the Simplified Janbu Method is a force equilibrium method, the variable M_{red} from the previous method is replaced here by F_{red}. Whereas M_{red} represents the net reduction in the driving *moments* in the slope due to the addition of geofoam, F_{red} represents the net reduction in driving *forces* in the slope due to the addition of geofoam. In the first method, Equations C.11 and C.12 were based on the factor of safety equation for the Ordinary Method of Slices given by Equation C.2 and were used to calculate M_{red}. In the same way, this second method uses Equations C.18, C.19, and C.20, which are based on the Simplified Janbu Method given by Equation C.3, to calculate F_{red}.

It may also be helpful to refer back to Figure C.3 when considering the equations understand what each of the variables actually means. Also, it is important to bear in mind that this optimization method is based on the assumption that the change in both the driving and resisting forces due to construction of the EPS-block geofoam fill can be accounted for in the F_{red} term used in Equation C.17. This method also assumes that the EPS fill will be placed in a soil mass having relatively uniform unit weight and shear strength parameters. As with the optimization method for circular slip surfaces, the set of equations given above has been set up so as to be readily adaptable for use in a spreadsheet. The next section will explain how to set up a spreadsheet to perform this optimization.

SUGGESTED SPREADSHEET SETUP

The optimization methods presented above were developed under the assumption that some sort of spreadsheet software would be available to most designers to assist in determining the optimum

volume and location for EPS-block geofoam fill. Theoretically, almost any spreadsheet software could be used; however, this description focuses specifically on Microsoft Excel® because this software package has seen extremely widespread use in U.S. industry and should be readily available to almost any designer. However, please note that the general approach provided here may be easily adapted to work with other spreadsheet software. Additionally, the approach outlined here might also serve as a blueprint for future work to develop an optimization method based on a more complex slope stability method, such as Spencer's Method, which satisfies full static equilibrium, and is therefore more accurate than either of the methods used for this procedure.

To successfully perform the optimization procedure described above, several sub-steps must be performed. These sub-steps are intended to fit into the overall design procedure presented in Figure 4.25, Step 3, regarding optimizing the volume and location of EPS-block geofoam fill. Thus, this sequence of sub-steps will pick up where the design procedure left off for Step 3.

Step 3.i: Obtain results of preliminary slope stability analysis

This step should have been performed as part of Step 1 of the design procedure. However, if the overall design procedure is not being used, a stability analysis of the slope prior to construction of the EPS-block geofoam fill must be performed in order to have the required optimization input parameters. A list of required input parameters for both optimization methods is provided in Table C.1 below.

Table C.1. Required input parameters for both of the proposed optimization methods.

Parameter	Description
FS_0	= factor of safety of the slope prior to the construction of the EPS-block geofoam fill
FS_1	= target factor of safety for the slope that is to be achieved after completion of the EPS-block geofoam fill
γ_{soil}	= unit weight of the soil or fill material that the EPS-block geofoam fill will displace
γ_{EPS}	= unit weight of EPS blocks to be used (selected in Step 2 of the design procedure)
γ_w	= unit weight of water = 62.4 lb/ft^3 (9.81 kN/m^3)
ϕ'	= effective friction angle of the soil at the slip surface
b	= width of each slice (See Figure C.3)
α	= angle from the horizontal to the inclination of the bottom of each slice (See Figure C.3)
*L	= length of the bottom of each slice (See Figure C.3)

*Not required for optimization method for non-circular slip surfaces

In addition to the parameters specified in Table C.1, it may also be necessary to obtain some additional information to calculate ΣM_{D0} for circular slip surfaces, or $(\Sigma B_0 + Q_0)$ for non-circular slip

surfaces. Remember that ΣM_{D0} represents the sum of disturbing moments in the Ordinary Method of Slices, and that $(\Sigma B_0 + Q_0)$ represents the sum of disturbing forces in the Simplified Janbu Method. Thus, it may be necessary to obtain the input parameters necessary to perform an Ordinary Method of Slices analysis or a Simplified Janbu analysis of the pre-construction slope. Equations C.2 and C.3, respectively, show exactly what parameters are required to perform each of these analyses. Further guidance on how to obtain either ΣM_{D0} or $(\Sigma B_0 + Q_0)$ is provided below in Step 3.iii.

It is important to note that the optimization procedures presented herein yield the optimum arrangement for an EPS fill to increase the factor of safety to some target value. So, this optimization procedure can be applied to any slope, existing or proposed. Much of the discussion above might possibly be interpreted as to imply that the slope for which the EPS fill is being designed must contain an existing failure surface before it can be optimized. This, however, is not the case.

For example, consider the case of a natural slope that is already stable. The slope contains no active slip surfaces, and its factor of safety can be assumed to be greater than 1.0. However, suppose that a new highway was proposed that required a large fill be placed on the side of the slope, and that this new fill would decrease the slope's factor of safety to less than 1.0. It would be possible to perform a slope stability analysis of the slope with the proposed fill in place, and then use results from this analysis to determine the optimum volume and location for an EPS-block geofoam fill that would replace a portion of the proposed soil fill, and ensure that the slope remained stable.

The same approach could also be used to design an EPS-block geofoam fill to be used in conjunction with other slope stabilization methods, such as lowering the ground water table, or altering the slope geometry, to increase the factor of safety. For example, suppose that a slope failure occurred due to an extremely high ground water level that developed after a period of heavy rains. It might be desirable to use a subsurface drainage system to lower the ground water table in the remediated slope, thus raising the factor of safety. However, suppose that to attain the required factor of safety for the repaired slope, the subsurface drainage system would need to be quite extensive. In this case, it might be desirable to install a smaller subsurface drainage system and use EPS-block geofoam fill to further increase the factor of safety until the target factor of safety is reached. In this case, it would be possible to perform a stability analysis of the slope, including the smaller subsurface drainage system and resulting lowered ground water table. Results of this analysis could then be used to perform the proposed optimization procedure to arrive at the optimum configuration for EPS-block geofoam fill to achieve the required factor of safety.

Step 3.ii: Copy or import stability analysis results into spreadsheet

The method used to perform this step will vary widely depending on what type of slope stability software was used for the preliminary analysis. Different software packages have different options for exporting or copying data. Thus, it is up to the designer to understand how to use this capability in the specific slope stability program used for the preliminary analysis.

Regardless of how the results are transferred to the spreadsheet, suggested spreadsheet formats are provided in Figures C.4 and C.5 for optimization methods for slides involving circular and non-circular slip surfaces, respectively. These suggested spreadsheet formats can also serve, along with Table C.1, as a list of the exact parameters that must be known to perform the optimization calculations. It should be noted that not all slope stability programs will have the capability to directly export all of these parameters. In all likelihood, some calculations will have to be performed within the spreadsheet itself to obtain all of the necessary information to begin the optimization procedure. For example, the values in columns G and J shown in Figure C.4 were calculated using values in columns H, I, and J, respectively. To clarify what each of these variables actually stand for, it may be helpful to refer back to Figure C.3.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T
2	γ_{soil} =	120	lb/ft³		d_{top} =	1	ft													
3	c' =	0	lb/ft²		d_{bottom} =	3	ft									*Ordinary Method of Slices Optimization*				
4	φ' =	20	deg			4	ft										FS_0 = 1.04			
5	γ_w =	62.4	lb/ft³														FS_t = 1.25			
6	γ'_{sat} =	57.6	lb/ft³														M_{req} = 11926.6			
7	γ_{EPS} =	6	lb/ft³														$\Sigma\Delta M_D - \Sigma\Delta M_R$ = 11926.6			
8																	Difference = 0.0			
9												*Ordinary Method of Slices*								
10												FS_{OMS} =	0.92				FS_{CHECK} = 1.24			
11	Slice	b	L	α	W	u	Z_w	Z_{bottom}	Z_{top}	H	H'	M_R	M_D	h	h'	h+h'	h_{MAX}	h'_{MAX}	ΔM_D	ΔM_R
12	No.	(ft)	(ft)	(deg)	(lb)	(lb/ft²)	(ft)	(ft)	(ft)	(ft)	(ft)	(lb)	(lb)	(ft)	(ft)	(ft)	(ft)	(ft)	(lb)	(lb)
13	1	4.56	9.48	61.23	2238.75	0.00	1440.85	1440.85	1444.94	7.72	0.00	392.1	1962.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00
14	2	4.56	7.90	54.76	6209.57	47.61	1434.23	1433.47	1444.82	13.46	0.76	1258.6	5071.4	6.36	0.00	6.36	12.46	0.00	2701.06	694.61
15	3	4.56	6.98	49.21	9353.89	367.41	1433.49	1427.60	1444.69	13.57	5.89	1825.7	7082.1	7.56	0.25	7.81	12.57	2.89	3073.91	948.56
16	4	4.56	6.36	44.24	11947.40	624.39	1432.74	1422.74	1444.57	13.82	10.01	2372.9	8335.3	6.32	4.78	11.11	12.82	7.01	4027.69	1150.63
17	5	4.56	5.92	39.66	14129.30	834.44	1431.99	1418.62	1444.45	14.14	13.37	2892.8	9018.1	0.00	8.48	8.48	13.14	10.37	2814.37	559.23
18	6	4.56	5.59	35.37	15981.90	1006.94	1431.25	1415.12	1444.32	14.51	16.14	3380.5	9251.6	0.00	11.52	11.52	13.51	13.14	3467.23	804.60
19	7	4.56	5.34	31.30	17558.80	1147.99	1430.51	1412.11	1444.20	14.92	18.40	3832.7	9122.3	0.00	0.00	0.00	13.92	15.40	0.00	0.00
20	8	4.56	5.14	27.40	18896.60	1261.78	1429.76	1409.54	1444.08	14.91	20.22	4247.0	8696.0	0.00	0.00	0.00	13.91	17.22	0.00	0.00
21	9	4.56	4.98	23.63	19604.00	1351.32	1429.02	1407.37	1443.09	14.00	21.66	4482.2	7858.4	0.00	0.00	0.00	13.00	18.66	0.00	0.00
22	10	4.56	4.85	19.97	19478.40	1418.82	1428.28	1405.54	1441.14	12.57	22.74	4450.0	6652.6	0.00	0.00	0.00	11.57	19.74	0.00	0.00
23	11	4.56	4.75	16.39	19166.90	1465.92	1427.53	1404.04	1439.07	11.10	23.49	4358.5	5409.2	0.00	0.00	0.00	10.10	20.49	0.00	0.00
24	12	4.56	4.68	12.88	18687.30	1493.84	1426.78	1402.85	1437.00	9.63	23.94	4213.5	4165.5	0.00	0.00	0.00	8.63	20.94	0.00	0.00
25	13	4.56	4.62	9.42	18047.60	1503.51	1426.04	1401.95	1434.93	8.16	24.09	4018.5	2952.4	0.00	0.00	0.00	7.16	21.09	0.00	0.00
26	14	4.56	4.59	5.99	17253.50	1495.66	1425.30	1401.33	1432.88	6.70	23.97	3776.9	1799.2	0.00	0.00	0.00	5.70	20.97	0.00	0.00
27	15	4.56	4.56	2.58	16308.60	1470.43	1424.55	1400.99	1430.80	5.25	23.56	3491.8	733.4	0.00	0.00	0.00	4.25	20.56	0.00	0.00
28	16	4.56	4.56	-0.82	15215.00	1428.34	1423.81	1400.92	1428.73	3.78	22.89	3166.9	-218.2	0.00	0.00	0.00	2.78	19.89	0.00	0.00
29	17	4.56	4.57	-4.22	13973.10	1369.33	1423.06	1401.12	1426.66	2.32	21.94	2805.5	-1029.1	0.00	0.00	0.00	1.32	18.94	0.00	0.00
30	18	4.56	4.60	-7.64	12581.70	1293.29	1422.32	1401.60	1424.59	0.92	20.73	2411.3	-1672.9	0.00	0.00	0.00	0.00	17.73	0.00	0.00
31	19	4.56	4.65	-11.09	11056.10	1199.87	1421.57	1402.35	1422.64	0.00	19.23	1994.4	-2125.6	0.00	0.00	0.00	0.00	16.23	0.00	0.00
32	20	4.56	4.71	-14.57	9746.01	1088.55	1420.83	1403.39	1421.20	0.00	17.44	1684.6	-2451.9	0.00	0.00	0.00	0.00	14.44	0.00	0.00
33	21	4.56	4.80	-18.11	8423.74	958.58	1420.09	1404.73	1420.11	0.00	15.36	1401.9	-2619.0	0.00	0.00	0.00	0.00	12.36	0.00	0.00
34	22	4.56	4.91	-21.73	6901.51	787.01	1419.99	1406.38	1419.99	0.00	12.61	1120.1	-2555.1	0.00	0.00	0.00	0.00	9.61	0.00	0.00
35	23	4.56	5.05	-25.44	5187.02	591.50	1417.85	1408.38	1417.85	0.00	9.48	818.3	-2228.1	0.00	0.00	0.00	0.00	6.48	0.00	0.00
36	24	4.56	5.23	-29.27	3270.57	372.96	1416.71	1410.74	1416.71	0.00	5.98	498.4	-1598.9	0.00	0.00	0.00	0.00	2.98	0.00	0.00
37	25	4.56	5.45	-33.25	1129.74	128.83	1414.07	1412.01	1416.14	0.00	2.06	165.1	-619.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00
38										*SUMMATIONS =*		65060.3	70991.9			45.28			16084.3	4157.6

Figure C.4. Suggested spreadsheet format based on NY Route 23A case history using circular slip surface optimization method based on Ordinary Method of Slices

Difference = 0.001
$FS_{Corrected}$ = 0.000

F_{sol} = 11571.40
(Σ×ΣΔB-ΣΔQ)/ΣΔA = 11571.40
Difference = 0.000

Preliminary Analysis

FS_{Bishop} = 1.370

FS_{Janbu} = 1.435

Slice No	b (ft)	L (ft)	α (deg)	W (lb)	S (lb)	N (lb)	-S cosα+N sinα (lb)	u (lb/ft²)	Z_t (ft)	Z_{bottom} (ft)	Z_{top} (ft)	H (ft)	H' (ft)	r_u	A (lb)	B (lb)	h (ft)	h' (ft)	h+h' (ft)	h_{water} (ft)	h'_{water} (ft)	r_{qa}	r_{qa}	ΔA (lb)	ΔB (lb)	ΔQ (lb)
1	9.69	16.92	55.07	7617.09	2795.08	9294.46	9020.30	0.00	83.02	83.02	89.85	6.84	0.00	0.433	5830.7	10903.6	0.00	0.00	0.00	0.00	0.00	0.427	0.414	0.0	0.0	461.9
2	9.69	16.92	55.07	23851.30	7313.23	29421.40	19935.66	0.00	69.15	69.15	89.68	20.52	0.00	0.433	15255.9	32721.8	9.43	0.00	9.43	17.52	0.00	0.427	0.427	2943.3	14872.7	6008.4
3	9.38	9.64	13.70	30172.00	6537.52	29456.80	625.05	221.59	64.62	61.67	86.83	24.21	3.55	0.996	7156.8	7205.8	0.00	0.00	0.00	21.21	0.55	0.992	0.992	0.0	0.0	798.6
4	9.38	9.64	13.70	28618.20	6314.04	28941.60	719.90	221.33	63.14	58.79	86.26	23.12	4.36	0.996	7147.0	7203.3	0.00	0.00	0.00	20.12	1.36	0.992	0.992	0.0	0.0	776.6
5	9.38	9.64	13.70	28028.20	5893.76	27418.90	795.85	221.06	61.65	56.51	82.54	20.89	5.15	0.996	7204.9	6831.9	0.00	0.00	0.00	17.89	2.15	0.992	0.992	0.0	0.0	721.3
6	9.38	9.64	13.70	26435.80	5412.91	25883.80	871.76	220.80	63.17	54.22	78.78	18.61	5.94	0.996	6851.0	6443.7	0.00	0.00	0.00	15.61	2.94	0.992	0.992	0.0	0.0	665.9
7	9.38	9.64	13.70	26438.20	5092.45	24939.90	958.53	420.54	59.88	51.94	75.89	17.18	6.74	0.996	6340.5	6275.0	0.00	0.00	0.00	14.18	3.74	0.992	0.992	0.0	0.0	634.8
8	8.92	8.92	1.46	24167.00	4831.51	24051.80	-4215.50	469.90	54.32	50.68	74.24	16.03	7.53	1.005	5769.0	617.6	0.00	0.00	0.00	13.02	4.53	1.005	1.005	0.0	0.0	577.6
9	8.92	8.92	1.46	23221.20	4633.13	23110.20	-4041.24	463.47	57.99	50.46	73.09	15.20	7.43	1.005	5532.1	591.4	0.00	0.00	0.00	12.20	4.43	1.005	1.005	0.0	0.0	553.9
10	8.92	8.92	1.46	22274.80	4434.74	22168.50	-3868.97	456.98	57.55	50.33	71.94	14.39	7.32	1.005	5295.2	566.2	0.00	0.00	0.00	11.39	4.32	1.005	1.005	0.0	0.0	530.1
11	8.92	8.92	1.46	20476.60	4046.18	20375.70	-3524.23	450.50	57.22	50.90	69.67	12.40	7.22	1.005	4738.2	511.3	0.00	0.00	0.00	9.40	4.22	1.005	1.005	0.0	0.0	474.4
12	8.92	8.92	1.46	16752.80	3293.61	16671.90	-2866.58	410.91	55.38	49.77	66.29	9.93	6.59	1.005	3885.2	433.2	0.00	0.00	0.00	6.93	3.59	1.005	1.005	0.0	0.0	389.0
13	8.81	8.87	6.68	14461.10	2889.04	14221.10	-1215.02	239.03	54.58	48.14	63.43	8.85	5.43	1.012	3472.2	1693.0	0.00	0.00	0.00	5.85	2.43	1.011	1.011	0.0	0.0	347.6
14	8.81	8.87	6.68	13895.30	2833.59	12674.50	-1141.94	282.94	52.81	48.11	60.85	8.04	4.69	1.012	3165.2	1503.7	0.00	0.00	0.00	5.04	1.69	1.011	1.011	0.0	0.0	318.9
15	8.81	8.87	6.68	11454.90	2405.77	11251.00	-1081.19	246.84	51.04	47.88	58.43	7.39	3.86	1.012	2800.4	1345.1	0.00	0.00	0.00	4.39	0.96	1.011	1.011	0.0	0.0	290.4
16	8.81	8.87	6.68	10265.40	2237.61	10093.20	-1048.80	200.75	49.27	46.85	56.21	6.94	3.22	1.012	2699.3	1204.2	0.00	0.00	0.00	3.94	0.22	1.011	1.011	0.0	0.0	269.2
17	8.81	8.87	6.68	9126.24	2071.50	8944.60	-1017.37	154.85	47.50	45.82	54.03	6.53	2.48	1.012	2489.8	1068.3	0.00	0.00	0.00	3.53	0.00	1.011	1.011	0.0	0.0	249.3
18	8.81	8.87	6.68	7965.04	1905.38	7795.98	-985.94	108.56	45.73	42.98	51.88	6.13	1.74	1.012	2290.0	932.5	0.00	0.00	0.00	3.13	0.00	1.011	1.011	0.0	0.0	229.3
19	8.81	8.87	6.68	6804.93	1739.26	6647.21	-954.52	62.40	43.96	42.98	49.69	5.72	1.00	1.012	2090.3	796.7	0.00	0.00	0.00	2.72	0.00	1.011	1.011	0.0	0.0	209.3
20	8.81	8.87	6.68	5644.83	1573.14	5498.67	-923.09	16.36	42.19	41.93	47.50	5.31	0.26	1.012	1890.7	660.9	0.00	0.00	0.00	2.31	0.00	1.011	1.011	0.0	0.0	189.3
21	8.81	8.87	6.68	4484.43	1349.37	4356.78	-833.61	0.00	43.90	40.80	45.33	4.43	0.00	1.012	1621.8	525.0	0.00	0.00	0.00	1.43	0.00	1.011	1.011	0.0	0.0	162.4
22	8.81	8.87	6.68	3324.22	1093.88	3216.62	-712.20	0.00	39.87	39.87	43.15	3.28	0.00	1.012	1214.7	389.2	0.00	0.00	0.00	0.28	0.00	1.011	1.011	0.0	0.0	121.6
23	8.81	8.87	6.68	2164.02	838.38	2086.46	-590.78	0.00	38.84	38.84	40.97	2.14	0.00	1.012	1007.6	253.4	0.00	0.00	0.00	0.00	0.00	1.011	1.011	0.0	0.0	100.9
24	8.81	8.87	6.68	1003.81	552.88	942.30	-469.36	0.00	37.81	37.81	38.80	0.99	0.00	1.012	700.5	117.5	0.00	0.00	0.00	0.00	0.00	1.011	1.011	0.0	0.0	70.1
25	10.39	10.61	11.69	249.99	479.44	195.99	-437.91	0.00	37.29	37.29	37.71	0.42	0.00	1.003	651.2	103.4	0.00	0.00	0.00	0.00	0.00	1.001	1.001	0.0	0.0	65.2
SUMMATIONS =							0.02								108590.0	91087.5			9.43					2943.26	14872.74	15230.8

Figure C.5. Suggested spreadsheet format based on Colorado Hwy. 160 case history using non-circular slip surface optimization method based on Simplified Janbu Method.

Step 3.iii: Use slice data to determine either $\Sigma \mathbf{M_{D0}}$ or $(\Sigma \mathbf{B_0} + \mathbf{Q_0})$ of slope prior to placement of EPS-block geofoam fill

The next step in the optimization procedure is to calculate the value of either ΣM_{D0} or $(\Sigma B_0 + Q_0)$ for the slope prior to construction of the EPS-block geofoam fill. These values will then be put into either Equation C.10 to determine M_{red} for the optimization method for circular slip surfaces, or Equation C.17 to determine F_{red} for the optimization method for non-circular slip surfaces.

As noted above, if the slope stability software used to perform the initial stability analysis allows the user to directly output the disturbing moment of force for each slice, then this capability may be used to determine either ΣM_{D0} or $(\Sigma B_0 + Q_0)$, depending on which optimization method is used. If the slope stability software does not have this capability, the slice data that is available must be exported and used to develop a spreadsheet solution to determine either ΣM_{D0} or $(\Sigma B_0 + Q_0)$. In other words, slice data exported from the software will be used to perform either an Ordinary Method of Slices analysis, or a Simplified Janbu analysis on the slope to determine the values for either ΣM_{D0} or $(\Sigma B_0 + Q_0)$. The suggested spreadsheet formats presented below provide additional guidance in setting up these analyses. If additional guidance is needed, Duncan and Wright's (2005) textbook on slope stability can be very helpful in setting up these and other slope stability analysis methods in a spreadsheet.

Whether values for ΣM_{D0} and $(\Sigma B_0 + Q_0)$ are obtained directly from slope stability software or calculated in the spreadsheet, it is desirable to obtain values for these parameters based on the same stability analysis methods used here. This means that ΣM_{D0} should be determined using the Ordinary Method of Slices, and $(\Sigma B_0 + Q_0)$ should be calculated based on the Simplified Janbu Method. However, this is not strictly necessary. Any analysis method could theoretically be used without introducing much error into the optimization. But, to be consistent with the derivations of the constraint equations used in the procedure developed here, it is recommended that these calculations be based on the appropriate method, if possible. Additionally, since a Simplified Janbu analysis will be required to determine the value of A_0 for Equation C.20 anyway, it makes sense to use results of this same analysis to supply the value of $(\Sigma B_0 + Q_0)$ to be used in Equation C.17.

While it is recommended that the values of ΣM_{D0} or $(\Sigma B_0 + Q_0)$ be determined using the appropriate slope stability analysis method, it is not a requirement that the value of FS_0 be based on the same method. For example, in order to determine the value of M_{red} for the optimization method for circular slip surfaces, it is necessary to determine both ΣM_{D0} and FS_0 for the slope prior to construction of EPS fill. The value of ΣM_{D0} should be determined using the Ordinary Method of Slices, since the optimization procedure for circular slip surfaces is based on this method, and derivations of Equations C.11 and C.12 are directly based on the Ordinary Method of Slices. However, the value of FS_0 may be based on any slope stability analysis method a designer wishes, because the derivation of Equation C.10 is not directly linked to a specific analysis method. The more accurate the input value of FS_0, the more accurate results of the optimization method will be. Therefore, it is advantageous to use a slope stability analysis method that satisfies full equilibrium, such as Spencer's Method, to determine FS_0. In most slope stability software packages, this would be simply a matter of selecting which analysis method is desired, and therefore would not introduce any difficulty into the procedure. However, if a situation

demands that some other slope stability analysis method be used to determine FS_0, it would not introduce any inconsistency into the optimization.

Step 3.iv: Set up optimization equations in spreadsheet

Figures C.4 and C.5 provide a suggested format that may be used to set up a spreadsheet to perform both the slope stability analysis described in Step 3.iii and the proposed optimization procedures for either circular or non-circular slip surfaces. To set up the optimization method for circular slip surfaces, the spreadsheet should be set up as shown in Figure C.4. Equations C.10 through C.12 can then be input into spreadsheet columns R, S, and T. Likewise, to set up the optimization method for non-circular slip surfaces, the spreadsheet can be set up as shown in Figure C.5, and Equations C.17 through C.21 input into columns W, X, Y, and Z. Initially, any values may be inserted for h and h' because the Solver tool will be used to change the values in those cells until the optimum is identified. Thus, it does not matter what values are entered into these cells to begin with.

The only additional issue relating to the set up of optimization procedures is that of determining appropriate values for h_{MAX} and h'_{MAX}. Again, it may be helpful to refer back to Figure C.3 to clarify what these variables represent. These parameters represent the maximum allowable height of geofoam that can be placed within a given slice above and below the ground water table, respectively. In general, these parameters can be calculated using Equations C.8 and C.9 for circular slip surfaces, or Equations C.15 and C.16 for non-circular slip surfaces.

Using these equations will require the designer to select appropriate values for d_{top} and d_{bottom}. These parameters define a sort of "buffer zone" both above and below the EPS-block geofoam fill. These buffer zones should constrain the optimization so that: (1) no EPS blocks will be placed above the ground surface, and (2) no EPS blocks will be placed at or below the failure surface. The first requirement is only logical for a real-world construction project. It would be useless to stack EPS blocks on top of the slope's surface. If any benefit is to be achieved in terms of increasing the factor of safety of the slope, the EPS block must be used to replace some of the soil within the slices, and must, therefore, be kept within the boundaries of the slices. The latter requirement is more important, and it serves two purposes.

The first purpose is to prevent the EPS-block geofoam fill from interrupting the slip surface, thereby avoiding the necessity to include shear strength of EPS blocks in the final slope stability analysis. This is an advantage because there is, as yet, no clear consensus on the best approach to modeling the shear strength of the EPS geofoam in a slope stability analysis.

The second purpose of this constraint is to prevent EPS block from being specified below the level of the failure surface, where they would have no effect on the stability of the failure mass being considered. It is left to the discretion of the designer to select values for d_{top} and d_{bottom}. Obviously, selecting higher values for these parameters will provide greater assurance that the above requirements will be met. However, excessively large values will result in the optimization procedure specifying slightly more geofoam than is truly necessary, because the optimization calculation will not be allowed to completely "fill up" some of the slices with geofoam. In view of these considerations, it is recommended that a value of 3 ft. (1m) be used for d_{top} and a value of 1 ft. (0.3m) be used for d_{bottom} in the absence of better information. For example, if it was known that the location of the slip surface was somewhat uncertain, it might be wise to specify a larger value for d_{bottom} to ensure that the resulting EPS-block geofoam fill would remain entirely within the actual failure mass, even if the true location of the critical slip surface is somewhat different from the estimated location used in the analysis.

Additionally, it would be possible for a designer to specify different values for d_{top} and d_{bottom} for each slice if a situation required it. For instance, if the location of the slip surface is known, with a greater level of precision in one portion of the slope than it is in another, it would be possible to use a

smaller d_{bottom} value in the area where location of the slip surface is more precisely known. This would, in theory, result in a slightly more accurate optimization. However, the impact of this alteration is so small that, for most projects, it would not be worth the trouble.

It should be noted that when setting up the h_{MAX} and h'_{MAX} values in the spreadsheet, negative values must not be allowed. This is very important, because the other constraints are set up in such a way that if negative values of h_{MAX} and h'_{MAX} are included, Solver will not be able to converge to a solution. Therefore, after entering equations for h_{MAX} and h'_{MAX} into the spreadsheet, the user should look back at the resulting values and manually replace any negative values with zeros. This will provide the necessary restrictions on the values of h and h' without causing any errors in Solver.

Step 3.v: Use Solver Add-In to perform optimization

Once the spreadsheet has been set up as shown in Figures C.4 or C.5, optimization can begin. To perform the calculations, it is recommended that the Solver tool included in Microsoft Excel™, or an equivalent tool from a different spreadsheet program, be used. Solver is accessed through the "Tools" menu in Microsoft Excel™. If the Solver tool does not appear under the "Tools" menu, it may be necessary to select "Add-Ins" from the "Tools" menu and install the Solver Add-In.

Once Solver has been successfully opened, the next step is to begin inputting optimization information. Solver is a very powerful and versatile tool, and it is beyond the scope of this report to explain how to utilize it fully. However, Figure C.6 shows an appropriate Solver setup to optimize the problem depicted in Figure C.4.

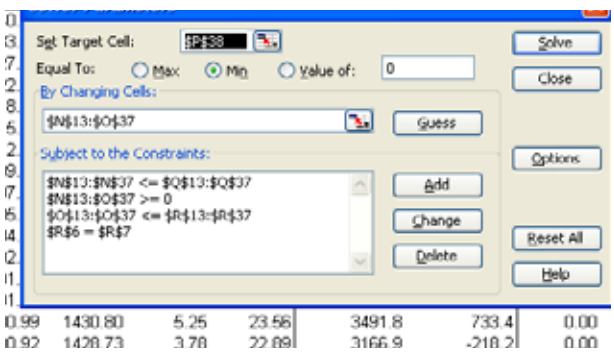


Figure C.6. Solver inputs to optimize EPS-block geofoam fill for NY Route 23A case history shown in Figure C.4.

Likewise, Figure C.7 shows an appropriate Solver setup to optimize the problem depicted in Figure C.5.

Figure C.7. Solver inputs to optimize EPS-block geofoam fill for Colorado Hwy 160 case history shown in Figure C.5.

Solver will set the target cell, which contains the summation of all heights of geofoam in all the slices, to a minimum. This is accomplished by trying different values in certain cells until the desired optimum is achieved. As shown in Figures C.6 and C.7, cells that will be changed in order to search for the optimum will contain the heights of EPS fill to be placed in the slices. The four constraint equations shown in Figure C.6 are based on the optimization constraints given by Equations C.7 through C.10 for the circular slip surface method. Likewise, the four constraints shown in Figure C.7 are based on the optimization constraints for the non-circular slip surface method given by Equations C.14 through C.17. If optimization equations have been input into the spreadsheet correctly, Solver should be able to identify an optimum arrangement for EPS-block geofoam fill.

In certain circumstances it may be desirable to select a specific area within the slope in which to locate EPS-block geofoam fill. In such cases, it would be possible to select only certain slices for Solver to use in calculating the optimum volume and location for EPS fill. For example, suppose that for the case history involving the NY Route 23A slide (Jutkofsky, 1998), the project requirements dictated that the EPS-block geofoam could not extend into slices 1, 2, and 3 shown in the suggested spreadsheet in Figure C.4. Had this been the case, it would have been possible to tell Solver to select all slices except for slices 1, 2, and 3, and to perform the optimization to determine the optimum volume and location for EPS geofoam fill without allowing any geofoam to be placed in these three slices. Referring to Figures C.6 and C.4, this objective could be accomplished by telling the Solver tool to "Set target cell $P$38 to a minimum by changing cells $N$16:$O$37" rather than allowing it to change cells $N$13:$O$37. This technique can be used to further constrain the location of optimized EPS-block geofoam fill. It should be noted that by not allowing the optimization to "use" certain slices, the resulting EPS fill configuration will no longer be a true optimum; however, if the project requirements dictate such a constraint, this may be a helpful technique for the designer to use.

EVALUATION OF PROPOSED OPTIMIZATION PROCEDURES

To evaluate the effectiveness of the procedures for optimizing volume and location of an EPS-block geofoam slope fill that are proposed in this report, two case histories were selected and the EPS fills optimized. Results were compared with the actual EPS-block geofoam fill system that was constructed for each project to see whether the optimization procedure proposed here would result in a significant reduction in the volume of EPS-block geofoam used. Additionally, for the case history that involved a circular failure surface, the Moment Reduction Method (Negussey and Srirajan, 2001) was used, and the results compared with both the actual EPS fill used and EPS fill developed using the proposed optimization method.

The first case history is State Route 23A outside the city of Jewett, NY, where an EPS-block geofoam fill was constructed to stabilize an existing landslide that involved a circular slip surface. The

second case history involves a stretch of Colorado Hwy 160 located between Mesa Verde National Park and the city of Durango, CO. Here an EPS-block geofoam fill was constructed to stabilize an existing landslide involving a non-circular slip surface. Results of these comparisons are presented in the next two sections.

NY Route 23A Case History

A preliminary analysis of the existing slope identified a circular slip surface beginning near the centerline of the existing NY Route 23A and day-lighting at the toe of the slope near Schoharie Creek. The slip surface was identified from the location of the surface scarp as well as by data gathered from inclinometers installed in the slope. Because the slope was already in a state of failure, the initial factor of safety was assumed to be 1.0 and soil parameters along the failure surface were back-calculated using the Simplified Bishop's Method and Janbu's Methods (Jutkofsky, 1998). These back-calculated soil parameters consisted of a unit weight of 120 lb./ft^3 (18.8 kN/m^3), an effective friction angle of 20 degrees, and an effective cohesion of 0 lb./ft^2. An interpreted cross-section of the existing slope, including the assumed failure surface, is shown below in Figure C.8.

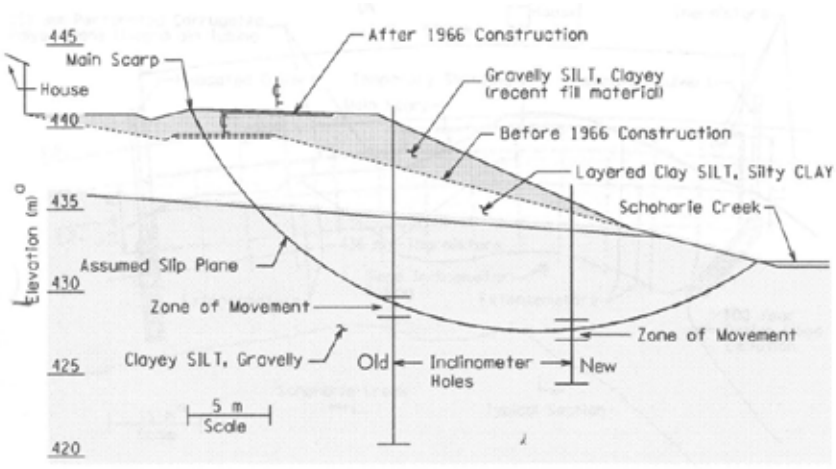


Figure C.8. Cross-section of NY Route 23A slope failure (Jutkofsky et al., 2000).

Developing Slope Model

Using Rocscience SLIDE 5.0 slope stability software, results of this analysis were duplicated using the back-calculated soil properties from Jutkofsky's report (Jutkofsky, 1998). The goal was to determine the factor of safety of the slope model and compare it to the factor of safety from the case history. The soil profile was assumed to consist of a uniform soil having the same back-calculated soil properties specified above. The ground water table and assumed slip surface given by Jutkofsky (1998) were then specified, and Rocscience SLIDE 5.0 was used to determine the factor of safety for assumed slip surface. A screenshot from the software depicting results of this analysis is shown in Figure C.9.

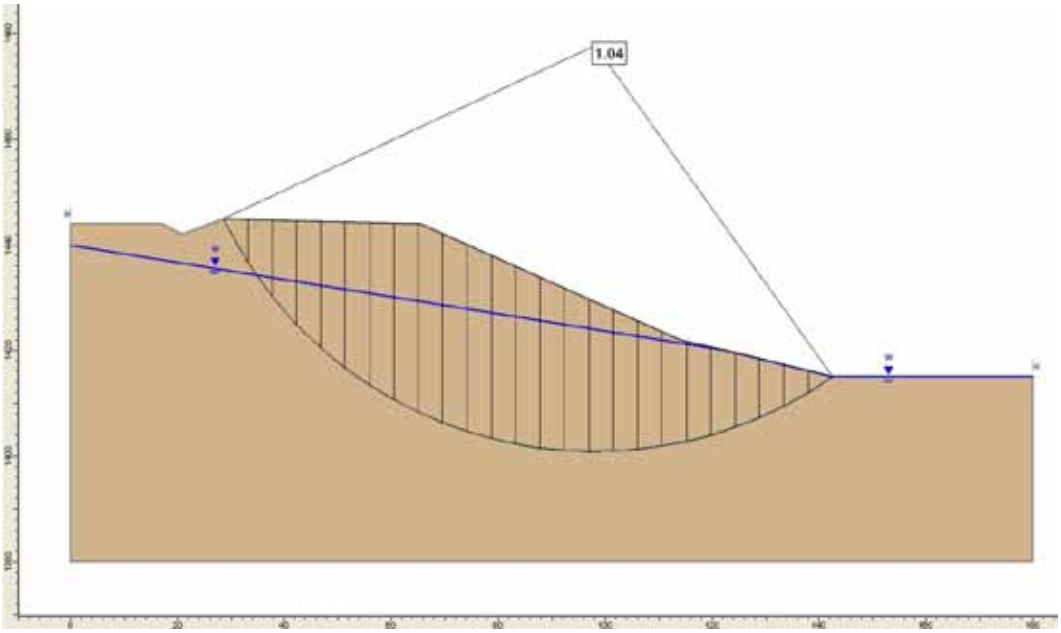


Figure C.9. Results of stability analysis of existing slope using Simplified Bishop's Method with back-calculated soil parameters (Jutkofsky, 1998).

As can be seen in Figure C.9, a slope stability analysis of the slope model using the Simplified Bishop's Method resulted in a factor of safety of 1.04, which is extremely close to the target value of 1.0, suggesting that the slope model shown in Figure C.9 was fairly accurate.

Next, an attempt was made to use the information provided by Jutkofsky (1998) to determine the factor of safety that was achieved by construction of the actual EPS-block geofoam fill. While Jutkofsky's (1998) report does contain several detailed construction drawings of the EPS-block geofoam fill itself, it does not provide much detail concerning the exact geometry of the remediated slope or location of the EPS-block geofoam fill within the slope cross-section. Jutkofsky's (1998) report indicates that the target factor of safety for the remediated slope was 1.25. However, the analysis performed for this report, depicted in Figure C.10, shows a factor of safety of approximately 1.13 for the remediated slope. Thus, the slope cross-section and analysis depicted in Figure C.10 should be considered an approximation, and not an exact representation of the results of the NY Route 23A remediation project. However, it does provide a baseline against which to compare EPS-block geofoam fill designs developed using the Moment Reduction Method and proposed optimization method.

It should also be noted that the actual remediation design for the NY Route 23A slide called for a slight change in geometry for the repaired slope. In the actual project, the upper portion of the earth slope was made slightly steeper than the one shown in Figure C.9. This difference in geometry was incorporated into the analysis of the actual EPS fill shown in Figure C.10, but was not incorporated into either the Moment Reduction Method or proposed optimization method. The reason for this is that both of these methods are intended to optimize the volume and location of EPS-block geofoam required to stabilize the slope that failed. Neither procedure considers any alterations to the slope geometry that might be made in addition to constructing the EPS-block geofoam fill.

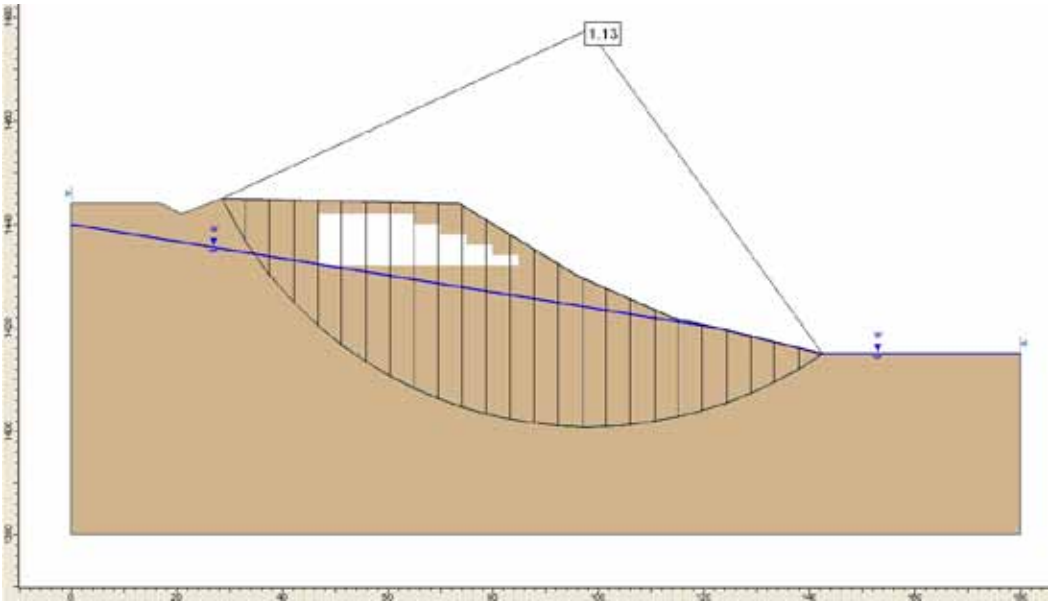


Figure C.10. Results of Simplified Bishop's Method stability analysis of actual EPS-block geofoam fill used to remediate NY Route 23A slope failure.

Moment Reduction Method

Once the slope model had been validated, the Moment Reduction Method proposed by Negussey and Srirajan (2001) was used to design an EPS-block geofoam fill to increase the factor of safety of the slope from 1.0 to 1.25. Although this method was not developed until after the NY Route 23A project had been completed, it is still useful to compare its results with those obtained from the proposed optimization procedure. The Moment Reduction Method was conducted based on the explanation of the process provided in Negussey and Srirajan (2001). Several slope stability analyses were performed, each using a different value for the threshold moment, M_T. This M_T value governs the volume of EPS fill that the Moment Reduction Method will specify, and thus, the factor of safety that will result. The curve relating M_T to the factor of safety is shown in Figure C.11.

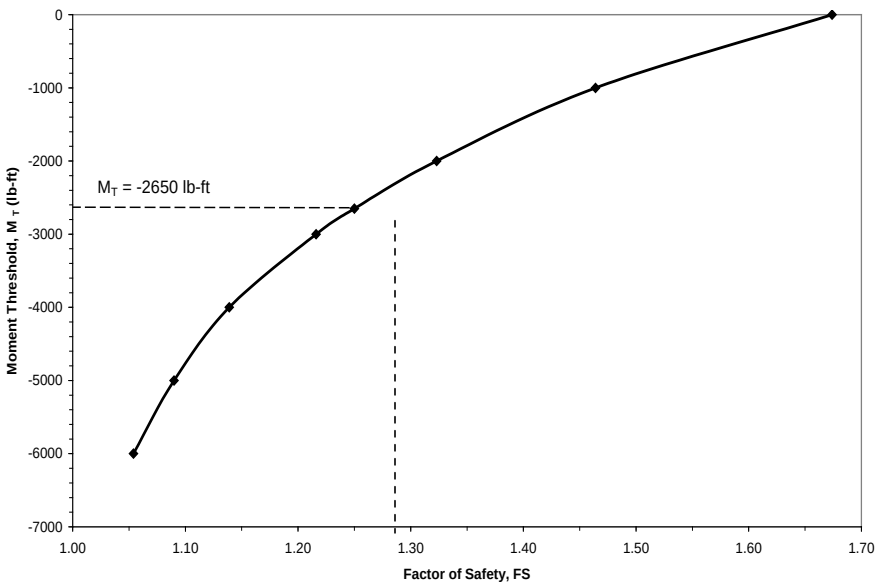


Figure C.11. Plot of threshold moment, M_T, versus factor of safety from the Moment Reduction Method.

Once the plot shown in Figure C.11 had been developed by trial and error, an M_T value of -2650 was identified as corresponding to the target factor of safety of 1.25. The Moment Reduction Method was then performed using this value of M_T, resulting in the EPS-block geofoam fill configuration and factor of safety shown below in Figure C.12.

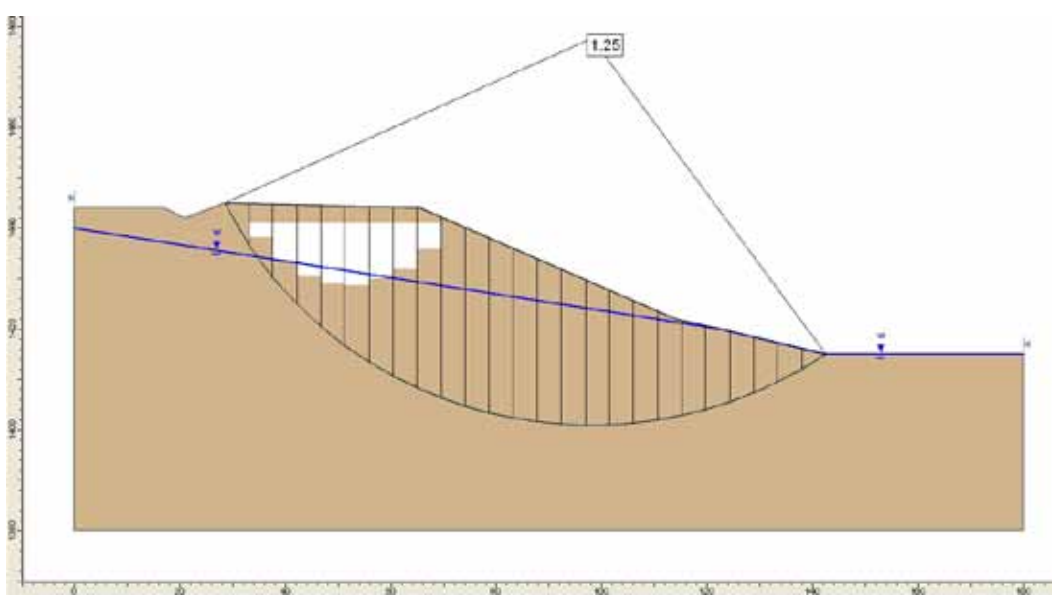


Figure C.12. Results of Simplified Bishop's stability analysis of EPS-block geofoam fill produced by the Moment Reduction Method.

Assuming that the EPS-block geofoam fill cross-section would simply be extended across the entire width of the slide (perpendicular to cross-section shown above) as was done in the actual case history, the Moment Reduction Method would call for a volume of approximately 3,790 yd^3 of EPS-block geofoam to raise the slope's factor of safety to the required 1.25. This is actually slightly more than the 3,685 yd^3 of EPS-block geofoam actually used on the project (Jutkofsky, 1998).

Optimization Procedure for Circular Failure Surfaces

Next, the optimum arrangement for an EPS-block geofoam slope fill was determined using the optimization procedure for circular failure surfaces described in Step 3 of the design procedure. In both cases, the initial factor of safety, FS_0, was assumed to be 1.0 and the target factor of safety, FS_1, was set at 1.25 to match the target factor of safety from the case history. The spreadsheet used to perform the optimization is shown in Figure C.4. Results of this analysis are shown below in Figure C.13.

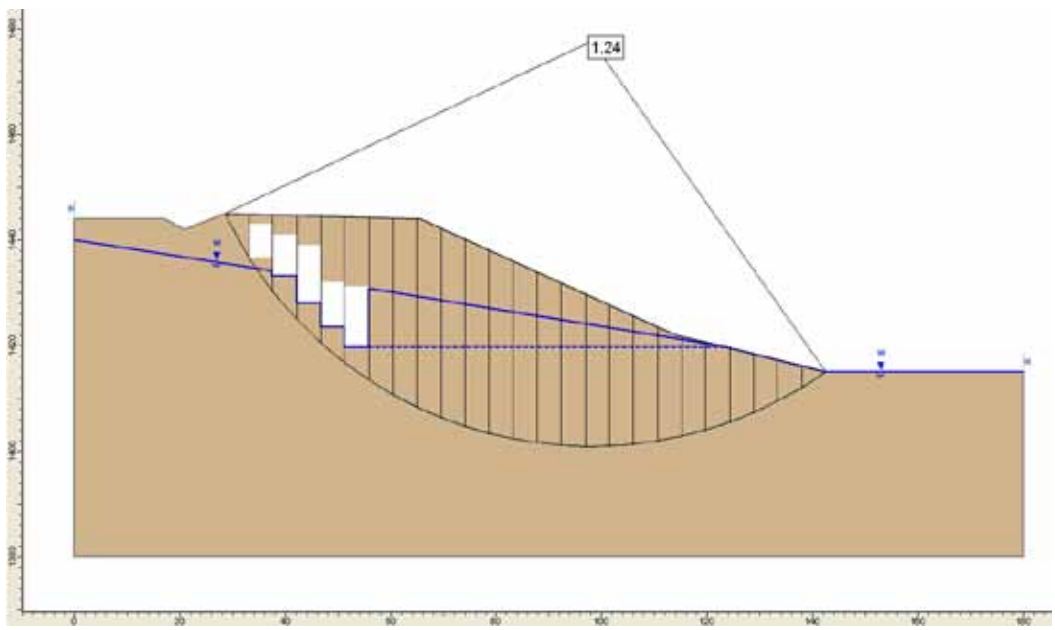


Figure C.13. Results of Simplified Bishop's stability analysis of EPS-block geofoam fill produced by optimization procedure for circular slip surfaces.

It should be pointed out that the unusual shape of the ground water level shown by the solid line in Figure C.13 represents the theoretical maximum possible ground water level in the finished slope, and the same ground water level used by Jutkofsky (1998). In reality, the ground water table would be diverted by the subsurface drainage system, and would probably follow a surface more akin to that depicted by the dashed line in Figure C.13. However, since a higher ground water level generally results in a less stable slope, it was concluded that the ground water level depicted by the solid line would most likely be the "worst case scenario." Therefore, to be conservative, it was this theoretical "worst case scenario" that was used in derivation of the optimization procedure. Thus, the factor of safety of 1.25 for the optimized EPS-block geofoam fill may be regarded as something of a lower bound. In a real slope, this configuration of the EPS-block geofoam fill would result in a slightly higher factor of safety for the slope than what is shown in Figure C.13.

Comparison of Results for Circular Slip Surface Optimization

Again, assuming that the EPS-block geofoam fill cross-section would extend laterally across the entire width of the slide (perpendicular to the cross-section shown above)—the proposed optimization procedure called for a volume of 2,365 yd^3 to raise the slope's factor of safety to the target value of 1.25. This was significantly less than either the volume specified by the Moment Reduction Method or the volume actually used on the project. A comparison of the different EPS-block geofoam fill volumes determined by each approach is shown below in Table C.2.

Table C.2. Comparison of EPS-block geofoam fill volumes from Moment Reduction Method and proposed optimization method against the actual volume used on NY 23A project.

	Figure No.	Target FS	Achieved FS	Total volume of EPS-block geofoam fill
Actual NY 23A Project	C.10	1.25	*1.13	3,685 yd^3
Moment Reduction Method	C.12	1.25	1.25	3,790 yd^3
Proposed Optimization Method	C.13	1.25	1.24	2,365 yd^3

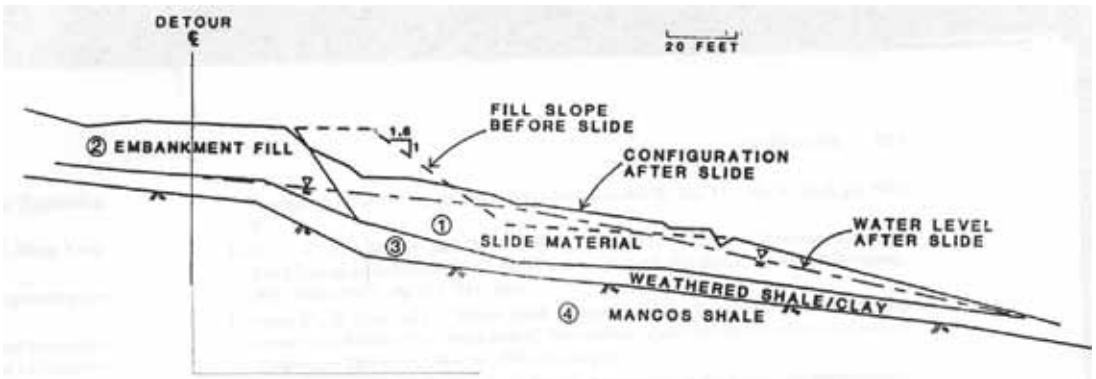
* Analysis based on approximation of remediated slope geometry.

As shown in Table C.2, the proposed optimization procedure indicates that a significantly lower volume of EPS-block geofoam could have been used to reach the required factor of safety of 1.25. The optimized EPS-block geofoam fill is approximately 1,320 yd^3 smaller than the volume of EPS-block geofoam fill actually used on the project. Using the average material cost of \$65/yd^3 (\$85/m^3) given by Jutkofsky (1998), use of the optimization procedure proposed here could have resulted in a significant material cost savings of almost \$86,000.

It should be noted that due to certain project requirements, the EPS-block geofoam fill arrangement developed using the proposed optimization procedure would probably not have been acceptable without some adjustments. The geometry of the EPS fill shown in Figure C.13 is too complex to be replicated in the field without excessive time and effort. Therefore, it would be necessary to adapt the optimized EPS fill as discussed in Step 4 of the design procedure. Additionally, the project required that the failed section of Route 23A remain passable to highway traffic throughout the duration of the project. To accomplish this, a sheet pile wall had to be driven near the head of the slide mass to support the roadway during excavation for the placement of EPS fill. Thus, the volume and location of the fill that was actually constructed was heavily influenced by other factors besides the basic requirements of slope stability. However, by comparing the actual case history with both the Moment Reduction Method and proposed optimization method, it becomes apparent that the proposed optimization method is not only a valid approach to determining location of EPS-block geofoam fill, since it was able to produce an EPS-block geofoam fill that achieved the target factor of safety; it also appears to be a better approximation of the truly optimum volume and location for EPS-block geofoam fill than either the Moment Reduction Method or the trial-and-error approach used on the actual NY Route 23A project.

Colorado Hwy 160 Case History

The initial analysis performed for the Colorado Hwy 160 slope failure indicated that the landslide occurred as a result of a high ground water table in the slope due to an extended period of wet weather in the region. It was determined that the slip surface was non-circular in shape, running primarily along the interface between the roadway fill and a relatively thin layer composed of weathered shale and clay. In addition to the active failure surface, two additional and older slip surfaces were identified underlying the entire area, beneath the active slip surface. A cross-section view of the failed slope, including the soil properties used in the analysis, is provided below in Figure C.14.



SOIL/ROCK PROPERTIES

SOIL/ROCK TYPE	TOTAL UNIT WEIGHT, PCF	MOISTURE %	L.L.	P.I.	C PSF	Ø
①	115	WET	-	-	50	15
②	115	16	35	14	500	20
③	120	14.1	36	15	1000	0
④	115	-	-	-	9000	0

Figure C.14. Cross-section and soil properties of Colorado Hwy. 160 slide remediation (Yeh and Gilmore, 1992).

In order to use this case history as a basis for comparison for the proposed optimization procedure, it was first necessary to replicate results of the stability analysis used on the actual project. The geometry of the slope prior to the failure was determined from information provided by Yeh and Gilmore (1992) and the back-calculated soil properties from Yeh and Gilmore (1992), were assigned to the appropriate materials. The reconstructed slope cross-section was then analyzed using Rocscience SLIDE 5.0 slope stability software. Results are shown in Figure C.15.

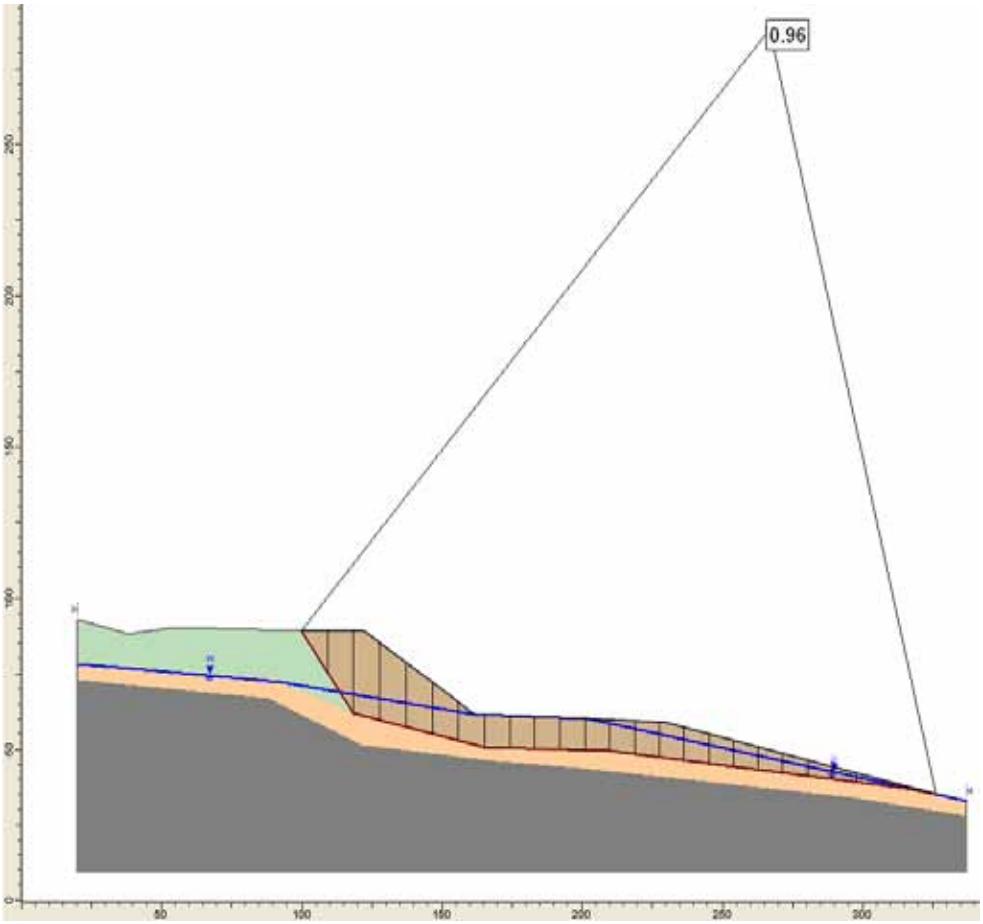


Figure C.15. Results of stability analysis of existing slope using Spencer's Method with back-calculated soil parameters (Yeh and Gilmore, 1992).

Since the soil properties used in the analysis were based on a back-analysis of the failed slope, the factor of safety of the slope should be approximately equal to 1.0. For the purposes of this analysis, it was concluded that the calculated factor of safety of 0.96 was accurate enough to allow the slope cross-section to be used here. Note that this analysis was performed using Spencer's Method, which satisfies full static equilibrium and is applicable to slides having both circular and non-circular slip surfaces. Spencer's Method was selected because it is generally accepted to be one of the most accurate slope stability analysis methods available, and because it was the procedure use by Yeh and Gilmore (1992).

Once the slope model had been validated, the next step in the analysis was to attempt to reconstruct the actual EPS-block geofoam fill that was used as part of the real project to determine the factor of safety actually achieved. Using the best available information from Yeh and Gilmore, (1992), a slope stability analysis of the actual EPS-block geofoam fill was performed. Results of this analysis are shown in Figure C.16.

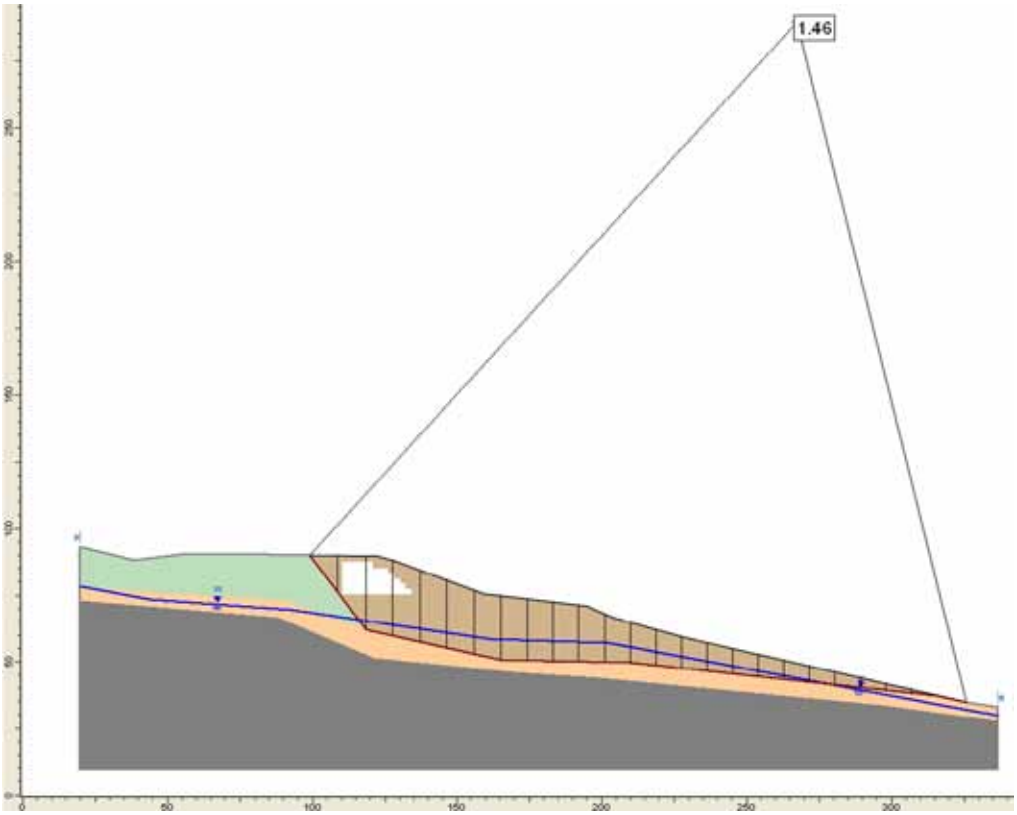


Figure C.16. Results of stability analysis of remediated slope using Spencer's Method with back-calculated soil parameters (Yeh and Gilmore, 1992).

As may be evident from a comparison between Figures C.15 and C.16, the Colorado Hwy 160 remediation project not only specified an EPS-block geofoam fill to help stabilize the slope, but also required lowering the ground water table by approximately 3 ft., and slightly altering the geometry of the slope. Reasons for these additional measures are discussed in greater detail in Yeh and Gilmore (1992). To distinguish between the change in the factor of safety due to EPS fill and change in the factor of safety due to changes in the slope geometry and ground water level, the slope cross-section shown in Figure C.16 was analyzed using Spencer's Method without including the EPS fill. Results of this analysis are shown in Figure C.17.

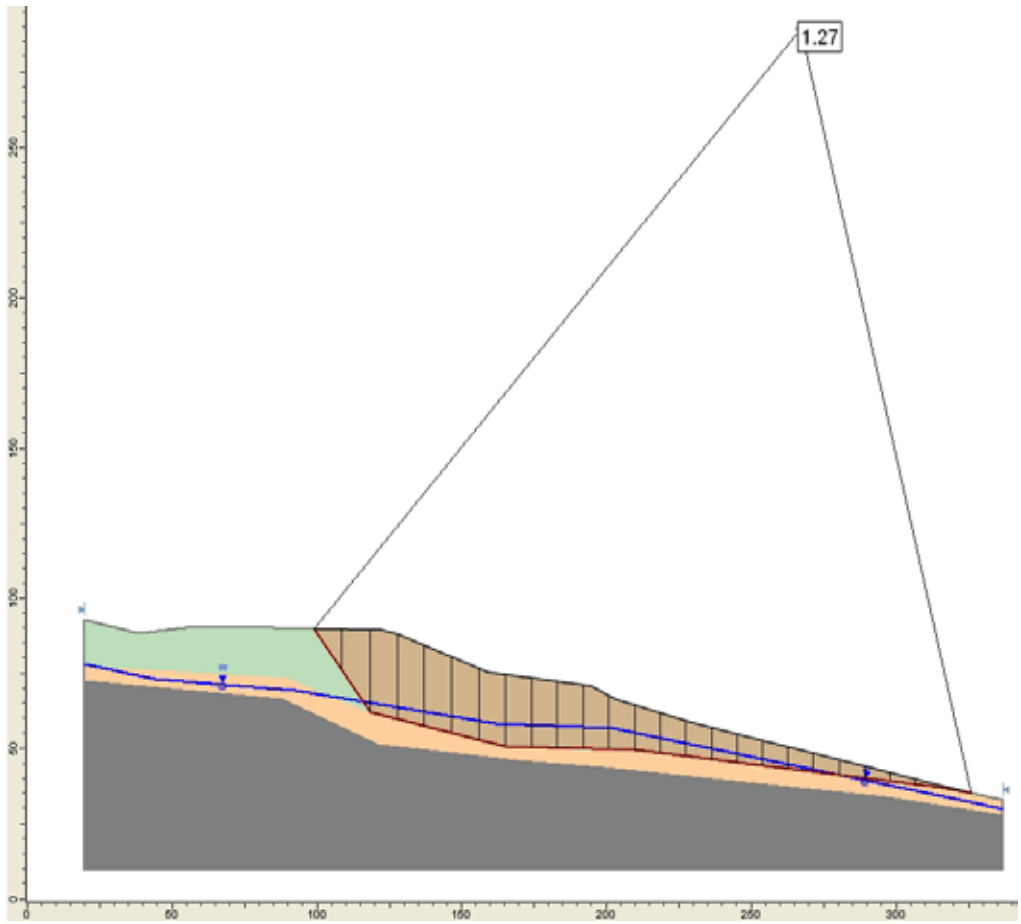


Figure C.17. Results of stability analysis of remediated slope using Spencer's Method without including any EPS fill.

As shown in Figure C.17, changes in slope geometry and ground water level raised the slope's factor of safety from 0.96 to 1.27 without using any EPS-block geofoam fill. According to Yeh and Gilmore (1992), the goal for the remediation was to increase the factor of safety by 20 percent. Thus, assuming the failed slope had a factor of safety of 1.0, the target factor of safety for the remediated slope would have been 1.20. The fact that changes in the slope geometry and ground water level alone produced a factor of safety greater than the target value indicates that EPS-block geofoam fill was probably not the primary mechanism for increasing the slope's factor of safety. Yeh and Gilmore (1992) seem to support this conclusion by indicating that the purpose of EPS fill in this project was to ensure that additional soil fill placed on the slope to obtain the altered geometry did not reactivate the two inactive landslide surfaces underlying the project site.

Optimization Procedure for Non-Circular Slip Surfaces

To evaluate the effectiveness of the proposed optimization procedure, an optimized EPS-block geofoam fill was obtained for the slope shown in Figure C.17. The goal for this analysis was to reproduce the same factor of safety of 1.46 achieved by EPS fill from the actual project, shown in Figure C.16, using a more efficient configuration for EPS-block geofoam. The value of FS_0, as used in Equation C.17 for this analysis was 1.27, since this was the factor of safety of the slope cross-section shown in Figure C.17. The target factor of safety, FS_1, was set at 1.46 to try to reach the same factor of safety that was achieved by the actual EPS fill used on the project.

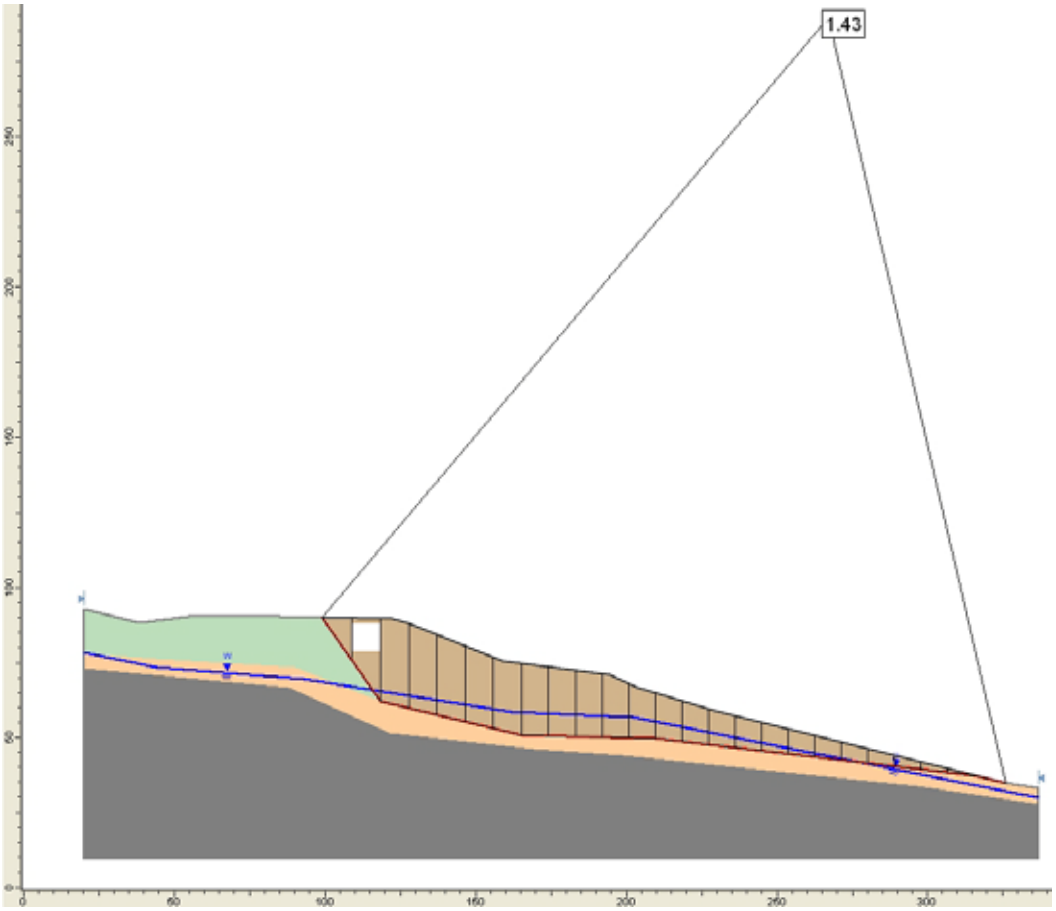


Figure C.18. Results of stability analysis using Spencer's Method on optimized EPS fill.

Comparison of Results for Non-Circular Slip Surface Optimization

As will be evident from a comparison between Figures C.16 and C.18, the proposed optimization procedure results in an EPS-block geofoam fill configuration that achieves the same factor of safety using a significantly smaller volume of EPS. The EPS fill used on the actual project consisted of approximately 648 yd^3 of EPS geofoam (Yeh and Gilmore, 1992) and resulted in a factor of safety of 1.46. Using the proposed optimization procedure, a factor of safety of 1.43 was achieved using only 352 yd^3 of EPS geofoam, as shown in Table C.3. The optimized EPS fill produced approximately the same factor obtained from the actual EPS fill using just over half the volume of EPS. With the material cost for EPS geofoam given by Yeh and Gilmore (1992) ranging from \$30 to \$50 per yd^3 (\$59 to \$65 per m^3), this reduction in volume of EPS fill translates into a material cost savings of between \$9,000 and \$15,000 for a project whose actual cost was estimated at roughly \$160,000. This potential cost savings clearly demonstrates the advantages of using the proposed optimization procedure over the traditional trial-and-error approach when designing an EPS-block geofoam slope system.

Table C.3. Comparison of EPS-block geofoam fill volume from the proposed optimization method against the actual volume used on Colorado Hwy. 160 project.

	Figure No.	Target FS	Achieved FS	Total volume of EPS-block geofoam fill
Actual Colorado Hwy. 160 Project	C.16	*1.20	1.46	648 yd^3
Proposed Optimization Method	C.18	1.46	1.43	352 yd^3

*Achieved primarily by altering slope geometry and lowering ground water level. The additional increase in the factor of safety is due to the fact that EPS fill is a secondary line of defense.

In addition to illustrating the potential cost savings associated with the proposed optimization procedure, this case history comparison also illustrates the fact that use of the optimization procedure is not limited to projects where EPS-block geofoam fill is the primary mechanism for stabilizing the slope. The optimization procedure can easily be used to design an EPS-block geofoam fill that will work in conjunction with other slope stabilization methods to produce the desired results.

CONCLUSIONS

By comparing results of the proposed optimization procedures with actual case histories taken from the field, it has been shown that optimizing the volume and location of the EPS-block geofoam slope fill can result in significantly lower project material costs. This is an advantage because EPS-block geofoam is typically more expensive than conventional fill materials. Thus, it is desirable to design the EPS-block geofoam fill to be as efficient as possible in terms of the volume of EPS that will be used. It should be noted that these cost comparisons are based on material costs only. On some projects, additional material costs associated with EPS blocks may be offset by project cost savings due to accelerated construction times made possible by use of EPS-block geofoam. However, even on these projects, optimizing volume and location of EPS blocks can produce significant additional material cost savings. The proposed optimization procedures for slides involving both circular and non-circular failure surfaces provide useful tools that can be used to identify optimum volume and location for EPS-block geofoam fill, which designers can then adapt to meet specific project requirements.

Although the optimization procedures presented here can be applied to slides involving both circular and non-circular failure surfaces, the optimization constraint equations assume that the failure mass consists of a homogeneous soil mass, with relatively uniform unit weight and shear strength characteristics. The more variation of unit weight and shear strength there is throughout the failure mass, the less accurate results of the optimization procedure will be. This weakness is common to both the optimization procedure presented here, as well as the Moment Reduction Method proposed by Negussey and Srirajan (2001). However, it would be theoretically possible to include multiple soil layers in the derivation of the optimization equations.

Aside from the procedure presented here, the only other alternative to the traditional trial-and-error approach is the Moment Reduction Method presented by Negussey and Srirajan (2001). While this method can be used to approximate the optimum volume and location for EPS fill, it is considerably more time-consuming to use than the proposed optimization methods. Additionally, the Moment Reduction Method is applicable only to slides involving circular failure surfaces, whereas the optimization methods presented here can be applied to both circular and non-circular failure surfaces.

Another advantage of the optimization procedures presented here is that ground water is explicitly taken into account in the derivation of each constraint equation. Thus, the optimization process not only accounts for the change in weight of the slice due to addition of EPS geofoam, but it also accounts for the lowering of the ground water level in that slice that occurs when EPS fill extends below the ground water table. Recall from Chapter 4 that a subsurface drainage system is required for EPS block placed below the ground water level to eliminate buoyancy and hydrostatic forces. Therefore, since this subsurface drainage is required by the design procedure, it was specifically accounted for in the derivation of each of the optimization constraints.

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Appendix D

Centrifugal Force Loads

APPENDIX D

CENTRIFUGAL FORCE LOADS

BACKGROUND

Although loads due to centrifugal forces are not typically considered in the design of earth structures, they may prove significant in design of slopes incorporating EPS-block geofoam. The AASHTO centrifugal load (CF) category is intended to account for the reaction forces exerted by vehicle tires on a curved highway bridge as the vehicle goes around the curve. The vehicle tires exert a force on the roadway to overcome the vehicle inertia and accelerate it around the curve. This, in turn, produces a reaction force acting on the roadway surface which is eventually transmitted to the subgrade. Note that the terminology commonly applied to this topic can be somewhat confusing, because it is heavily dependent on the frame of reference being discussed. The force exerted by vehicle tires that overcomes the vehicle's inertia is technically a centripetal or "center-seeking" force; that is, it is oriented in such a way as to push the vehicle toward the center of the curve, as shown in Figure D.1. The force of interest for the design of EPS-block slopes is the reaction force corresponding to this centripetal force. It is equal in magnitude and opposite in direction, pushing the roadway away from the center of the curve as shown in Figure D.1, hence the term centrifugal, or "center-fleeing." Thus, the AASHTO designation is technically the proper way of describing the loads in question.

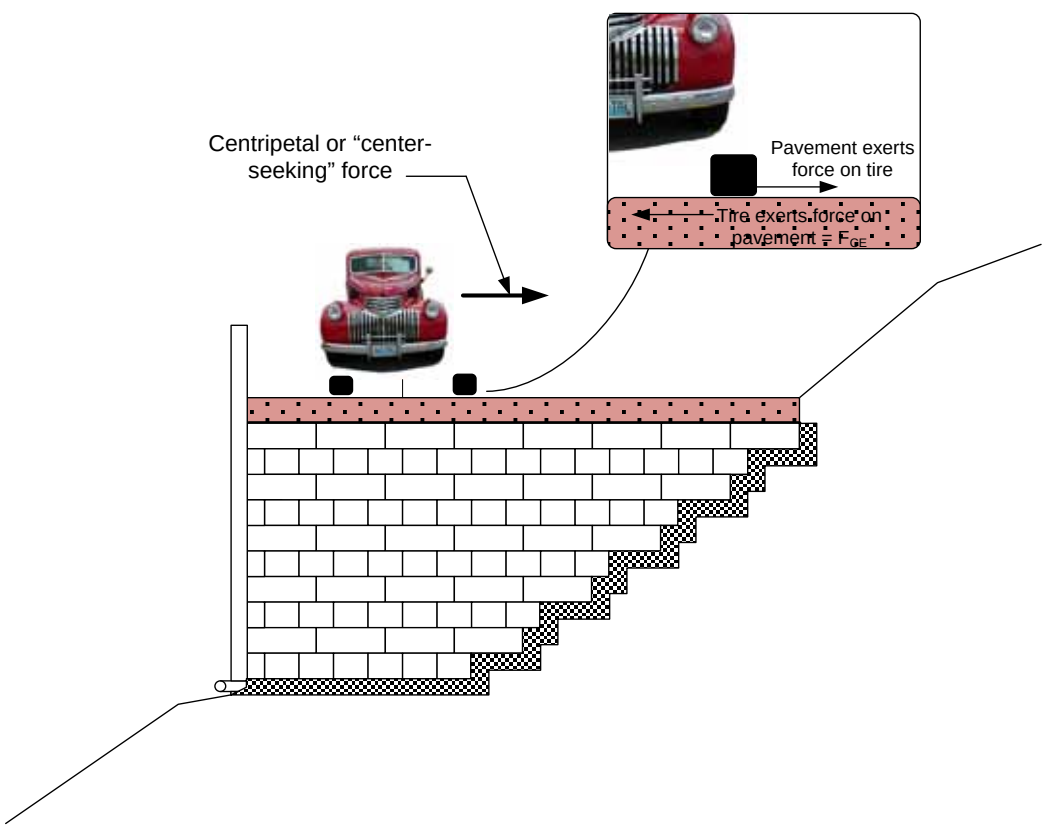

Figure D.1. Centrifugal forces exerted by vehicle on pavement system.

These centrifugal loads are dependent on the volume of traffic the roadway is designed to carry, as well as the design speed of the roadway. An interstate highway designed to carry large volumes of traffic at high design speeds will obviously exert much greater centrifugal loads on its underlying subgrade than a low-traffic rural road. For most earth structures, the sum of the reaction forces developed at the roadway is so small compared to the mass of the underlying subgrade that centrifugal loading can be safely ignored; however, because EPS-block geofoam has an extremely low density, the inertia of the fill mass may not be large enough to justify neglecting centrifugal forces developed at a curved roadway surface. Therefore, a study on the impact of typical centrifugal loads on an EPS-block geofoam fill mass is required. Just as with seismic loading, any lateral loads applied to EPS-block geofoam fill must be given special consideration to prevent shifting and shearing at the interfaces between layers of blocks.

Because centrifugal loads on the roadway are directed away from the center of the curve, the centrifugal forces acting on the roadway may not always tend to act as a destabilizing force. This is a key difference between use of EPS-block geofoam in slopes, as opposed to stand-alone embankments. If the roadway curve is oriented in such a way that the center of the curve lies on the side of the roadway away from the slope, centrifugal forces generated by curving vehicles may actually push EPS fill back into the slope. This is, in essence, a stabilizing force acting on the slope. In view of this, it is recommended that should vehicle centrifugal forces be found to be relevant for EPS-block geofoam slope fills, they should be considered only in the case of a curved roadway for which the center of the curve lies on the side of the roadway toward the slope. For instances where the roadway curve has its center on the side of the road opposite the slope, effects of the centrifugal forces should not be taken into account. This practice will ensure a safe, conservative design for the slope and EPS fill.

According to AASHTO (1996) specifications, the centrifugal force exerted on a roadway by a moving vehicle can be calculated as a percentage of the live load associated with the roadway using Equation D.1 shown below:

$$C = 0.00117 S^2 D = \frac{6.68 S^2}{R} \qquad \textbf{(D.1)}$$

where

C = centrifugal force in percentage of the live load, without impact
S = design speed of roadway in miles per hour
D = degree of curve
R = radius of the curve in feet

Once C has been calculated by Equation D.1, the magnitude of the force may be calculated using Equation D.2 (AASHTO, 1996):

$$F_{CE} = \frac{C}{100} \times LL \qquad \textbf{(D.2)}$$

Note that LL represents the weight of the AASHTO design truck that is most applicable to the roadway under consideration, as specified in Section 3.7 of AASHTO (1996). It is not the same as the live load that is considered elsewhere in the recommended design procedure, which may be approximated by 2 ft. (610mm) of 120 lb./ft^3 (18.9 kN/m^3) fill. This is an important point, because the AASHTO manual (1996) does not emphasize this distinction between the general live load and the live load used to calculate centrifugal forces acting on the roadway, making it easy to confuse the two. If the roadway is superelevated, this superelevation must be accounted for by multiplying F_{CE} times the cosine of the angle of inclination of the roadway surface due to superelevation.

The centrifugal force F_{CE} should be applied at a height of 6 ft. (1.8m) above the surface of the roadway (AASHTO, 1996). Therefore, the magnitude of the overturning moment due to centrifugal forces may be calculated using Equation D.3 shown below:

$$M_O = F_{CE}(H + 6) \qquad \textbf{(D.3)}$$

where

M_O = overturning moment
F_{CE} = centrifugal force acting on roadway
H = height of the EPS-block fill + height of pavement system in feet

This equation is based on an example design calculation that was performed for the Boston CA/T project (Parsons Brinckerhoff, undated). This M_O should be compared to the potential resisting moment due to the weight of the structure to determine the factor of safety against overturning, as shown in Equation D.4 in the next section. Note that Equations D.1 and D.3 are based on Imperial units.

ANALYSIS

Failure Mechanisms

Vehicle centrifugal forces could potentially affect both the internal and external stability of an EPS-block geofoam fill system. The presence of an additional horizontal force could, in theory, have an effect on static slope stability, seismic stability and overturning, and internal seismic stability. In fact, centrifugal force loads are, in many ways, very much akin to seismic forces in that they are both relatively short-duration loads oriented primarily in a horizontal direction. The chief difference between centrifugal forces and seismic forces is related to their magnitudes. Because of this similarity, it was concluded that many of the same failure mechanisms considered as part of the seismic analysis (i.e., Steps 6 and 7 of the updated design procedure shown in Figure 4.25) might be considered as potential failure mechanisms for centrifugal force loads. That is not to say that seismic loads and centrifugal loads should be considered as acting in conjunction with each other. The possibility of such loading actually occurring simultaneously would be extremely remote, as is evidenced by AASHTO recommended load combinations (1996). However, the action of centrifugal forces on the slope system may be similar to that of seismic forces, but considerably lower in magnitude. Therefore, the same failure mechanisms that are considered as part of the seismic analysis are also considered in the evaluation of the significance of centrifugal forces.

The failure mechanisms considered for seismic design of EPS-block geofoam slope fill systems include external seismic stability, external seismic overturning, internal seismic stability (sliding), and internal seismic load-bearing. With the exception of this last mechanism, each of the other three failure mechanisms could theoretically be applicable for centrifugal force loads. Seismic load-bearing is excluded because it is related to the tendency of a lightweight fill to “rock” back and forth under the influence of seismic vibrations, as shown in Figure 4.14. For a given roadway, centrifugal force loads would all be oriented in the same direction, eliminating the mechanism which tends to produce this rocking under earthquake loads.

With regard to the remaining failure mechanisms, it was concluded that centrifugal forces are not likely to have a significant impact on external stability of the slope simply because the magnitude of the forces involved is too small. The centrifugal force, which was given by Equation D.2, is calculated as a percentage of the weight of the AASHTO design truck, which has a maximum weight of 40,000 lbs. (180 kN) (AASHTO, 1996). Thus, the magnitude of the centrifugal force, regardless of roadway geometry, must be less than or equal to this 40,000 lb. (180 kN), which is roughly equivalent to the weight of 12.5 yd^3 ($9.5m^3$) of 120 $lb./ft^3$ (18.9 kN/m^3) soil. In other words, the maximum possible magnitude of a centrifugal force load due to the AASHTO design truck is only a little larger than the magnitude of the

weight of the soil carried by a typical dump truck. Thus, it was concluded that a force of this magnitude would be unlikely to have any significant impact on overall external stability of a slope supporting a roadway. Therefore, having eliminated both overall external stability and internal load-bearing as potential failure mechanisms, the remaining two potential failure mechanisms that must be evaluated to determine whether centrifugal forces should be considered as part of the design process are horizontal sliding and overturning. These two failure mechanisms are also evaluated as part of external and internal seismic stability analysis, Steps 6 and 7 of the design procedure shown in Figure 4.25. Therefore, it may be possible to compare centrifugal forces and seismic forces and perform the evaluation of horizontal sliding and overturning based on the larger of the two forces.

Horizontal Sliding

Horizontal sliding encompasses both sliding of the entire fill system as well as horizontal sliding at any interfaces within the system. For the sake of simplicity, the analysis assumed that, in the case of a sliding failure, sliding would occur first at the interface within the EPS-block geofoam fill system having the lowest interface friction angle. In other words, rather than evaluate the potential for horizontal sliding at each interface throughout the fill system, only the most critical sliding interface was analyzed.

The sliding analysis was set up very much like a typical slope stability analysis in that the factor of safety against sliding, $FS_{Sliding}$, was defined in terms of force equilibrium, as shown below in Equation D.4. F_{CE} was calculated using Equation D.2 shown above, using an approach taken from an example design calculation from the CA/T project (Parsons Brinckerhoff, undated). It may be helpful to refer to Figure D.2 to clarify what each of these variables represents:

$$FS_{Sliding} = \frac{F_R}{F_{CE}} = \frac{W_{Pavement} \tan(\phi'_{crit})}{F_{CE}} \qquad \textbf{(D.4)}$$

where

F_R = force resisting sliding motion

F_{CE} = vehicle centrifugal force acting on fill system, as given by Equation D.2

$W_{Pavement}$ = weight of pavement system

ϕ'_{crit} = friction angle at critical sliding interface

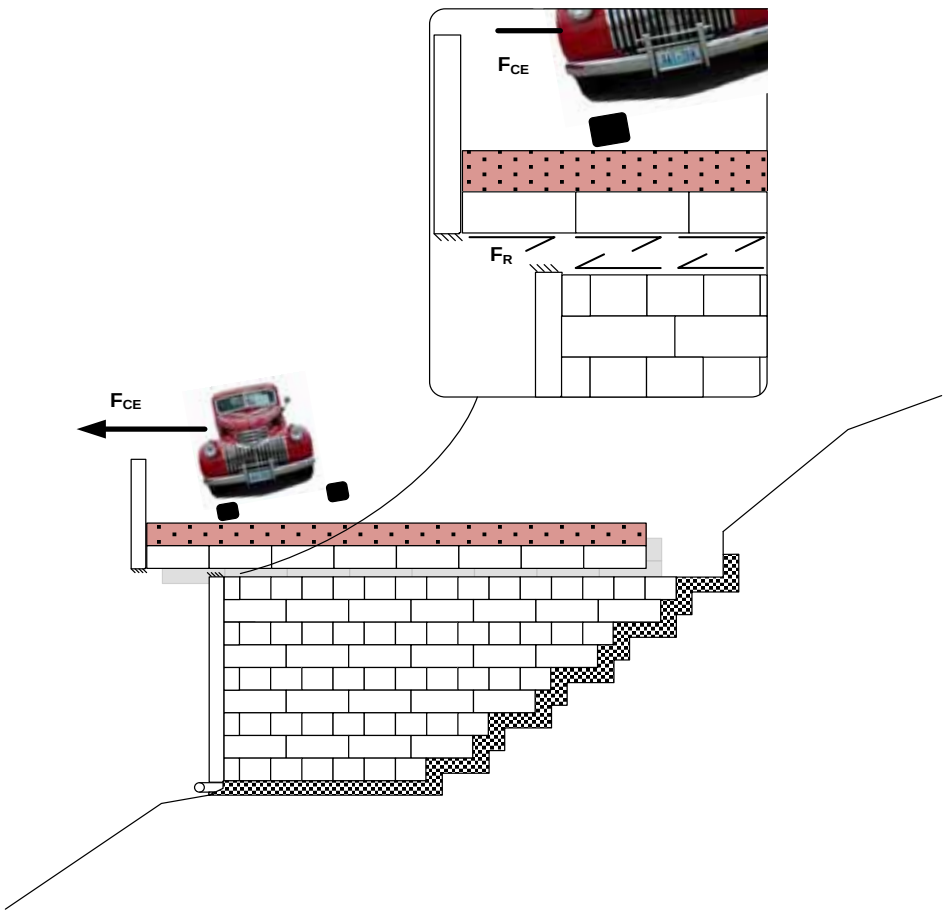

Figure D.2. Cross-section view of model used to evaluate horizontal sliding due to centrifugal forces.

In considering the definition of factor of safety shown in Equation D.4, it should be noted that the resisting force, F_R, is a function only of the critical friction angle, ϕ'_{crit}, and weight of the pavement system, $W_{Pavement}$. This $W_{Pavement}$ term is intended to represent the weight of a strip of pavement system spanning the entire width of the roadway, and having the same length as the AASHTO design truck used to calculate F_{CE}. The AASHTO *H* design truck has a length of 14 ft. (4.3m). The length of *HS* design trucks may vary from 14 to 30 ft. (4.3m to 9.1m) as specified by AASHTO (1996). Thus, to be conservative, the pavement strip considered for this analysis was assumed to be 14 ft. (4.3m) in length, as shown below in Figure D.3. The roadway was assumed to consist of 2 lanes, each having a width of 10 ft. (3m).

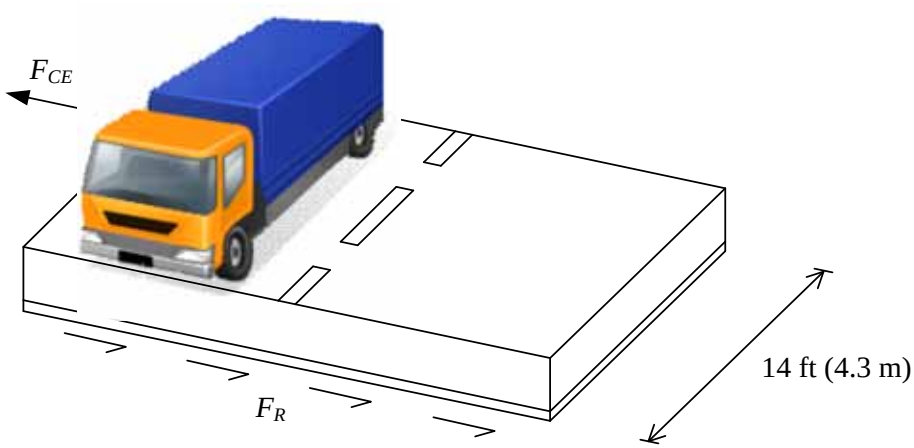

Figure D.3. Pavement block used to determine $W_{Pavement}$ in Eq. D.4.

By assuming that the resisting force, F_R, is solely due to the weight of the pavement block beneath the design truck, the additional friction due to the weight of the truck pressing down on the sliding surface is neglected. This not only avoids complicating the analysis by trying to estimate the vertical stress due to truck tire loads at various depths within the fill; it also ensures that results of the analysis will be conservative.

As noted above, this analysis assumed that sliding would occur at the interface within the EPS-block geofoam fill system having the lowest interface friction angle. Based on information regarding interface shear resistance presented in Tables 3.1 and 3.2, it was decided that a friction angle of 16° would be used for the analysis. This was the value was given by Atmatzidis et al., (2001) as the peak interface friction angle between EPS blocks and a High Density Polyethylene (HDPE) geomembrane, and it represented the lowest peak friction angle for any interface found in the literature search.

Note that this value of 16° was identified as a *peak* interface friction angle, not a *residual* friction angle. This is an important distinction, because residual shear resistance values are typically smaller than peak shear resistance values. Also, use of a residual interface shear resistance value was recommended for seismic sliding analysis in the proposed design procedure. However, for the purposes of this analysis, it was assumed that a peak shear resistance value would be appropriate, because centrifugal loads would be applied for a very short duration as a vehicle traverses the roadway curve. This duration is shorter than even that of a typical seismic load, which could last for several seconds.

Therefore, because the duration of centrifugal loads is so short, even by comparison with seismic forces, it was assumed that a peak interface friction angle would be appropriate, because it is unlikely that such a short-duration load would have time to produce sufficient displacement to fully develop a residual shear resistance. Thus, a sliding analysis based the lowest available peak interface friction value for materials that are commonly used in EPS-block geofoam fill construction was assumed to be a conservative, lower bound estimate of $FS_{Sliding}$ that is likely to occur in a typical EPS-block geofoam slope fill.

Once a suitable model for horizontal sliding failure had been developed, a sensitivity study was performed using spreadsheet software to evaluate $FS_{Sliding}$. Exhibit 3-15 from AASHTO (2004), shown below in Table D.1, was used as the basis for a spreadsheet calculation to evaluate $FS_{Sliding}$ over the full range of allowable AASHTO roadway curvatures and superelevations.

Table D.1. AASHTO recommended minimum curve radii (AASHTO, 2004).

U.S. Customary			Metric		
Design Speed, S (mph)	Superelevation, e (%)	Minimum Curve Radius, R (ft)	Design Speed, S (km/h)	Superelevation, e (%)	Minimum Curve Radius, R (m)
10	4	16	15	4	4
15	4	42	20	4	8
20	4	86	30	4	22
25	4	154	40	4	47
30	4	250	50	4	86
35	4	371	60	4	135
40	4	533	70	4	203
45	4	711	80	4	280
50	4	926	90	4	375
55	4	1190	100	4	492
60	4	1500			
10	6	15	15	6	4
15	6	39	20	6	8
20	6	81	30	6	21
25	6	144	40	6	43
30	6	231	50	6	79
35	6	340	60	6	123
40	6	485	70	6	184
45	6	643	80	6	252
50	6	833	90	6	336
55	6	1060	100	6	437
60	6	1330	110	6	560
65	6	1660	120	6	756
70	6	2040	130	6	951
75	6	2500			
80	6	3050			
10	8	14	15	8	4
15	8	38	20	8	7
20	8	76	30	8	20
25	8	134	40	8	41
30	8	214	50	8	73
35	8	314	60	8	113
40	8	444	70	8	168
45	8	587	80	8	229
50	8	758	90	8	304
55	8	960	100	8	394
60	8	1200	110	8	501
65	8	1480	120	8	667
70	8	1810	130	8	832
75	8	2210			
80	8	2670			
10	10	14	15	10	4
15	10	36	20	10	7
20	10	72	30	10	19
25	10	126	40	10	38
30	10	200	50	10	68
35	10	292	60	10	105
40	10	410	70	10	154
45	10	540	80	10	210
50	10	694	90	10	277
55	10	877	100	10	358
60	10	1090	110	10	454
65	10	1340	120	10	597
70	10	1630	130	10	739
75	10	1970			
80	10	2370			

Table D.1. (continued) AASHTO recommended minimum curve radii (AASHTO, 2004).

U.S. Customary			Metric		
Design Speed, S (mph)	Superelevation, e (%)	Minimum Curve Radius, R (ft)	Design Speed, S (km/h)	Superelevation, e (%)	Minimum Curve Radius, R (m)
10	12	13	30	12	18
15	12	34	40	12	36
20	12	68	50	12	64
25	12	119	60	12	98
30	12	188	70	12	143
35	12	272	80	12	194
40	12	381	90	12	255
45	12	500	100	12	328
50	12	641	110	12	414
55	12	807	120	12	540
60	12	1000	130	12	665
65	12	1220			
70	12	1480			
75	12	1790			
80	12	2130			

Overturning

As previously noted, AASHTO (1996) states that F_{CE} may be assumed to act in a horizontal direction at a location 6 ft. (1.8m) above the roadway. This assumption is the basis for Equation D.5 below, which can be used to calculate the overturning moment due to vehicle centrifugal forces about the bottom of the EPS-block geofoam fill system. This overturning moment will be resisted by the weight of the EPS-block geofoam fill system itself. Therefore, just as the factor of safety against horizontal sliding was defined in Equation D.4 as the total resisting force divided by the driving force due to centrifugal loads, the factor of safety against overturning can be similarly defined as shown in Equation D.5. It may be helpful to refer to Figure D.4 to clarify what each of these variables represents:

$$FS_{OT} = \frac{\sum M_R}{M_{OT}} = \frac{W_{EPS}\,\overline{X}_{EPS} + W_{Pavement}\,\overline{X}_{Pavement} + M_{add}}{M_{OT}} \qquad \textbf{(D.5)}$$

where

W_{EPS} = weight of EPS-block geofoam fill

$\overline{X}_{EPS}$ = horizontal distance from centroid of EPS fill to toe of the EPS fill

$W_{Pavement}$ = weight of pavement system

$\overline{X}_{Pavement}$ = horizontal distance from centroid of pavement system to toe of the EPS fill

M_{add} = any additional resisting moment that may be present in system, such as any moment due to anchors supporting facing system, etc

M_{OT} = overturning moment due to centrifugal force loads, as given by Equation D.3

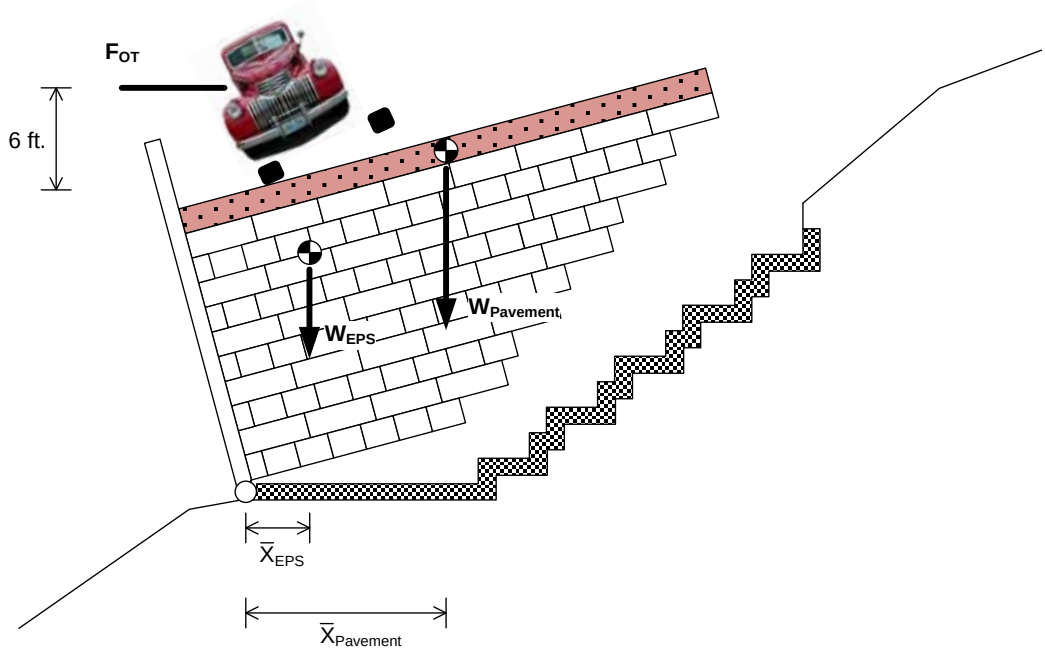

Figure D.4. Cross-section view of model used to evaluate centrifugal force overturning.

Because the magnitude of each of the moments used in Equation D.5 is so dependent on the specific geometry of the EPS-block geofoam slope fill system, it would not be worthwhile to try to make a generalized lower-bound estimate of FS_{OT} for different configurations of EPS-block geofoam fill. Each individual project will have different geometry and values for M_{add}. Therefore, it is recommended that the potential for overturning due to vehicle centrifugal forces be evaluated on a case-by-case basis using the model described above.

Results of Analysis

From the spreadsheet analysis described above, a series of plots was developed showing the variation of $FS_{Sliding}$ with the design speed of the roadway for each of the four AASHTO design truck loads (AASHTO, 1996). For each design speed, the value of F_{CE} was calculated based on the minimum allowable curve radius for various levels of superelevation (*e*) as given in Exhibit 3-15 of AASHTO (AASHTO, 2004), shown in Table D.1 earlier in this chapter. These plots are shown in Figure D.5.

In examining the plots shown in Figure D.5, the reader may notice these plots show a trend that is somewhat counterintuitive. Each of the four plots shows the factor of safety increasing with the design speed. Initially, this trend may appear to conflict with Equation D.4, which indicates that the factor of safety should be inversely proportional to design speed. However, recall that this analysis was based on AASHTO's recommended minimum curve radius for each value of design speed and superelevation. This trend of increasing factor of safety with increasing design speed is simply a reflection of the recommended minimum curve radius specified by AASHTO (2004). The same can be said about the trend of decreasing factor of safety with increasing superelevation. Both are somewhat counterintuitive, but simply reflect the AASHTO geometric requirements upon which the analysis was based.

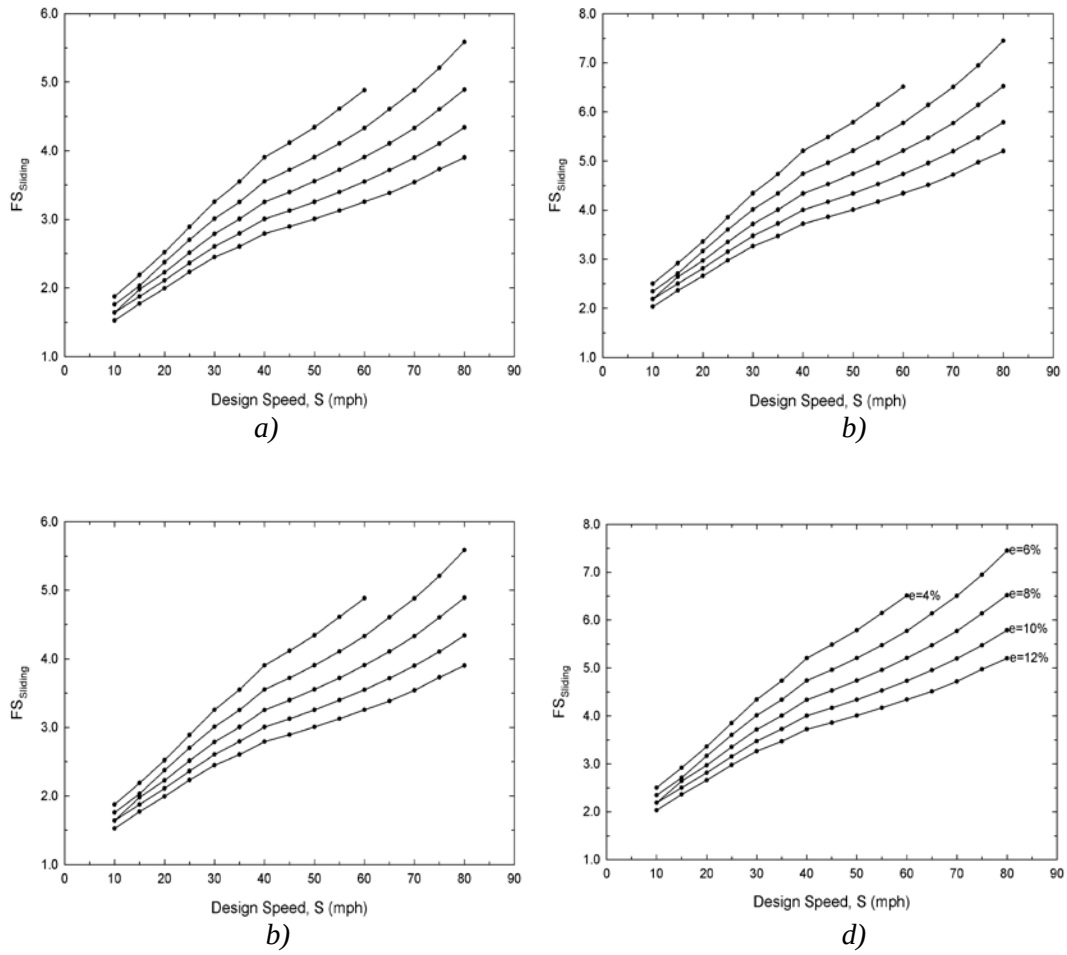

Figure D.5. Results of centrifugal force sliding analysis for a) AASHTO H-20 design truck, b) AASHTO H-15 design truck, c) AASHTO HS-20 design truck, d) AASHTO HS-15 design truck.

CONCLUSIONS

As shown in Figure D.5, the lowest value calculated for $FS_{Sliding}$ corresponded to a 10 mph (16 km/hr.) design speed and a superelevation of 12 percent for AASHTO H-20 and HS-20 design trucks. The calculated value of $FS_{Sliding}$ was approximately 1.52. For the sake of comparison, it should be noted that the typical recommended minimum *FS* value for internal sliding due to seismic loads given in the proposed design procedure was 1.2. Thus, even the lowest calculated value of $FS_{Sliding}$ was significantly higher than the required FS for internal seismic stability.

Therefore, based on results of the analysis described above, it was concluded that for most EPS-block geofoam slope projects, it is very unlikely that sliding due to centrifugal force loads would occur. However, it is recommended that any projects involving a geosynthetic layer in the upper portion of the fill system, such as a geomembrane separation layer between the EPS-block geofoam fill and pavement system, should implement a testing program to determine the interface friction angle between EPS-block geofoam and the particular geosynthetic that is to be used. If this interface friction angle turns out to be less that 16°, it may be advisable to analyze the potential for sliding due to centrifugal forces for the project before proceeding with construction. It may also be advisable to perform a centrifugal force sliding analysis for projects involving very narrow roadways with a short radius curve. The analysis above assumed the shear force that resisted sliding resulted from the weight of a pavement block roughly

20 ft. (6m) wide. Thus, any roadway narrower than 20 ft. (6m) should be checked to ensure that the weight of the pavement block beneath the applicable AASHTO design truck has sufficient weight to develop the necessary shear resistance at the critical interface.

For the failure mechanism of overturning due to vehicle centrifugal forces, it was not practical to try to identify a generalized worst-case-scenario for analysis, as was done for horizontal sliding, because the magnitude of each of the moments involved is so heavily dependent on the geometry of the EPS-block geofoam fill system. Obviously, vertical-sided EPS fills will be more likely to overturn than sloped-sided fills. On the other hand, projects that utilize a thicker pavement system will have a much higher resisting moment, making them significantly more stable. Thus, it is recommended that projects for which the EPS-block geofoam fill is not confined on the down-slope side by a conventional fill material (i.e. slope-sided fills) should be checked to ensure that FS_{OT} is greater than or equal to 1.2. This calculation is relatively simple and will ensure that the design has an adequate factor of safety against overturning. However, for cases where seismic design is performed, and specifically, where the failure mechanism of seismic overturning is evaluated, it may not be necessary to calculate the FS_{OT} for vehicle centrifugal forces, since the seismic loading condition will generally be more critical than the centrifugal force loading condition.

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Appendix E

Design Example

INTRODUCTION

This appendix will present a comprehensive design example illustrating the use of the design procedure included in the recommended design guideline included in Appendix B and described in detail in Chapter 4. This example will also demonstrate how the optimization procedure described in Appendix C can be incorporated into the design procedure as a whole. The majority of the background material used in this design example was developed specifically for use in this example. However, some of the information not relating directly to the design of the EPS-block geofoam slope fill (e.g., pavement system design information) was based on the design example provided in the NCHRP Project 24-11(01) report for stand-alone embankments over soft ground (Stark et al. 2004).

This example will demonstrate how to use the design procedure shown in Figure 4.25 to design an EPS-block geofoam fill that will help to stabilize a failed slope that supports a major highway. The slope failure has rendered approximately 100 ft. of the 2 lane highway impassable to traffic. EPS-block geofoam will be used to repair the slide and get the roadway operational as soon as possible. Drained-strength conditions are assumed in the slope stability analysis for this example. It should be noted that in certain projects, such as a project that will require the new road grade to be raised, slope stability analyses based on undrained strengths will also be required.

STEP 1: BACKGROUND INVESTIGATION

- Site Evaluation
  - Major highway located near the head of slide mass.
  - The failed slope consists of fairly uniform brown silty clay.
  - The back-calculated soil parameters for the failed slope are:
    - Brown silty clay
    - γ_{soil} = 110 lb/ft^3
    - c' = 0 lb/ft^2
    - φ' = 25 degrees.
  - The slip surface along which the slope failed is circular in shape.
  - The width of the highway is 24 ft (2 lanes at 12 ft. per lane).
  - Design life for the project has been set at 75 years.

- Performance Criteria
  - Supervising agency has a design policy that requires the following factors of safety:
    - Static slope stability for remediated slope: FS ≥ 1.25.
    - Seismic slope stability for remediated slope: FS ≥ 1.05. Site Class B. For external seismic stability failure mechanisms of horizontal sliding, overturning and the internal seismic stability failure mechanisms of horizontal sliding use FS ≥ 1.2.
    - Max allowable settlement for roadway: 0.5 ft.
    - Bearing capacity: FS ≥ 3.
  - Supervising agency does not have any policies or design codes relating specifically to EPS-block geofoam. Therefore, use the following recommended factors of safety.
    - Internal load bearing of EPS blocks: FS ≥ 1.2.

- Initial Slope Stability Analysis
  - Stability analysis (Spencer's Method) of the active slip surface using back-calculated soil properties, original groundwater level, and a roadway surcharge of 390 lb/ft^2, which corresponds to a 3 ft. pavement layer of 130 lb/ft^3 material, indicates a factor of safety of approximately 1 for the slope just prior to failure, as shown in Figure E.1.

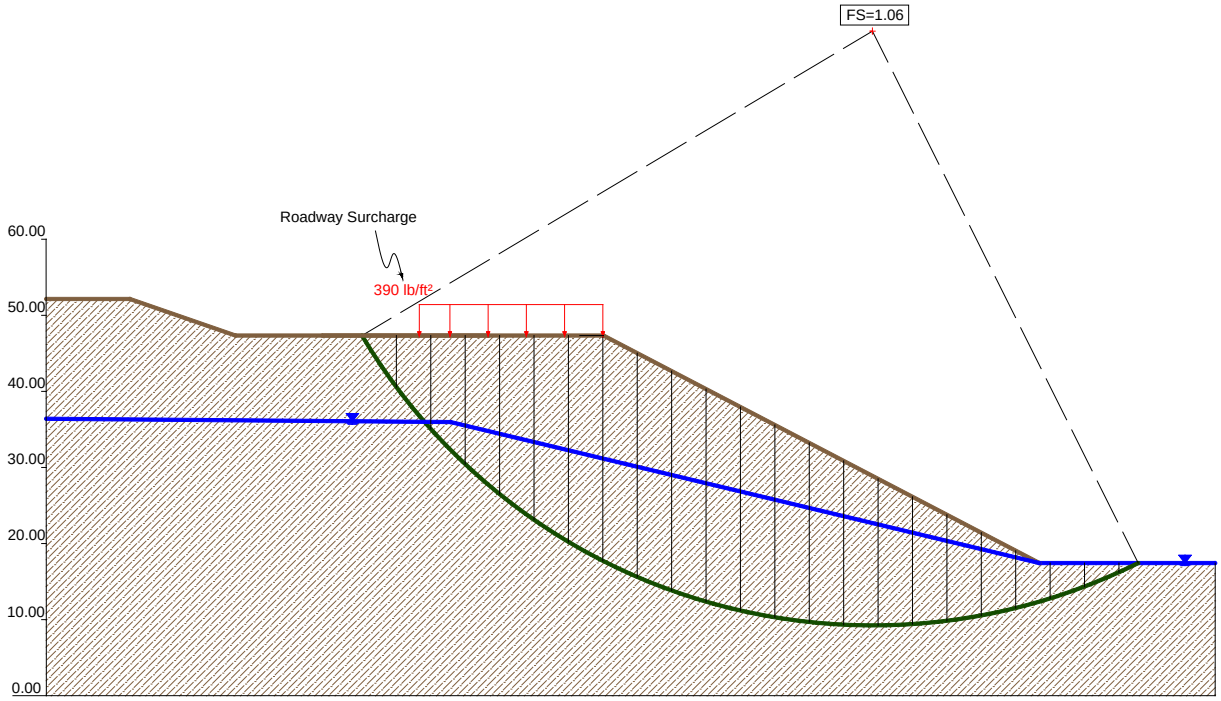


Figure E.1. Results of stability analysis of existing slope using Spencer's Method with back-calculated soil parameters.

STEP 2: SELECT PRELIMINARY TYPE OF EPS AND ASSUME PRELIMINARY PAVEMENT SYSTEM

- Assume EPS blocks with a dry unit weight of 1.25 lb/ft^3 (i.e., EPS50) and a corresponding long-term unit weight of 1.9 lb/ft^3 to account for potential long-term water absorption as indicated in Chapter 3. A moist unit weight of 1.9 lb/ft^3 will be used for some failure mechanisms based on the recommendation included in Chapters 3 and 4 that each failure mechanism be analyzed using the unit weight corresponding to the "worst case scenario."

- Assume pavement system will be equivalent to a 3 ft. layer of 130 lb/ft^3 material. This will result in an equivalent surcharge of 390 lb/ft^2

STEP 3: OPTIMIZE VOLUME AND LOCATION OF EPS FILL

The slope needs to be reconstructed to match its original geometry. To accomplish this, some additional fill material will have to be placed near the head of the slide to provide support for the roadway. This fill material will most likely be taken from a borrow pit located near the project site, and will consist of the same brown silty clay present at the site. Since the slope needs to be reconstructed to match the original geometry, the slope cross-section shown in Figure E.1 will be used in the optimization procedure for circular slip surfaces to identify the optimum volume and location for the EPS fill.

The Solver inputs used to perform the optimization are shown below in Figure E.2. The spreadsheet to which these Solver parameters refer is shown in Figure E.3. Note that the initial factor of safety, FS_0, used in the optimization procedure is the factor of safety obtained based on the Ordinary

Method of Slices, which is calculated by the optimization procedure spreadsheet and is 1.01 and not the factor of safety of the existing slope shown in Figure E.1, which is based on Spencer's Method. The target factor of safety, FS_1, will be set at 1.25 to match the supervising agency's static slope stability requirements. The moist unit weight of 1.9 lb/ft^3 will be used for the EPS.

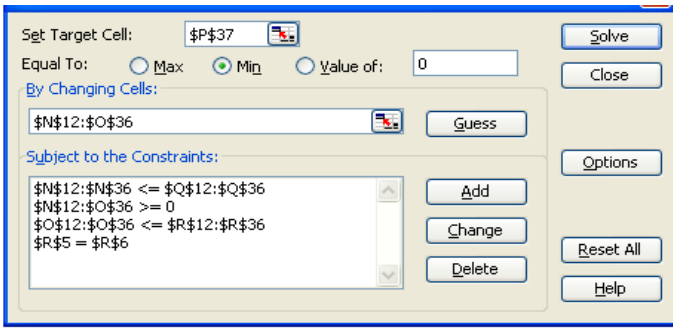


Figure E.2. Solver inputs to optimize EPS-block geofoam fill for example slope based on spreadsheet shown in Figure B.3.

1																				
2	γ_{soil} =	110	lb/ft^3			d_{top} =	0.5	ft									Ordinary Method of Slices Optimization			
3	c' =	0	lb/ft^2			d_{bottom} =	0.5	ft									FS_0 =	1.01		
4	φ' =	25	deg														FS_1 =	1.25		
5	γ_w =	62.4	lb/ft^3														M_{red} =	10927.2		
6	γ_{EPS} =	1.9	lb/ft^3														$\Delta M_D - \Delta M_R$ =	10927.2		
7												Ordinary Method of					Difference =	0.0		
8												FS =	1.01							
9																	FS_{CHECK} =	1.25		
10	Slice	b	L	α	W	u	Z_{bottom}	Z_{top}	Z_w	H	H'	M_D	M_R	h	h'	h+h'	h_{MAX}	h'_{MAX}	ΔM_D	ΔM_R
11	No.	(ft)	(ft)	(deg)	(lbs)	(lb/ft^2)	(ft)	(ft)	(ft)	(ft)	(ft)	(lbs)	(lbs)	(ft)	(ft)	(ft)	(ft)	(ft)	(lbs)	(lbs)
12	1	4.50	8.13	56.39	1678.05	0.00	43.95	47.34	43.95	3.39	0.00	1397.5	433.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
13	2	4.50	7.11	50.76	4717.35	0.00	37.81	47.34	37.81	9.53	0.00	3653.6	1391.5	9.03	0.00	9.03	9.03	0.00	3402.12	1295.71
14	3	4.50	6.45	45.75	7222.05	207.79	32.75	47.34	36.08	11.26	3.33	5173.2	2045.6	10.76	0.00	10.76	10.76	0.00	3749.27	1703.13
15	4	4.50	5.98	41.17	9340.65	407.16	28.47	47.34	35.00	12.35	6.53	6148.9	2635.4	0.00	6.03	6.03	11.85	6.03	1929.37	434.73
16	5	4.50	5.63	36.88	11152.35	567.84	24.81	47.34	33.91	13.43	9.10	6693.0	3206.0	0.00	8.60	8.60	12.93	8.60	2510.67	659.00
17	6	4.50	5.35	32.83	12706.65	696.07	21.67	47.34	32.83	14.52	11.16	6888.9	3752.7	0.00	10.66	10.66	14.02	10.66	2810.02	859.75
18	7	4.50	5.14	28.95	14038.20	796.22	18.98	47.34	31.74	15.60	12.76	6795.1	4266.9	0.00	9.76	9.76	15.10	12.26	2297.39	819.30
19	8	4.50	5.97	25.22	14607.45	872.66	16.67	46.18	30.66	15.53	13.99	6224.2	4174.0	0.00	0.00	0	15.03	13.49	0.00	0.00
20	9	4.50	4.84	21.60	14414.40	926.64	14.72	43.84	29.57	14.27	14.85	5306.3	4441.6	0.00	0.00	0	13.77	14.35	0.00	0.00
21	10	4.50	4.73	18.06	14048.10	960.02	13.10	41.48	28.49	13.00	15.39	4355.1	4314.0	0.00	0.00	0	12.50	14.89	0.00	0.00
22	11	4.50	4.65	14.60	13533.30	974.69	11.78	39.12	27.40	11.72	15.62	3411.3	4127.7	0.00	0.00	0	11.22	15.12	0.00	0.00
23	12	4.50	4.59	12.88	12909.60	975.62	10.68	36.76	26.32	10.45	15.64	2877.7	3884.0	0.00	0.00	0	9.94	15.14	0.00	0.00
24	13	4.50	4.54	6.08	12117.60	955.34	9.92	34.40	25.23	9.17	15.31	1283.5	3618.9	0.00	0.00	0	8.67	14.81	0.00	0.00
25	14	4.50	4.51	4.48	11152.35	913.22	9.51	32.04	24.15	7.90	14.64	871.1	3275.7	0.00	0.00	0	7.40	14.14	0.00	0.00
26	15	4.50	4.50	1.15	10093.05	859.25	9.29	29.68	23.06	6.62	13.77	202.6	2903.2	0.00	0.00	0	6.12	13.27	0.00	0.00
27	16	4.50	4.50	-2.17	8905.05	789.05	9.33	27.32	21.98	5.35	12.65	-337.2	2496.2	0.00	0.00	0	4.85	12.15	0.00	0.00
28	17	4.50	4.52	-5.50	7588.35	702.62	9.63	24.96	20.89	4.07	11.26	-727.3	2054.9	0.00	0.00	0	3.57	10.76	0.00	0.00
29	18	4.50	4.55	-8.85	6138.00	599.66	10.20	22.60	19.81	2.79	9.61	-944.3	1585.9	0.00	0.00	0	0.00	9.11	0.00	0.00
30	19	4.50	4.60	-12.24	4554.00	479.23	11.04	20.24	18.72	1.52	7.68	-965.5	1093.5	0.00	0.00	0	0.00	7.18	0.00	0.00
31	20	4.50	4.67	-15.66	2836.35	341.95	12.15	17.88	17.63	0.25	5.48	-765.6	583.1	0.00	0.00	0	0.00	4.98	0.00	0.00
32	21	4.50	4.76	-19.14	1915.65	241.49	13.57	17.44	17.44	0.00	3.87	-628.1	365.5	0.00	0.00	0	0.00	3.37	0.00	0.00
33	22	4.50	4.88	-22.70	1064.25	134.16	15.29	17.44	17.44	0.00	2.15	-410.7	198.0	0.00	0.00	0	0.00	1.65	0.00	0.00
34	23	2.53	2.80	-25.54	169.76	38.06	16.83	17.44	17.44	0.00	0.61	-73.2	31.0	0.00	0.00	0	0.00	0.11	0.00	0.00
35									*SUMMATIONS:*			56430.0	56878.5			54.8			16698.84	5771.61
36																				

Figure E.3. Spreadsheet used in the optimization procedure.

The optimization procedure depicted in Figures E.2 and E.3 results in the EPS-block geofoam fill configuration shown below in Figure E.4.

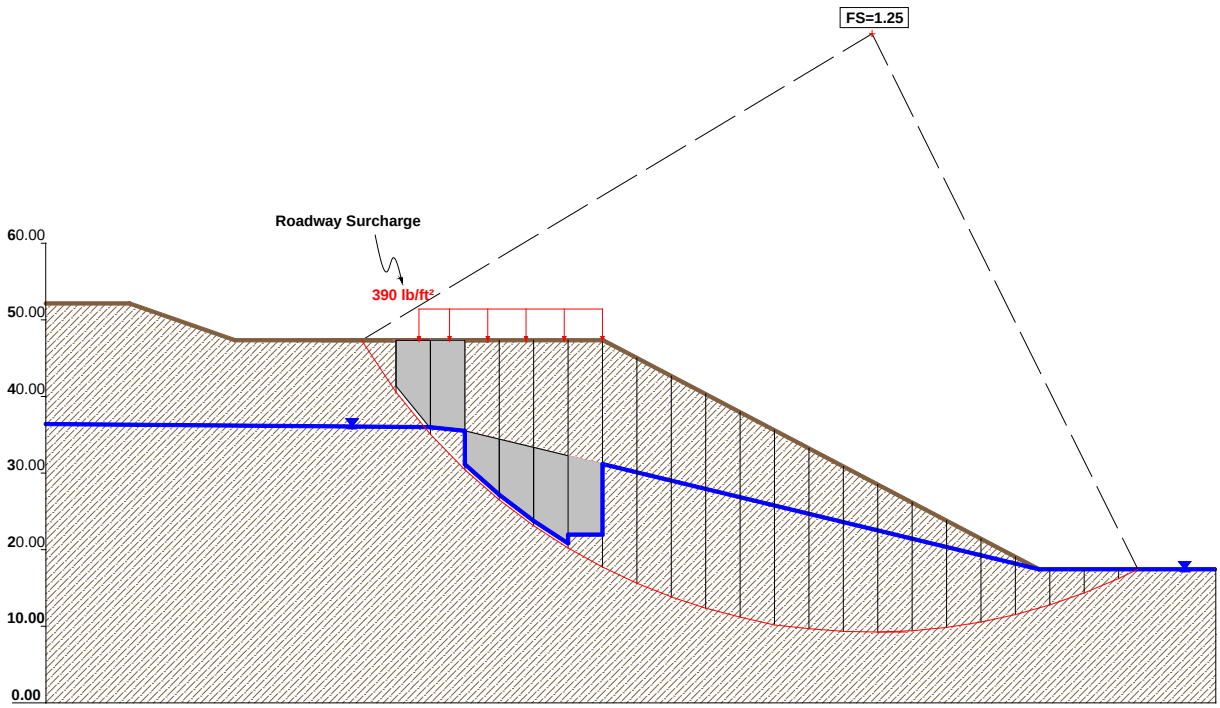


Figure E.4. Stability analysis using Spencer's Method of the optimized EPS-block geofoam fill for design example.

STEP 4: MODIFY OPTIMIZED EPS FILL AS NEEDED FOR CONSTRUCTABILITY

Because the optimized EPS-block geofoam fill from Step 3 has such a complex geometry, it will be advantageous to simplify the configuration to make it easier to construct and to minimize the extent of the required subsurface drainage system. Recall that the recommended design procedure summarized in Chapter 4 is based on including a drainage system to prevent water from accumulating above the bottom of the EPS blocks and to alleviate seepage pressures between the adjacent upper slope material and the EPS blocks. Additionally, it may be helpful to account for the fact that EPS blocks are typically manufactured to certain standard dimensions (e.g., 2 ft x 4 ft x 8 ft). Modifying the EPS fill with these dimensions in mind can help reduce the amount of cutting and trimming required for construction. Note that the actual dimensions of the EPS blocks used in the field will probably be dictated by the molder and specified in the EPS block shop drawings. However, since it is helpful to have an idea of the block dimensions during the initial design process, the preliminary block dimensions previously mentioned may be used.

To modify the EPS fill shown in Figure E.4, the lower portion of the EPS, which is located below the groundwater table, may simply be raised to a higher elevation. This not only reduces the extent of the required subsurface drainage system, but it also results in a much simpler geometry for the EPS fill. However, by lifting this lower portion of the EPS fill up above the groundwater level, the increase in factor of safety due to the lowering of the groundwater table will be negated. Therefore, it will be necessary to increase the volume of the EPS fill slightly to ensure that the factor of safety is still adequate.

Since it is desirable to design the EPS fill taking into account the preliminary block dimensions, the dimensions of the EPS fill can simply be "rounded up" to the nearest whole block dimension. Taking this approach to the EPS-block geofoam fill shown in Figure E.4 results in the modified EPS fill configuration shown below in Figure E.5. Once these modifications have been made, the modified EPS fill can be analyzed for static slope stability in Step 5.

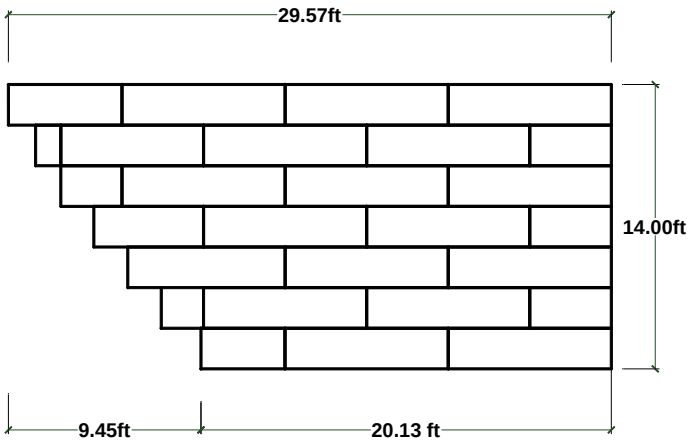


Figure E.5. Modified EPS-block geofoam fill for design example.

STEP 5: STATIC SLOPE STABILITY (EXTERNAL)

Overall Static Slope Stability

The next step in the design is to re-analyze the EPS fill developed in Step 4 to ensure that the required factor of safety of 1.25 can still be achieved. The results of this analysis, which was performed using Spencer's Method, are shown below in Figure E.6.

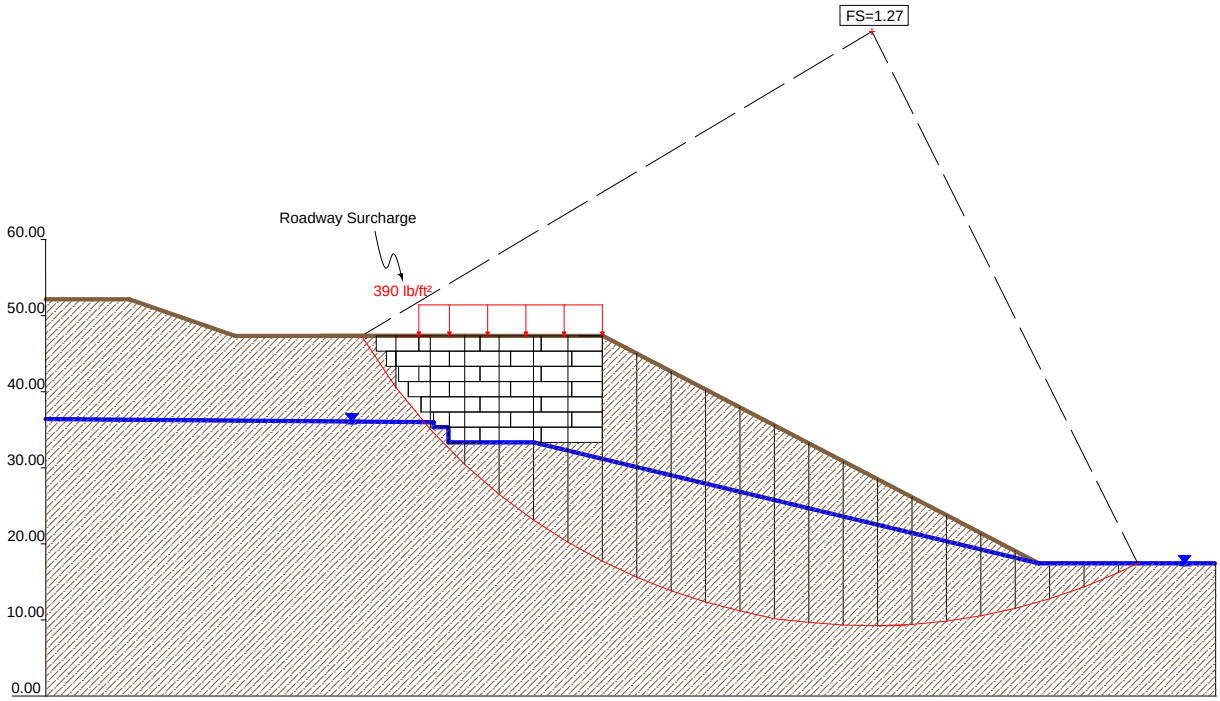


Figure E.6. Results of static slope stability analysis using Spencer's Method for EPS fill developed in Step 4 of the design procedure.

The backfill material that is typically required between the EPS blocks and the adjacent slope material is analyzed to ensure that the factor of safety will be adequate. For this example, the backfill material was assumed to be crushed stone with a unit weight of 120 lb/ft^3, an effective cohesion of 0, and an effective friction angle of 40 degrees. The results of this analysis are depicted below in Figure E.7. Note that the purpose of this analysis is to ensure that the preliminary backfill configuration will have an adequate factor of safety. Once the preliminary backfill configuration has been established and checked, the designer will be able to determine the extent of the excavation that will be necessary to allow this backfill configuration to be constructed

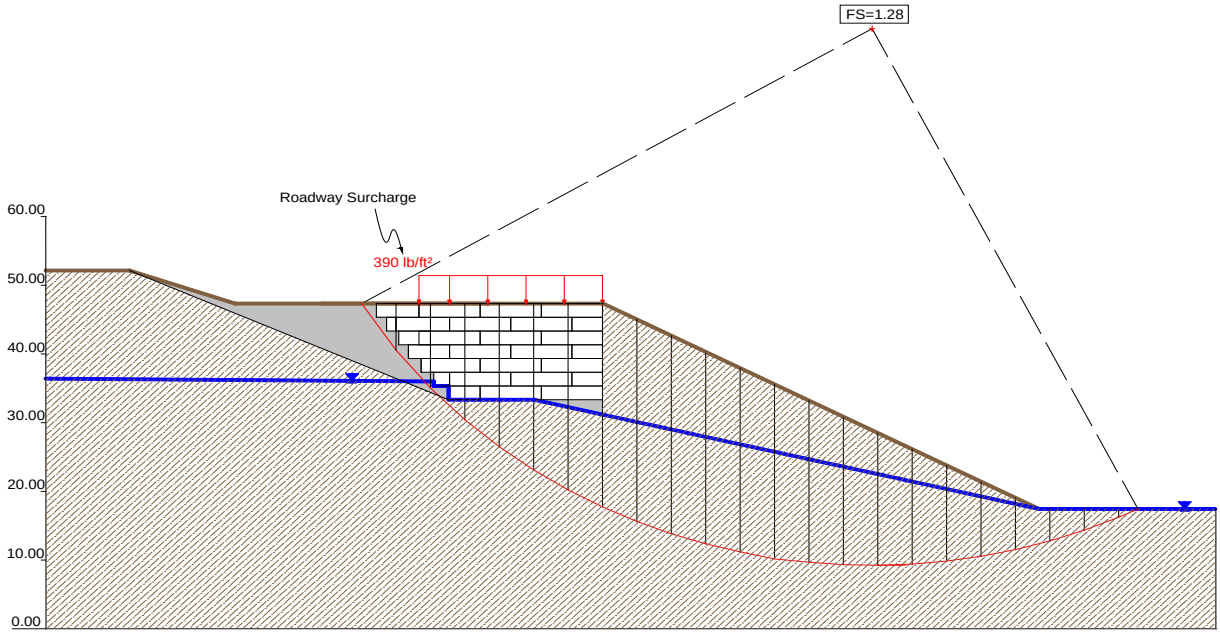


Figure E.7. Results of static slope stability analysis using Spencer's Method for modified EPS fill including assumed backfill.

Once the preliminary backfill configuration has been checked and the limits of the excavation have been determined, the excavated slope (prior to the construction of the EPS fill or the placement of the backfill) must be analyzed to ensure that it is self-stable as shown in Figure E.8. It is also necessary to perform a slope stability analysis to ensure that the existing slope can be cut into a self-stable configuration prior to the construction of the EPS fill. This analysis serves both to ensure that the excavation will remain stable until the EPS-block geofoam fill can be constructed and to ensure that the adjacent slope will not exert excessive lateral pressures on the EPS fill. Recall from the discussion of design loads in Chapter 4 that the design procedure is based on the assumption that the EPS-block geofoam fill will not be subjected to lateral earth pressures from the adjacent slope material.

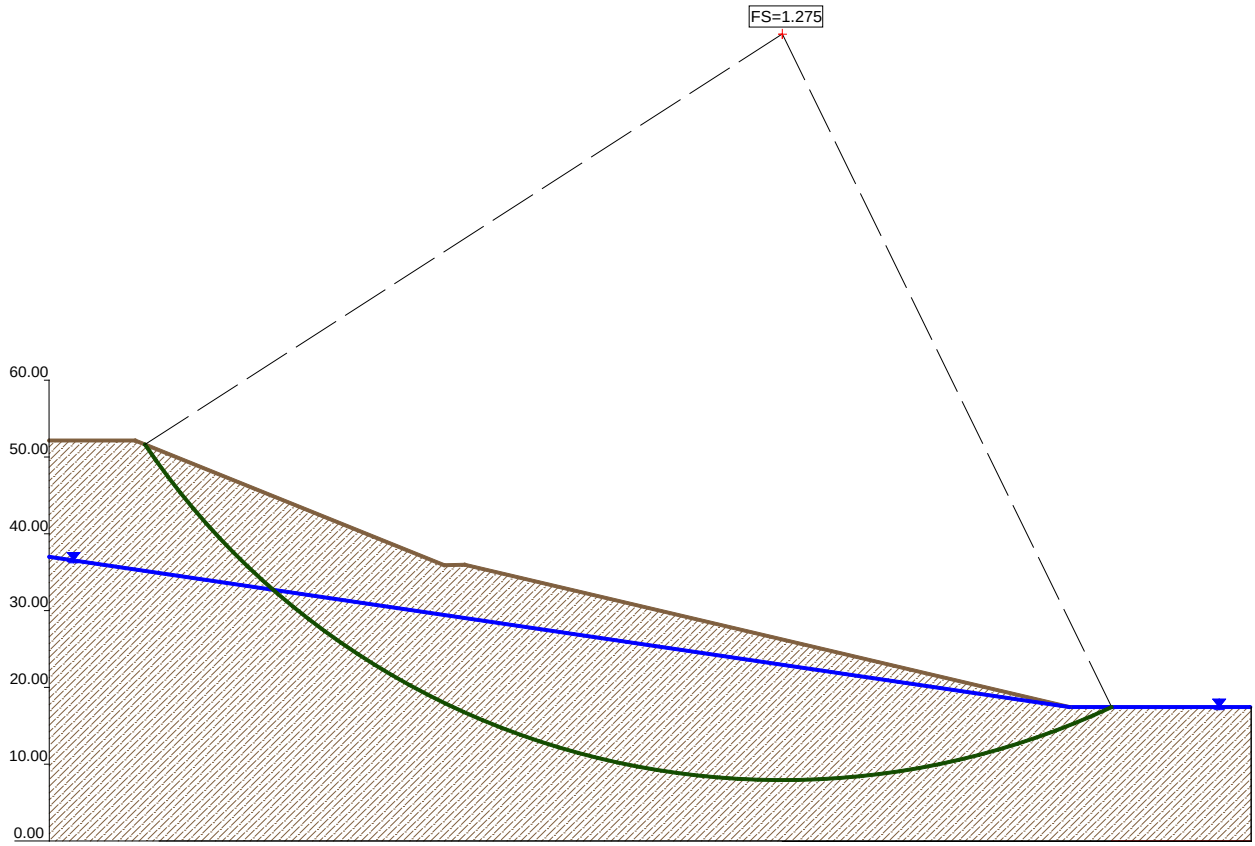


Figure E.8. Results of static slope stability analysis using Spencer's Method for excavated slope prior to construction of EPS-block geofoam fill.

The analysis depicted in Figure E.8 shows that the backfill configuration in Figure E.7 and the cut slope configuration in Figure E.8 is acceptable because the factor of safety is greater than 1.25. As shown in Figure E.8, groundwater may be encountered during the excavation procedure. Therefore, dewatering may be required during construction. Therefore, in addition to the permanent drainage system, a temporary dewatering system will be needed.

As noted in Chapter 4, the recommended design procedure for geofoam slopes is based on a self-stable adjacent upper slope material under static conditions to prevent earth pressures on the EPS fill mass that can result in horizontal sliding between blocks. However, backfill material, which typically consists of cohesionless material, is typically required between the EPS fill mass and the adjacent natural slope material. Therefore, under static conditions, the EPS fill mass will be subjected to lateral earth pressures from the backfill material.

Evaluate Static Horizontal Sliding Due to the Backfill Material

Use Figure 4.30 to evaluate the factor of safety against horizontal sliding but without the seismic coefficients. For the static condition, the equation shown in Figure 4.30 becomes:

$$FS = \frac{\mu(W_{EPS} + W_{Pavement})}{\Sigma P_i} \qquad \textbf{(E.1)}$$

where

μ = coefficient of friction between EPS blocks and foundation soil = $\tan\delta$

Referring to Table 3.2, a δ value of 22° appears to be an appropriate preliminary value for an EPS-silty clay soil interface. Note that this value is adequate for preliminary design, but it would be advisable to perform interface shear tests using actual soil samples from the project site to determine a more accurate value of δ for the final design.

Therefore, $\mu = \tan(22) = 0.4$

W_{EPS} = weight of EPS fill per linear foot of fill = $(A_{EPS})(\gamma_{EPS})$

A_{EPS} = cross-sectional area of EPS fill (See Figure E.5)= 350 ft^2

γ_{EPS} = unit weight of EPS used for project. Note that for this failure mechanism, it is conservative to use the dry unit weight rather than the moist unit weight of 1.9 lb/ft^3.

= 1.25 lb/ft^3

W_{EPS} = (350ft^2)(1.25 lb/ft^3) = 437.5 lb/ft

$W_{Pavement}$ = weight of pavement system per linear foot (See Step 2)

= $A_{Pavement}\gamma_{Pavement}$

$A_{Pavement}$ = cross-sectional area of pavement system

= (2 lanes wide)(12 ft per lane)(3 ft thick) = 72 ft^2

$\gamma_{Pavement}$ = unit weight of pavement system = 130 lb/ft^3 from Step 2

$W_{Pavement}$ = (72 ft^2)(130 lb/ft^3) = 9,360 lb/ft

Determine active pressure coefficient using Equation 4.23 and Figure 4.28.

$$K_A = \frac{\cos^2(\varphi-\omega)}{\cos^2\omega\cos(\delta+\omega)\left[1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-i)}{\cos(\delta+\omega)\cos(i-\omega)}}\right]^2}$$

$$= \frac{\cos^2(40-(-38))}{\cos^2(-38)\cos(20+(-38))\left[1+\sqrt{\frac{\sin(40+20)\sin(40-0)}{\cos(20+(-38))\cos(0-(-38))}}\right]^2}$$

$$= 0.021$$

where:

$\phi' =$ friction angle of backfill $= 40^o$

$\omega =$ slope of the back of the geofoam block along the block edges $= 38^o$ as shown in Figure E.9.

$i =$ slope of the backfill surface $= 0°$

$$\delta = \frac{\phi'}{2} = 20^o$$

Determine static active earth pressure distribution using Figure 4.29 as shown by Figure E.9.

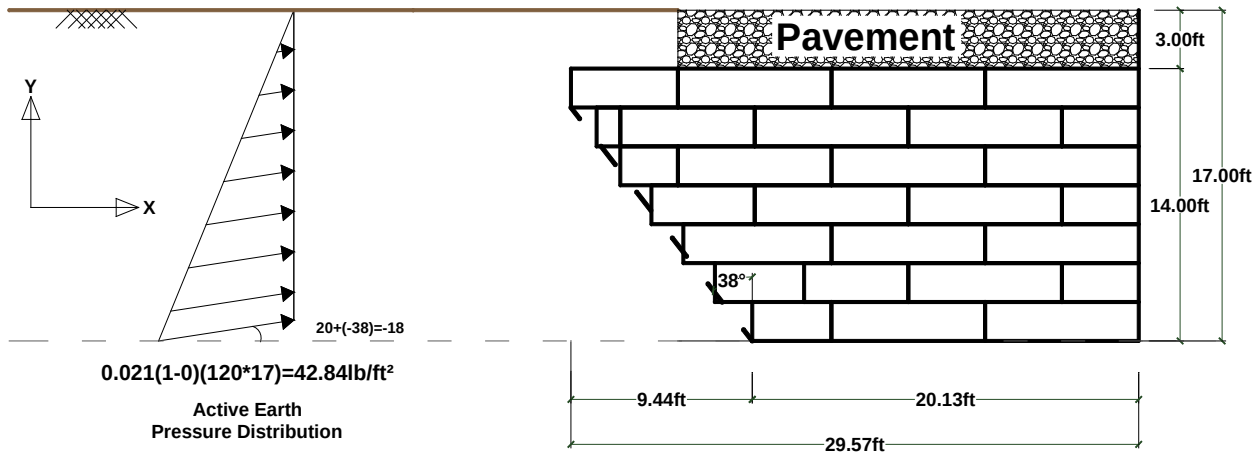


Figure E.9. Calculation of active earth pressure due to the backfill on the EPS blocks for horizontal sliding analysis.

Determine the active force, P_A.

P_A=(1/2)(static active earth pressure)(height of EPS+ pavement)

$$P_A = \frac{(42.84)*17}{2} = 364.14 lb/\ ft - wall$$

$$P_{AX} = P_A * \cos(\delta + \omega)$$

The slope of the back of the geofoam block along the block edges, ω, is approximately 38 degrees.

$$P_{AX} = P_A * \cos(\delta + \omega) = 364.14 * \cos(20 + (-38)) = 346.3 lb/\ ft - wall$$

Since (ω+δ) = (20+(-38))= -18, the vertical component of P_A, P_{AY} is acting downward. Therefore, for the horizontal sliding failure mechanism, P_{AY} is conservatively neglected.

$$\sum P_i = P_{AX} = 346.3\ lb/\ ft - wall$$

From Equation E.1,

$$FS = \frac{(0.4)(437.5 lb/\ ft + 9360 lb/\ ft)}{346.3} = 11.32 > 1.2$$

The factor of safety against horizontal sliding is acceptable.

Evaluate Static Overturning Due to the Backfill Material

Determine the earth pressure forces as shown in Figure E.10.

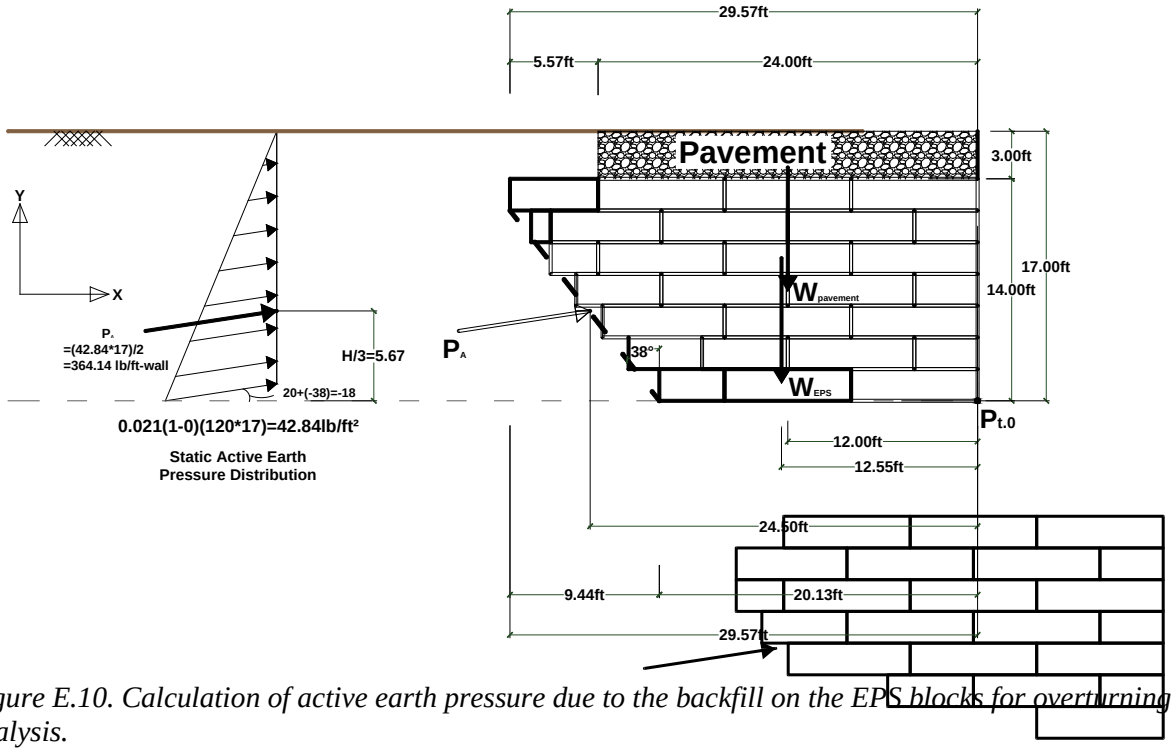


Figure E.10. Calculation of active earth pressure due to the backfill on the EPS blocks for overturning analysis.

The locations of the forces from the weight of the pavement and the EPS blocks are at the center of gravity.

Determine the sum of the moments that tend to cause overturning about Pt.O, $\sum M_o$.

$$\sum M_o = P_A * \cos(\omega + \delta)(H/3)$$
$$= 364.14 * (\cos(20 + (-38))(17/3)$$
$$= 1963\ lb \cdot ft / ft - wall$$

Determine the sum of the moments that tend to resist the overturning moments about Pt. O, $\sum M_R$.

$$\sum M_R = W_{Pavement} * 12 + W_{EPS} * 12.55 + +(P_A \sin(\omega + \delta))(24.5)$$
$$= 9360 * 12 + 437.5 * 12.55 + (364.14 * (\sin(20 + (-38))(24.5)$$
$$= 115,054 lb.ft / ft - wall$$
$$FS = \frac{\sum M_R}{\sum M_o} = \frac{115054}{1963} = 58.6 > 1.2$$

The factor of safety against overturning is acceptable.

STEP 6: SEISMIC STABILITY (EXTERNAL)

Determine the Horizontal Seismic Coefficient for Pseudo-Static Analysis

Once the static slope stability analysis is complete, the seismic stability analysis can proceed. The spectral response acceleration with a seven percent probability of exceedance in 75 years is recommended by ASHHTO 2010. Using the USGS seismic hazard curves for the bridges and assuming a site class of B, the site-modified acceleration response spectrum was developed. The site- modified response spectrum is shown below in Figure E.9, which gives the design spectral acceleration, S_a, as a function of the resonant period of the structure, T. From Figure E.11, S_a is 0.06g for external seismic stability.

Therefore, the horizontal seismic coefficient for pseudo-static analysis is

$$k_h = \frac{S_a}{g} = \frac{0.06g}{g} = 0.06$$

After k_h is determined, perform pseudo-static limit equilibrium stability analysis for the various failure mechanisms.

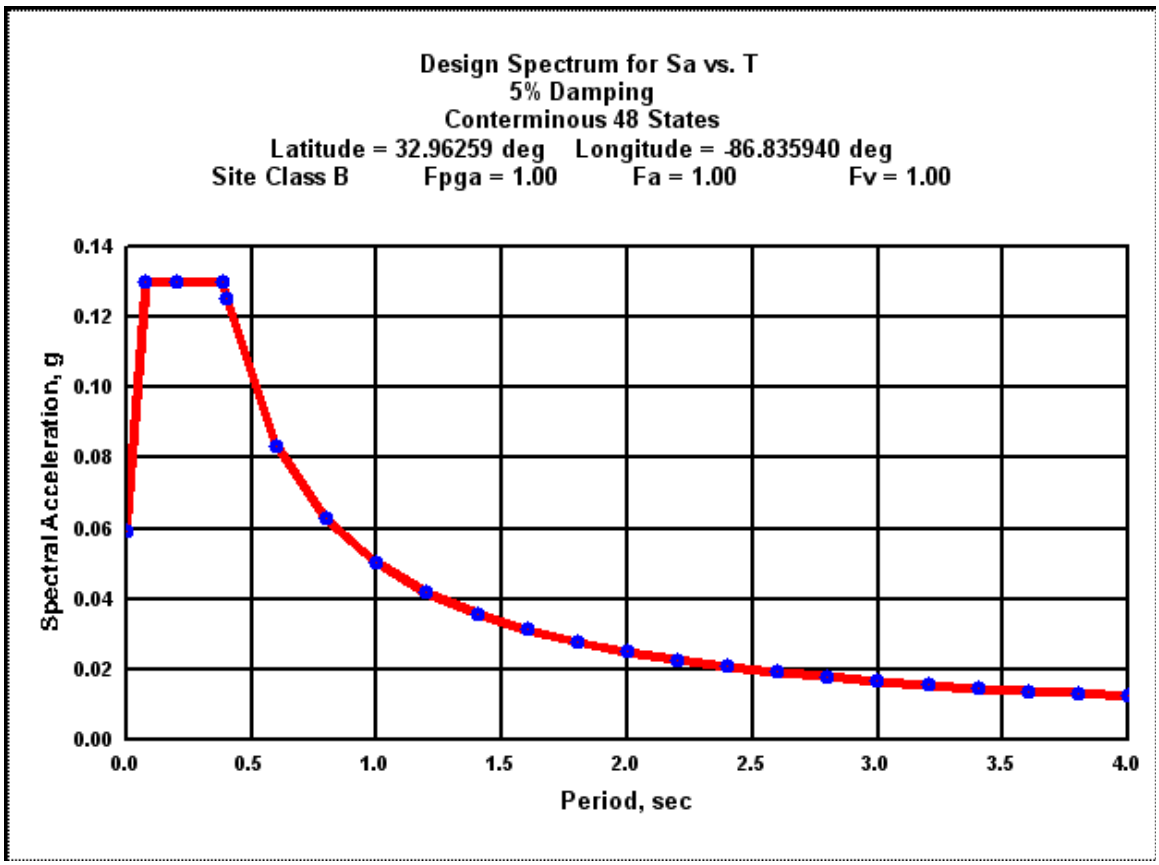


Figure E.11. Site-modified acceleration response spectrum for project site.

Evaluate overall seismic slope stability

Using the seismic coefficient from Figure E.11, re-analyze the slope and the EPS fill to determine the seismic factor of safety. Figure E.12 shows the results of the seismic stability analysis. As shown in Figure E.12, the factor of safety for seismic slope stability is 1.06, which is satisfies the required factor of safety 1.05. Thus, the proposed slope configuration is acceptable.

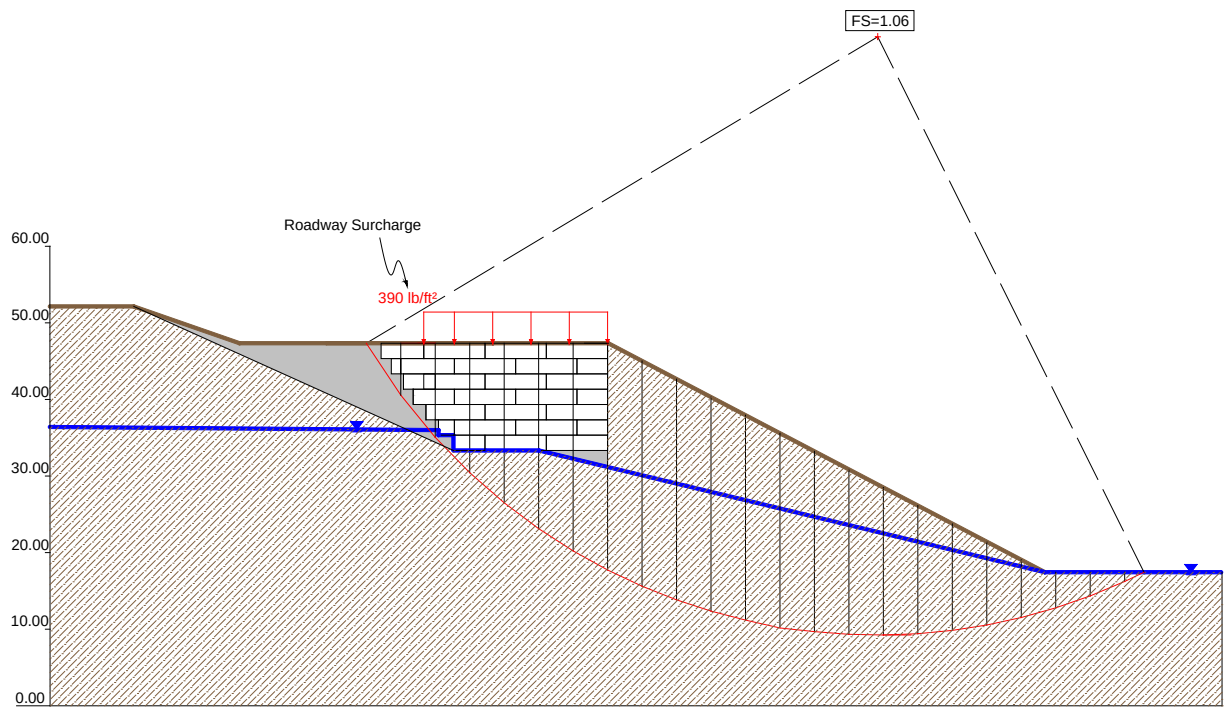


Figure E.12. Results of seismic slope stability analysis using Spencer's Method for EPS fill developed in Step 4 of the design procedure.

Evaluate Seismic Horizontal Sliding

Use Figure 4.30 to evaluate the factor of safety against seismic horizontal sliding. As shown in Figure 4.30, the factor of safety against seismic horizontal sliding is:

$$FS = \frac{\mu(W_{EPS} + W_{Pavement})}{W_{EPS}k_h + W_{Pavement}k_h + \Sigma P_i} \qquad \textbf{(E.2)}$$

From the pervious evaluation of static horizontal sliding in Step 5,

$\mu = 0.4$

W_{EPS} = 437.5 lb/ft

$W_{Pavement}$ =9360lb/ft

k_h = 0.06 as previously determined in this step.

Determine active and dynamic earth pressures using Figure 4.28 and 4.29.

The shear strength of the natural slope material of $\phi' = 25°$ is used instead of the shear strength of the granular backfill material because the extent of granular backfill material is typically limited and because the shear strength of the granular backfill will typically exceed $\phi' = 25°$. Therefore, the analysis is conservative.

From Equation 4.21,

$$K_{AE} = \frac{\cos^2(\varphi - \psi - \omega)}{\cos\psi \cos^2\omega \cos(\delta + \omega + \psi)[1 + \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - \psi - i)}{\cos(\delta + \psi + \omega)\cos(i - \omega)}}]^2}$$

$$= \frac{\cos^2(25 - 3.43 - (-38))}{\cos 3.43 \cos^2 38 \cos(12.5 + (-38) + 3.43)[1 + \sqrt{\frac{\sin(25 + 12.5)\sin(25 - 3.43 - 0)}{\cos(12.5 + 3.43 + (-38))\cos(0 - (-38))}}]^2}$$

$$= 0.18$$

where

$\phi' =$ friction angle of natural slope $= 25^o$

$\omega =$ slope of the back of the geofoam block along the block edges $= 38^o$

$i =$ slope of the backfill surface $= 0°$

$$\delta = \frac{\phi'}{2} = 12.5$$

$$\psi = \arctan(\frac{k_h}{1 - k_v}) = \arctan(\frac{0.06}{1 - 0}) = 3.43$$

From Equation 4.23,

$$K_A = \frac{\cos^2(\varphi - \omega)}{\cos^2\omega \cos(\delta + \omega)\left[1 + \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - i)}{\cos(\delta + \omega)\cos(i - \omega)}}\right]^2}$$

$$= \frac{\cos^2(25 - (-38))}{\cos^2(-38)\cos(12.5 + (-38))\left[1 + \sqrt{\frac{\sin(25 + 12.5)\sin(25 - 0)}{\cos(12.5 + (-38))\cos(0 - (-38))}}\right]^2}$$

$$= 0.14$$

The static active and seismic earth pressures are determined from Figure 4.29 and are shown in Figure E.13.

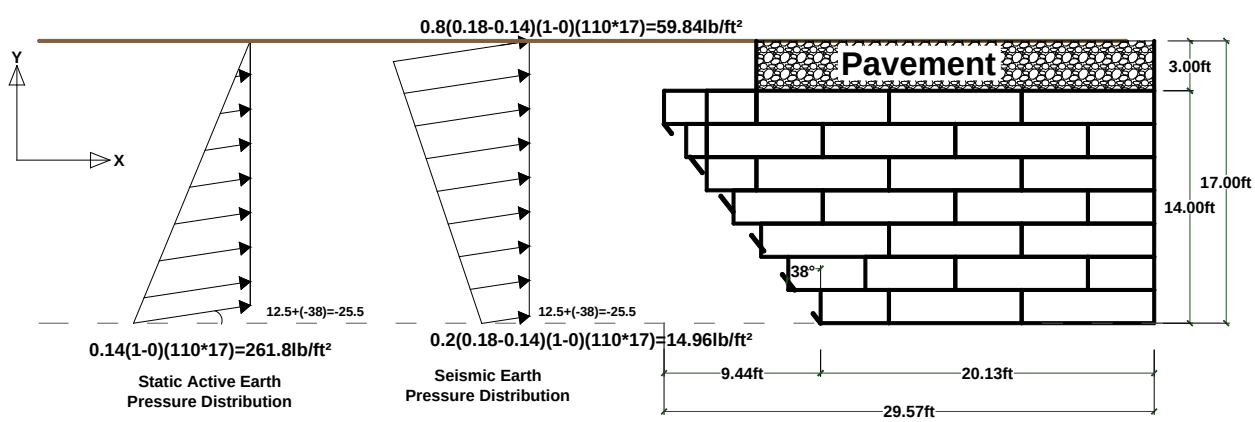


Figure E.13. Static active and seismic earth pressures.

$\Sigma P_i = P_A + P_E$

where

$P_A =$ static active earth pressure

$P_E =$ seismic force

$$\Sigma P_i = (\frac{1}{2})(\text{static active earth pressure})(\text{height of EPS} + \text{pavement}) +$$

$$(\frac{1}{2})(\text{seismic earth pressure})(\text{height of EPS} + \text{pavement})$$

$$= \frac{(261.8)*17}{2} + \frac{(59.84+14.96)*17}{2} = 2861 lb/\ ft-wall$$

$\Sigma P_{Xi} = \Sigma P_i * \cos(\delta + \omega)$

The slope of the back of the geofoam block along the block edges, ω,is approximately 38 degrees.

$\Sigma P_{Xi} = \Sigma P_i * \cos(\delta + \omega) = 2861 * \cos(12.5 + (-38)) = 2582.4\ lb/\ ft-wall$

Since (ω+δ) = (12.5+(-38))= -25.5, the vertical components of P_A and P_E act downward.

$\Sigma P_{Yi} = \Sigma P_i * \sin(\delta + \omega) = 2861 * \sin(12.5 + (-38)) = -1231.7\ lb/\ ft-wall$

From equation E.2,

$$FS = \frac{(0.4)(437.5 lb/\ ft + 9360 lb/\ ft + 1231.7 lb/\ ft)}{(437.5 lb/\ ft)(0.06) + (9360 lb/\ ft)(0.06) + 2582.4} = 1.4 > 1.2$$

Therefore, the factor of safety against external seismic horizontal sliding satisfies the required value of 1.2. Note that the preceding calculation conservatively neglected the additional sliding resistance due to the wedge of soil located to the right of the EPS fill in Figure E.12.

Evaluate Seismic Overturning.

Since the EPS-block geofoam fill for this slope is relatively short and is confined by a significant amount of soil on both sides, this failure mechanism will probably not be critical. Figure E.14 provides the static active and seismic earth pressures and forces.

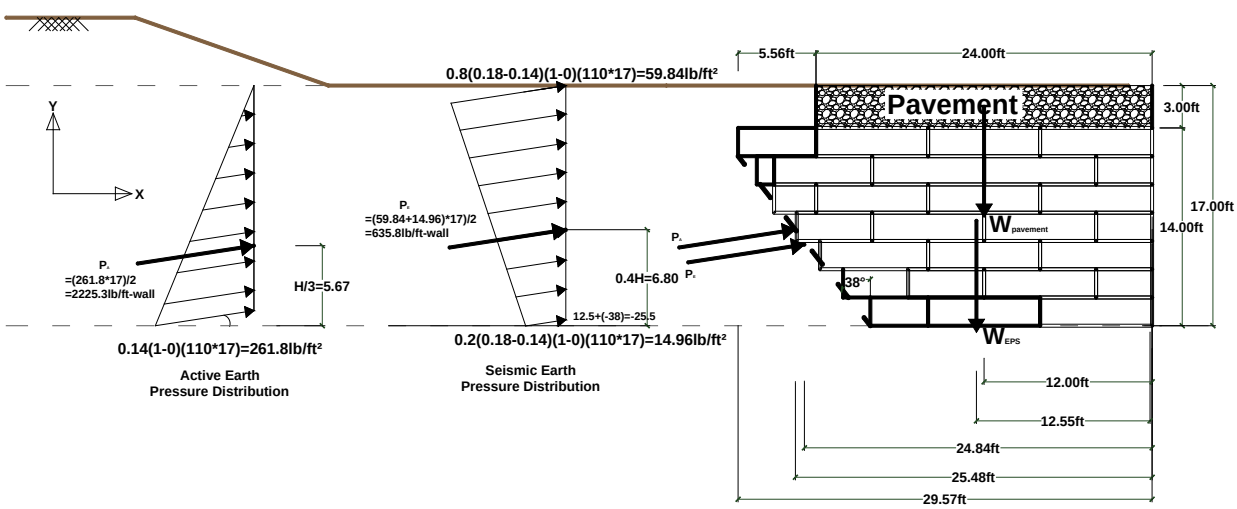


Figure E.14. Calculation of static active and seismic earth pressures and forces for overturning analysis.

$$\sum M_o = P_A\left(\cos(\omega+\delta)\right)(H/3) + P_E\left(\cos(\omega+\delta)\right)(0.4H)$$
$$= 2225.3*(\cos(12.5+(-38))(17/3) + 635.8*(\cos(12.5+(-38))*(0.4*17)$$
$$= 15,283\ lb\cdot ft/\ ft-wall$$

Since (ω+δ) = (12.5+(-38))= -25.5°, the vertical components of P_A and P_E act downward.

$$\sum M_R = W_{Pavement}*12 + W_{EPS}*12.55 + (P_A\sin(\omega+\delta))(24.84) + (P_E\sin(\omega+\delta))(25.48)$$
$$= 9360*12 + 437.5*12.55 + (2225.3*(\sin(12.5+(-38))(24.84) +$$
$$(635.8*(\sin(12.5+(-38)))(25.48)$$
$$= 87,039\ lb\cdot ft/\ ft-wall$$
$$FS = \frac{\sum M_R}{\sum M_o} = \frac{87039}{15283} = 5.7 > 1.2$$

Therefore, the factor of safety against seismic overturning satisfies the required value of 1.2.

Evaluate Seismic Bearing Capacity

From Equation 4.45, the ultimate bearing capacity, q_{ult}, of the foundation material beneath the EPS-block geofoam fill is

$$q_{ult} = c'N_c s_c d_c i_c b_{ci} g_c + \sigma'_{vD} N_q s_q d_q i_q b_q g_q + 0.5\gamma'_{soil} BN_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

Since there is eccentric load in this problem, the following condition should be satisfied.
Moment over center of the base of foundation, Point C as shown in Figure E.15 is

$M = -P_{AX}*5.67-P_{EX}*6.8+ W_{EPS}*(12.55-10.07)+W_{Pavement}*(12-10.07)+ P_{AY}*(24.84-10.07)$
$+P_{EY}*(25.48-10.07)$
$= 14,508.7$ *lb.ft/ft-wall*

$P = W_{EPS}+W_{Pavement}+ P_{AY}+P_{EY}$
$= 437.5+9360+958+274$
$= 11029.2$*lb/ft*

$$e = \frac{M}{P} = \frac{14508.7}{11029.2} = 1.31 < \frac{B}{6} = \frac{20.13}{6} = 3.36 \Rightarrow O.K$$

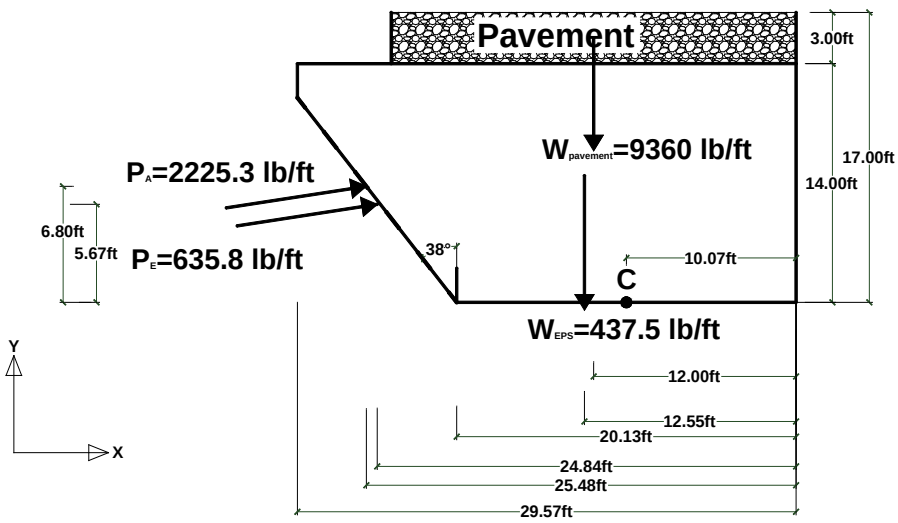


Figure E.15 free body diagram

Determine the effective width, *B'*.

$B' = B - 2e_B = 20.13 - 2*1.31 = 17.51\,ft$

$c' = 0.$

To calculate γ'_{soil}, it is first necessary to determine if groundwater Case 1, 2 or Case 3 is applicable as summarized in Chapter 4. From Figure E.5 and E.7, B=20.13, D = 17 ft, the average D_w = 17 ft. Therefore, Case 1 is applicable because $D_w \le D$ and Equation 4.46 is applicable.

From Equation 4.46,

$$\gamma'_{soil} = \gamma_{soil} - \gamma_w$$
$$= 110\ lb/\ ft^3 - 62.4\,lb/\ ft^3$$
$$= 47.6\ lb/\ ft^3$$

$$\sigma'_{vD} = \gamma_{soil}(D)$$

Therefore,

$$\sigma'_{vD} = 110\,lb/\ ft^3 \times 17\ ft = 1870\,lb/\ ft^2$$

N_q = 10.7 for ϕ' = 25°. Recall that ϕ' = 25° is the shear strength of the soil slope. Values of N_c, N_q, and N_γ can be obtained from any foundation design textbook.

$s_q, i_q, b_q, s_\gamma, i_\gamma, b_\gamma, d_\gamma$ are all equal 1 for this example problem. The shape factors, s_c, s_q, s_γ should be based on the effective footing dimensions.

The depth factors are based on the actual footing dimensions.

$$d_q = 1 + 2k\tan\varphi'(1-\sin\varphi')^2$$

since $\frac{D}{B} = 0.84 \le 1$, then $k = \frac{D}{B} = 0.84$

$$d_q = 1 + 2*0.84*\tan(25)*(1-\sin(25))^2$$
$$= 1.26$$

$g_q = g_\gamma = (1 - \tan\beta)^2$. For the slope shown in Figure E.7, the value of β, the slope inclination, is approximately 28°. Therefore, the values of both g_q and g_γ are approximately 0.22.

N_γ = 10.9 for ϕ' = 25°.

i_q, i_γ are load inclination factors and are all equal 1 for this example problem.

Equation 4.45 becomes

$$q_{ult} = c' N_c s_c d_c i_c b_{ci} g_c + \sigma'_{vD} N_q s_q d_q i_q b_q g_q + 0.5\gamma'_{soil} B' N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

$$= 0 + (1870 lb/ft^2)(10.7)(0.22)(1.26) + 0.5(47.6 lb/ft^3)(17.51\, ft)(10.9)(0.22)$$

$$= 6545.8 lb/ft^2$$

$$q_a = \frac{q_{ult}}{FS} = \frac{6545.8}{3} = 2182 lb/ft^2$$

In the seismic condition the allowable bearing capacity can be increased by 33% (FHWA GEC No.3).

$$q_a = 2182 lb/ft^2 * 1.33 = 2902\ lb/ft^2$$

Next, determine the bearing pressure acting on the foundation soil. Per FHWA GEC No. 6, the applied pressure due to the eccentric loading is assumed to be an equivalent uniform pressure over the effective area. Therefore,

$$q_{\max} = \frac{P}{B' * L} = \frac{11029.2}{17.51*1} = 629.9 lb/ft^2$$

$$q_{\max} = 629.9 lb/ft^2 < q_a = 2902 lb/ft^2$$

Therefore, factor of safety against seismic bearing capacity failure is adequate based on an effective stress analysis. Should also check seismic bearing capacity based on a total stress analysis. The effective stress analysis performed above is based on the assumption that the silty clay foundation soil is not a sensitive soil and, therefore, no loss of shear strength will occur during the earthquake. Note that this factor of safety value is conservative because it does not take into account any stress dissipation of the pavement system stresses through the EPS fill.

Evaluate Seismic Settlement

Since the EPS-block geofoam fill will exert less vertical stress on the foundation soil than the original slope material, it is not likely that seismic shaking will have a major impact on the settlement.

STEP 7: SEISMIC STABILITY (INTERNAL)

Step 7i : Estimate the Seismic—Response Acceleration of the Existing Ground Surface or Base (Subgrade Level) of the EPS fill mass.

This was performed in Step 6. The site- modified response spectrum, which gives the design spectral acceleration, S_a, as a function of the resonant period of the structure, *T*, is shown in Figure E.11. From Figure E.11, S_a is 0.06g and the corresponding horizontal seismic coefficient is 0.06 as previously determined in Step 6.

Step 7ii: Estimate the Seismic-Response Acceleration at the Top of the EPS Fill Mass.

Calculate resonant period of the EPS fill mass using Equation 4.24.

$$T_0 = 2\pi\left\{\frac{\sigma'_{v_0} H}{E_{t_i} g}\left[4\left(\frac{H}{B}\right)^2 + \left(\frac{12}{5}\right)(1+\upsilon)\right]\right\}^{1/2}$$

σ'_{vo} = vertical effective stress at the top of the EPS blocks due to dead loads only. Therefore, use the assumed pavement system from Step 2.

= (3 ft)(130 lb/ft3) = 390 lb/ft²

A = cross-sectional area of EPS. From Figure E.5

$= 350\ ft^2 = B'H = H'B$

H = height of EPS-block geofoam fill= *14 ft*

H'= equivalent height of EPS-block geofoam fill (yielding A=350 ft²with B) =11.8 *ft*

E_{ti} = initial tangent Young's Modulus of the EPS

= 725 lb/in² = 104400 lb/ft² (from Table F.2)

g = gravitational constant = *32.2 ft/s²*

B = width of EPS-block geofoam fill = *29.57ft*

B' = equivalent width of EPS-block geofoam fill (yielding A=350 ft²with H) = *25 ft*

υ = Poisson's ratio for the EPS

= 0.1

Determine the resonant period of the EPS fill mass for both Model 1 and Model 2 shown in Figure 4.36 using Equation 4.26.

$$T_1 = 2\pi\left\{\frac{(390\ lb/\ ft^2)(14\ ft)}{(104400\ lb/\ ft^2)(32.2\ ft/s^2)}\left[4\left(\frac{14\ ft}{25\ ft}\right)^2 + \left(\frac{12}{5}\right)(1+0.1)\right]\right\}^{1/2} = 0.5\ \text{sec}$$

$$T_2 = 2\pi\left\{\frac{(390\ lb/\ ft^2)(11.8\ ft)}{(104400\ lb/\ ft^2)(32.2\ ft/s^2)}\left[4\left(\frac{11.8\ ft}{29.57\ ft}\right)^2 + \left(\frac{12}{5}\right)(1+0.1)\right]\right\}^{1/2} = 0.42\,\text{sec}$$

Use both of the above resonant periods of the EPS fill to determine the spectral acceleration value for the EPS fill from the site- modified response spectrum as shown in Figure E.16.

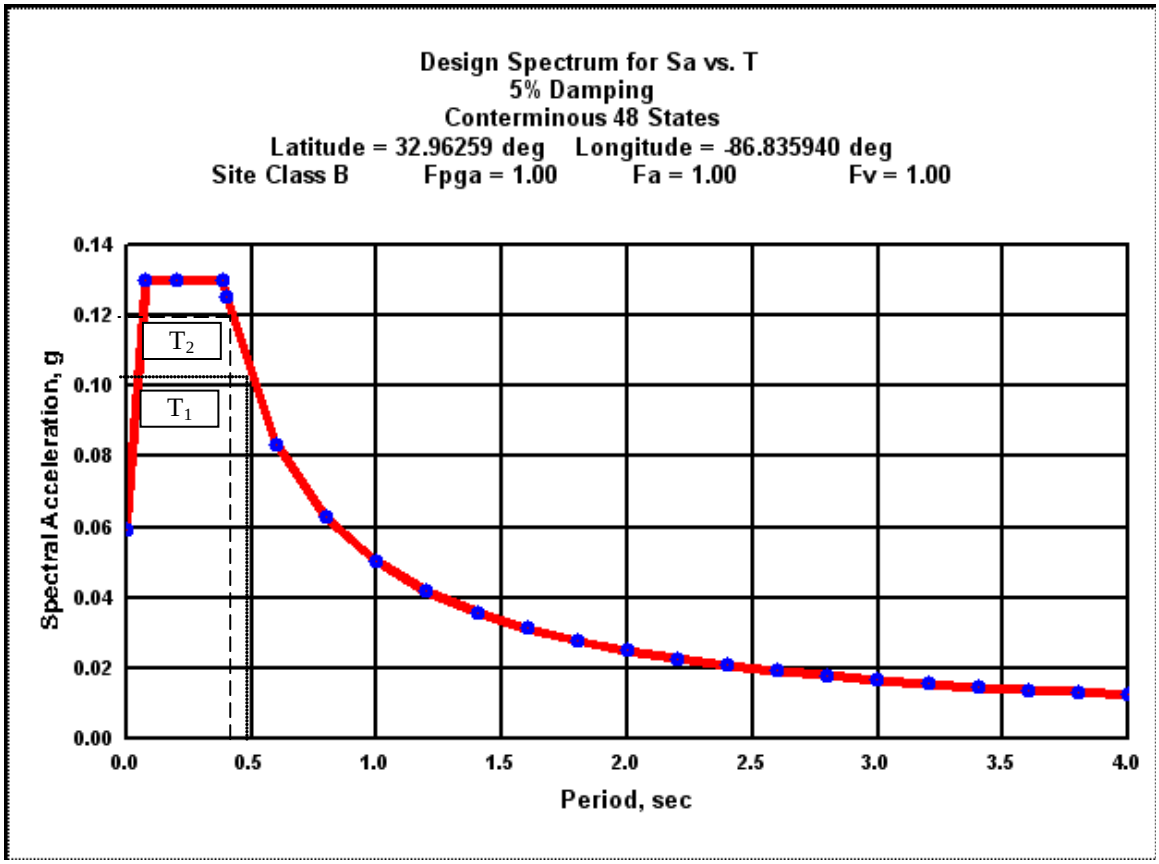


Figure E.16. Determining the spectral acceleration value from the site-modified acceleration response spectrum.

The critical spectral acceleration is the larger value of the two resonant periods, which is $S_a = 0.12\ g$ as shown in Figure E.16. This spectral acceleration value represents the acceleration that the top of the EPS-block geofoam fill will experience during the design earthquake. Therefore, the horizontal seismic coefficient for pseudo-static analysis is

$$k_h = \frac{S_a}{g} = \frac{0.12g}{g} = 0.12$$

Step 7iii: Perform Pseudo-Static Limit Equilibrium Stability Analyses of the various Failure Mechanisms.

Check internal horizontal sliding between the pavement system and the upper layer of blocks and between layers of blocks. First check horizontal sliding between the pavement system and the upper layer of blocks.

Using the same model that was used for external seismic sliding (Figure 4.30), evaluate the factor of safety against internal seismic sliding at 1) the interface between the pavement system and the EPS blocks and 2) the interfaces between layers of EPS blocks throughout the fill. For the purpose of this example, only the interface between the pavement system and the EPS blocks and the interface between the top two layers of EPS blocks will be presented. However, in actual practice, horizontal sliding should be checked at several different levels through the EPS fill. The reason for this is that, since concentrated vertical stresses such as those associated with the pavement system tend to dissipate with depth, some of the

lower interfaces between layers of EPS blocks may have less vertical stress applied to them and may therefore be more likely to experience horizontal sliding.

Evaluate factor of safety against sliding between pavement system and top layer of EPS blocks. This slip surface is designated as Failure Surface 1. Figure E.17 shows the geometry for Failure Surface 1.

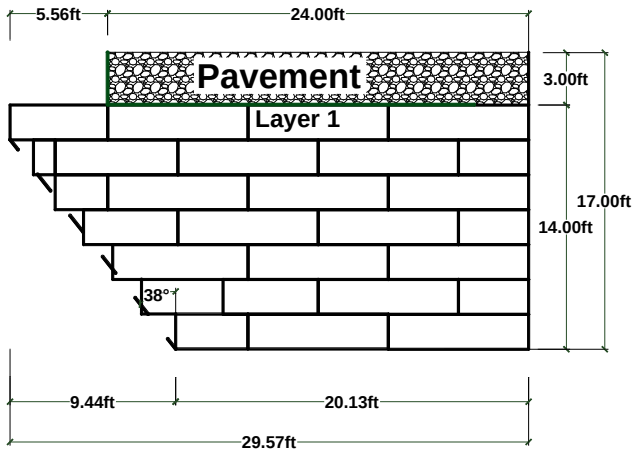


Figure E.17. Geometry for Failure Surface 1.

Equation E.2 can be used to determine the factor of safety against seismic horizontal sliding along the pavement system and EPS interface.

μ = coefficient of friction between pavement system and EPS.

= tan (55.5°) using δ value from Table 3.2 for interface between gravel and EPS blocks. Note that, just as in external horizontal sliding, the δ value used here is adequate for preliminary design, but lab tests should be performed using the actual material from the project to determine a more accurate δ value for the final design.

μ = 1.46

W_{EPS} = 0 for this case since there are no EPS blocks overlying the interface being considered.

$W_{Pavement}$ = 9,360 lb/ft from pervious evaluation of static horizontal sliding in Step 5.

k_h = 0.12 as previously obtained from Figure E.16.

Determine active and dynamic earth pressures using Figures 4.28 and 4.29. From Equation 4.21,

$$K_{AE} = \frac{\cos^2(\varphi - \psi - \omega)}{\cos\psi \cos^2\omega \cos(\delta + \omega + \psi)[1 + \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - \psi - i)}{\cos(\delta + \psi + \omega)\cos(i - \omega)}}]^2}$$

$$= \frac{\cos^2(25 - 3.43 - (-38))}{\cos 3.43 \cos^2 38 \cos(12.5 + (-38) + 3.43)[1 + \sqrt{\frac{\sin(25 + 12.5)\sin(25 - 3.43 - 0)}{\cos(12.5 + 3.43 + (-38))\cos(0 - (-38))}}]^2}$$

$$= 0.18$$

where

$\phi' =$ friction angle of natural slope $= 25^{\circ}$

$\omega =$ slope of the back of the geofoam block along the block edges $= 38^{\circ}$

$i =$ slope of the back of the surface $= 0°$

$$\delta = \frac{\phi'}{2} = 12.5°$$

$$\psi = \arctan(\frac{k_h}{1 - k_v}) = \arctan(\frac{0.06}{1 - 0}) = 3.43°$$

From Equation 4.23,

$$K_A = \frac{\cos^2(\varphi - \omega)}{\cos^2\omega \cos(\delta + \omega)\left[1 + \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - i)}{\cos(\delta + \omega)\cos(i - \omega)}}\right]^2}$$

$$= \frac{\cos^2(25 - (-38))}{\cos^2(-38)\cos(12.5 + (-38))\left[1 + \sqrt{\frac{\sin(25 + 12.5)\sin(25 - 0)}{\cos(12.5 + (-38))\cos(0 - (-38))}}\right]^2}$$

$$= 0.14$$

The static active and seismic earth pressures are shown in Figure E.18.

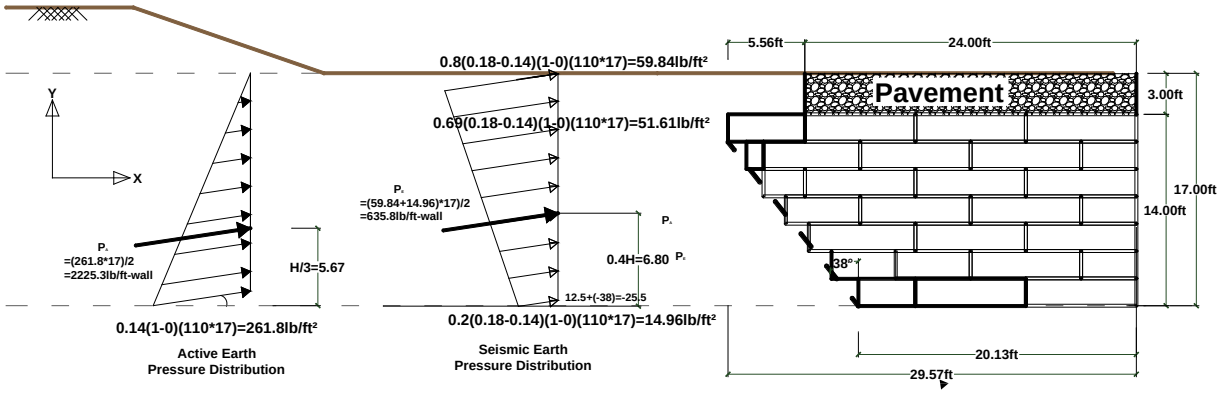
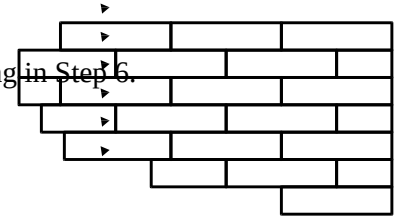


Figure E.18. Static active and seismic earth pressures.



$\Sigma P_i = P_A + P_E$ as previously defined in external seismic horizontal sliding in Step 6.

$$= (\frac{1}{2})(\text{static active earth pressure})(\text{height of pavement}) +$$

$$(\frac{1}{2})(\text{seismic earth pressure})(\text{height of pavement})$$

$$= \frac{(261.8*(3/17))*17}{2} + \frac{(59.84 + 51.61)*3}{2} = 559.88 lb/\ ft - wall$$

$$P_{Xi} = \Sigma P_i * \cos(\delta + \omega) = 559.88 * \cos(12.5 + (-38)) = 505.33 lb/\ ft - wall$$

From Equation E.2,

$$FS = \frac{\mu(W_{EPS} + W_{Pavement})}{W_{EPS} k_h + W_{Pavement} k_h + \Sigma P_i}$$

$$FS = \frac{1.46(0 + 9360 lb/\ ft)}{(0)(0.12) + (9360 lb/\ ft)(0.12) + 505.33} = 8.4 > 1.2$$

Therefore, the factor of safety against horizontal sliding for Failure Surface 1, i.e., between the pavement system and the first EPS block layer meets the required value of 1.2.

Now evaluate factor of safety against sliding between the top two layers of EPS blocks. This slip surface is designated as Failure Surface 2. Figure E.19 shows the geometry for Failure Surface 2.

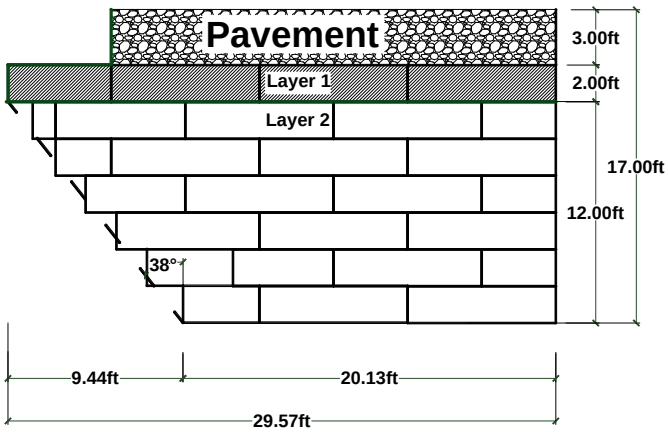


Figure E.19. Geometry for Failure Surface 2.

Equation E.2 can be used to determine the factor of safety against seismic horizontal sliding along Failure Surface 2.

μ = coefficient of friction between EPS and EPS.

= tan (30°) using δ value from Table 3.1 for interface between two EPS blocks.

= 0.58

W_{EPS} = weight of EPS fill per linear foot of fill = $(A_{EPS})\gamma_{EPS}$

A_{EPS} = cross-sectional area of the EPS blocks located above the sliding interface being considered. As shown in Figure E.18.

= (2 ft)(29.57 ft) = 59.14 ft^2

γ_{EPS} = dry unit weight of EPS is conservatively used.

= 1.9 lb/ft^3

W_{EPS} = (59.14 ft^2)(1.9 lb/ft^3) = 112.37 lb/ft

$W_{Pavement}$ = 9,360 lb/ft from the pervious evaluation of static horizontal sliding in Step 5.

k_h = 0.12 as previously obtained from Figure E.16.

Determine active and dynamic earth pressures using Figure 4.28 and 4.29.

K_A and K_{AE} are the same as for the failure surface 1.

$K_A = 0.14$

$K_{AE} = 0.18$

The static active and seismic earth pressures are shown in Figure E.20.

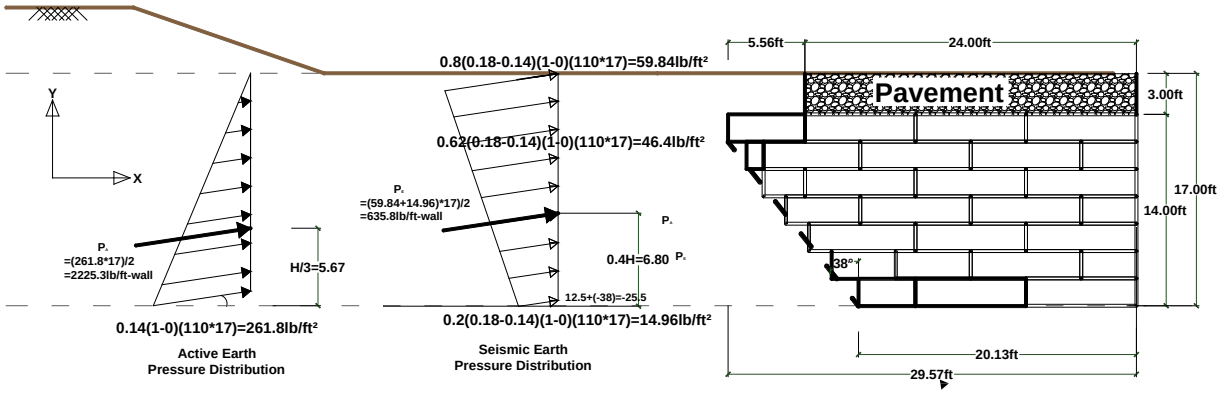
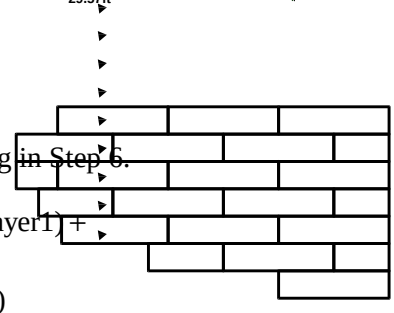


Figure E.20. Static active and seismic earth pressures.



$\Sigma P_i = P_A + P_E$ as previously defined in external seismic horizontal sliding in Step 6.

$$= (\frac{1}{2})(\text{static active earth pressure})(\text{height of pavement} + \text{height of layer1}) +$$

$$(\frac{1}{2})(\text{seismic earth pressure})(\text{height of pavement} + \text{height of layer1})$$

$$= \frac{(261.8*(5/17))*17}{2} + \frac{(59.84 + 46.4)*5}{2} = 920.1 lb/\ ft - wall$$

$$P_{Xi} = \Sigma P_i * \cos(\delta + \omega) = 920.1 * \cos(12.5 + (-38)) = 830.47 lb/\ ft - wall$$

From Equation E.2,

$$FS = \frac{\mu(W_{EPS} + W_{Pavement})}{W_{EPS} k_h + W_{Pavement} k_h + \Sigma P_i}$$

Value of horizontal seismic coefficient varies linearly by the depth and it can be estimated by linearly interpolating between horizontal seismic coefficient at the top (0.12 in this example) and base (0.06 in this example).

At a depth of 3ft the horizontal seismic coefficient is

$$k_h = 0.06 + (0.06 - 0.06 * \frac{3}{17}) = 0.11$$

$$FS = \frac{1.46(112.37 + 9360 lb/\ ft)}{(112.37)(0.11) + (9360 lb/\ ft)(0.12) + 830.47} = 7 > 1.2$$

Therefore, the factor of safety against horizontal sliding for Failure Surface 2, i.e., between the top two layers of EPS blocks, meets the required value of 1.2.

For the purpose of this example, only the interface between the pavement system and the EPS blocks and the interface between the top two layers of EPS blocks was determined. However, in actual practice, horizontal sliding should be checked at several different levels through the EPS fill. The reason for this is that, since concentrated vertical stresses such as those associated with the pavement system tend to

dissipate with depth, some of the lower interfaces between layer of EPS blocks may have less vertical stress applied to them and may therefore be more likely to experience horizontal sliding.

Check Seismic Load Bearing

Since the EPS-block geofoam fill mass is completely buried within the slope and confined by soil on all sides, the seismic rocking shown in Figure 4.14 will probably not be possible. Therefore, this failure mechanism is not considered.

STEP 8: PAVEMENT SYSTEM DESIGN

The pavement design requirements are the same as the example problem provided in the NCHRP 24-11(01) report for stand-alone geofoam embankments over soft ground (Stark et al. 2004) including wheel loadings and design level of reliability. The supervising agency prefers that flexible pavements be used whenever possible. The roadway is a low-volume road. The traffic loads for this section of the highway have been estimated at roughly 300,000 equivalent single axle loads (ESAL). The pavement system is to be designed based on a 75% level of reliability.

- From the flexible pavement design catalog for low-volume roads included in Table 3 of the NCHRP 24-11(01) Report 529 for stand-alone geofoam embankments over soft ground the design structural number, SN_{REQ}, for *EPS50* block is found to be 5.6.
- The pavement design manual used by the supervising agency provides the following values for the pavement materials:
    - Asphalt concrete: $a_1 = 0.44$
    - Crushed stone based: $a_2 = 0.14$
- Try a pavement system consisting of 29 in. (432 mm) of crushed stone base course with a 7 in. (178 mm) layer of asphalt concrete.

$$SN = a_1 D_1 + a_2 D_2 = (0.14 \times 29\,in) + (0.44 \times 7\,in) = 7.14 > SN_{REQ}$$

The structural number of this pavement system exceeds the required structural number for the EPS-block geofoam subgrade. Therefore, the pavement system is acceptable and may be used in the remainder of the design.

STEP 9: EVALUATION OF EFFECTS OF PAVEMENT SYSTEM DESIGN ON PREVIOUS FAILURE MECHANISMS

Next, the pavement system designed in Step 8 must be compared to the assumed pavement system from Step 2 to determine whether the design from Step 8 constitutes a significant change.

Remember that Step 2 assumed that the pavement system could be approximated by 3 ft of material having a unit weight of 130 lb/ft^3. Therefore, the vertical stress due to the dead load at the bottom of the assumed pavement system would be

$$\sigma_{v\_Step2} = (3\ ft)(130\,lb/\ ft^3) = 390\ lb/\ ft^2$$

Assuming a unit weight of 130 lb/ft^3 for asphalt and a unit weight of 130 lb/ft^3 for the crushed stone, the vertical stress at the bottom of the actual pavement system designed in Step 8 can be calculated as

$$\sigma_{v\_Step8} = (0.58\ ft \times 130\,lb/\ ft^3) + (2.42\ ft \times 130\,lb/\ ft^3) = 390\ lb/\ ft^2$$

Based on the calculations shown above, the pavement system that was designed in Step 8 will exert the same stress as the pavement system that was assumed in Step 2. Therefore, the design may proceed to Step 10.

As noted in Figure 4.25, if the stress of the pavement system determined in this Step 9 is different than the estimated one used in Step 2, static and seismic slope stability must be rechecked and, therefore, the design procedure would revert back to Step 5.

STEP 10: LOAD BEARING (INTERNAL)

For this example, additional stresses due to seismic loads are not considered because they are not significant.

Step 10.i: Estimate traffic loads.

For this roadway, use AASHTO H 20-44 standard loading. Therefore,
Rear Axle Load = 24,000 lbs.

Live load per dual tire set, $LL_D = \frac{24{,}000\ lbs}{2}$ = 12,000 lbs.

Step 10.ii: Add impact allowance to traffic loads.

Use impact coefficient, I, of 0.3.
Using AASHTO (1996) approach, calculate the impact-corrected traffic load, Q_D.

$$Q_D = LL_D(1+I) = (12000\ lbs)(1+0.3) = 15{,}600\ lbs$$

Step 10.iii: Estimate traffic stresses at top of EPS blocks.

Determine circular contact area, A_{CD}, from methods included in the NCHRP 24-11(01) Report 529 for stand-alone geofoam embankments over soft ground using Q_D from Step 10.ii and σ_{LL}, which is based on the traffic load, LL_D, from Step 10.i.

$$A_{CD} = \frac{Q_D}{\sigma_{LL}} = \frac{15600\ lbs}{397\ lb/ft^2} = 39.3\ ft^2$$

Determine equivalent rectangular loaded area using relationships from the NCHRP 24-11(01) Report 529 for stand-alone geofoam embankments over soft ground. To convert the circular contact area, A_{CD}, to its equivalent rectangular loaded area, the parameter L' must first be calculated. From this parameter, the equivalent loaded length, *L*, and the equivalent loaded width, *B*, can be calculated using the relationships given below. Note that these are empirical relationships based on SI units. Therefore, A_{CD} must be converted from ft^2 to m^2.

$$68.4\ ft^2 = 3.65\ m^2$$

$$L' = \sqrt{\frac{A_{CD}}{0.5227}} = \sqrt{\frac{3.65\ m^2}{0.5227}} = 2.64\ m$$

$$L = 0.8714 \times L' = (0.8714)(2.64\ m) = 2.3\ m = 7.5\ ft$$

$$B = 0.6 \times L' = (0.6)(2.64\,m) = 1.58\,m = 5.2\ ft$$

If the center-to-center wheel spacing on the design truck is $\leq B$, stress overlap will occur between the wheel loads. For this example, B = 5.2 ft. The center-to-center wheel spacing for the AASHTO H 20-44 design truck is 4.0 ft. Therefore, stress overlap will occur. This overlap distance will be approximately equal to $(5.2\ ft) - (4.0\ ft) = 1.2\ ft.$ Therefore, the combined rectangular width =

$(2 \times B) - 1.2\ ft = (2 \times 5.2\ ft) - 1.2\ ft = 9.2\ ft.$

The combined rectangular area = $9.2\ ft \times 7.5\ ft = 69\ ft^2$

The combined load of the two dual tire sets = $2 \times 15{,}600\, lbs = 31{,}200\, lbs$

Therefore, the combined vertical stress = $\sigma_{LL} = \dfrac{31{,}200\, lbs}{69\ ft^2} = 452.2\, lb/\ ft^2$

Because the actual center-to-center spacing between one interior dual tire set and an exterior dual tire set is greater than B, stress overlap between the tire sets does not occur.

Step10.iv: Estimate gravity stresses at top of EPS blocks.

From Step 8,

$\sigma_{DL} = \sigma_{v\_Step8} = (0.58\ ft \times 130\, lb/\ ft^3) + (2.42\ ft \times 130\, lb/\ ft^3) = 390\ lb/\ ft^2$

Step 10.v: Calculate total stresses at top of EPS blocks.

$\sigma_{Total} = \sigma_{DL} + \sigma_{LL} = 390\, lb/\ ft^2 + 452.2\, lb/\ ft^2 = 842.2\, lb/\ ft^2$

Step 10.vi: Determine minimum required elastic limit stress for top layer of EPS blocks.

Using a factor of safety of 1.2 against load bearing failure,

$\sigma_{e\ req'd} = 1.2 \times 842.2\, lb/\ ft^2 = 1010.6\, lb/\ ft^2 = 7.02\, lb/in^2$

Step 10.vii: Calculate factor of safety against load bearing failure using the elastic limit stress of preliminary EPS type from Step 2 (See Table F.2 of the report).

In Step 2, EPS50 from Table F.2 of the report was selected. This type of EPS has a unit weight of 1.25 lb/ft^3 and an elastic limit stress of 7.2 lb/in^2. Therefore, the design will proceed using the original EPS50 blocks.

Step 10.viii: If EPS blocks with a suitable elastic limit stress are not available, attempt to reduce total stresses at top of EPS blocks.

EPS blocks with a suitable elastic limit stress are available.

Step 10.ix: Estimate traffic stresses at various depths within the EPS blocks.

The remaining sub-steps in Step 10 are intended to evaluate the possibility of using EPS blocks with lower densities in the lower portions of the EPS fill to try to reduce material costs. However, since the EPS-block geofoam fill is only 14 ft in height, it is unlikely that any significant cost savings would be achieved. Ensuring that the EPS blocks were placed correctly according to their densities would require more time and effort during the construction process than would be justified by the potential material cost savings. However, if for some reason it became necessary to use EPS blocks with different densities in

the fill, the stress dissipation of the pavement system could be estimated using the 2:1 method to determine where the lower density EPS blocks could be placed.

The load bearing analysis above did not consider the additional inertia forces from any earthquake. These inertia forces may increase the total stresses within the EPS blocks. However, if desired, a separate load bearing analysis can be performed that includes the earthquake inertia forces. Such an analysis may indicate that EPS blocks with a higher elastic limit stress than the one selected in Step 10.vii maybe required.

STEP 11: SETTLEMENT (EXTERNAL)

The foundation soil upon which the EPs-block geofoam fill will be constructed consists of a brown silty clay. Since the EPS fill is being constructed to repair an existing roadway, the vertical effective stress applied to the foundation soil after the EPS fill is complete will be less than the vertical effective stress on the foundation from the original roadway fill. In other words, the construction of the EPS-block geofoam slope fill will result in a net decrease in the vertical stress applied to the foundation soil. This means that the foundation soil will be overconsolidated and will probably experience very little settlement. However, settlement should still be checked to ensure that the requirements of the supervising agency are satisfied.

Calculate the total settlement of the roadway using Equation 4.40.

$$S_{total} = S_{if} + S_i + S_p + S_s + S_{cf}$$

S_{if} = immediate (elastic) settlement of the EPS fill mass.

The total vertical stress applied at the top of the EPS fill is

$$\sigma_{Total} = 842.2\,lb/\ ft^2 = 5.85\,lb/in^2$$

From Table F.2 of the report, the initial secant Young's modulus, E_i, of the EPS50 blocks is $725\,lb/in^2$.

Therefore, the elastic strain, ε, of the EPS blocks can be calculated by

$$\varepsilon = \frac{\sigma_{Total}}{E_i} = \frac{5.85\,lb/in^2}{725\,lb/in^2} = 0.0081$$

Note that this calculation of strain conservatively assumes that the full magnitude of σ_{Total} will be applied through the entire height of the EPS fill. In reality, σ_{Total} would dissipate with depth.

From the strain calculated above, the total elastic deformation of the EPS fill can be calculated by

$$\Delta L = \varepsilon L$$

Where

ΔL = the change in height of the EPS fill

L = the original height of the EPS fill

Therefore, the immediate (elastic) deformation of the EPS fill mass can be calculated by

$$S_{if} = \Delta L = \varepsilon L = (0.0081)(14\ ft) = 0.1134\ ft = 1.36\,in$$

S_i = immediate (elastic) settlement of the foundation soil

A conventional elastic settlement analysis is performed on the foundation soil. Elastic settlement of clayey soils is usually relatively small compared to consolidation (EOP) settlement. The elastic settlement analysis indicates that approximately 0.1 in. of elastic settlement can be expected from the foundation soil.

S_p = end-of-primary (EOP) consolidation settlement of the foundation soil

A conventional settlement analysis of the silty clay foundation soil indicates that the EOP consolidation settlement of the foundation soil will be approximately 0.5 in. It makes sense that this value should be very small because the EPS-block geofoam fill will actually decrease the vertical stress applied to the foundation soil.

S_s = secondary consolidation of the foundation soil

As noted above, the EPS-block geofoam fill actually decreases the vertical stress applied to the foundation soil. Because of this, secondary consolidation of the foundation soil will be very small and may be safely neglected.

S_{cf} = long-term vertical deformation (creep) of the EPS fill mass

Based on the current state of knowledge regarding the creep behavior of EPS-block geofoam, EPS blocks loaded at less than their elastic limit stress can be expected to undergo strains of 1% at most over the design life of the structure.

Therefore, the S_{cf} can be conservatively estimated as

$$S_{cf} = (0.01)(14\ ft) = 0.14\ ft = 1.7\ in$$

The total settlement of the roadway, S_{total}, can be calculated as shown on the next page.

$$S_{total} = S_{if} + S_i + S_p + S_s + S_{cf}$$
$$= 1.36\,in + 0.1\,in + 0.5\,in + 0 + 1.7\,in = 3.66\,in. < 6\,in$$

Therefore, the predicted settlement of the roadway is within the acceptable limits set by the supervising agency.

STEP 12: BEARING CAPACITY (EXTERNAL)

From Equation 4.45, the ultimate bearing capacity, q_{ult}, of the foundation material beneath the EPS-block geofoam fill is

$$q_{ult} = c' N_c s_c d_c i_c b_{ci} g_c + \sigma'_{vD} N_q s_q d_q i_q b_q g_q + 0.5\gamma'_{soil} B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

Since there are eccentric loads in this problem, eccentricity, *e*, should be checked.

Determine moment over center of the base of foundation, Point C, as shown in Figure E.20.

M= W_{EPS}*(12.55-10.07) + $W_{Pavement}$*(12-10.07)= 19199 lb.ft/ft-wall*

Determine the total applied vertical loads.

P= applied normal load = W_{EPS} + $W_{Pavement}$ = *437.5+9360= 9797.5lb/ft*

$$e = \frac{M}{P} = \frac{19199}{9797.5} = 1.96 < \frac{B}{6} = \frac{20.13}{6} = 3.36 \Rightarrow O.K$$

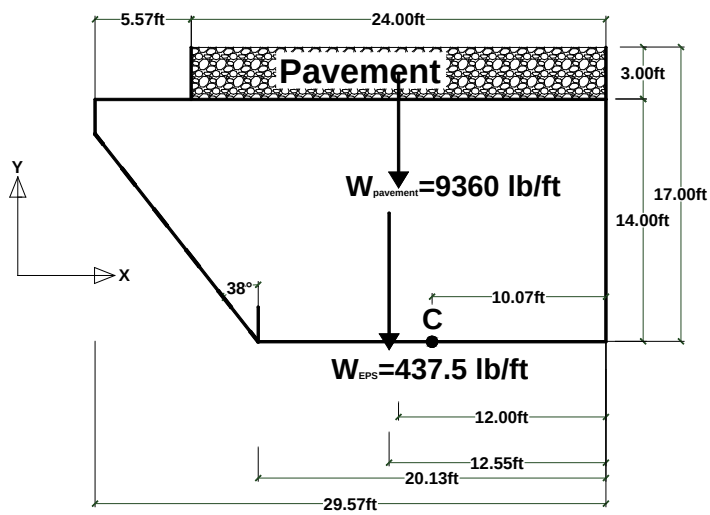


Figure E.21 Free body diagram for bearing capacity analysis.

The effect of the eccentric load is to distribute the load over a smaller area than the entire footing area. The smaller area can be taken into account by reducing the footing dimensions. These reduced footing dimensions are called effective width and effective length. Determine the effective width, *B'*.

$B' = B - 2e_B = 20.13 - 2*1.96 = 16.21\,ft$

Equation 4.45 becomes

$q_{ult} = c' N_c s_c d_c i_c b_{ci} g_c + \sigma'_{vD} N_q s_q d_q i_q b_q g_q + 0.5\gamma'_{soil} B' N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$

$c' = 0.$

To calculate γ'_{soil}, it is first necessary to determine if groundwater Case 1, 2 or Case 3 is applicable as summarized in Chapter 4. From Figure E.5 and E.7, B=20.13, D = 17 ft, the average D_w = 17 ft.

Therefore, Case 1 is applicable because $D_w \le D$ and Equation 4.46 is applicable.

From Equation 4.46,

$$\gamma'_{soil} = \gamma_{soil} - \gamma_w$$
$$= 110\ lb/\ ft^3 - 62.4\,lb/\ ft^3$$
$$= 47.6\ lb/\ ft^3$$

$\sigma'_{vD} = \gamma_{soil}(D)$

Therefore,

$\sigma'_{vD} = 110\,lb/\ ft^3 \times 17\,ft = 1870\,lb/\ ft^2$

N_q = 10.7 for ϕ' = 25°. Recall that ϕ' = 25° is the shear strength of the soil slope.

N_γ = 10.9 for ϕ' = 25°

Note: Values of N_c, N_q, and N_γ can be obtained from any foundation design textbook.

$s_q, i_q, b_q, s_\gamma, i_\gamma, b_\gamma, d_\gamma$ are all equal 1 for this example problem. The shape factors, s_c, s_q, s_γ should be based on the effective footing dimensions.

The depth factors are based on the actual footing dimensions.

$$d_q = 1 + 2k\tan\varphi'(1-\sin\varphi')^2$$

since $\frac{D}{B} = 0.84 \le 1$, then $k = \frac{D}{B} = 0.84$

$$d_q = 1 + 2*0.84*\tan(25)*(1-\sin(25))^2$$

$$=1.26$$

$g_q = g_\gamma = (1-\tan\beta)^2$. For the slope shown in Figure E.7, the value of β, the slope inclination, is approximately 28°. Therefore, the values of both g_q and g_γ are approximately 0.22.

$$q_{ult} = c'N_c s_c d_c i_c b_{ci} g_c + \sigma'_{vD} N_q s_q d_q i_q b_q g_q + 0.5\gamma'_{soil} B' N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

$$= 0 + (1870 lb/ft^2)(10.7)(1.26)(0.22) + 0.5(47.6 lb/ft^3)(16.21\,ft)(10.9)(0.22)$$

$$= 6471.6 lb/ft^2$$

$$q_a = \frac{q_{ult}}{FS} = \frac{6471.6}{3} = 2157.2 lb/ft^2$$

Next, determine the bearing pressure acting on the foundation soil. Per FHWA GEC No. 6, the applied pressure due to the eccentric loading is assumed to be an equivalent uniform pressure over the effective area. Therefore,

$$q = \frac{P}{B'*L} = \frac{9797.5}{16.21*1} = 604.41 lb/ft^2$$

$$q = 604.41 lb/ft^2 < q_a = 2157.2 lb/ft^2$$

Therefore, factor of safety against bearing capacity failure is adequate. For this example, additional stresses due to seismic loads are not considered because they are not significant.

STEP 13: FINAL DESIGN DETAILS

Additional, items that may need to be considered are summarized below. The list below is not comprehensive and the final design details that will need to be considered will vary based on project and site requirements.

- Design subsurface drainage system to ensure that the groundwater level cannot rise above the level of the bottom of the EPS blocks.
- Design roadway hardware for the highway according to specifications of supervising agency.
- The EPS-block geofoam fill developed in Steps 3 and 4 is completely buried within the slope. Therefore, no protective facing system is necessary. However, provisions should be made for re-seeding the reconstructed soil slope to prevent erosion according to the specifications of the supervising agency.

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Appendix F

Recommended Standard and Commentary
(AASHTO FORMAT)

F.1. SCOPE

F.1.1 This is a combined material, product and construction standard covering block-molded expanded polystyrene (EPS block) used as a geofoam geosynthetic product (EPS-block geofoam) in applications involving road embankments and other fills/backfills constructed on slopes as well as for other slope-stabilization designs. This is a material-purchasing standard and project-specific review of its use is required.

F.1.2 This is not a design standard, but includes technical information used in design. This standard is intended to be used in conjunction with the document titled "Recommended Design Guideline" found in Appendix B of the report for the National Cooperative Highway Research Program (NCHRP) Project 24-11(02) titled "Guidelines for Geofoam Applications in Slope Stability Projects." All information concerning this report should be obtained from the Transportation Research Board (TRB).

F.2. REFERENCED DOCUMENTS

F.2.1 AASHTO Standards: none

F.2.2 ASTM Standards:[1]

C165-05 - Standard Test Method for Measuring Compressive Properties of Thermal Insulations

C203-05a - Standard Test Methods for Breaking Load and Flexural Properties of Block-Type Thermal Insulation

C303-02 - Standard Test Method for Dimensions and Density of Preformed Block and Board-Type Thermal Insulation

C578-06 - Standard Specification for Rigid, Cellular Polystyrene Thermal Insulation

D1621-04a - Standard Test Method for Compressive Properties Of Rigid Cellular Plastics

D2863-06a - Standard Test Method for Measuring the Minimum Oxygen Concentration to Support Candle-Like Combustion of Plastics (Oxygen Index)

F.2.3 NCHRP reports:[2]

NCHRP Project No. 24-11(01) - Guidelines for Geofoam Applications in Embankment Projects

NCHRP Project No. 24-11(02) - Guidelines for Geofoam Applications in Slope Stability Projects

F.3. TERMINOLOGY

For the purposes of this standard, the following terminology is used herein:

- **Project**: The proposed site- and application-specific construction work involving use of EPS-block geofoam to which this document is being applied.
- **Owner**: The government agency having contractual authority over the Project at the time of its execution. This may or may not be the government agency having final ownership of the work performed for the Project and/or legal jurisdiction over the operation and maintenance of the work resulting from the Project. For example, a state department of transportation (DOT) may oversee construction of the Project on behalf of a local

[1] Available from ASTM International; P.O. Box C700; West Conshohocken, PA 19428-2959, U.S.A.

[2] Available from the Transportation Research Board; 2101 Constitution Avenue, N.W.; Washington, DC 20418-0007, U.S.A.

(county, town) jurisdiction that will actually own and/or maintain the completed work. In this case, the DOT would be considered the Owner for the purposes of this document.

- **Designer**: The government agency or private-business entity having legal responsibility for the professional-engineering design of those portions of the Project that include the EPS-block geofoam.
- **Owner's Agent**: The government agency or private-business entity having direct responsibility for quality assurance (material, product and construction inspection and testing) on behalf of the Owner during construction of those portions of the Project that involve the EPS-block geofoam. This may or may not be the Designer.
- **Contractor**: The business entity having the direct contractual relationship with the Owner for the overall Project as well as the overall legal responsibility for acceptability of the overall construction performed for the Project, including, but not limited to, the acceptability of all EPS-block geofoam.
- **Molder**: A business entity actually manufacturing EPS blocks used as the EPS-block geofoam for the Project. There may be more than one Molder involved in a project. The Contractor may contract directly with each Molder; with only one Molder (hereinafter defined as the Primary Molder,) who then subcontracts with one or more additional Molders; or with a Supplier as defined below.
- **Supplier**: A business entity that is not a Molder that has a contractual relationship with the Contractor for the supply of the EPS-block geofoam to the Project. The Supplier is typically a distributor of construction and/or geosynthetic products manufactured by others.

F.4. PRODUCT MANUFACTURING QUALITY CONTROL REQUIREMENTS

F.4.1 Manufacturing quality control (MQC) of the EPS-block geofoam product is the ultimate responsibility of a Molder, although MQC may be coordinated through the Primary Molder or Supplier if one exists on a Project. The purpose of this section is to define the parameters for use in developing a MQC plan. These parameters will also be those measured as part of the manufacturing quality assurance (MQA) process to be conducted by the Owner's Agent. MQA requirements are detailed in sections F.5, F.6, and F.8 of this standard.

F.4.2 All EPS-block geofoam as delivered to the Project for installation shall satisfy the minimum product flammability requirements specified in ASTM C578 using the test methodology specified in ASTM D2863.

F.4.3 All EPS-block geofoam shall consist entirely of expanded polystyrene except as noted within this document. The default basic component of EPS-block geofoam is virgin raw material ('expandable polystyrene' a.k.a. 'bead' or 'resin') of a nature sufficient to produce EPS blocks that meet the minimum flammability requirements specified in Section F.4.2 of this standard. Complete material documentation for all expandable polystyrene to be used on the Project must be disclosed as part of the Phase I MQA pre-construction pre-certification process described in Section F.6 of this standard. This documentation must state the source (nation of origin) and specifications (including, but not limited to, bead size, flame retardancy, and relative content of pentane blowing agent) of all expandable polystyrene. This documentation must also indicate complete quality and safety compliance of the expandable polystyrene as would normally be required for its use in producing EPS for building construction in the U.S.A. Should any changes in the source and/or specifications of expandable polystyrene occur during the course of the Project, updated information must be supplied to and acknowledged by the Owner's Agent prior to the implementation of any change.

At the discretion of a Molder, the EPS-block geofoam may be manufactured using a mixture of materials that includes recycled EPS ('regrind') content. If regrind is to be used, this shall be clearly stated as part of the Phase I MQA pre-construction pre-certification process described in Section F.6 of this standard. The source of the regrind (e.g. block- versus shape-molded EPS, in-plant versus post-consumer)

shall also be stated clearly, and the Molder, Primary Molder or Supplier must demonstrate to the satisfaction of the Owner's Agent that the source of the regrind can be depended on to provide only clean expanded polystyrene for the duration of the project. Furthermore, it must be demonstrated to the satisfaction of the Owner's Agent that the use of regrind will not compromise the ability of the finished EPS blocks to meet the minimum flammability requirements specified in Section F.4.2 of this standard. Should any changes relative to regrind usage occur during the course of the Project, updated information must be supplied to and acknowledged by the Owner's Agent prior to the implementation of any change.

In addition, if a Molder plans to use any optional chemical additive in the finished EPS-block product that is not required for generic EPS-block manufacture, e.g. a chemical additive for insect control, the nature and safety issues associated with use of such additive(s) must be stated clearly by the Molder, Primary Molder or Supplier prior to molding any blocks for the Project as part of the Phase I MQA pre-construction pre-certification process described in Section F.6 of this standard. In addition, the Molder, Primary Molder or Supplier must demonstrate to the satisfaction of the Owner's Agent that the proposed additive(s) will not compromise the ability of the finished EPS blocks to meet the minimum flammability requirements specified in Section F.4.2 of this standard, and pose no environmental hazard in either the short- or long-term. Finally, the Molder, Primary Molder or Supplier shall provide written documentation that will indemnify and hold harmless the Owner against all environmental risks associated with the additive(s) that may exist at present or might develop in the future. Should any changes relative to optional chemical additive usage occur during the course of the Project, updated information must be supplied to and acknowledged by the Owner's Agent prior to implementing any change.

F.4.4 All EPS-block geofoam shall be manufactured using a vacuum-assisted mold. Written documentation and technical information concerning the mold to be used shall be submitted as part of the Phase I MQA pre-construction pre-certification process described in Section F.6 of this standard. Should any changes in mold use occur during the course of the project, updated information must be supplied to and acknowledged by the Owner's Agent prior to any change. Note that any change in molds may, at the discretion of the Owner's Agent, require a completely new pre-certification process as described in Section F.6 of this standard.

The Owner's Agent shall be allowed to inspect the facilities to be used for producing EPS blocks for the Project upon reasonable advance request and during normal business days and hours. The Owner's Agent shall also be allowed to photograph and/or videograph these facilities during this inspection. This is solely for Project documentation and information, and any photographs and/or videographs will not be made available to anyone not involved in the Project without prior written consent of the owner of the facilities used for producing EPS blocks for the Project. The owner of the facilities used for producing EPS blocks for the Project will be entitled to receive one copy of all photographs and/or videographs at no cost within a reasonable period of time after the inspection upon written request to the Owner's Agent.

Any anticipated use of oil or any other type of additive intended to assist the molding process shall be disclosed in writing as part of the Phase I MQA pre-construction pre-certification process described in Section F.6 of this standard. The type and percentage of oil and/or additive must also be noted in this written disclosure. In addition, the Molder, Primary Molder or Supplier shall demonstrate to the satisfaction of the Owner's Agent by using appropriate ASTM standards referenced in Section F.2 of this standard, and tests performed by a certified, independent testing laboratory that the minimum flammability requirements specified in Section F.4.2 of this standard are not compromised by the presence of the oil and/or other additives. Should the Molder, Primary Molder or Supplier desire to make any changes regarding the use of oil or other additive during the course of the project, updated information concerning flammability shall be supplied to and approved by the Owner's Agent prior to making any changes in the molding process. Note that any change in oil/additive usage may, at the discretion of the Owner's Agent, require a completely new pre-certification process described in Section F.6 of this standard.

F.4.5 All EPS-block geofoam shall be adequately aged (seasoned) prior to shipment to the Project site. For the purposes of this standard, aging is defined as storage of molded EPS blocks within a facility suitable for the intended purpose as subsequently defined herein for a minimum of 72 hours at

normal ambient indoor temperature after an EPS block is released from the mold. Aging shall be done within a facility that protects the EPS blocks from being exposed to moisture as well as UV radiation. The facility in which EPS blocks are stored for aging shall also be such that adequate space is allowed between blocks and positive air circulation and venting of the facility provided so as to foster the outgassing of residual blowing agent and trapped condensate from within the blocks, and to allow blocks to stabilize chemically and thermally.

The Owner's Agent shall be allowed to inspect the facilities to be used for aging EPS blocks upon reasonable advance request and during normal business days and hours. The Owner's Agent shall also be allowed to photograph and/or videograph these facilities during this inspection. This is solely for Project documentation and information, and any photographs and/or videographs will not be made available to anyone not involved in the Project without prior written consent of the owner of the facilities used for aging purposes. The owner of the facilities used for aging purposes will be entitled to receive one copy of all photographs and/or videographs at no cost within a reasonable period of time after the inspection upon written request to the Owner's Agent.

The Molder, Primary Molder or Supplier may request an aging period of less than 72 hours if the EPS blocks are aged within an appropriate heated storage space and the Molder, Primary Molder or Supplier demonstrates to the satisfaction of the Owner's Agent that the alternative aging treatment produces blocks that equal or exceed the safety and quality of blocks subjected to the normal 72-hour-minimum aging period.

Should the Molder, Primary Molder or Supplier desire to make any changes during the course of the project regarding the facilities and/or protocol to be used for aging EPS blocks, detailed information concerning those changes shall be supplied to and approved by the Owner's Agent prior to making any such changes.

F.4.6 Table F.1 indicates the AASHTO material type designations used for the different densities/unit weights of EPS blocks that are covered by this standard. Only these material type designations shall be used in any correspondence or other communication related to the Project. For a given material type, the dry density/unit weight of each EPS block (as measured for the overall block as a whole) after the period of aging as defined in Section F.4.5 of this standard shall equal or exceed that shown in Table F.1. The dry density/unit weight shall be determined by measuring the mass/weight of a block using a scale and dividing the mass/weight by the volume of the block. This volume shall be determined by obtaining linear-dimensional measurements of the block in accordance with ASTM test method C303.

Table F.1. AASHTO Material Type Designations for EPS-Block Geofoam.

Material Designation	**Minimum Allowable Density (Unit Weight), kg/m^3 (lbf/ft^3)**	
AASHTO (provisional)	**Each Block as a Whole**	**Any Test MQC/MQA Specimen**
EPS40	16 (1.0)	16 (1.0)
EPS50	20 (1.25)	20 (1.25)
EPS70	24 (1.5)	24 (1.5)
EPS100	32 (2.0)	32 (2.0)
EPS130	40 (2.5)	40 (2.5)
EPS160	48 (3.0)	48 (3.0)

F.4.7 Table F.2 gives the minimum allowable values of various EPS material properties corresponding to each AASHTO material type shown in Table F.1. **It is imperative to note that there is no guarantee, expressed, implied or suggested, that the minimum required block density for a given grade of EPS will result in EPS that will meet the required minimum values of material properties as stated in Table F.2. For the purposes of this standard the minimum material-property values specified in Table F.2 are to be assumed to be independent of each other. A Molder, Primary Molder or Supplier must make their own independent assessment of block density required to meet or exceed all material-property values specific in Table F.2 for a given grade of EPS.**

These EPS material properties stated in Table F.2 are to be obtained by testing specimens prepared from samples taken from actual EPS blocks produced for the Project covered by this standard for either MQC by the Molder, Primary Molder or Supplier or MQA by the Owner's Agent as described in Section F.8 of this standard.

All EPS test specimens shall be aged and environmentally stabilized prior to all testing as specified in ASTM C165. Dry density/unit weight, compressive strength, and flexural strength shall be measured using the general protocols specified in ASTM C303, C165, and C203, respectively, and in accordance with specific test guidelines as specified in ASTM C578.

The specimens used for compressive testing shall be cubic in shape with a 50 millimeter (2 inch) face width. A strain rate of 10% per minute shall be used for the compressive strength tests. Both the elastic-limit stress and initial secant Young's modulus shall be determined in the same test used to measure compressive strength. The elastic-limit stress is defined herein as the measured compressive normal stress at a compressive normal strain of 1%. The initial secant Young's modulus is defined herein as the average slope of the compressive stress versus compressive strain curve between 0% and 1% compressive normal strain. Note that compression-test curves that exhibit an initial upward concavity due to seating or other testing problems must be corrected in accordance with the protocol specified in ASTM C165 to establish a new, fictitious origin (zero compressive stress-zero compressive strain point) for the purposes of calculating the elastic-limit stress and initial secant Young's modulus.

Table F.2. Minimum Allowable Values of MQC/MQA Parameters for Individual Test Specimens.

Material Designation	**Dry Density (Dry Unit Weight), kg/m^3 (lbs./ft^3)**	**Compressive Strength, kPa (lbs./in^2)**	**Flexural Strength, kPa (lbs./in^2)**	**Elastic-Limit Stress, kPa (lbs./in^2)**	**Initial Secant Young's Modulus, MN/m^2 (lbs./in^2)**
EPS40	16 (1.0)	85 (12.5)	185 (27)	40 (5.8)	4 (580)
EPS50	20 (1.25)	120 (17.5)	240 (35)	50 (7.2)	5 (725)
EPS70	24 (1.5)	155 (22.5)	300 (43)	70 (10.1)	7 (1015)
EPS100	32 (2.0)	230 (33.5)	380 (55)	100 (14.5)	10 (1450)
EPS130	40 (2.5)	275 (40)	415 (60)	130 (18.8)	13 (1885)
EPS160	48 (3.0)	345 (50)	520 (75)	160 (23.2)	16 (2320)

F.4.8 Each EPS block shall meet dimensional tolerances as determined in three distinct areas:

- Variation in linear dimensions as defined in Section F.4.9 of this standard.
- Squareness (deviation from perpendicularity of block faces) as defined in Section F.4.10 of this standard.
- Flatness (overall warp of block faces) as defined in Section F.4.11 of this standard.

F.4.9 The thickness, width, and length dimensions of an EPS block are defined herein as the minimum, intermediate, and maximum overall dimensions of the block, respectively, as measured along a block face. Each of these three dimensions of each block shall not deviate from the theoretical dimensions

shown on the plans or shop drawings for the Project by more than ±0.5%, but not to exceed 5 millimetres (0.25 inches).

F.4.10 The intersection of any two faces of an EPS block shall be perpendicular, i.e. form an angle of 90 degrees, unless indicated to be otherwise on the plans or shop drawings for the Project, in which case the angle formed must be as shown on said plans or shop drawings. The deviation from 90 degrees or the indicated angle, if different, shall not exceed 0.5%.

F.4.11 Any one face of a block shall not deviate from theoretical planarity of that face by more than 0.5%.

F.5. PRODUCT MANUFACTURING QUALITY ASSURANCE REQUIREMENTS: GENERAL

F.5.1 Manufacturing quality assurance (MQA) of EPS-block geofoam will be conducted to verify the MQC procedures of the Molder, Primary Molder or Supplier. The Owner's Agent will have primary responsibility for all MQA unless the Owner notifies the Contractor otherwise. The Owner's Agent shall communicate directly only with the Contractor in matters and questions of MQA unless all parties agree otherwise and specify in writing alternative lines of communication in the interest of efficiency, e.g., between the Owner's Agent and Molder, Primary Molder or Supplier directly.

F.5.2 MQA of EPS-block geofoam will consist of two phases. Phase I MQA consists of pre-certification of the Molder, Primary Molder or Supplier and shall be conducted prior to shipment of any EPS blocks to the project site. Phase I MQA is covered in Section F.6 of this standard. Phase II MQA shall be conducted as the EPS blocks are delivered to the Project and is discussed in Section F.8 of this standard. Table F.3 provides a summary of the MQA procedures.

F.5.3 As part of the MQA process, the Owner's Agent shall be allowed to inspect the manufacturing facilities of the Molder(s) as described in Section F.4.4. of this standard. The Owner's Agent reserves the right to request more than one inspection during the course of the Project should conditions warrant. These inspections may be combined with or in addition to the inspection of aging facilities as described in Section F.4.5 of this standard.

F.6. PRODUCT MANUFACTURING QUALITY ASSURANCE REQUIREMENTS: PHASE I

F.6.1 No EPS blocks shall be shipped to the Project site until such time as all parts of Phase I MQA as specified in this section of the standard have been completed in the order listed.

F.6.2 The Contractor shall first indicate in writing to the Owner's Agent whether the Molder has a third-party certification program in force. When there are multiple molders, each must have such a program in order for third-party certification to be indicated, and the Primary Molder must take responsibility for coordinating the third-party certification of all molders. Alternatively, if there is a Supplier on the Project, third-party certification of one or multiple molders may be coordinated through the Supplier.

F.6.3 If third-party certification is offered, this notification shall be accompanied by documentation that indicates the organization providing the third-party certification and describes in detail the steps to be taken by this organization to verify the Molder's compliance with the specific requirements of this specification. Acceptance of the Molder's third-party certification by the Owner's Agent will waive the need for pre-construction product submittal and testing as specified in Section F.6.4 of this standard. When there are multiple molders, third-party certification must be acceptable for each and every molder, otherwise it will be denied for each and every molder.

F.6.4 If the Molder does not have third-party certification or if the certification is deemed unacceptable by the Owner's Agent, the Contractor shall deliver a minimum of three full-size EPS blocks for each AASHTO EPS-block geofoam type to be used on the Project to a location specified by the Owner's Agent. When there are multiple molders, there shall be three blocks from each Molder. These blocks shall in all respects be the same as blocks to be supplied to the Project, including required aging as described in Section F.4.5. The Owner's Agent will weigh, measure, sample, and test a random number of

blocks to evaluate the ability of the molder(s) to produce EPS-block geofoam of quality as specified herein. The sampling and testing protocol will be the same as for Phase II MQA as discussed in Section C.8 of this standard.

F.6.5 Independent of whether or not there is an acceptable third-party certification for the Project, the Contractor shall submit written certification from the Molder, Primary Molder or Supplier indicating that all EPS blocks supplied to the Project will meet the requirements specified in this standard.

F.6.6 The Contractor shall submit a comprehensive, detailed plan for shipping, handling, and storing EPS blocks to the Owner's Agent for review and comment. This plan shall take into account considerations outlined in sections F.7 and F.10 of this standard and include specific details concerning the type of transport vehicle(s) to be used (flatbed, closed-body, etc.) to transport EPS blocks from the Molder, or each molder if more than one, to the Project site, and how the EPS blocks will be handled and stored on site. This plan shall also include details concerning protective measures to be used during shipping and handling to avoid damage to the blocks, especially punctures and crushing of edges, sides, and corners which are particular areas of concern. Timber cribbing with straps, tarps attached to the trucks, or other effective means may be proposed to secure the blocks to a flat-bed vehicle. Alternatively, closed-body vehicles may be used. Should the Contractor desire to make any changes to this shipping, handling, and storage plan during the course of the Project, updated information must be supplied to and acknowledged by the Owner's Agent prior to making any changes. In any event, the Contractor is ultimately responsible for proper shipping, handling and storage of the EPS blocks to prevent damage to them.

F.6.7 If required by the contract documents, the Contractor shall submit shop drawings indicating the proposed location and layout of all EPS blocks to be placed during the Project. When there are multiple molders, the areas to be covered by each molder shall be clearly identified. These drawings shall be reviewed by the Owner's Agent. The block layout shall be designed so that the following general design details are taken into account:

- The plane on which a given layer of blocks is placed must be parallel to the road surface in a direction parallel to the longitudinal axis of the road alignment.
- There must be a minimum of two layers of blocks at all locations, except where a layer of blocks terminates horizontally there may be a portion of the final block in that layer that has no EPS block above or below it.
- Within a given layer of blocks, the longitudinal axes of all blocks must be oriented so as to be parallel to each other.
- Within a given layer of blocks, the vertical joints between the adjacent ends of blocks within a given row of blocks ("row" is defined herein as a series of blocks placed end to end) must be offset horizontally to the greatest extent practicable relative to blocks in adjacent rows.
- The longitudinal axes of blocks for layers above and/or below a given layer must be oriented perpendicular to the longitudinal axes of blocks within that given layer.
- The longitudinal axes of the uppermost layer of blocks must be oriented perpendicular to the longitudinal axis of the road alignment.

F.6.8 Prior to delivery of any EPS-block geofoam to the Project site, a meeting shall be held between, as a minimum, the Owner's Agent and Contractor. The Supplier and/or Molder/Primary Molder of the EPS-block geofoam may also attend at the Contractor's discretion to facilitate answering any questions first-hand. The purpose of this meeting shall be to review the Phase I MQA items and results, and discuss the Phase II MQA, as well as other aspects of construction to ensure that all parties are familiar with the requirements of this standard. The timing and sequencing of delivering EPS blocks to the Project site shall also be discussed at this meeting. It is imperative that the Contractor's scheduling needs for EPS blocks at the Project site be consistent with the ability of the Molder/Primary Molder to supply EPS blocks that meet all requirements of this standard, especially but not limited to proper aging.

At the satisfactory conclusion of this meeting, the Contractor shall be allowed to begin on-site receipt, storage (if desired) and placement of the EPS-block geofoam.

F.7. PRODUCT SHIPMENT

F.7.1 Product shipment is the direct responsibility of the Molder, Primary Molder or Supplier. The Contractor has indirect responsibility for product shipment by virtue of their contractual relationship with the Owner. For the purposes of this standard, it will be assumed that direct custody of the EPS block is transferred from the Molder, Primary Molder or Supplier to the Contractor at such time the Contractor unloads the delivery vehicles of the Molder, Primary Molder or Supplier at the Project site.

F.7.2 Each EPS block shall be labeled to indicate the name of the Molder (if there is more than one supplying the Project), the date the block was molded, the mass/weight of the entire block (in kilograms or pounds) as measured after a satisfactory period of aging as specified in Section F.4.5, the dimensions of the block in millimeters or inches, and the actual dry density/unit weight in kilograms per cubic meter or pounds per cubic feet. Additional markings using alphanumeric characters, colors, and/or symbols shall be applied as necessary by the Molder, Primary Molder or Supplier to indicate the location of placement of each block relative to the design drawings or shop drawing indicated in Section F.6.7 of this standard, as well as the density/unit weight of the block if multiple block densities/unit weights are to be supplied for the Project. If multiple block densities/unit weights are to be supplied, the use of no marking shall be considered an acceptable marking for one of the densities/unit weights as long as it is used for the lower (lowest) density/unit weight EPS blocks supplied to the Project. If there is more than one molder supplying the Project all molders must use the same marking system for consistency.

F.7.3 At all stages of manufacturing and shipment the EPS blocks shall be handled in a manner so as to minimize physical damage to the blocks. No method of lifting or transporting the blocks that creates dents or holes in the block surfaces or losses of portions of the block shall be allowed under any conditions. The approved shipping, handling, and storage plan described in Section F.6.6 of this standard shall be followed throughout the entire Project without exception.

F.7.4 Properly aged EPS blocks are not, in general, an inherently dangerous or toxic material, so there are no particular safety issues to be observed in shipping EPS blocks other than normal safety protocols and protection against heat and flame, with the additional warning that food and beverage products for human consumption must not be placed directly on an EPS surface due to the fire-retardant chemical incorporated into the expandable polystyrene used to make the EPS. However, special, additional safety issues may exist depending on optional, proprietary additive(s) used by a Molder as defined in Section F.4.3.

Extra caution shall be exercised around exposed EPS blocks during wet or cold weather. Surfaces of EPS blocks tend to be more slippery wet than dry. In addition, when air temperatures approach or go below freezing, a thin layer of ice that can be difficult to see can readily develop on the exposed surfaces of EPS blocks if the dew point is sufficiently high. Thus, the surfaces of EPS blocks can pose particular slip hazards in this condition.

F.8. PRODUCT MANUFACTURING QUALITY ASSURANCE REQUIREMENTS: PHASE II

F.8.1 Phase II MQA will be performed by the Owner's Agent as EPS blocks are delivered to the Project site. Phase II MQA will consist of four subphases, IIa through IId inclusive. The Contractor shall cooperate with and assist the Owner's Agent in implementing Phase II MQA.

F.8.2 Phase IIa MQA will consist of on-site visual inspection by the Owner's Agent of each block delivered to the Project site to check for damage, as well as visually verify the labeled information as described in Section F.7.1 of this standard on each block. Any blocks with damage or not meeting requirements of this standard will be rejected on the spot, marked "unacceptable," be placed in an area separate from those blocks that are accepted, and eventually returned to the Molder, Primary Molder or Supplier.

F.8.3 Phase IIb MQA will consist of on-site verification that the minimum block dry density/unit weight as specified for the Project, as well as the physical tolerances specified in sections F.4.8 through F.4.11, inclusive, of this standard have been satisfied. At least one truckload of EPS blocks will be subjected to Phase IIb MQA, with additional blocks checked if initial measurements indicate lack of compliance. A truckload of EPS blocks is defined herein as either a full-length box- or flat-bed trailer of typical dimensions, i.e., approximately 12 meters (40 feet) or more in length, fully loaded with EPS blocks. The Contractor shall supply a scale for use of the Owner's Agent at the Project site with sufficient capacity and precision for weighing EPS blocks. This scale shall be recently calibrated and certification of such calibration made available to the Owner's Agent. The Contractor shall assist the Owner's Agent in moving and measuring EPS blocks for all aspects of Phase IIb MQA.

F.8.4 Phase IIc MQA will consist of sampling EPS blocks and laboratory testing of specimens prepared from these samples. As a minimum, sampling will be done at the locations shown in Figure F.1. Laboratory tests will check for compliance with the EPS material properties given in Table F.2. The Contractor shall cooperate with and assist the Owner's Agent with obtaining the necessary samples. Testing will be performed by or under the direction of the Owner's Agent. For each density/unit weight of EPS used on a Project, at least one block will be selected for sampling from the first truckload of EPS blocks of that density delivered to the Project site. Additional blocks may be selected for sampling during the course of the Project at the discretion of the Owner's Agent at a rate of sampling not to exceed one sample for every 250 cubic meters (325 cubic yards) of EPS delivered. Portions of sampled blocks that are otherwise acceptable can be used as desired by the Contractor. The Owner's Agent will make every reasonable effort to conduct the laboratory testing expeditiously. However, if unsatisfactory test results are obtained, the Contractor may be directed to remove potentially defective EPS blocks and replace them with blocks of acceptable quality at no additional expense to the Owner.

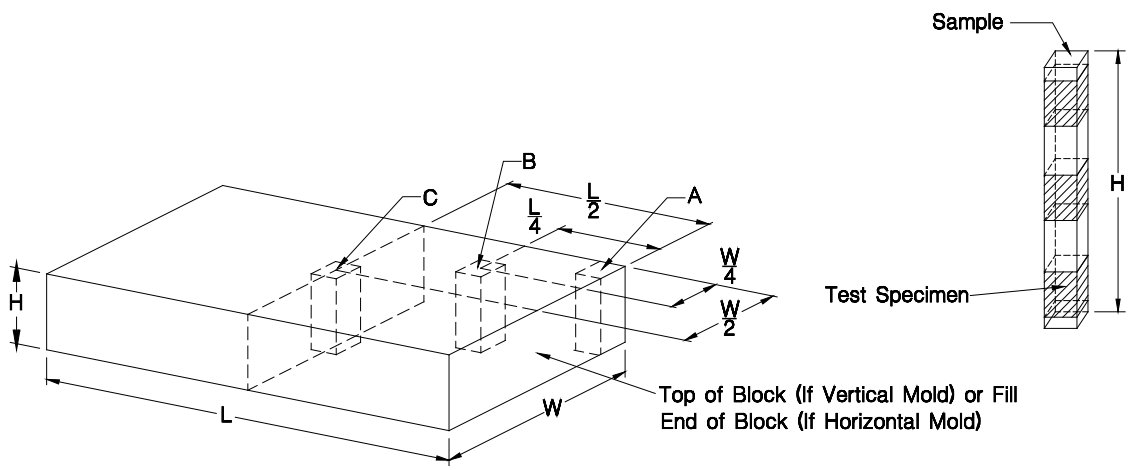


Figure F.1. Required minimum locations for EPS-block sampling and test specimens.

F.8.5 Phase IId MQA will consist of preparation of an as-built drawing or drawings, as well as additional record-keeping to document the location of all EPS blocks placed for the project. The Contractor shall cooperate with and assist the Owner's Agent with this work.

F.9. CONSTRUCTION QUALITY REQUIREMENTS

F.9.1 The Contractor shall be directly responsible for all construction quality control (CQC). Items covered by CQC include unloading and storing EPS blocks at the Project site and all earthwork and related activities necessary for placement of the EPS-block geofoam. Items of particular relevance to the placement of EPS-block geofoam are given in sections F.11 through F.13, inclusive, of this standard.

F.9.2 The Owner's Agent will be responsible for providing construction quality assurance (CQA) of the Contractor's construction activities.

F.10. PRODUCT HANDLING AND STORAGE

F.10.1 The Contractor is directly responsible for all handling and storage (if necessary) of EPS blocks at the Project site.

F.10.2 At all stages of construction the EPS blocks shall be handled in a manner so as to minimize physical damage to the blocks. No method of lifting or transporting the blocks that creates dents or holes in the block surfaces or losses of portions of the block shall be allowed under any conditions. In addition, liquid petroleum products such as gasoline, diesel fuel, or kerosene must not be allowed to come into contact with EPS at any time. The approved shipping, handling, and storage plan described in Section F.6.6 of this standard shall be followed throughout the entire Project without exception.

F.10.3 If the EPS blocks are to be stockpiled at the Project site until placement, a secure storage area shall be designated for this purpose. The storage area shall be away from any heat source or construction activity that produces heat or flame. In addition, personal tobacco smoking shall not be allowed in the storage area with appropriate signage posted to that effect. EPS blocks in temporary on-site storage shall be secured with sandbags and similar 'soft' weights to prevent their being dislodged by wind. The blocks shall not be covered in any manner that might allow the build-up of heat beneath the cover. The blocks shall not be trafficked by any vehicle or equipment. In addition, foot traffic by persons shall be kept to a minimum.

F.10.4 Properly aged EPS blocks are not in general inherently dangerous or toxic, so there are no particular safety issues to be observed other than normal construction safety and protection against heat and flame as specified in Section F.10.3 of this standard, with the additional warning that food and beverage products for human consumption must not be placed directly on an EPS surface due to the fire-retardant chemical incorporated into the expandable polystyrene used to make the EPS. However, special, additional safety issues may exist depending on optional, proprietary additive(s) used by a Molder as defined in Section F.4.3.

Extra caution shall be exercised around exposed EPS blocks during wet or cold weather. Surfaces of EPS blocks tend to be more slippery wet than dry. In addition, when air temperatures approach or go below freezing, a thin layer of ice that can be difficult to see can readily develop on the exposed surfaces of EPS blocks if the dew point is sufficiently high. Thus, the surfaces of EPS blocks can pose particular slip hazards in this condition.

F.11. SITE PREPARATION

F.11.1 If required by the contract documents, the natural soil subgrade shall be cleared of vegetation and any large or sharp-edged soil particles, and made reasonably planar (smooth) prior to placing a geotextile and/or sand bedding layer. If no sand bedding layer is used, the natural subgrade shall be cleared such that there is no vegetation or particles of soil or rock larger than coarse gravel in size exposed at the surface.

F.11.2 Regardless of the subgrade material (natural soils or sand bed), the subgrade surface on which the EPS blocks will be placed shall be sufficiently planar (smooth) prior to placement of the first block layer. The required smoothness is defined as a vertical deviation of no more than ±10 millimeters (0.4 inches) over any 3 meters (9.8 feet) distance.

F.11.3 There shall be no debris of any kind on the subgrade surface at the time EPS blocks are placed.

F.11.4 Unless directed otherwise by the Owner's Agent, there shall be no standing water or accumulated snow or ice on the subgrade within the area where EPS blocks are placed at the time of block placement.

F.11.5 EPS blocks shall not be placed on a frozen subgrade except in the case of construction over continuous or discontinuous permafrost terrain or as directed by the Owner's Agent.

F.12. PLACEMENT OF EPS-BLOCK GEOFOAM

F.12.1 EPS blocks shall be placed at the locations shown on either the contract drawings or approved shop drawings submitted by the Contractor. Particular care is required if EPS blocks of different density/unit weight are to be used on the Project to ensure that blocks of the appropriate density/unit weight are placed in the correct location.

F.12.2 EPS blocks shall be placed so that all vertical and horizontal joints between blocks are tight.

F.12.3 The surfaces of the EPS blocks shall not be directly traversed by any vehicle or construction equipment during or after placement of the blocks.

F.12.4 Blocks shall not be placed above blocks in which ice has developed on the surface.

F.12.5 With the exception of sand bags or similar 'soft' weights used to temporarily restrain EPS blocks against wind, no construction material other than that shown on the contract drawings shall be placed or stockpiled on the EPS blocks.

F.12.6 At no time shall heat or open flame be used in proximity to the EPS blocks so as to cause melting or combustion of the EPS.

F.12.7 At no time shall liquid petroleum products such as gasoline, diesel fuel, or kerosene be poured or spilled on EPS blocks so as to cause dissolution of the EPS.

F.12.8 The final surface of the EPS blocks shall be covered as indicated in the contract documents. Care shall be exercised during placement of the cover material so as not to cause any damage to EPS blocks.

F.13. PAVEMENT CONSTRUCTION

F.13.1 The pavement system is defined for the purposes of this standard as all material placed above the assemblage of EPS blocks within the limits of the roadway, including any shoulders.

F.13.2 The pavement system shall be constructed above the assemblage of EPS-block geofoam as indicated in the contract documents.

F.13.3 No vehicles or construction equipment shall traverse directly on the uppermost surface of the assemblage of EPS blocks or any separation layer (portland-cement concrete slab, geotextile, etc.) placed between the uppermost surface of the assemblage of EPS blocks and the pavement system. Soil or aggregate for the pavement system layers shall be pushed onto the uppermost surface of the assemblage of EPS blocks or the separation layer (if any) using appropriate equipment such as a bulldozer or front-end loader. A minimum of 300 millimeters (12 inches) of soil or aggregate shall cover the uppermost surface of the assemblage of EPS blocks or separation layer (if any) before compaction commences.

Table F.3. Manufacturing Quality Assurance (MQA) Procedure for EPS-Block Geofoam Used for the Function of Lightweight Fill in Road Embankments on Slopes and Slope Stabilization.

Phase	Sub-phase	Start of Phase	Description	Requirements	Possible Actions
I	-	Prior to shipment to Project site	Pre-certification of Molder	With third-party certification approved by Owner's Agent: • Molder, Primary Molder or Supplier will identify the organization providing this service. • Molder, Primary Molder or Supplier will provide detailed information as to the procedure and tests used by this organization to verify compliance with the specific requirements of this standard. • Molder, Primary Molder or Supplier shall provide written certification that all EPS blocks supplied to the Project will meet the requirements specified in this standard. • The Contractor shall submit a comprehensive, detailed plan for shipping, handling, and storing EPS blocks to the Owner's Agent for review and comment. • If required by the contract documents, the Contractor shall submit shop drawings indicating the proposed location and layout of all EPS blocks to be placed during the Project. • The Contractor (and, optionally, the Molder, Primary Molder or Supplier as well) shall meet with the Owner's Agent to review the Phase I MQA items and results, and discuss the Phase II MQA as well as other aspects of construction to ensure that all parties are familiar with the requirements of this standard.	• Acceptance of Molder's third-party certification by Owner's Agent will waive the need for pre-construction product submittal and testing. • No EPS blocks shall be shipped to the Project until such time as all parts of Phase I MQA have been completed.

				No approved third-party certification: • Contractor shall deliver a minimum of three full-size EPS blocks for each AASHTO EPS-block geofoam type to be used on the project to a location specified by the Owner's Agent. This is to be done for each Molder if there is more than one. • Owner's Agent will weigh, measure, sample, and test a random number of blocks. Sampling and testing protocol will be the same as for Phase IIc MQA. • Molder, Primary Molder or Supplier shall provide written certification that all EPS blocks supplied for the Project will meet the requirements specified in this standard. • The Contractor shall submit a comprehensive, detailed plan for shipping, handling, and storing EPS blocks to the Owner's Agent for review and comment. • If required by the contract documents, the Contractor shall submit shop drawings indicating the proposed location and layout of all EPS blocks to be placed during the Project. • The Contractor (and, optionally, the Molder, Primary Molder or Supplier as well) shall meet with the Owner's Agent to review the Phase I MQA items and results, and discuss the Phase II MQA as well as other aspects of construction to ensure that all parties are familiar with the requirements of this standard.	

II	a	As the EPS blocks are delivered to the Project site	On-site visual inspection of each block delivered to the Project site to check for damage as well as visually verify the labeled information on each block	Approved third-party certification: • Each truckload. Owner's Agent will inventory each block. No approved third-party certification: • Each truckload. Owner's agent will inventory each block.	• Any blocks with significant physical damage or not meeting specifications will be rejected on the spot, placed in an area separate from those blocks that are accepted, marked "unacceptable," and returned to the Molder, Primary Molder or Supplier.
II	b	As the EPS blocks are delivered to the project site	On-site verification that the minimum block dry density as well as the physical tolerances meet specifications	Approved third-party certification: • Each truckload. Initially, only one block per load. No approved third-party certification: • Each truckload. Each block for the first load then at least one block per load for subsequent truckloads.	• If the selected block meets specifications with respect to its size and shape, and the mass agrees with that marked on the block, no further checking of the load for these material properties is required and the shipment is approved conditionally until the Phase IIc test results verify that the blocks meet the requirement of this standard. • If the selected block does not meet specifications with respect to its size and/or shape then other blocks in the truckload should be checked and none used until the additional checking has determined which blocks are unsatisfactory. • At the completion of this subphase, the Contractor should be conditionally (until the Phase IIc test results verify that the blocks meet specifications) allowed to proceed with installing blocks.

II	c	As the EPS blocks are delivered to the project site	Confirming the EPS material properties related to stiffness as well as other quality-control strength parameters	Approved third-party certification: • Level of testing at discretion of Owner's Agent. For example, can be omitted entirely on a small Project, can perform testing only at the beginning of a Project, or can be done on an ongoing basis throughout a Project. In any event should not exceed what is done when there is no third-party certification. No approved third-party certification: • Performed on all projects throughout the entire duration of the Project • For each AASHTO EPS-block geofoam type at least one block will be selected for sampling from the first truckload. • Additional blocks may be selected at a rate of sampling not exceeding one sample for every 250 cubic meters (325 cubic yards). • Sampling to be performed per the locations indicated in Figure F.1. • Laboratory tests should be performed to check for compliance with the material properties shown in Table F.2 to include the elastic-limit stress, initial secant Young's modulus, compressive strength, and flexural strength.	• Portions of sampled blocks that are not damaged or otherwise compromised by the sampling can be used as desired by the Contractor. • If unsatisfactory test results are obtained, the Contractor may be directed to remove potentially defective EPS blocks and replace them with blocks of acceptable quality at no additional expense to the Owner.
II	d	As the EPS blocks are placed	As-built drawing(s)	• Owner's Agent with the cooperation of the Contractor will prepare as-built drawing(s) as well as perform additional record keeping to document the location of all EPS blocks placed for the project.	

Note: A truckload of EPS blocks is intended to mean either a full length box- or flat-bed trailer of typical dimensions, i.e., approximately 12 meters (40 feet) or more in length, fully loaded with EPS blocks. The volume of EPS in such a truckload would typically be of the order of 50 to 100 cubic meters (65 to 130 cubic yards).

COMMENTARY TO PROVISIONAL STANDARD

EPS-Block Geofoam Standard for Lightweight Fill in Road Embankments on Slopes and Slope Stabilization

Contents

F.1. SCOPE

The stated intended applications of this document are intentionally relatively limited, as they are dictated by the specific NCHRP research project for which this document was generated. Although there is no warranty or guarantee as to applicability, most users with some experience with EPS-block geofoam should be able to adapt the contents of this document to many small-strain, load-bearing geofoam functional applications that use block-molded EPS.

F.2. REFERENCED DOCUMENTS

ASTM reference standards for both material properties and laboratory testing protocols are crucial to successful manufacturing quality control (MQC) and manufacturing quality assurance (MQA) of EPS-block geofoam as defined in this standard. Experience indicates unequivocally that laboratory test results for block-molded EPS are very sensitive to several laboratory test parameters, as well as environmental factors such as temperature, humidity, and barometric pressure in the laboratory space. Consequently, following relevant ASTM test protocols for all MQC and MQA testing indicated in this document should be considered mandatory, and not optional or suggested.

F.3. TERMINOLOGY

Compared to other manufactured products used in engineered construction, EPS molders (manufacturers) typically play a much more active and direct role in projects that use EPS-block geofoam, including preparing and supplying shop drawings in many cases. Therefore, clearly understanding who all the participants are and what roles they serve in a typical project is very important.

F.4. PRODUCT MANUFACTURING QUALITY CONTROL REQUIREMENTS

There are several aspects of this section of the standard that should be understood and appreciated by all concerned so that the various requirements of this section of the standard are followed and interpreted properly in practice:

- There are numerous steps in the overall manufacturing process of EPS blocks that can significantly influence the small-strain stiffness of the final EPS-block product even if the final density/unit weight of the EPS is not affected. Small-strain stiffness, which is defined as the Young's modulus (slope) of the compressive stress-strain curve between 0% and 1% stain, is now universally recognized as being the single most important EPS material property for the types of EPS-block geofoam applications covered by this standard. On the other hand, other, traditional EPS material properties such as density/unit weight and compressive strength are in and of themselves largely irrelevant for the EPS-block geofoam applications covered by this standard, although these properties can be useful index properties in certain restricted situations, which is why they have been retained in this standard.

- The use of 'regrind' (recycled scrap EPS) is prominent relative to molding variables that affect the small-strain stiffness of EPS. In simple terms, regrind is never beneficial with regard to small-strain stiffness of EPS. All other things being equal, for given density/unit weight of EPS, the greater the relative regrind content, the smaller the small-strain Young's modulus of EPS.

- Related to the fact that variations in EPS molding practice can affect the geotechnically-relevant properties of EPS is increasing awareness that the nature and source of expandable polystyrene (a.k.a. bead or resin), the basic raw-material component of all EPS, can be important as well. Increasingly, expandable polystyrene is manufactured outside the U.S., and may not consistently have the quality compared to what has come to be taken for granted in the past from the relatively few major U.S. sources of expandable polystyrene who have decades of experience making expandable polystyrene. Therefore, because of these recent market shifts, it has now become desirable to pre-qualify sources of expandable polystyrene to ensure that they meet minimum product quality and safety standards that have long been met by domestic sources.

- The specified 72-hour-minimum aging requirement is not something that should be taken lightly, ignored, or modified without well-founded justification. Proper aging of EPS blocks is both a safety and technical necessity. Pentane blowing agent, which is combustible in proper gas-air mixtures, continues to outgas from EPS blocks for some time after a block is released from a mold. Failure to age a block under controlled conditions to allow this natural outgassing could result in a flame or explosion hazard. In addition, an EPS block is at an elevated temperature and somewhat softened after its release from a mold. It also contains condensed steam from the molding process. Failure to allow a block to age so that it can stabilize thermally and chemically, as well as dry-out, can result in a block that fails one or more aspects of MQA testing.

- Perhaps the single most important aspect of this section of the standard is that the specified minimum value of EPS material properties specified herein are interpreted correctly. In this regard it is vitally important that it is clearly understood that this standard is a **performance** standard, and not a prescriptive standard. As such, this standard indicates the minimum required values of various EPS material properties that must be met but not the specifics of how they are to be met. The specific molding processes as to how required EPS material properties are met are left entirely up to a Molder. Specifically, it cannot be stressed too strongly that the EPS material-property values shown in Table F.2 have <u>no</u> inherent relationship to each other. There is absolutely no guarantee, expressed or implied, by this standard that the various EPS material strengths and moduli can be obtained at the corresponding values of material density/unit weight given in this table. This is because there are simply far too many variables in the many steps of the EPS molding process that allow for a single correlation between final material density/unit weight and measured material properties.

F.5. PRODUCT MANUFACTURING QUALITY ASSURANCE REQUIREMENTS: GENERAL

MQA by a technically qualified agent who both represents the Owner's interests and is independent of all others involved in the Project is considered mandatory on all projects regardless of their size. Experience has shown conclusively that even when a Molder, Primary Molder or Supplier may offer third-party certification of MQC, such certification does not represent an appropriate substitute for MQA.

F.6. PRODUCT MANUFACTURING QUALITY ASSURANCE REQUIREMENTS: PHASE I

The overall purpose of Phase I MQA is to establish, before actual production and construction begins, the ability of the Molder/Primary Molder/Supplier to supply EPS-block geofoam of the quality and quantity required for the Project in a damage-free condition. This includes both a basic ability to produce

EPS blocks to the relatively stringent technical requirements of geofoam applications as well as to have an appropriate MQC plan in place. Note that there are slightly different requirements for a molder that has an established third-party certification protocol in place versus a molder than has no such plan. It is important to emphasize that experience has clearly demonstrated that having third-party certification is no guarantee of EPS-block geofoam quality. However, third-party certification does indicate that the molder offering such certification is at least cognizant of and reasonably familiar with formal MQC protocols. Consequently experience indicates that molders that do not have pre-existing third-party certification programs need to be vetted more rigorously than those that do.

Another very important aspect of Phase I MQA is to establish clearly that the ability of the Molder, Primary Molder, or Supplier to deliver EPS blocks that meet all Project requirements throughout the course of the Project is consistent with the anticipated rate of Contractor need for EPS blocks. Because EPS blocks can be physically placed at a Project site much faster than they can be produced, it has been an all-too-common occurrence, especially on larger projects, that demand outstrips supply. This has led to unacceptable situations where EPS blocks have been supplied to a Project site that have been rushed through production (especially with regard to minimum-required aging times), cutting corners for both quality and safety in the process. Such situations are wholly avoidable with proper Project planning and scheduling so that the Molder/Primary Molder/Supplier can produce EPS blocks days, weeks, or even months in advance to meet delivery requirements. It cannot be emphasized too strongly that the 72-hour-minimum aging time is not something that can be shortened or waived entirely without extensive testing to justify such action.

F.7. PRODUCT SHIPMENT

EPS-block geofoam is like most geosynthetics in that the greatest potential for permanent product damage occurs prior to or during placement in the ground. Therefore, EPS-block geofoam cannot be handled like other construction materials and simply loaded on trucks and dumped, stockpiled haphazardly, and moved roughly without considerable attention paid to how shipping, handling, and storage is performed. This section of the standard highlights the key considerations that must be given for shipping and associated handling EPS blocks up to the point that they are unloaded at the Project site, and the chain of direct custody passes from the Molder, Primary Molder or Supplier to the Contractor. The contents of this section of the standard are to be incorporated, as a minimum, into the shipping, handling and storage plan to be submitted by the Contractor as part of Phase I MQA.

F.8. PRODUCT MANUFACTURING QUALITY ASSURANCE REQUIREMENTS: PHASE II

Overall, Phase II MQA consists of ongoing verification by the Owner's Agent of molder MQC throughout the life of a Project. Within the basic framework of sub-phases defined in this section of the standard, experience has shown that there can be considerable latitude and professional judgment applied by the Owner's Agent during the course of executing Phase II MQA. Projects with multiple block densities/unit weights, multiple molders, and/or of extended duration with concomitant greater potential for product variability typically required more checking, sampling, and testing than simple projects of short duration. Although it is inherently impossible to quantify, the subjective level of confidence in a Molder or Supplier is also an important factor as well in how specifically an Owner's Agent executes Phase II MQA. Because block-molded EPS is a generic, commodity product, in most cases the Contractor will purchase the EPS blocks from the lowest-cost source, possibly one that has little or no prior experience supplying EPS blocks for geofoam applications and/or little or no prior experience with rigorous MQC protocols. In such cases the Owner's Agent needs to exercise professional judgment and apply a higher level of Phase II MQA, at least at the earlier stages of a project, to ensure compliance with the requirements of this standard.

F.9. CONSTRUCTION QUALITY REQUIREMENTS

The chain of custody for the EPS blocks is transferred from the Molder, Primary Molder, or Supplier to the Contractor at such time as the Contractor unloads the EPS blocks from delivery vehicles at the Project site. The Contractor is thus responsible for all activities related to handling, storage and placement of the EPS blocks at the Project site. Consequently, there is both quality control and quality assurance related to these construction activities.

F.10. PRODUCT STORAGE

EPS-block geofoam is like most geosynthetics in that the greatest potential for permanent product damage occurs prior to or during placement in the ground. Therefore EPS-block geofoam cannot be handled like other construction materials and simply loaded on trucks and dumped, stockpiled haphazardly, and moved roughly without considerable attention paid to how shipping, handling, and storage is performed. This section of the standard highlights the key considerations that must be given for handling and stockpiling (if desired) EPS blocks beginning with the point that they are unloaded at the Project site and the chain of direct custody passes from the Molder, Primary Molder or Supplier to the Contractor. The contents of this section of the standard are to be incorporated, as a minimum, into the shipping, handling and storage plan to be submitted by the Contractor as part of Phase I MQA.

F.11. SITE PREPARATION

Experience indicates that proper preparation of the ground surface on which the first layer of EPS blocks is placed is critically important to satisfactory placement of all subsequent layers. This is because EPS blocks are relatively long, stiff objects that do not accommodate uneven surfaces well and tend to rock when placed on an uneven surface. Consequently, unevenly placed blocks in the first layer will only tend to amplify the unevenness in all succeeding layers and produce unacceptable results. Therefore care taken to prepare the subgrade on which the first layer of EPS blocks is placed will produce significant benefits for the entire assemblage of EPS blocks.

F.12. PLACEMENT OF EPS-BLOCK GEOFOAM

As noted in the commentary for Sections F.7 and F.10 of this standard, EPS-block geofoam is like any other geosynthetic in that it is most susceptible to damage during the installation process. Therefore the Contractor needs to take care not to cause avoidable damage to EPS blocks during this vulnerable window during construction.

F.13. PAVEMENT CONSTRUCTION

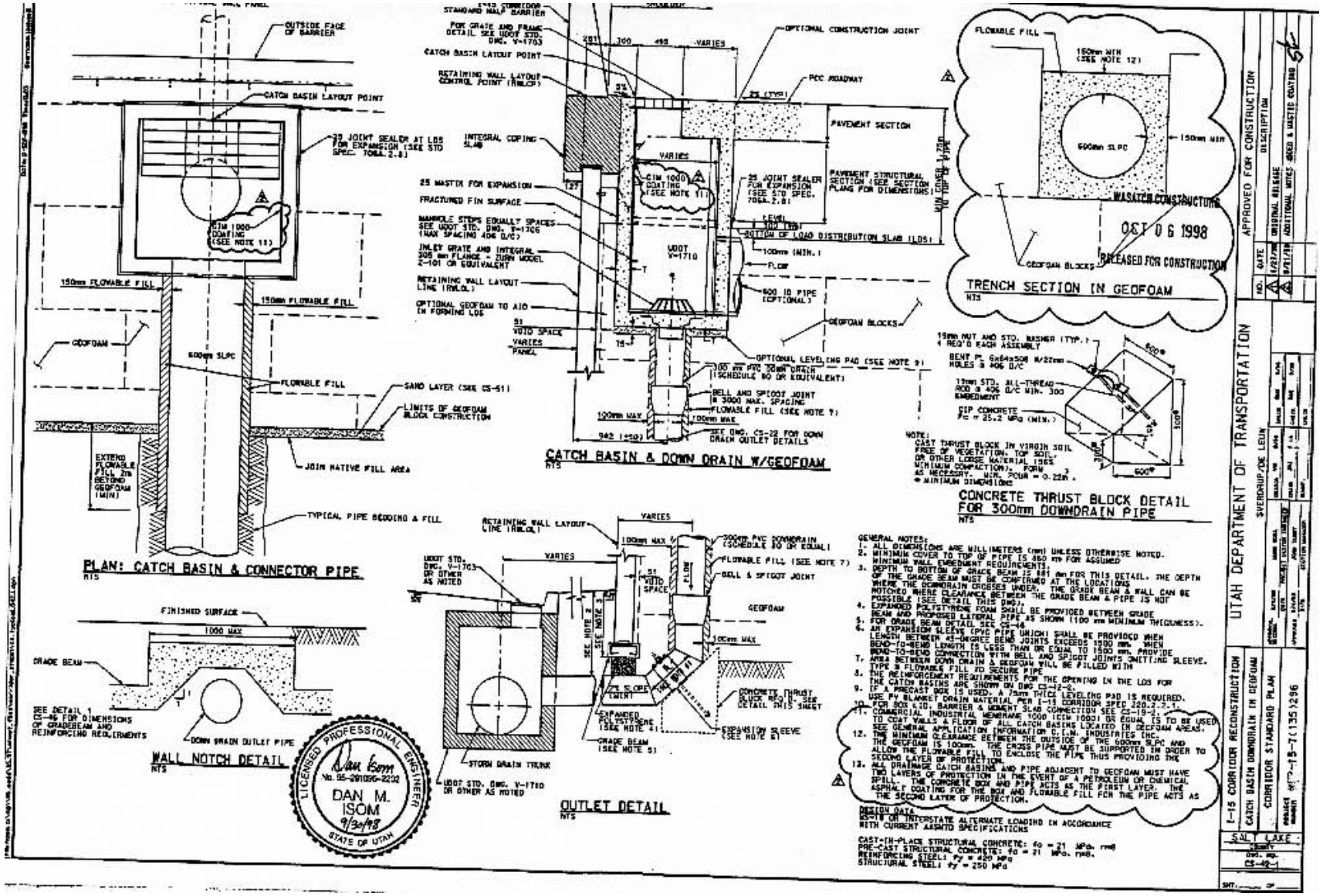
For the same reasons as discussed in the preceding section of this commentary, the Contractor needs to exercise care when placing the initial portion of the pavement system on top of EPS blocks or any portland-cement concrete slab, geotextile, etc. that was placed on the EPS blocks. This is because earthmoving and/or compaction equipment can damage EPS blocks unless proper precautions are observed.

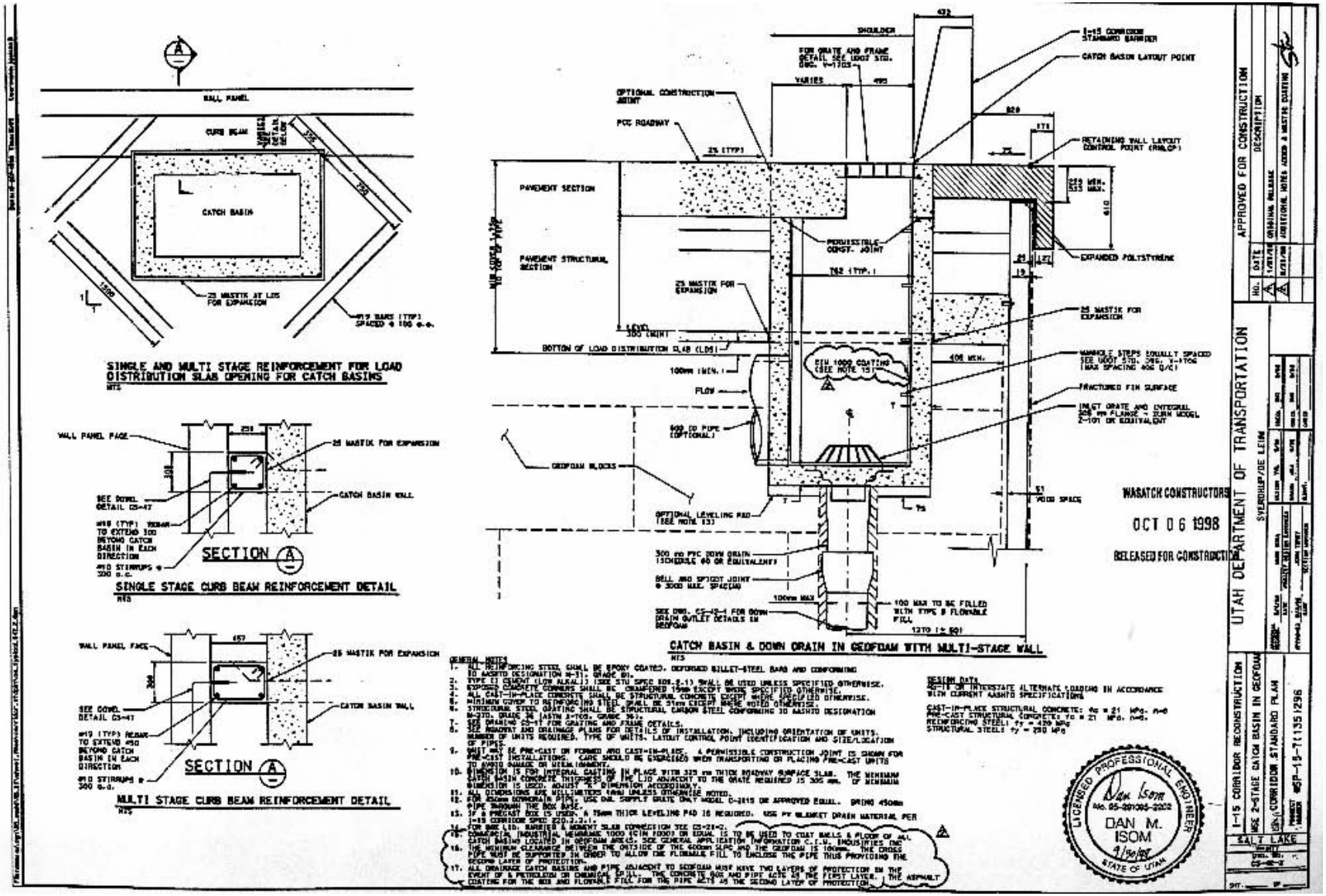
Appendix G

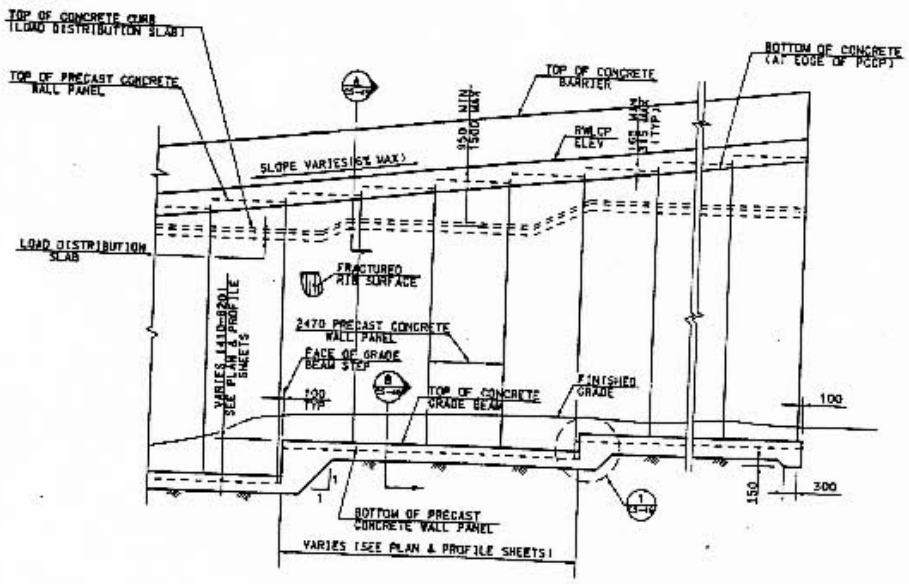
Example Plan Details

Project: I-15 Reconstruction Project Salt Lake City, UT

Source: Utah Department of Transportation


PLAN: CATCH BASIN & CONNECTOR PIPE
CATCH BASIN & DOWN DRAIN W/GEOFOAM
TRENCH SECTION IN GEOFOAM
CONCRETE THRUST BLOCK DETAIL FOR 300mm DOWNDRAIN PIPE
WALL NOTCH DETAIL
OUTLET DETAIL
GENERAL NOTES:
DESIGN DATA
UTAH DEPARTMENT OF TRANSPORTATION
I-15 CORRIDOR RECONSTRUCTION
CATCH BASIN DOWNDRAIN IN GEOFOAM
CORRIDOR STANDARD PLAN
SALT LAKE
LICENSED PROFESSIONAL ENGINEER
DAN M. ISOM
STATE OF UTAH
RELEASED FOR CONSTRUCTION
OCT 0 6 1998
APPROVED FOR CONSTRUCTION


SINGLE AND MULTI STAGE REINFORCEMENT FOR LOAD DISTRIBUTION SLAB OPENING FOR CATCH BASINS
SINGLE STAGE CURB BEAM REINFORCEMENT DETAIL
MULTI STAGE CURB BEAM REINFORCEMENT DETAIL
CATCH BASIN & DOWN DRAIN IN GEOFOAM WITH MULTI-STAGE WALL
WASATCH CONSTRUCTORS
OCT 06 1998
RELEASED FOR CONSTRUCTION
APPROVED FOR CONSTRUCTION
UTAH DEPARTMENT OF TRANSPORTATION
I-15 CORRIDOR RECONSTRUCTION
CORRIDOR STANDARD PLAN
SALT LAKE
DAN M. ISOM
STATE OF UTAH



ELEVATION-GEOFOAM (GPS) WALL

NTS

WASATCH CONSTRUCTORS

MAR 2 0 1998

RELEASED FOR CONSTRUCTION

NOTE:

1. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.

2. FOR DETAILS UNDER APPROACH SLABS AND BRIDGES SEE CS-49

2 DELETED NOT APPROVED FOR CONSTRUCTION NOTE

LICENSED PROFESSIONAL ENGINEER
No 96-324447-2202
JOHN M. TERRY
3/18/98
STATE OF UTAH

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	3/17/98	RELEASE FOR GEOFOAM WALL ONLY.
2	3/23/98	APPROVED FOR CONSTRUCTION AT BRIDGES

UTAH DEPARTMENT OF TRANSPORTATION

SVENDRUP/DE LEUW

DESIGN JMW 11/97 CHECK JMT 11/97
DRAWN MTR 11/97 CHECK JCD 11/97
REVIEW JOHN MILLER
PROJECT ENGR. KEITH BERGSTROM
JOHN JONES
WESTON BARNES

I-15 CORRIDOR RECONSTRUCTION

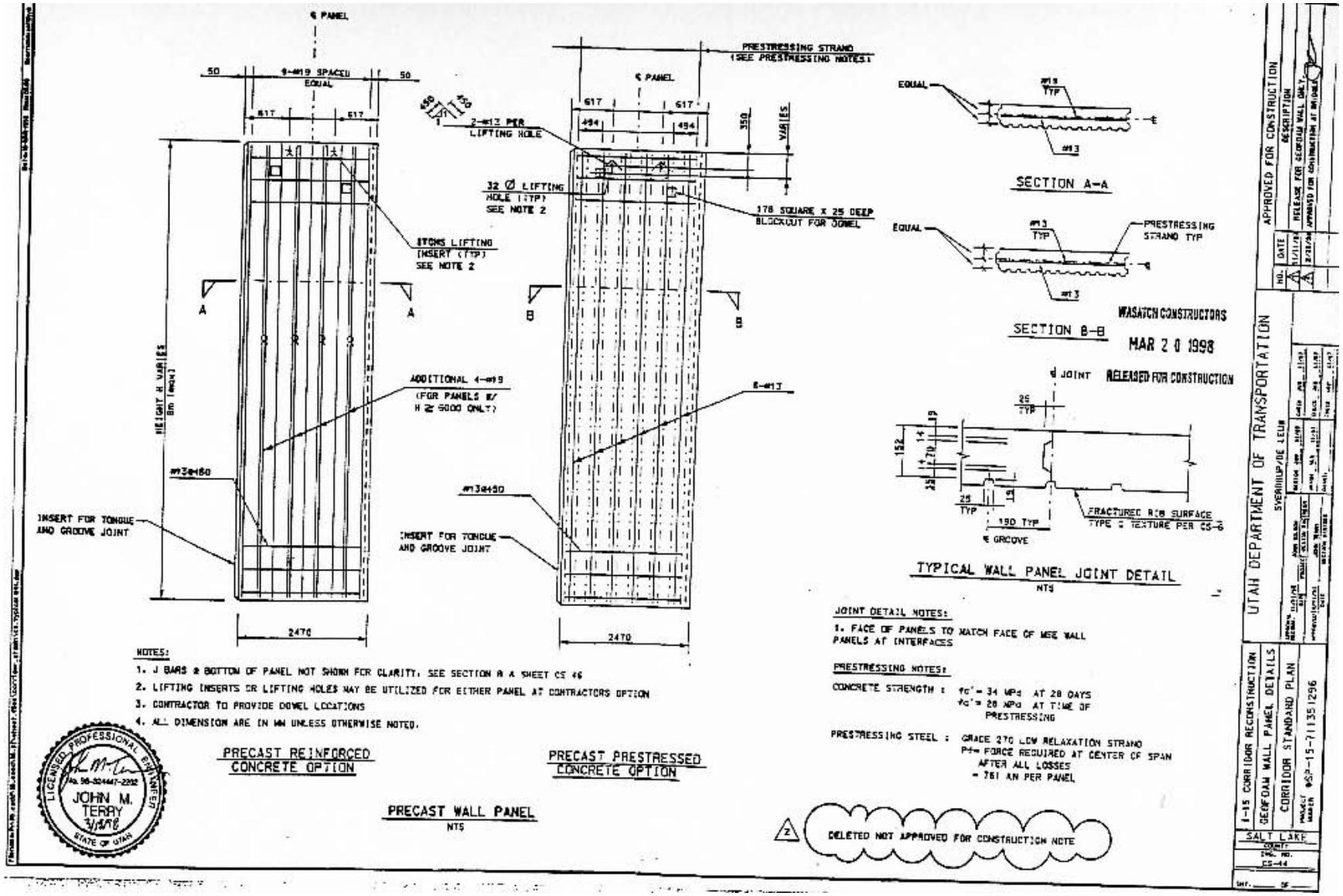
ELEVATION - GEOFOAM WALLS

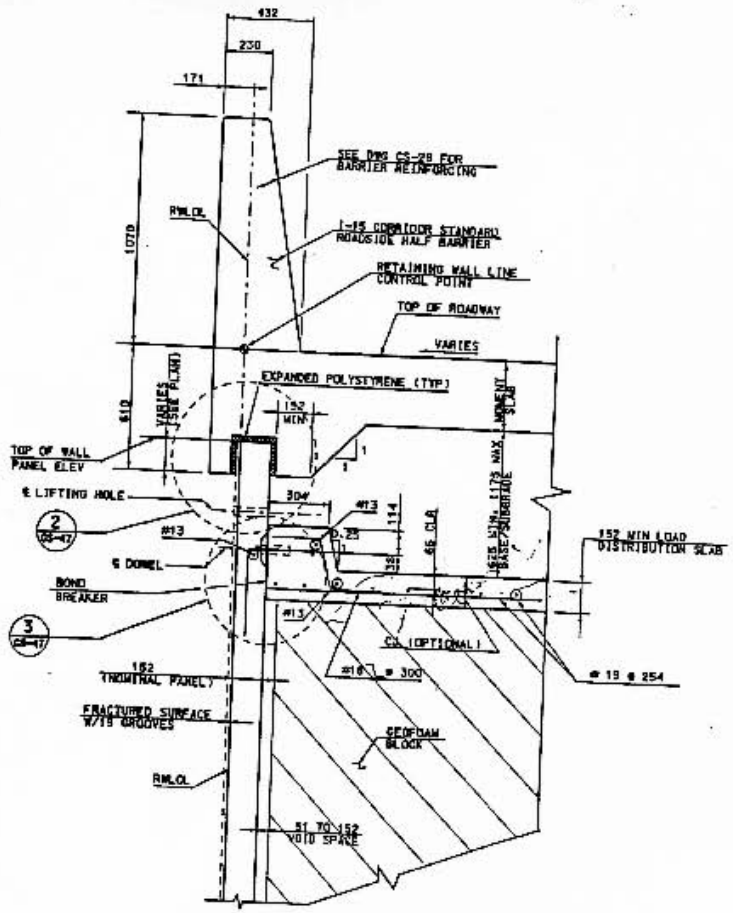
CORRIDOR STANDARD PLAN

PROJECT NUMBER HSP-15-7(135)296

SALT LAKE COUNTY

SHT. NO. CS-43


℄ PANEL
50
9-#19 SPACED EQUAL
50
617
617
HEIGHT H VARIES 8m (MAX)
#13@450
INSERT FOR TONGUE AND GROOVE JOINT
2470
A
A
PRESTRESSING STRAND (SEE PRESTRESSING NOTES)
℄ PANEL
617
617
494
494
2-#13 PER LIFTING HOLE
32 Ø LIFTING HOLE (TYP) SEE NOTE 2
178 SQUARE X 25 DEEP BLOCKOUT FOR DOWEL
350
VARIES
STONS LIFTING INSERT (TYP) SEE NOTE 2
ADDITIONAL 4-#19 (FOR PANELS W/ H ≥ 5000 ONLY)
6-#13
#13@450
INSERT FOR TONGUE AND GROOVE JOINT
2470
B
B
EQUAL
#13 TYP
#13
SECTION A-A
EQUAL
#13 TYP
PRESTRESSING STRAND TYP
#13
SECTION B-B
℄ JOINT
25 TYP
152
14
70
14
25
19
25 TYP
19
190 TYP
℄ GROOVE
FRACTURED RIB SURFACE TYPE 2 TEXTURE PER CS-6
TYPICAL WALL PANEL JOINT DETAIL
NTS
JOINT DETAIL NOTES:
1. FACE OF PANELS TO MATCH FACE OF MSE WALL PANELS AT INTERFACES
PRESTRESSING NOTES:
CONCRETE STRENGTH : f'c = 34 MPa AT 28 DAYS
f'ci = 28 MPa AT TIME OF PRESTRESSING
PRESTRESSING STEEL : GRADE 270 LOW RELAXATION STRAND
Pf = FORCE REQUIRED AT CENTER OF SPAN AFTER ALL LOSSES
= 761 kN PER PANEL
NOTES:
1. J BARS @ BOTTOM OF PANEL NOT SHOWN FOR CLARITY. SEE SECTION B A SHEET CS 46
2. LIFTING INSERTS OR LIFTING HOLES MAY BE UTILIZED FOR EITHER PANEL AT CONTRACTORS OPTION
3. CONTRACTOR TO PROVIDE DOWEL LOCATIONS
4. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.
PRECAST REINFORCED CONCRETE OPTION
PRECAST PRESTRESSED CONCRETE OPTION
PRECAST WALL PANEL
NTS
LICENSED PROFESSIONAL ENGINEER
No. 98-324447-2202
JOHN M. TERRY
3/18/98
STATE OF UTAH
2
DELETED NOT APPROVED FOR CONSTRUCTION NOTE
WASATCH CONSTRUCTORS
MAR 20 1998
RELEASED FOR CONSTRUCTION
APPROVED FOR CONSTRUCTION
NO. DATE DESCRIPTION
1 1/21/98 RELEASE FOR GEOFOAM WALL ONLY
2 2/23/98 APPROVED FOR CONSTRUCTION AT BRIDGE
UTAH DEPARTMENT OF TRANSPORTATION
SVERDRUP/DE LEUW
I-15 CORRIDOR RECONSTRUCTION
GEOFOAM WALL PANEL DETAILS
CORRIDOR STANDARD PLAN
PROJECT NUMBER H52-15-7(1)351296
SALT LAKE COUNTY
FED. RD. CS-44
SHT. OF



LOAD DISTRIBUTION SLAB RESTRAINT

SECTION A

NTS

WASATCH CONSTRUCTORS

APR 13 1998

RELEASED FOR CONSTRUCTION

NOTES:

1. LOAD DISTRIBUTION SLAB DESIGNED FOR HS 20 LOADING.
2. WHEEL LOADS ARE NOT PERMITTED WITHIN 1500 OF FREE EDGE OF LOAD DISTRIBUTION SLAB PRIOR TO PLACING PCCP SLAB.
3. LIFTING HOLES TO BE DRY PACKED.
4. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.
5. SEE CS-28 FOR MOMENT SLAB REINFORCING.
6. FOR DETAILS UNDER APPROACH SLABS AND BRIDGES SEE CS-49
7. TRANSVERSE CONSTRUCTION JOINTS ARE ALLOWED IN THE LOAD DISTRIBUTION SLAB AT THE OPTION OF THE CONTRACTOR.
8. ALL CRACKS OVER 0.5 MM SHALL BE SEALED WITH A HIGH MOLECULAR WEIGHT METHYL METHACRYLATE SEALANT (TRANSPO T-70 OR EQUAL.
9. CONCRETE SHALL BE CLASS AA(AE) EXCEPT AS MODIFIED FOR F'C=27.5MPa (4,000 PSI). NO CONSTRUCTION LOADS SHALL BE PERMITTED ON THE LOAD DISTRIBUTION SLAB UNTIL AFTER 7 DAYS OF CURING.

LICENSED PROFESSIONAL ENGINEER
No. 96-224447-2202
JOHN M. TERRY
STATE OF UTAH

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	1/15/97	RELEASE FOR GEOFOAM WALL ONLY.
2	2/26/98	APPROVED FOR CONSTRUCTION AT BRIDGES
3	3/10/98	ADD NOTES

UTAH DEPARTMENT OF TRANSPORTATION

SVERDRUP/DE LEUW

I-15 CORRIDOR RECONSTRUCTION

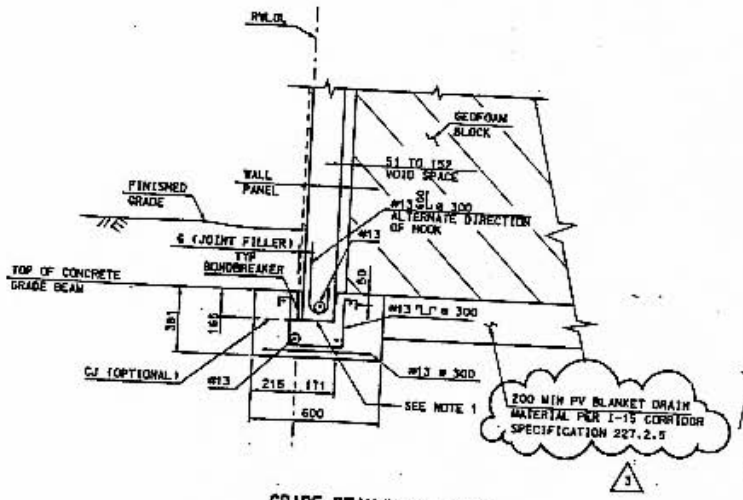
GEOFOAM WALL RESTRAINT DETAILS

CORRIDOR STANDARD PLAN

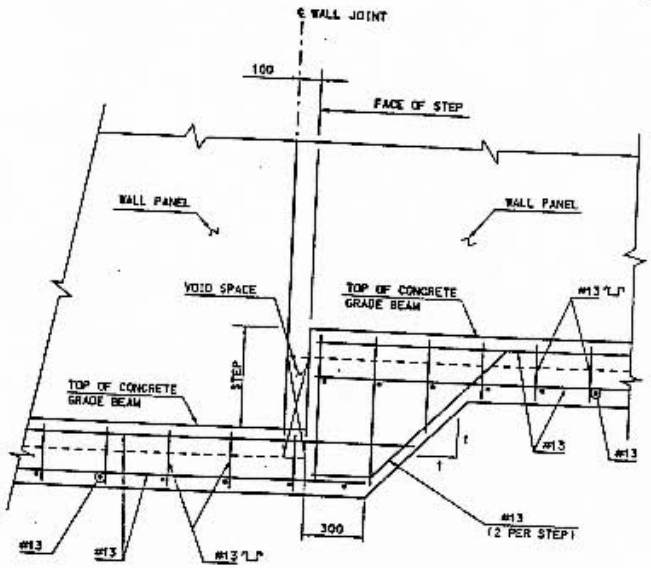
PROJECT NUMBER #SP-15-7(135)296

SALT LAKE COUNTY

DWG. NO. CS-48



GRADE BEAM/WALL PANEL
SECTION
NTS
8



TYPICAL GRADE BEAM STEP
DETAIL
NTS
1

NOTES:

1. SHIM BOTTOM OF WALL PANELS AS REQUIRED TO ALIGN VERTICAL WALL JOINTS.
2. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.

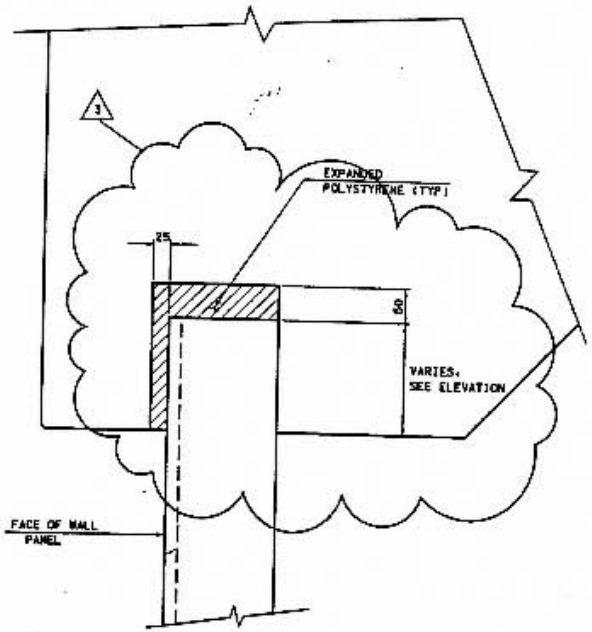
LICENSED PROFESSIONAL ENGINEER
No. 05-324447-2202
JOHN M. TERRY
STATE OF UTAH

WASATCH CONSTRUCTORS
APR - 3 1998
RELEASED FOR CONSTRUCTION

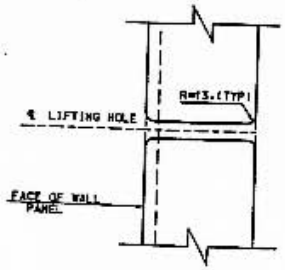
APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	3/31/98	RELEASE FOR GEOFOAM WALL ONLY.
2	4/29/98	APPROVED FOR CONSTRUCTION AT BRIDGES
3	1/28/99	NOTE REVISION

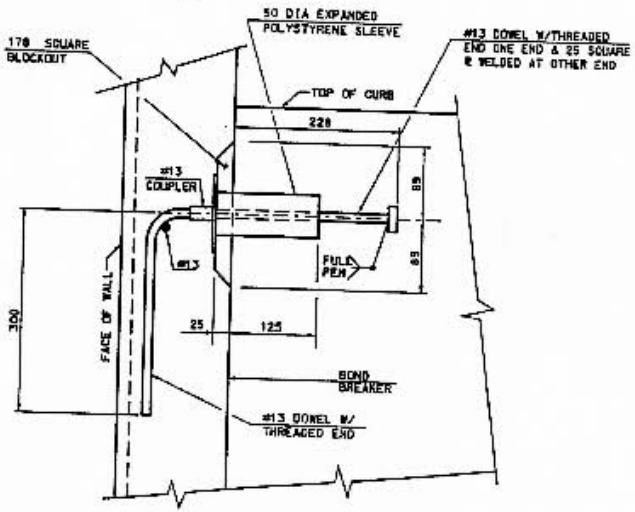
UTAH DEPARTMENT OF TRANSPORTATION
SVERDRUP/DE LEUW
I-15 CORRIDOR RECONSTRUCTION
GEOFOAM WALL GRADE BEAM DETAILS
CORRIDOR STANDARD PLANS
PROJECT NUMBER NSP-15-7(135)296
SALT LAKE COUNTY
PROJ. NO. 15



EXTERNAL KEY
DETAIL 2/1
NTS



LIFTING HOLE
NTS



WALL TO LDS CONNECTION
DETAIL 3/1
NTS

NOTE:

1. ALL DIMENSIONS ARE IN MM UNLESS OTHERWISE NOTED.

WASATCH CONSTRUCTORS

JUL 3 1 1998

RELEASED FOR CONSTRUCTION

LICENSED PROFESSIONAL ENGINEER
No. 96-324447-2202
JOHN M. TERRY
7/23/98
STATE OF UTAH

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	11/11/97	RELEASE FOR BIDDING WALL ONLY
2	02/20/98	APPROVED FOR CONSTRUCTION AT BRIDGES
3	07/30/98	REVISED GPS DETAILS

UTAH DEPARTMENT OF TRANSPORTATION

SVERDRUP/DE LEUW

I-15 CORRIDOR RECONSTRUCTION

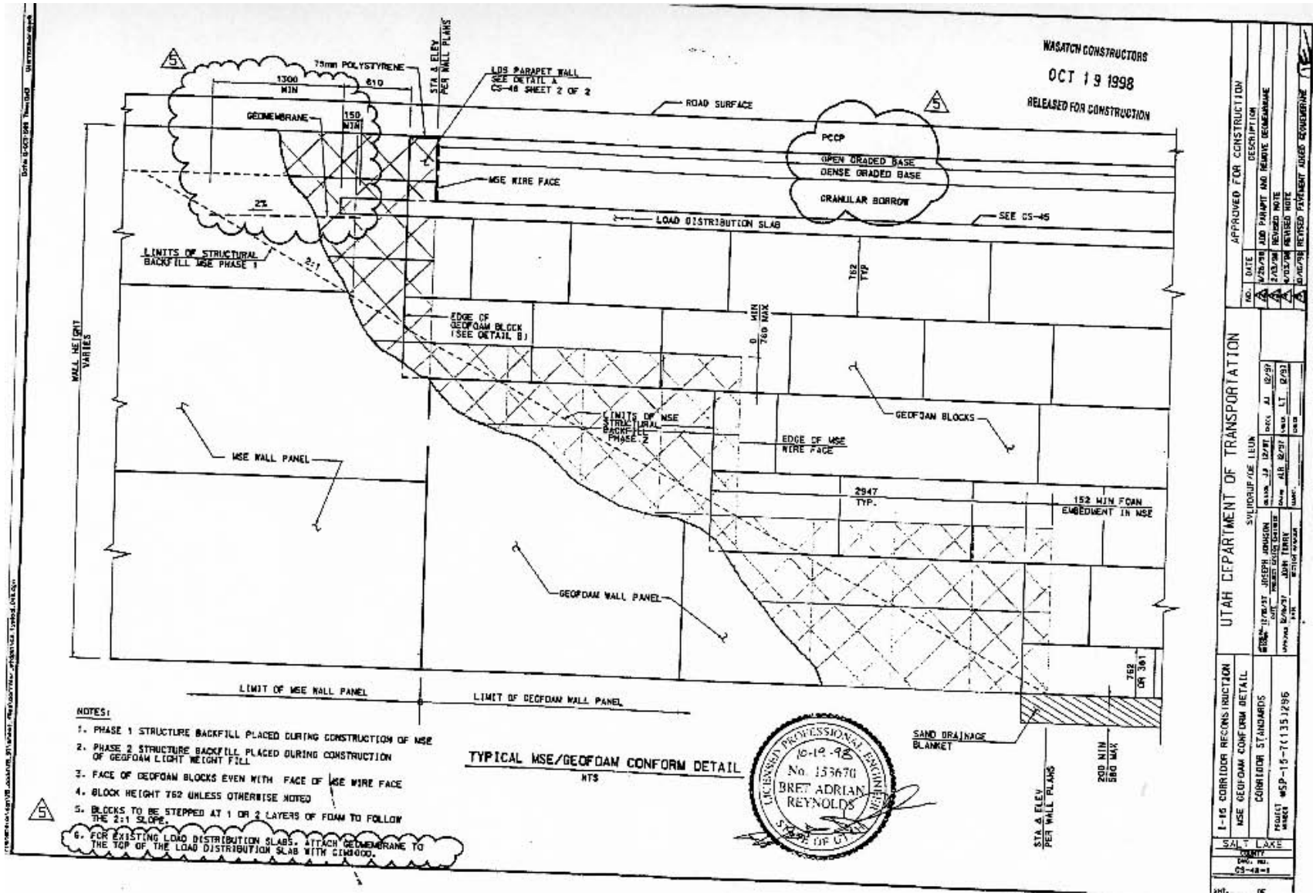
DEDFDAM WALL CONNECTION DETAILS

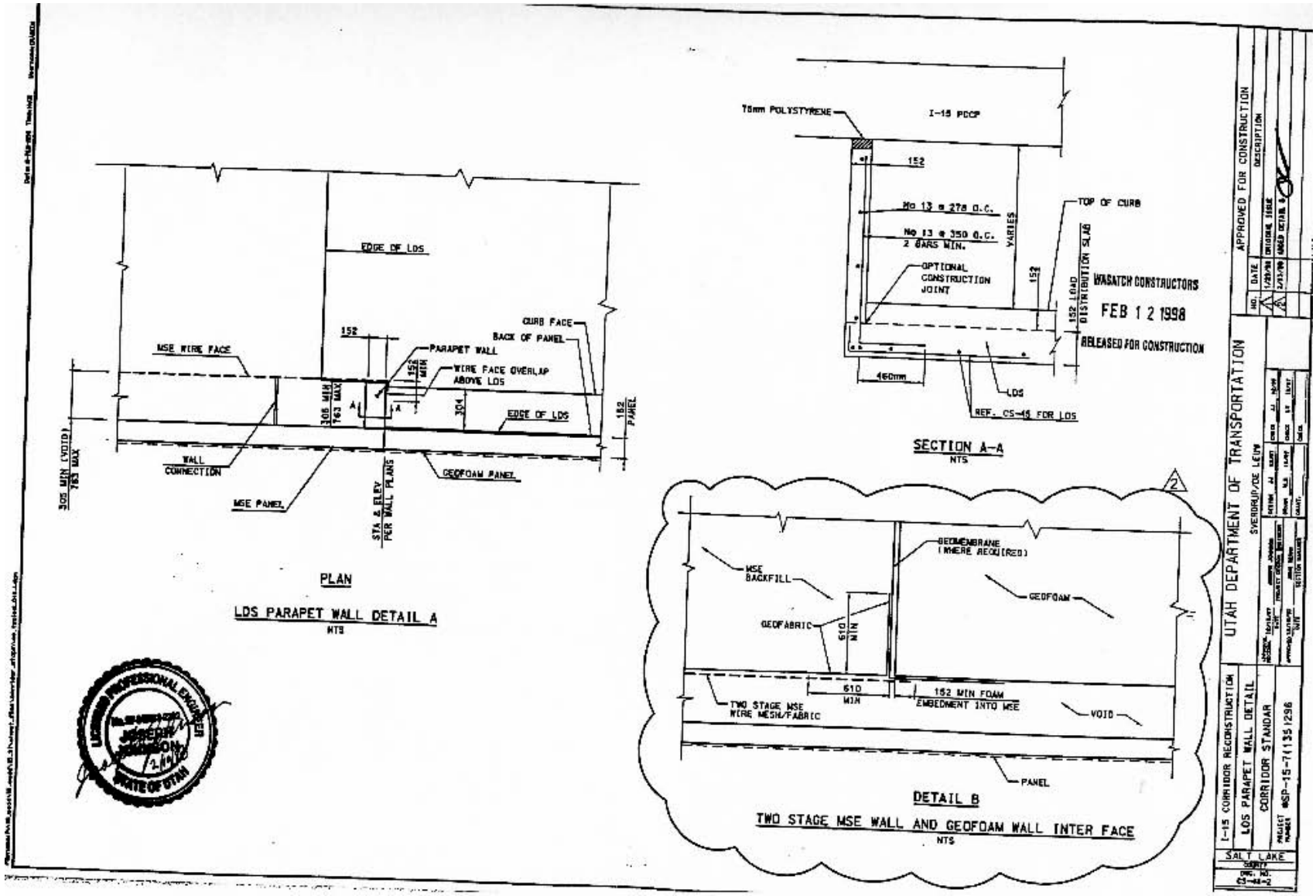
CORRIDOR STANDARD PLANS

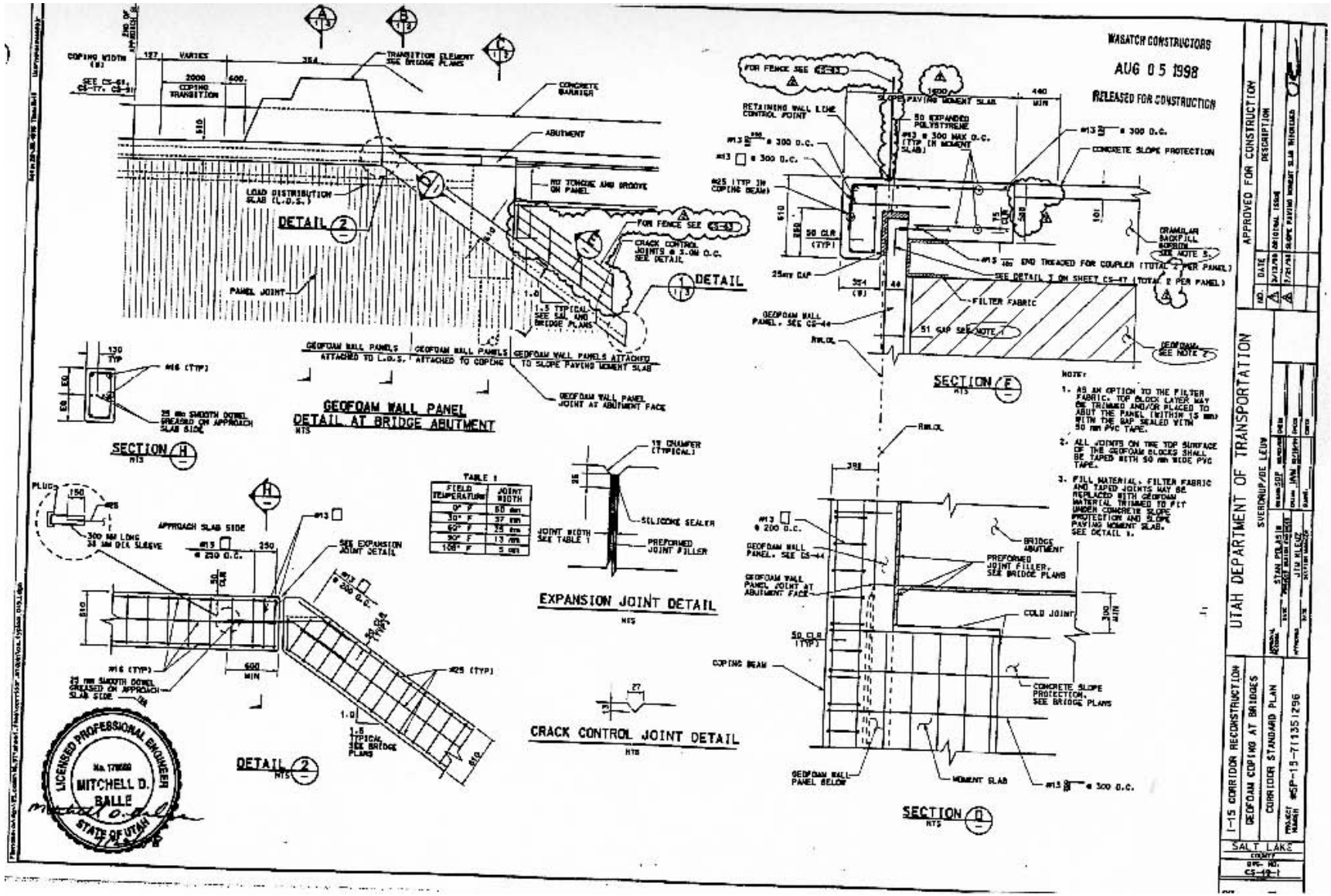
PROJECT NUMBER NSP-15-7(135)296

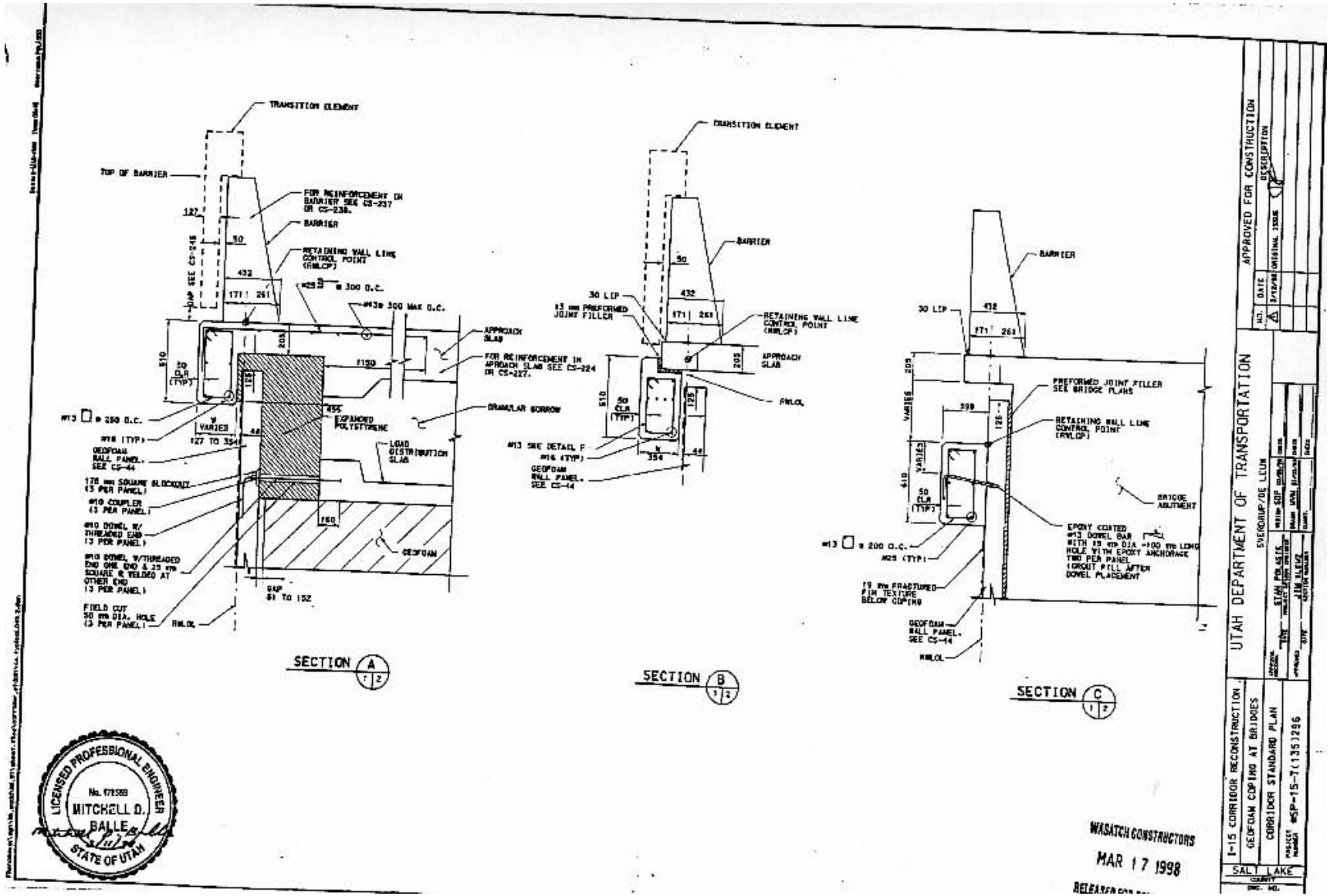
SALT LAKE COUNTY

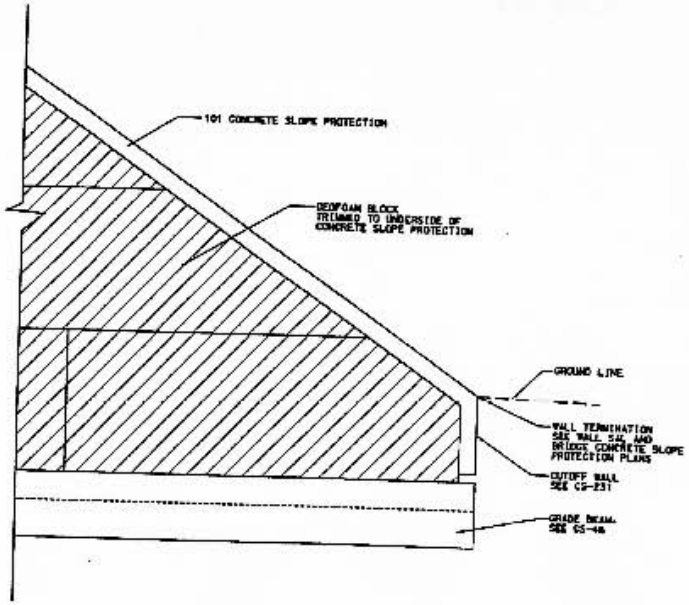
DWG. NO. CS-47


WASATCH CONSTRUCTORS
OCT 19 1998
RELEASED FOR CONSTRUCTION
75mm POLYSTYRENE
1300 MIN
610
150 MIN
GEOMEMBRANE
STA & ELEV PER WALL PLANS
LDS PARAPET WALL SEE DETAIL A CS-48 SHEET 2 OF 2
ROAD SURFACE
PCCP
OPEN GRADED BASE
DENSE GRADED BASE
GRANULAR BORROW
MSE WIRE FACE
2%
LOAD DISTRIBUTION SLAB
SEE CS-45
LIMITS OF STRUCTURAL BACKFILL MSE PHASE 1
2:1
762 TYP
EDGE OF GEOFOAM BLOCK (SEE DETAIL B)
0 MIN 760 MAX
WALL HEIGHT VARIES
LIMITS OF MSE STRUCTURAL BACKFILL PHASE 2
GEOFOAM BLOCKS
EDGE OF MSE WIRE FACE
MSE WALL PANEL
2947 TYP.
152 MIN FOAM EMBEDMENT IN MSE
GEOFOAM WALL PANEL
762 OR 381
LIMIT OF MSE WALL PANEL
LIMIT OF GEOFOAM WALL PANEL
SAND DRAINAGE BLANKET
200 MIN 580 MAX
STA & ELEV PER WALL PLANS
NOTES:
1. PHASE 1 STRUCTURE BACKFILL PLACED DURING CONSTRUCTION OF MSE
2. PHASE 2 STRUCTURE BACKFILL PLACED DURING CONSTRUCTION OF GEOFOAM LIGHT WEIGHT FILL
3. FACE OF GEOFOAM BLOCKS EVEN WITH FACE OF MSE WIRE FACE
4. BLOCK HEIGHT 762 UNLESS OTHERWISE NOTED
5. BLOCKS TO BE STEPPED AT 1 OR 2 LAYERS OF FOAM TO FOLLOW THE 2:1 SLOPE.
6. FOR EXISTING LOAD DISTRIBUTION SLABS, ATTACH GEOMEMBRANE TO THE TOP OF THE LOAD DISTRIBUTION SLAB WITH GIMBOOD.
TYPICAL MSE/GEOFOAM CONFORM DETAIL
NTS
LICENSED PROFESSIONAL ENGINEER
10-19-98
No. 153670
BRET ADRIAN REYNOLDS
STATE OF UTAH
APPROVED FOR CONSTRUCTION
NO. DATE DESCRIPTION
1 9/25/98 ADD PARAPET AND REMOVE GEOMEMBRANE
2 2/13/98 REVISED NOTE
3 4/03/98 REVISED NOTE
4 10/16/98 REVISED PAVEMENT ADJACENT GEOMEMBRANE
UTAH DEPARTMENT OF TRANSPORTATION
SYLVESTER/DE LEUW
JOSEPH JOHNSON
JOHN TURNER
I-15 CORRIDOR RECONSTRUCTION
MSE GEOFOAM CONFORM DETAIL
CORRIDOR STANDARDS
PROJECT NUMBER MSP-15-7(113)296
SALT LAKE COUNTY
DWG. NO. CS-48-1


EDGE OF LDS
MSE WIRE FACE
152
PARAPET WALL
WIRE FACE OVERLAP ABOVE LDS
CURB FACE
BACK OF PANEL
EDGE OF LDS
WALL CONNECTION
MSE PANEL
STA & ELEV PER WALL PLANS
GEOFOAM PANEL
152 PANEL
PLAN
LDS PARAPET WALL DETAIL A
NTS
75mm POLYSTYRENE
I-15 PCCP
No 13 @ 275 O.C.
No 13 @ 350 O.C.
2 BARS MIN.
VARIES
TOP OF CURB
OPTIONAL CONSTRUCTION JOINT
152 DISTRIBUTION SLAB
WASATCH CONSTRUCTORS
FEB 1 2 1998
RELEASED FOR CONSTRUCTION
460mm
LDS
REF. CS-45 FOR LDS
SECTION A-A
NTS
GEOMEMBRANE (WHERE REQUIRED)
MSE BACKFILL
GEOFOAM
GEOFABRIC
610 MIN
TWO STAGE MSE WIRE MESH/FABRIC
152 MIN FOAM EMBEDMENT INTO MSE
VOID
PANEL
DETAIL B
TWO STAGE MSE WALL AND GEOFOAM WALL INTER FACE
NTS
LICENSED PROFESSIONAL ENGINEER
STATE OF UTAH
APPROVED FOR CONSTRUCTION
UTAH DEPARTMENT OF TRANSPORTATION
I-15 CORRIDOR RECONSTRUCTION
LDS PARAPET WALL DETAIL
CORRIDOR STANDAR
SALT LAKE COUNTY

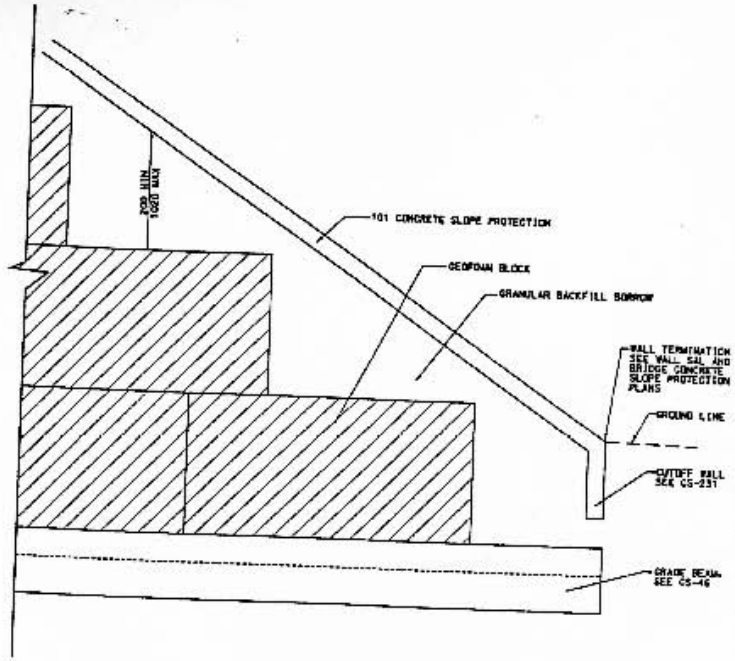

WASATCH CONSTRUCTORS
AUG 05 1998
RELEASED FOR CONSTRUCTION
APPROVED FOR CONSTRUCTION
UTAH DEPARTMENT OF TRANSPORTATION
I-15 CORRIDOR RECONSTRUCTION
GEOFOAM COPING AT BRIDGES
CORRIDOR STANDARD PLAN
SALT LAKE
GEOFOAM WALL PANEL
DETAIL AT BRIDGE ABUTMENT
EXPANSION JOINT DETAIL
CRACK CONTROL JOINT DETAIL
SECTION H
SECTION E
SECTION D
DETAIL 2
LICENSED PROFESSIONAL ENGINEER
MITCHELL D. BALLE
STATE OF UTAH


TRANSITION ELEMENT
TOP OF BARRIER
FOR REINFORCEMENT IN BARRIER SEE CS-237 OR CS-238.
BARRIER
RETAINING WALL LINE CONTROL POINT (RWLCP)
APPROACH SLAB
FOR REINFORCEMENT IN APPROACH SLAB SEE CS-224 OR CS-227.
GRANULAR BORROW
EXPANDED POLYSTYRENE
LOAD DISTRIBUTION SLAB
GEOFOAM WALL PANEL, SEE CS-14
178 mm SQUARE BLOCKOUT (3 PER PANEL)
#10 COUPLER (3 PER PANEL)
#10 DOWEL W/ THREADED END (3 PER PANEL)
#10 DOWEL W/THREADED END ONE END & 25 mm SQUARE & WELDED AT OTHER END (3 PER PANEL)
FIELD CUT 50 mm DIA. HOLE (3 PER PANEL)
RWLOL
GEOFOAM
GAP 51 TO 152
SECTION A
TRANSITION ELEMENT
BARRIER
30 LCP
13 mm PREFORMED JOINT FILLER
RETAINING WALL LINE CONTROL POINT (RWLCP)
APPROACH SLAB
RWLOL
#13 SEE DETAIL F
GEOFOAM WALL PANEL, SEE CS-14
SECTION B
BARRIER
30 LCP
PREFORMED JOINT FILLER SEE BRIDGE PLANS
RETAINING WALL LINE CONTROL POINT (RWLCP)
BRIDGE ABUTMENT
EPOXY COATED #13 DOWEL BAR WITH 19 mm DIA. ±100 mm LONG HOLE WITH EPOXY ANCHORAGE TWO PER PANEL (GROUT FILL AFTER DOWEL PLACEMENT)
19 mm FRACTURED FIN TEXTURE BELOW COPING
GEOFOAM WALL PANEL, SEE CS-14
RWLOL
SECTION C
LICENSED PROFESSIONAL ENGINEER
No. 175589
MITCHELL D. BALLE
STATE OF UTAH
WASATCH CONSTRUCTORS
MAR 17 1998
APPROVED FOR CONSTRUCTION
UTAH DEPARTMENT OF TRANSPORTATION
I-15 CORRIDOR RECONSTRUCTION
GEOFOAM COPING AT BRIDGES
CORRIDOR STANDARD PLAN
SALT LAKE



DETAIL G – TRIMMED GEOFOAM OPTION

DETAIL 1/1 3
NOT TO SCALE



DETAIL G – BACKFILL OPTION

DETAIL 1/1 3
NOT TO SCALE

LICENSED PROFESSIONAL ENGINEER
No. 178559
MITCHELL D. BALLE
STATE OF UTAH

WASATCH CONSTRUCTORS

MAR 17 1998

RELEASED FOR CONSTRUCTION

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
A	3/12/98	ORIGINAL ISSUE

UTAH DEPARTMENT OF TRANSPORTATION

SVERDRUP/DE LEUW

STAN DOLATIK
PROJECT ENGINEER

JIM BLEDSOE
SECTION MANAGER

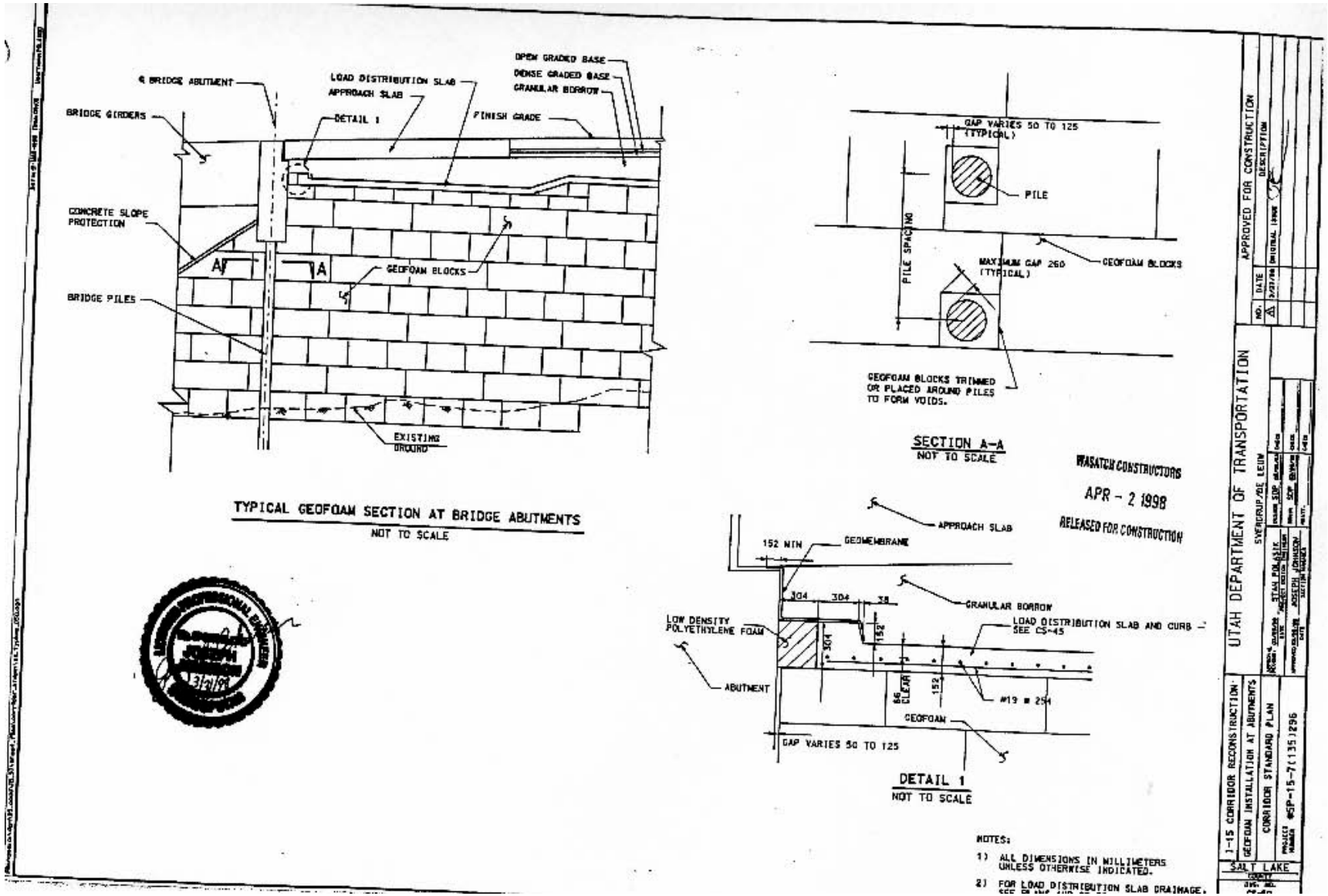
I-15 CORRIDOR RECONSTRUCTION
GEOFOAM COPING AT BRIDGES

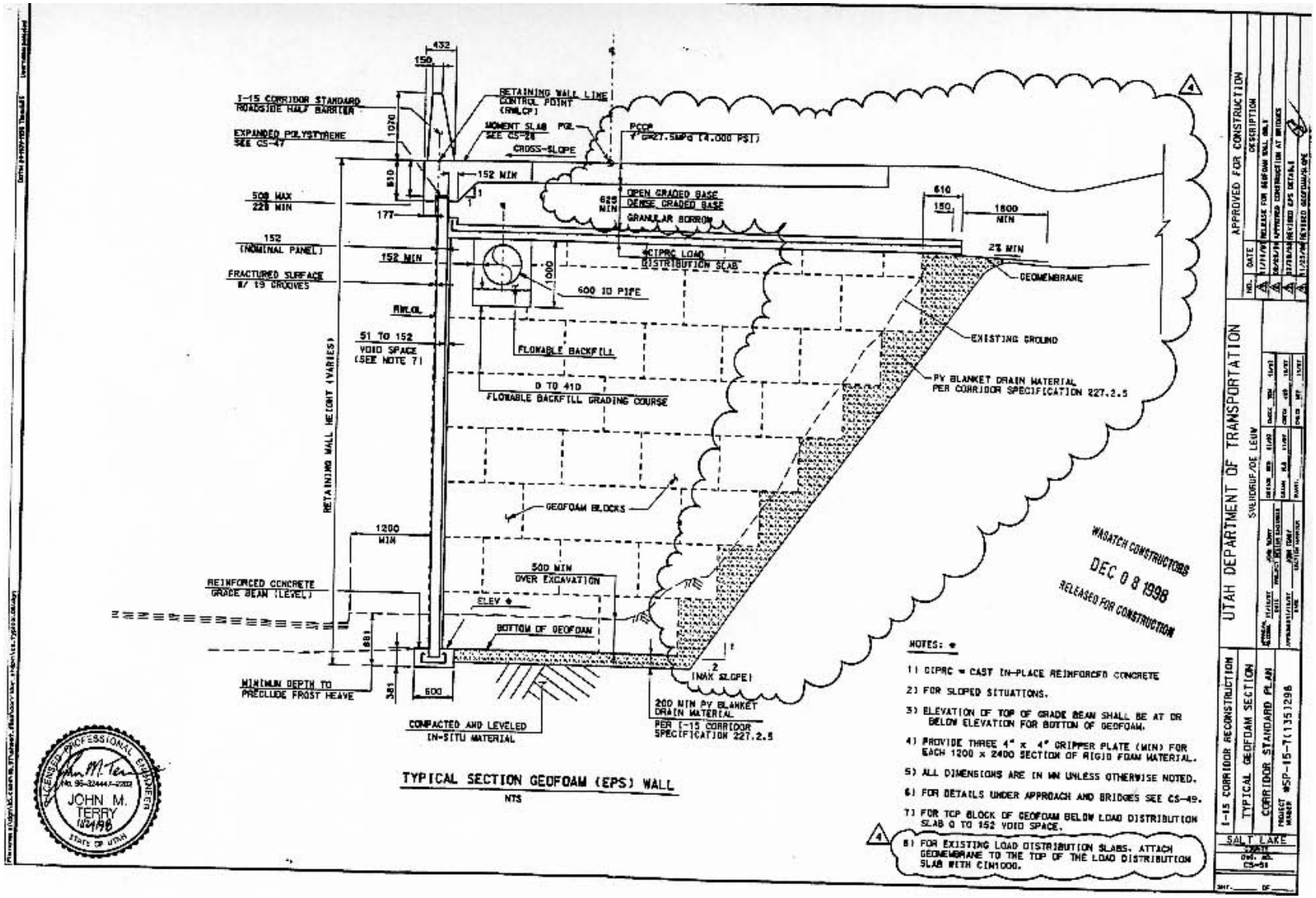
CORRIDOR STANDARD PLAN

PROJECT NO. NSP-15-7(135)296

SALT LAKE COUNTY

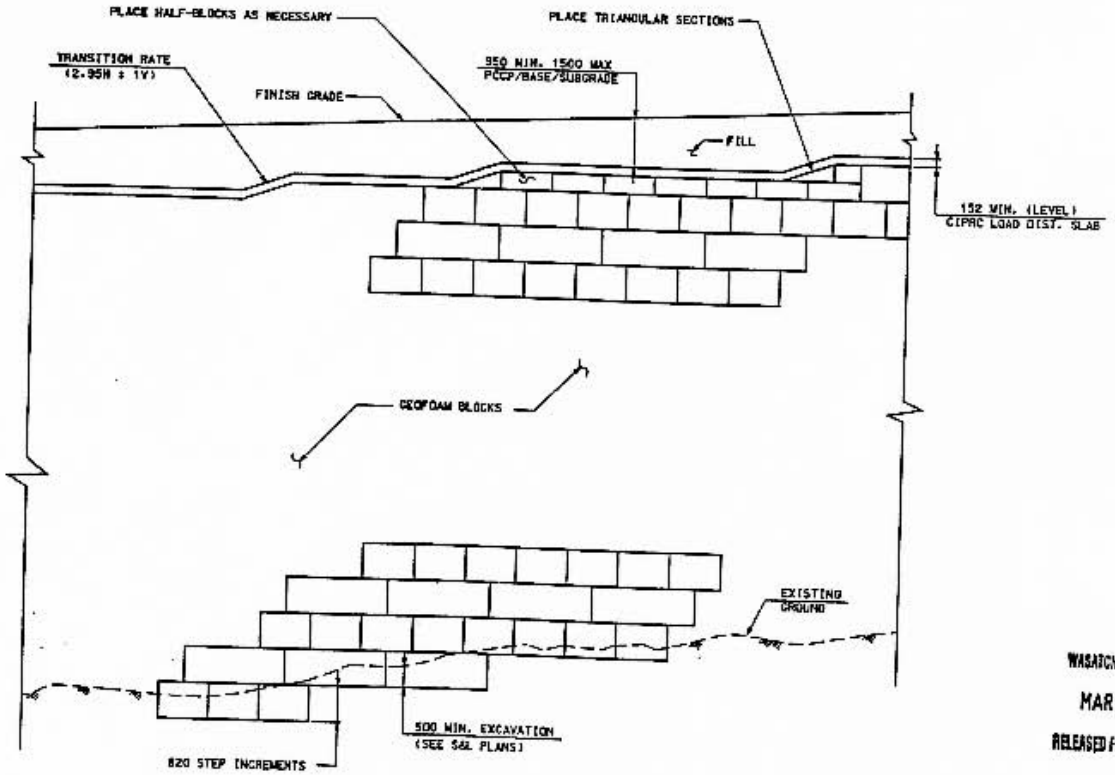
DWG. NO. CS-49-3


℄ BRIDGE ABUTMENT
BRIDGE GIRDERS
LOAD DISTRIBUTION SLAB
APPROACH SLAB
DETAIL 1
OPEN GRADED BASE
DENSE GRADED BASE
GRANULAR BORROW
FINISH GRADE
CONCRETE SLOPE PROTECTION
A
A
GEOFOAM BLOCKS
BRIDGE PILES
EXISTING GROUND
TYPICAL GEOFOAM SECTION AT BRIDGE ABUTMENTS
NOT TO SCALE
GAP VARIES 50 TO 125 (TYPICAL)
PILE
PILE SPACING
MAXIMUM GAP 250 (TYPICAL)
GEOFOAM BLOCKS
GEOFOAM BLOCKS TRIMMED OR PLACED AROUND PILES TO FORM VOIDS.
SECTION A-A
NOT TO SCALE
WASATCH CONSTRUCTORS
APR - 2 1998
RELEASED FOR CONSTRUCTION
APPROACH SLAB
152 MIN
GEOMEMBRANE
304
304
38
GRANULAR BORROW
LOW DENSITY POLYETHYLENE FOAM
LOAD DISTRIBUTION SLAB AND CURB SEE CS-45
304
152
ABUTMENT
86 CLEAR
152
#19 @ 254
GEOFOAM
GAP VARIES 50 TO 125
DETAIL 1
NOT TO SCALE
NOTES:
1) ALL DIMENSIONS IN MILLIMETERS UNLESS OTHERWISE INDICATED.
2) FOR LOAD DISTRIBUTION SLAB DRAINAGE,
APPROVED FOR CONSTRUCTION
NO. DATE DESCRIPTION
UTAH DEPARTMENT OF TRANSPORTATION
SVERDRUP/AGE LEIN
I-15 CORRIDOR RECONSTRUCTION:
GEOFOAM INSTALLATION AT ABUTMENTS
CORRIDOR STANDARD PLAN
PROJECT NUMBER #SP-15-7(135)296
SALT LAKE
COUNTY



TYPICAL SECTION GEOFOAM (EPS) WALL

NTS



TYPICAL LONGITUDINAL GEOFOAM SECTION
NOT TO SCALE

WASATCH CONSTRUCTORS
MAR 2 0 1998
RELEASED FOR CONSTRUCTION

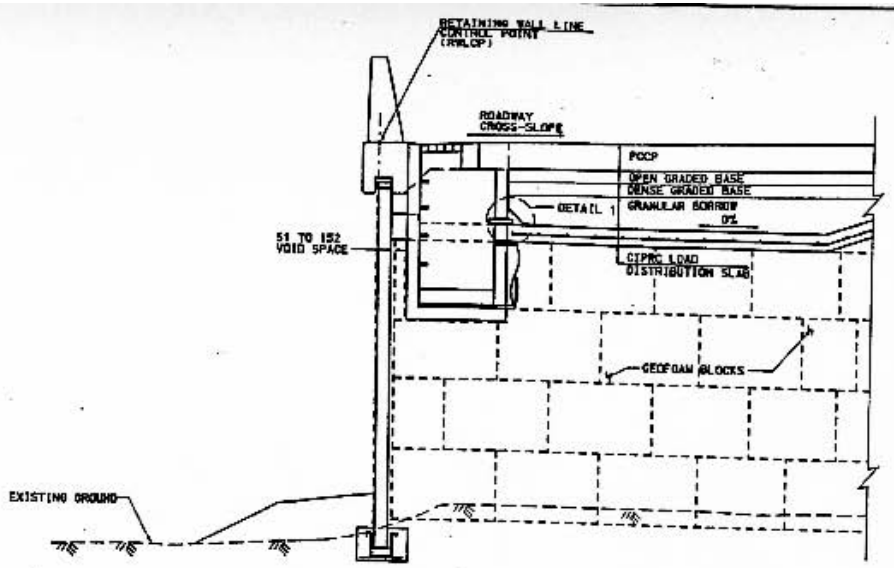
LICENSED PROFESSIONAL ENGINEER
JOHN M. TERRY
3/18/98
STATE OF UTAH

APPROVED FOR CONSTRUCTION

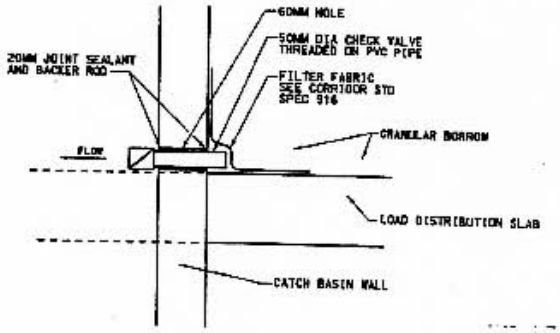
NO.	DATE	DESCRIPTION
1	3/18/98	ORIGINAL ISSUE

UTAH DEPARTMENT OF TRANSPORTATION
SVERDRUP/DE LEUW

I-15 CORRIDOR RECONSTRUCTION
TYPICAL LONGITUDINAL GEOFOAM SECTION
CORRIDOR STANDARD PLAN
PROJECT NUMBER MSP-15-7(135)298
SALT LAKE COUNTY
DWG. NO. CS-69



TYPICAL LOAD DISTRIBUTION SLAB DRAIN
NOT TO SCALE



DETAIL 1
NOT TO SCALE

WASATCH CONSTRUCTORS

NOV 11 1998

RELEASED FOR CONSTRUCTION

NOTES:

1) ALL DIMENSIONS IN MILLIMETERS UNLESS OTHERWISE INDICATED.

LICENSED PROFESSIONAL ENGINEER
No. 95-291065-2202
DAN M. ISOM
8/10/98
STATE OF UTAH

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	11/10/98	ORIGINAL ISSUE

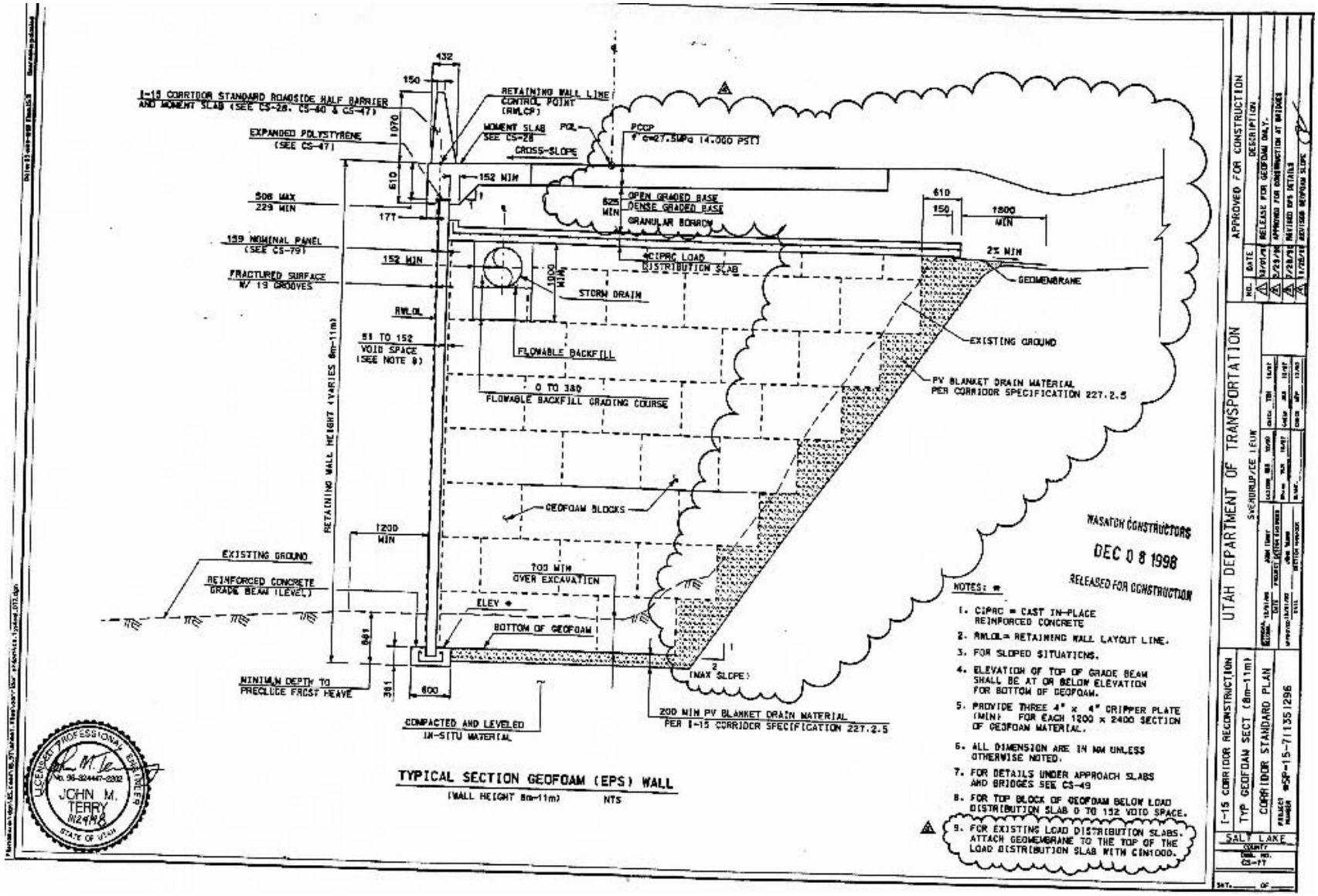
UTAH DEPARTMENT OF TRANSPORTATION

SVERDRUP/IE LLC

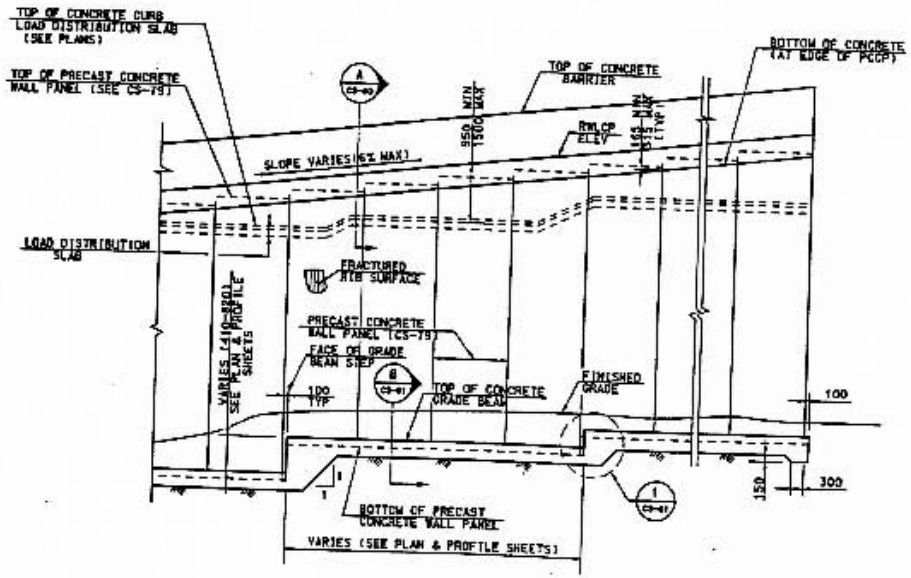
DESIGN: SEAN POLASEK
PROJECT DESIGN ENGINEER
REVIEW: JIM KLEMZ
SECTION MANAGER
DRAWN: DMG
CHECK:

I-15 CORRIDOR RECONSTRUCTION
LOAD DISTRIBUTION SLAB DRAIN
CORRIDOR STANDARD PLAN
PROJECT NUMBER: MSP-15-7(135)296

SALT LAKE COUNTY
ENG. NO. CS-52
SHT. ___ OF ___



TYPICAL SECTION GEOFOAM (EPS) WALL
(WALL HEIGHT 8m-11m) NTS



ELEVATION-GEOFOAM (EPS) WALL
(WALL HEIGHT 8m-11M) NTS

WASATCH CONSTRUCTORS

MAR 2 0 1998

RELEASED FOR CONSTRUCTION

NOTE:

1. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.
2. FOR DETAILS UNDER APPROACH SLABS AND BRIDGES SEE CS-49

2 DELETED NOT APPROVED FOR CONSTRUCTION NOTE

LICENSED PROFESSIONAL ENGINEER
JOHN M. TERRY
STATE OF UTAH

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	3/20/98	RELEASE FOR GEOFOAM WALL ONLY.
2	8/21/98	APPROVAL FOR CONSTRUCTION AT BRIDGES

UTAH DEPARTMENT OF TRANSPORTATION

SVERDRUP/DE LEUW

I-15 CORRIDOR RECONSTRUCTION

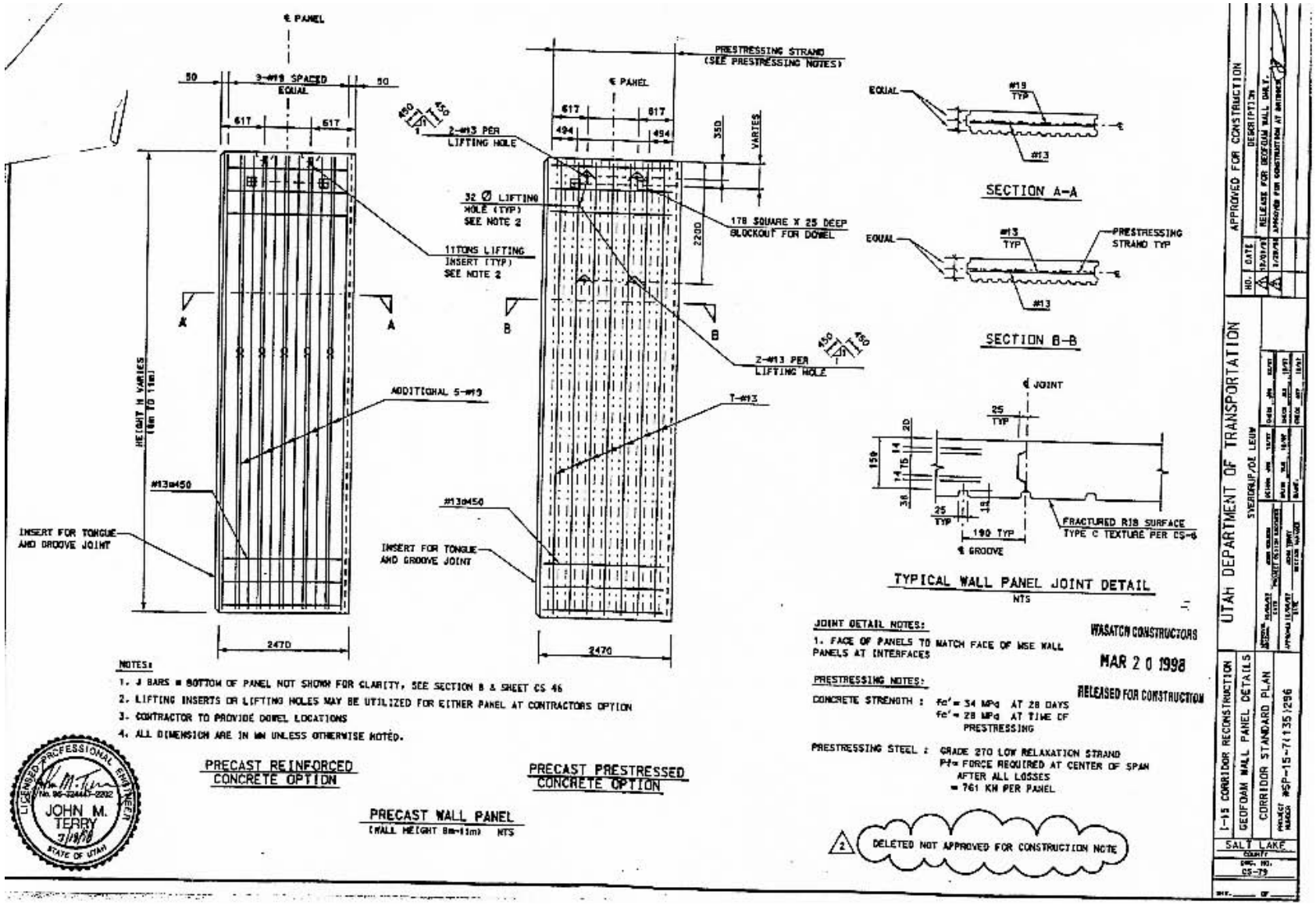
ELEV-GEOFOAM WALL (8m-11m)

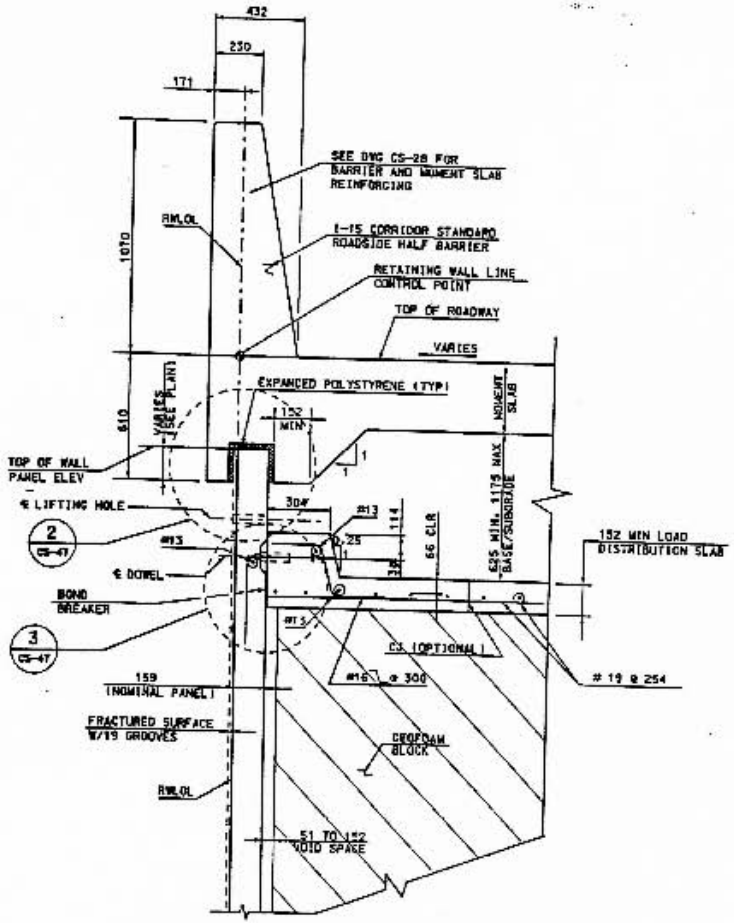
CORRIDOR STANDARD PLAN

PROJECT NUMBER MSP-15-7(135)296

SALT LAKE COUNTY

DWG. NO. CS-78


℄ PANEL
50
3-#19 SPACED EQUAL
50
617
617
HEIGHT H VARIES (6m TO 11m)
A
A
#13@450
INSERT FOR TONGUE AND GROOVE JOINT
ADDITIONAL 5-#19
2470
PRESTRESSING STRAND (SEE PRESTRESSING NOTES)
℄ PANEL
617
617
494
494
450
450
2-#13 PER LIFTING HOLE
350
VARIES
32 Ø LIFTING HOLE (TYP) SEE NOTE 2
178 SQUARE X 25 DEEP BLOCKOUT FOR DOWEL
2200
11TONS LIFTING INSERT (TYP) SEE NOTE 2
B
B
450
450
2-#13 PER LIFTING HOLE
T-#13
#13@450
INSERT FOR TONGUE AND GROOVE JOINT
2470
EQUAL
#19 TYP
#13
SECTION A-A
EQUAL
#13 TYP
PRESTRESSING STRAND TYP
#13
SECTION B-B
℄ JOINT
25 TYP
20
14
150
14
15
36
25 TYP
18
190 TYP
℄ GROOVE
FRACTURED RIB SURFACE TYPE C TEXTURE PER CS-6
TYPICAL WALL PANEL JOINT DETAIL
NTS
JOINT DETAIL NOTES:
1. FACE OF PANELS TO MATCH FACE OF MSE WALL PANELS AT INTERFACES
PRESTRESSING NOTES:
CONCRETE STRENGTH : fc' = 34 MPa AT 28 DAYS
fc' = 28 MPa AT TIME OF PRESTRESSING
PRESTRESSING STEEL : GRADE 270 LOW RELAXATION STRAND
Pf = FORCE REQUIRED AT CENTER OF SPAN AFTER ALL LOSSES = 761 KN PER PANEL
WASATCH CONSTRUCTORS
MAR 2 0 1998
RELEASED FOR CONSTRUCTION
NOTES:
1. J BARS = BOTTOM OF PANEL NOT SHOWN FOR CLARITY, SEE SECTION B & SHEET CS 46
2. LIFTING INSERTS OR LIFTING HOLES MAY BE UTILIZED FOR EITHER PANEL AT CONTRACTORS OPTION
3. CONTRACTOR TO PROVIDE DOWEL LOCATIONS
4. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.
PRECAST REINFORCED CONCRETE OPTION
PRECAST PRESTRESSED CONCRETE OPTION
PRECAST WALL PANEL
(WALL HEIGHT 6m-11m) NTS
LICENSED PROFESSIONAL ENGINEER
JOHN M. TERRY
STATE OF UTAH
DELETED NOT APPROVED FOR CONSTRUCTION NOTE
APPROVED FOR CONSTRUCTION
NO. DATE DESCRIPTION
RELEASE FOR GEOFOAM WALL DWG.
APPROVED FOR CONSTRUCTION AT BRIDGE
UTAH DEPARTMENT OF TRANSPORTATION
SVENDRUP/DE LEUW
I-15 CORRIDOR RECONSTRUCTION
GEOFOAM WALL PANEL DETAILS
CORRIDOR STANDARD PLAN
PROJECT NUMBER #SP-15-7(135)296
SALT LAKE COUNTY
DWG. NO. CS-79



LOAD DISTRIBUTION SLAB RESTRAINT SECTION
(WALL HEIGHT 6m-11m) NTS A

WASATCH CONSTRUCTORS
APR 13 1998
RELEASED FOR CONSTRUCTION

NOTES:

1. LOAD DISTRIBUTION SLAB DESIGNED FOR HS-20 LOADING.
2. WHEEL LOADS ARE NOT PERMITTED WITHIN 1500 OF FREE EDGE OF LOAD DISTRIBUTION SLAB PRIOR TO PLACING PCCP SLAB.
3. LIFTING HOLES TO BE DRY PACKED.
4. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.
5. SEE CS-28 FOR MOMENT SLAB REINFORCING.
6. FOR DETAILS UNDER APPROACH SLABS AND BRIDGES SEE CS-49.
7. TRANSVERSE CONSTRUCTION JOINTS ARE ALLOWED IN THE LOAD DISTRIBUTION SLAB AT THE OPTION OF THE CONTRACTOR.
8. ALL CRACKS OVER 0.5 MM SHALL BE SEALED WITH A HIGH MOLECULAR WEIGHT METHYL METHACRYLATE SEALANT (TRANSPO T-70 OR EQUAL).
9. CONCRETE SHALL BE CLASS AA(AE) EXCEPT AS MODIFIED FOR f'c=27.5 MPa (4,000 PSI). NO CONSTRUCTION LOADS SHALL BE PERMITTED ON THE LOAD DISTRIBUTION SLAB UNTIL AFTER 7 DAYS OF CURING.

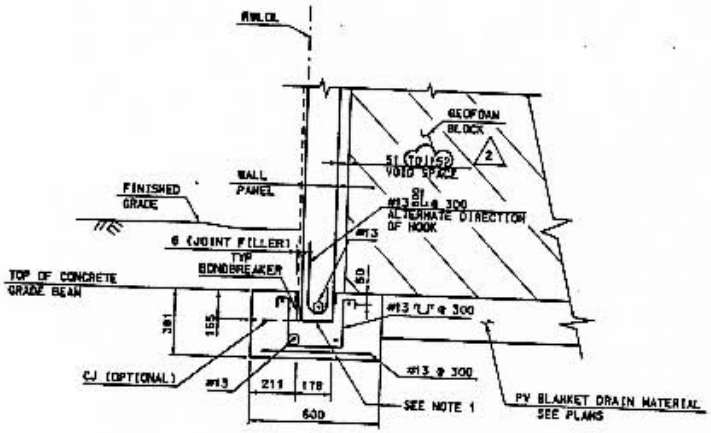
LICENSED PROFESSIONAL ENGINEER
No. 90-324487-2202
JOHN M. TERRY
STATE OF UTAH

APPROVED FOR CONSTRUCTION

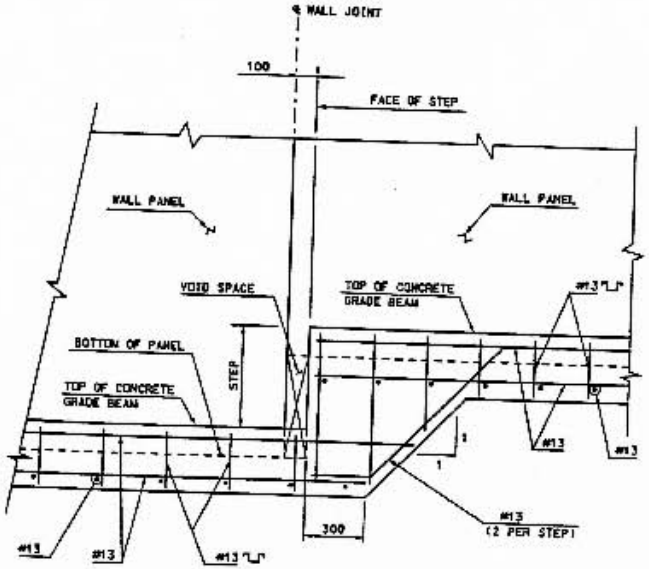
NO.	DATE	DESCRIPTION
1	12/24/97	RELEASE FOR GEOFOAM WALL ONLY.
2	1/27/98	APPROVED FOR CONSTRUCTION AT MIDDLE
3	04/10/98	ADD NOTES

UTAH DEPARTMENT OF TRANSPORTATION
SVERDRUP/DE LEUW

I-15 CORRIDOR RECONSTRUCTION
GEOFOAM WALL RESTRAINT DETAILS (8-11m)
CORRIDOR STANDARD PLAN
PROJECT NO. NHSP-15-7(135)296
SALT LAKE COUNTY
STR. NO. CS-40



GRADE BEAM/WALL PANEL SECTION (WALL HEIGHT 8m-11m) NTS 8/—



TYPICAL GRADE BEAM STEP DETAIL (WALL HEIGHT 8m-11m) NTS 1/—

NOTES:

1. SHIM BOTTOM OF WALL PANELS AS REQUIRED TO ALIGN VERTICAL WALL JOINTS.
2. ALL DIMENSION ARE IN MM UNLESS OTHERWISE NOTED.
3. FOR DETAILS UNDER APPROACH BRIDGES SEE CS-49

2 DELETED NOT APPROVED FOR CONSTRUCTION NOTE

WASATCH CONSTRUCTORS

MAR 2 0 1998

RELEASED FOR CONSTRUCTION

LICENSED PROFESSIONAL ENGINEER
No. 98-324447-2202
JOHN M. TERRY
STATE OF UTAH

APPROVED FOR CONSTRUCTION

NO.	DATE	DESCRIPTION
1	1/20/98	RELEASE FOR GEOFOAM WALL ONLY.
2	2/23/98	APPROVED FOR CONSTRUCTION AT BRIDGES

UTAH DEPARTMENT OF TRANSPORTATION

SVERDRUP/CE LEIM

I-15 CORRIDOR RECONSTRUCTION

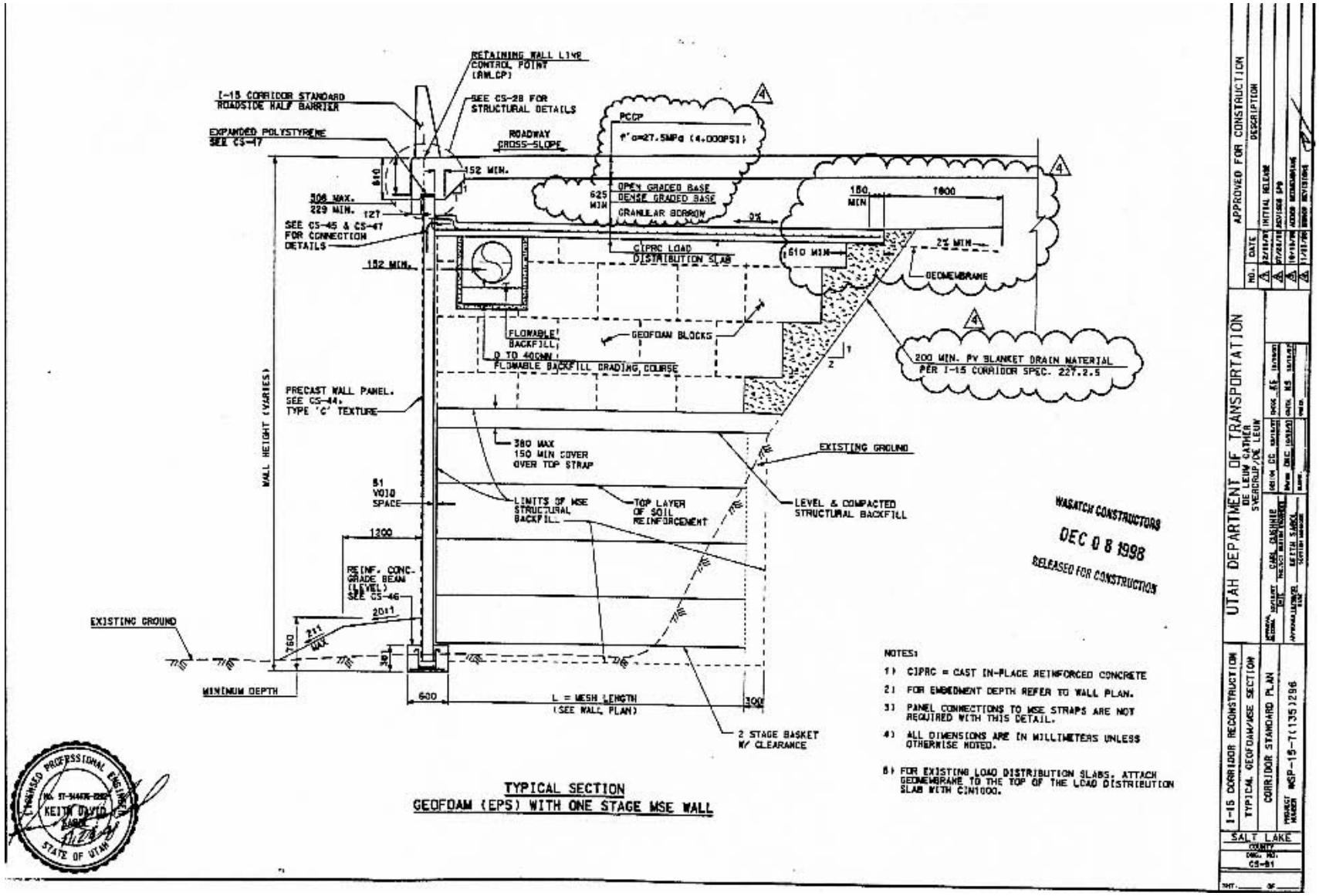
GEOFOAM WALL GRADE BEAM DETAILS (8-11m)

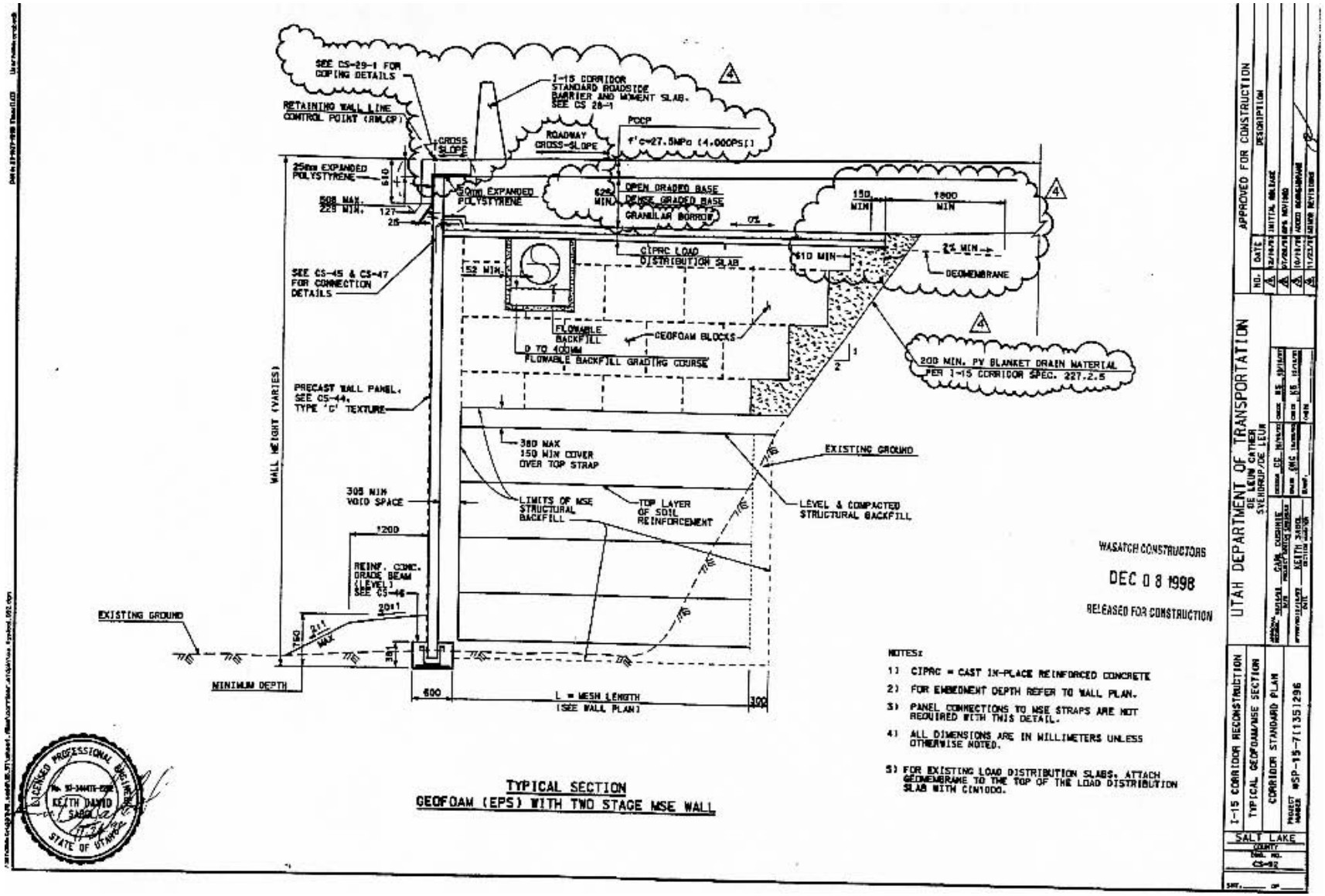
CORRIDOR STANDARD PLANS

PROJECT NUMBER S-I5-7(135)296

SALT LAKE COUNTY

DWG. NO. CS-61


RETAINING WALL LINE CONTROL POINT (RWLCP)
I-15 CORRIDOR STANDARD ROADSIDE HALF BARRIER
SEE CS-28 FOR STRUCTURAL DETAILS
EXPANDED POLYSTYRENE SEE CS-47
ROADWAY CROSS-SLOPE
PCCP
f'c=27.5MPa (4,000PSI)
152 MIN.
810
306 MAX.
229 MIN.
127
SEE CS-45 & CS-47 FOR CONNECTION DETAILS
625 MIN
OPEN GRADED BASE
DENSE GRADED BASE
GRANULAR BORROW
2%
150 MIN
1800
2% MIN
CIPRC LOAD DISTRIBUTION SLAB
610 MIN
GEOMEMBRANE
152 MIN.
FLOWABLE BACKFILL
0 TO 40CMM
FLOWABLE BACKFILL GRADING COURSE
GEOFOAM BLOCKS
200 MIN. PV BLANKET DRAIN MATERIAL PER I-15 CORRIDOR SPEC. 227.2.5
PRECAST WALL PANEL. SEE CS-44. TYPE 'C' TEXTURE
WALL HEIGHT (VARIES)
380 MAX
150 MIN COVER OVER TOP STRAP
EXISTING GROUND
51 VOID SPACE
LIMITS OF MSE STRUCTURAL BACKFILL
TOP LAYER OF SOIL REINFORCEMENT
LEVEL & COMPACTED STRUCTURAL BACKFILL
1300
REINF. CONC. GRADE BEAM (LEVEL) SEE CS-46
2011
EXISTING GROUND
760
211 MAX
30
MINIMUM DEPTH
600
L = MESH LENGTH (SEE WALL PLAN)
300
2 STAGE BASKET W/ CLEARANCE
WASATCH CONSTRUCTORS
DEC 0 8 1998
RELEASED FOR CONSTRUCTION
NOTES:
1) CIPRC = CAST IN-PLACE REINFORCED CONCRETE
2) FOR EMBEDMENT DEPTH REFER TO WALL PLAN.
3) PANEL CONNECTIONS TO MSE STRAPS ARE NOT REQUIRED WITH THIS DETAIL.
4) ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE NOTED.
5) FOR EXISTING LOAD DISTRIBUTION SLABS, ATTACH GEOMEMBRANE TO THE TOP OF THE LOAD DISTRIBUTION SLAB WITH CIN1000.
TYPICAL SECTION
GEOFOAM (EPS) WITH ONE STAGE MSE WALL
LICENSED PROFESSIONAL ENGINEER
KEITH DAVID
STATE OF UTAH
APPROVED FOR CONSTRUCTION
UTAH DEPARTMENT OF TRANSPORTATION
I-15 CORRIDOR RECONSTRUCTION
TYPICAL GEOFOAM/MSE SECTION
CORRIDOR STANDARD PLAN
SALT LAKE
CS-81



TYPICAL SECTION
GEOFOAM (EPS) WITH TWO STAGE MSE WALL

Project: North Abutment of the Route 1 Bridge over I-95 as part of the Woodrow Wilson Bridge Replacement Project, Alexandria, VA

Source: Virginia Department of Transportation

DESIGN FEATURES RELATING TO CONSTRUCTION OR TO REGULATION AND CONTROL OF TRAFFIC MAY BE SUBJECT TO CHANGE AS DEEMED NECESSARY BY THE DEPARTMENT

DATE						
12/07/04	1	95	0095-96A-106,C-501	24A(1)

GENERAL NOTES FOR EPS STRUCTURE

DESIGN

The design of expanded polystyrene (EPS) structures is in accordance with the following codes, standards and specifications.

Specifications:
Construction: Virginia Department of Transportation Road and Bridge Specifications, 2002.

Design: AASHTO Standard Specifications for Highway Bridges, 1996; 1997 and 1998 Interim Specifications; and VDOT Modifications.

Special Provisions for EPS - Block Fill Material and Geomembranes.

All designs are based on the Service Load Method (Allowable Stress Design) unless noted otherwise below:

Parapets and concrete barrier walls are designed by ultimate strength design (Load Factor Design - LFD).

Foundations are designed by ultimate strength design (Load Factor Design - LFD).

GENERAL

All dimensions shown indicate the final configuration after construction is complete. All dimensions shown on the drawings are the true plan horizontal projected dimensions unless noted.

Geotechnical data and discussion of subsurface conditions included in geotechnical reports are factual data only and are not a warranty of subsurface conditions. Boring logs depict data (especially groundwater conditions) at the specific location of the borings at the time made and may or may not represent subsurface conditions encountered during construction.

The accuracy and completeness of subsurface conditions cannot be guaranteed. The Contractor may make the appropriate field investigations to verify the subsurface conditions.

If found necessary and with the approval of the Engineer, foundations may be altered to suit conditions encountered during construction.

All unsuitable material within the limits of the foundations shall be removed and disposed as directed by the Engineer.

GEOMEMBRANE

The geomembrane used shall comply with and be installed in accordance with the Special Provision for Geomembranes.

EXPANDED POLYSTRENE - EPS BLOCKS

EPS material properties below are minimum allowable values representing the design and quality control parameters for individual test specimens and not for the block as a whole.

Compressive and flexural strength, minimum elastic limit stress and initial tangent young's modulus are based on minimum density, and are both used in analyses and design.

AASHTO MATERIAL TYPE	ELASTIC LIMIT STRESS psf	INITIAL TANGENT YOUNG'S MODULUS psf	FLEXURAL STRENGTH psf	COMPRESSIVE STRENGTH psf
EPS TO	10.2	1015	40	15

Layout depicted in drawings reflects EPS block with 8'x4'x2' length x width x height).

EPS material supplier may elect to provide blocks with alternate dimensions.

EPS material supplier shall prepare EPS shop drawings of block placement for review and approval by the Engineer prior to construction.

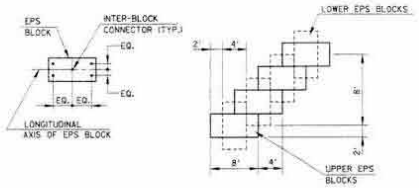
LAYOUT OF EPS BLOCKS

Layout of EPS blocks depicted in these drawings is representative of the finished layout. The Contractor is responsible to develop the specific layout to meet the requirements as indicated in the Typical Sections, Cross-sections, Wall 9 Details, B656 abutment details, and the guidelines indicated herein.

LAYOUT OF EPS BLOCKS (CONT'D)

EPS design shall be limited only to the use of EPS material Type EPS TO for all EPS - blocks fill structures.

The following guidelines shall be satisfied on all EPS - blocks:



Blocks shall be placed with their smallest dimension oriented perpendicular to roadway surface when viewed in elevation.

A minimum of two layers of block must always be used in all EPS - block fill structures.

Block must be placed in a pattern that results in minimal or no continuity of the vertical joints between blocks unless otherwise noted.

Longitudinal axes of EPS blocks within any layer shall be perpendicular to the longitudinal axes of the EPS block in the layers above and below.

To the extent possible, the uppermost layout of EPS blocks shall be oriented such that its long (maximum) dimension is transverse to traffic (i.e., perpendicular to the roadway alignment when viewed in plan).

The top surface of the assemblage of EPS blocks should always be parallel to the final pavement surface in the longitudinal direction. Thus any desired change in elevation (grade) along the roadway alignment must be accommodated by sloping the subgrade surface as necessary prior to placement of the first layer of EPS blocks and/or stepping the subgrade in conjunction with cutting the upper course of the EPS blocks.

All blocks must be placed in close direct contact against adjacent blocks on all sides.

Mechanical inter-block connectors (such as galvanized steel barbed plates) shall be placed on all horizontal surfaces between layers of EPS blocks to enhance the inter-block friction resistance to loads between blocks. A suggested layout of inter-block connectors for an 8'x4'x2' EPS block is as shown above.

SUGGESTED CONSTRUCTION SEQUENCE

The suggested construction sequence for the EPS embankment is as follows:

1) Grade and compact the area for sand layer placement
2) Install abutment piles (bridge B656) and Pier 8 (B635).
3) Install sand layer/wall strip footings
4) Install EPS embankment to the bottom of the load distribution slab (LDS). In the area of the abutment, install EPS to the bottom of abutment footing
5) Erect the abutment facing panels
6) Construct the abutment and embedded wall panel connection hardware
7) Continue EPS installation to bottom of the LDS
8) Erect the side wall panels
9) Place LDS reinforcing and concrete
10) Install utilities
11) Construct the pavement structure

CAST-IN-PLACE CONCRETE

Concrete in the barrier walls and parapets shall be Class A4.
Concrete in the load distribution slab shall be Class A4.

PRECAST WALL PANEL CONCRETE

Reinforced Option : Concrete shall be Class A5.
Prestressed Option: Concrete shall be Class A5.
Concrete shall have a minimum strength of 4000 psi at time of prestressing release.
Wall Panel Footings: Concrete shall be Class A4.

REINFORCING STEEL

Deformed reinforcing bars shall conform to ASTM A615, Grade 60.
All reinforcing bar dimensions on the detailed drawings are to center of bars except where otherwise noted and are subject to fabrication and construction tolerances.

PRESTRESSING STEEL

All prestressing strands shall be ½" dia. low relaxation, grade 270k and uncoated.
Jacking force per strand shall be 30,970 lbs.

CONSTRUCTION

The Contractor shall be responsible for the stability of the precast concrete wall sections during transportation, off-loading, storage and erection.

The final excavated subgrade below the EPS fill shall be proofrolled with four passes of a small to medium smooth-drum vibratory compactor (2,000 to 8,000 pounds at the drum). If unstable subgrade conditions develop, cease vibration and use a static roller. The final subgrade surface must be smooth and plane in accordance with the contract documents and parallel to the final top of EPS surface shown on the plans.

Lightweight equipment shall be utilized when placing embankment fill on the EPS sideslopes.

ESTIMATED QUANTITIES

ITEM	UNITS	QUANTITY
Load Distribution Slab (6" depth) *	SY	4,667
Precast Wall Panels **	SF	4,964
Barrier Wall Parapet ***	LF	44
EPS Fill	CY	20,240
Grade 8 Fine Aggregate	TON	3,091
Geomembrane	SY	2,045
Select Material, Type I, min. CBR-30	TON	84
Lightweight Aggregate Backfill	CY	1,196

* Measurement for payment is based on the plan area of the LDS. The price bid shall include all LDS steps and perimeter curbs. Also included shall be all costs in connection with providing formwork; furnishing, placing and curing concrete; placing reinforcing steel; furnishing and placing hardware for connection to wall panel; accommodating drainage system components; placing 1" mastic for expansion at catch basins; sealing cracks with high molecular weight methyl methacrylate; and all other work required to install Load Distribution Slab.

** Measurement for payment is based on the area of the walls from the top of the grade beam to the top of coping. The price bid shall include all costs for all materials, labor and equipment required or incidental to precast or precast/prestressed panel system. The payment shall include providing the specified finish, delivery, installation of the wall panels, all reinforcing steel and the connection of the Load Distribution Slab to the abutment of bridge B656, the integral coping at the top of the panel, and the grade beam. Payment shall also include placement of expanded polystyrene between Wall Panel and Integral coping slab as shown on the plans.

*** Shall include all costs in connection with providing formwork, furnishing, placing, finishing and curing concrete, placing reinforcing steel and all other work required to install the Barrier Wall Parapet.

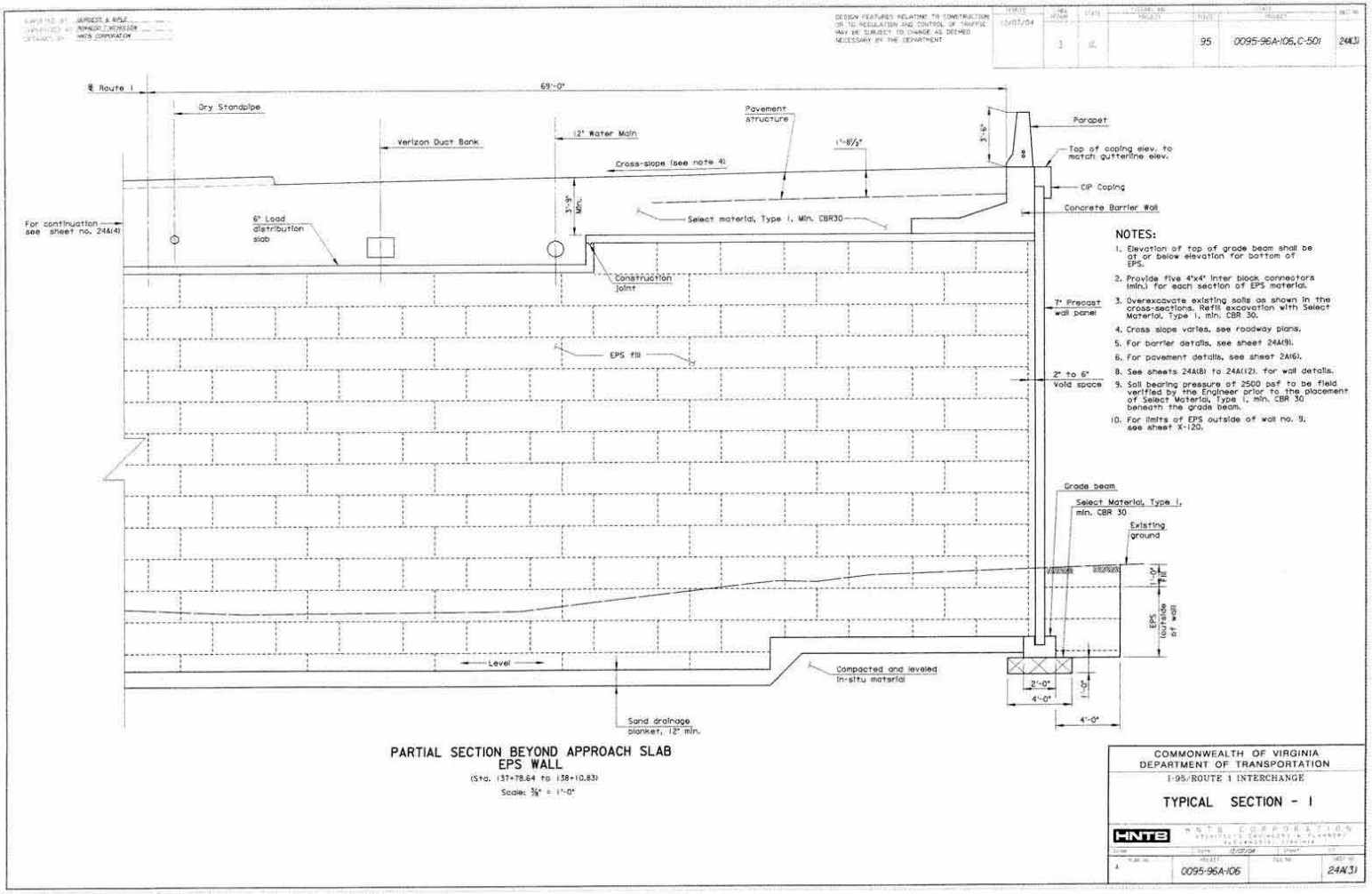
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION

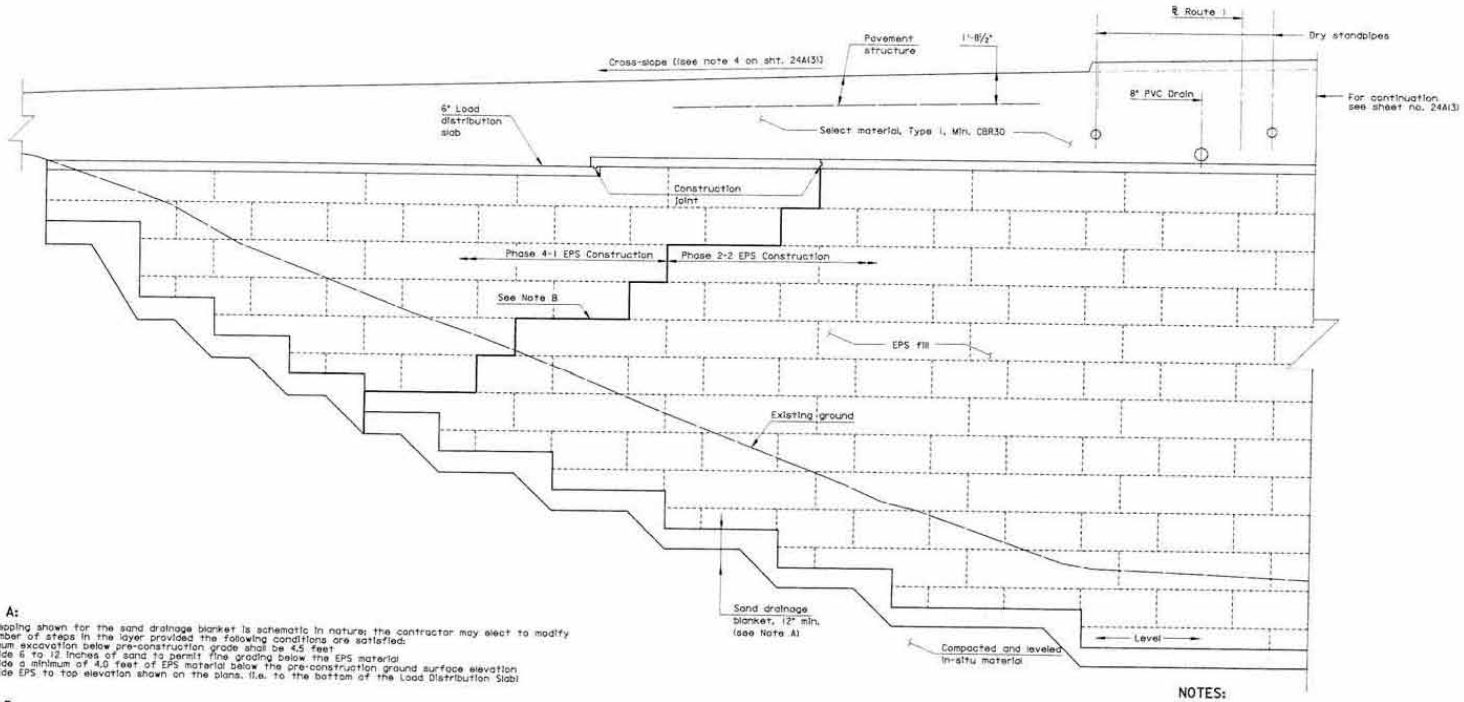
I-95/ROUTE 1 INTERCHANGE

GENERAL NOTES AND ESTIMATED QUANTITIES

HNTB

1	0095-96A-106		24A(1)


℄ Route 1
69'-0"
Dry Standpipe
Verizon Duct Bank
12" Water Main
Pavement structure
1'-8½"
3'-6"
Parapet
Top of coping elev. to match gutterline elev.
Cross-slope (see note 4)
CIP Coping
Concrete Barrier Wall
3'-9" Min.
Select material, Type I, Min. CBR30
For continuation see sheet no. 24A(4)
6" Load distribution slab
Construction joint
7" Precast wall panel
EPS fill
2" to 6" Void space
Grade beam
Select Material, Type I, min. CBR 30
Existing ground
1'-0" Fill
EPS (outside of wall)
Level
Compacted and leveled in-situ material
2'-0"
4'-0"
1'-0"
4'-0"
Sand drainage blanket, 12" min.
NOTES:
1. Elevation of top of grade beam shall be at or below elevation for bottom of EPS.
2. Provide five 4"x4" inter block connectors (min.) for each section of EPS material.
3. Overexcavate existing soils as shown in the cross-sections. Refill excavation with Select Material, Type I, min. CBR 30.
4. Cross slope varies, see roadway plans.
5. For barrier details, see sheet 24A(9).
6. For pavement details, see sheet 24A(6).
8. See sheets 24A(8) to 24A(12), for wall details.
9. Soil bearing pressure of 2500 psf to be field verified by the Engineer prior to the placement of Select Material, Type I, min. CBR 30 beneath the grade beam.
10. For limits of EPS outside of wall no. 9, see sheet X-120.
PARTIAL SECTION BEYOND APPROACH SLAB
EPS WALL
(Sta. 137+78.64 to 138+10.83)
Scale: ⅜" = 1'-0"
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
I-95/ROUTE 1 INTERCHANGE
TYPICAL SECTION - I
HNTB
0095-96A-106
24A(3)



NOTE A:

The stepping shown for the sand drainage blanket is schematic in nature; the contractor may elect to modify the number of steps in the layer provided the following conditions are satisfied:

1. Minimum excavation below pre-construction grade shall be 4.5 feet
2. Provide 6 to 12 inches of sand to permit fine grading below the EPS material
3. Provide a minimum of 4.0 feet of EPS material below the pre-construction ground surface elevation
4. Provide EPS to top elevation shown on the plans. (i.e. to the bottom of the Load Distribution Slab)

NOTE B:

The exposed edge of the EPS placed in Phase 2-2 shall be protected by a temporary membrane meeting the requirements of the Geomembrane Special Provisions and a minimum of 2' of fill. Prior to commencing with the Phase 4-4 EPS construction, the fill and temporary membrane shall be removed from the "stepped edges" of the Phase 2-2 EPS fill. All debris shall be removed from the exposed EPS prior to commencing with the Phase 4-1 EPS placement.

PARTIAL SECTION
EPS FILL
(Sta. 137+50(±) to Sta. 141+50)
Scale: 3/8" = 1'-0"

NOTES:

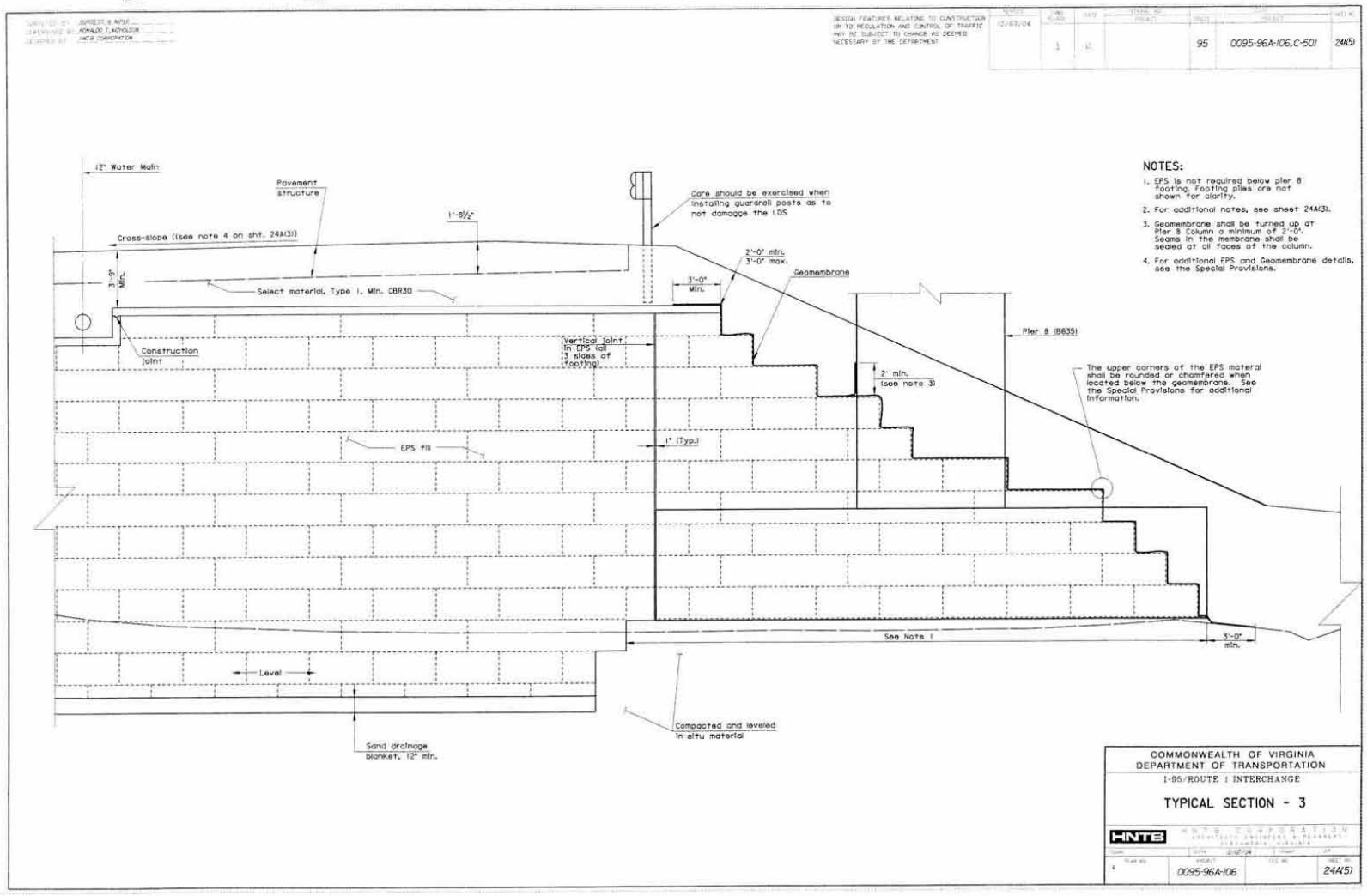
1. See sheet no. 24A(3), for notes.

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
I-95/ROUTE 1 INTERCHANGE

TYPICAL SECTION - 2

HNTB

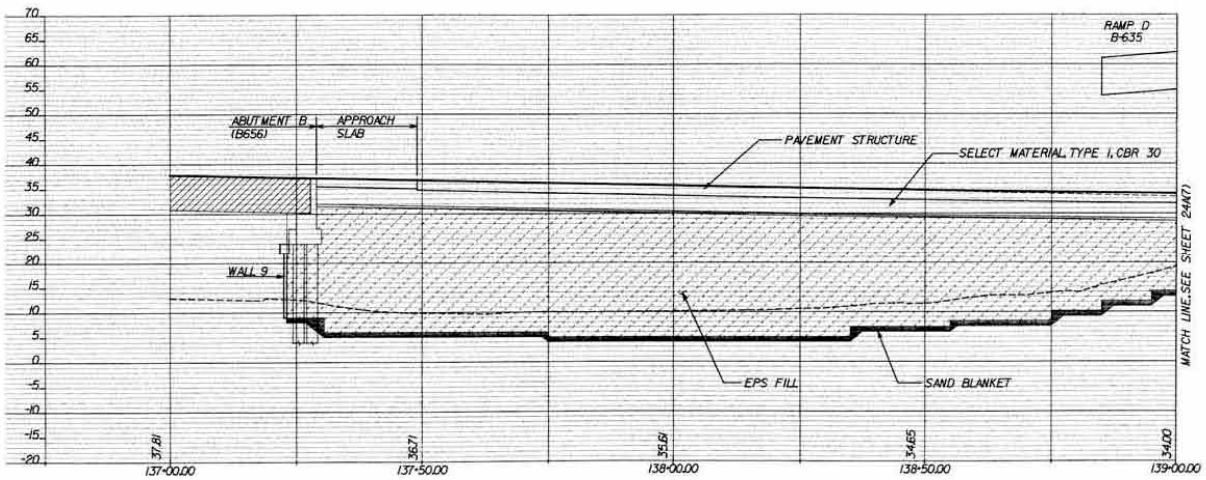
0095-96A-106			24A(4)


12" Water Main
Pavement structure
1'-8½"
Cross-slope (see note 4 on sht. 24A(3))
3'-9" Min.
Select material, Type I, Min. CBR30
Care should be exercised when installing guardrail posts as to not damage the LDS
2'-0" min.
3'-0" max.
Geomembrane
3'-0" Min.
Pier 8 (B635)
Construction joint
Vertical joint in EPS (at 3 sides of footing)
2' min. (see note 3)
The upper corners of the EPS material shall be rounded or chamfered when located below the geomembrane. See the Special Provisions for additional information.
EPS fill
1" (Typ.)
See Note 1
3'-0" min.
Level
Compacted and leveled in-situ material
Sand drainage blanket, 12" min.
NOTES:
1. EPS is not required below pier 8 footing. Footing piles are not shown for clarity.
2. For additional notes, see sheet 24A(3).
3. Geomembrane shall be turned up at Pier 8 Column a minimum of 2'-0". Seams in the membrane shall be sealed at all faces of the column.
4. For additional EPS and Geomembrane details, see the Special Provisions.
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
I-95/ROUTE 1 INTERCHANGE
TYPICAL SECTION - 3
HNTB
0095-96A-106
24A(5)

SURVEYED BY: BURGESS & NIPLE
SUPERVISED BY: FERALDO T. NICHOLSON
DESIGNED BY: HNTB CORPORATION

DESIGN FEATURES RELATING TO CONSTRUCTION OR TO REGULATION AND CONTROL OF TRAFFIC MAY BE SUBJECT TO CHANGE AS DEEMED NECESSARY BY THE DEPARTMENT

REVISED	FHWA REGION	STATE	FEDERAL AID PROJECT	STATE ROUTE	STATE PROJECT	SHEET NO
12/07/04	1	VA		95	0095-96A-106, C-501	24A(6)



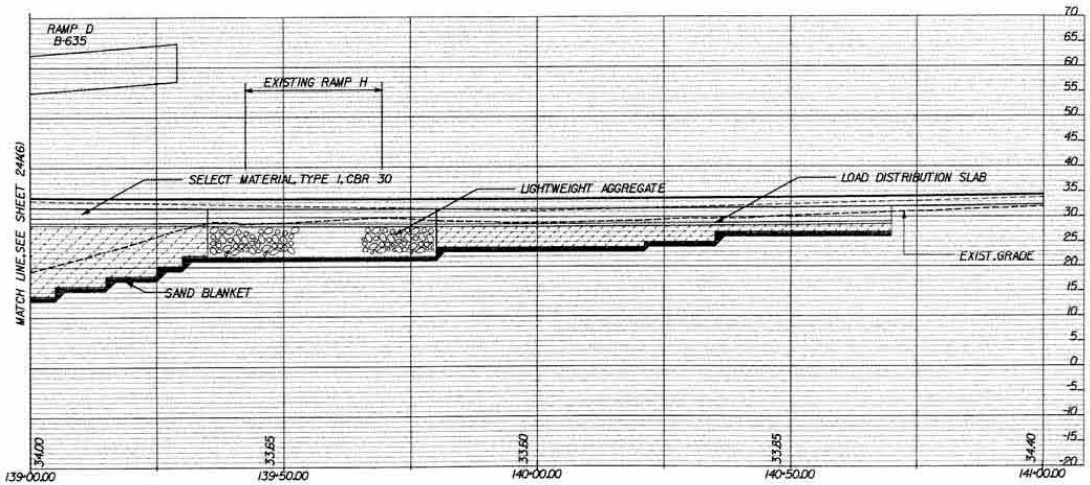
LONGITUDINAL SECTION-1
ROUTE 1
STA. 137+00.00 TO STA. 139+00.00
HORZ.: 1"=10'-0" VERT.: 1"=10'-0"

PLAN NO.	PROJECT	FILE NO.	SHEET NO.
	0095-96A-106		24A(6)

SURVEYED BY BURGESS & NIPLE
SUPERVISED BY RONALDO T. MICHALSKI
DESIGNED BY HATB CORPORATION

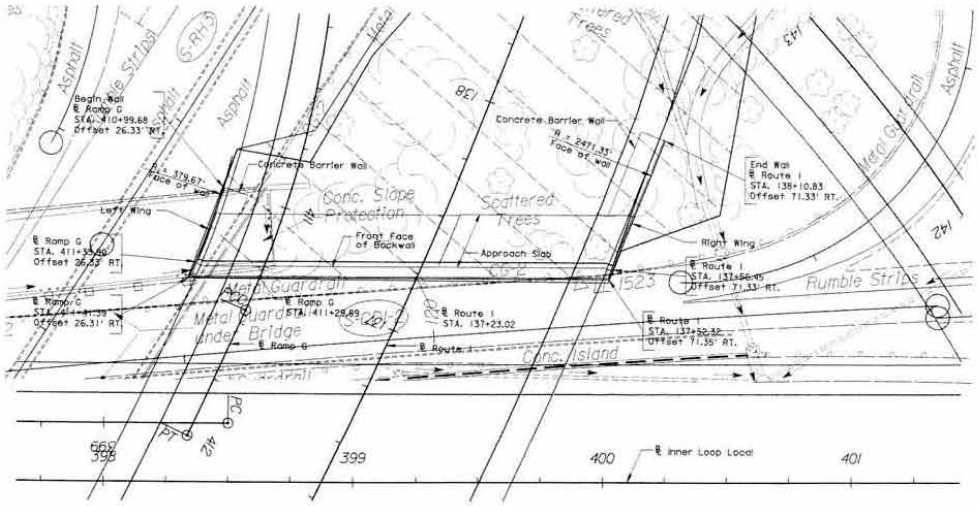
DESIGN FEATURES RELATING TO CONSTRUCTION OR TO REGULATION AND CONTROL OF TRAFFIC MAY BE SUBJECT TO CHANGE AS DEEMED NECESSARY BY THE DEPARTMENT

REVISED	FHWA REGION	STATE	FEDERAL AID PROJECT	STATE ROUTE	STATE PROJECT	SHEET NO.
12/27/04	3	VA.		95	0095-96A-106, C-501	24A71



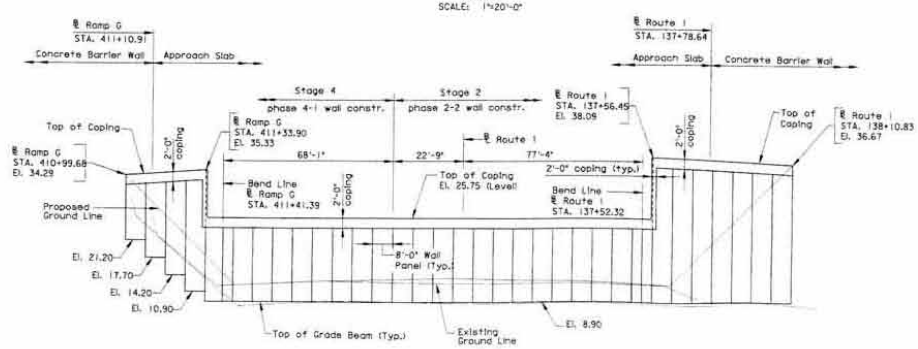
LONGITUDINAL SECTION-2
ROUTE 1
STA. 139+00.00 TO STA. 141+00.00
HORIZ. 1"=10'-0" VERT. 1"=10'-0"

PLAN NO.	PROJECT	FILE NO.	SHEET NO.
A	0095-96A-106		24A71



PLAN

SCALE: 1"=20'-0"



DEVELOPED ELEVATION

HORIZ. SCALE: 1"=20'-0"
VERT. SCALE: 1"=10'-0"

Notes:

1. Soil bearing pressure of 2500 psf to be field verified by the Engineer prior to the placement of Select Material, Type I, min. CBR 30 beneath the grade beam.
2. See Roadway Plans for Roadway Geometry.
3. Offset dimensions are to front face of wall panels.
4. Two spare conduits, 2" dia., are to be placed in barrier.
5. Lighting conduit in barrier. See lighting plans 321(A-35) and B656 plans 491(-234).

Limits of EPS and excavation

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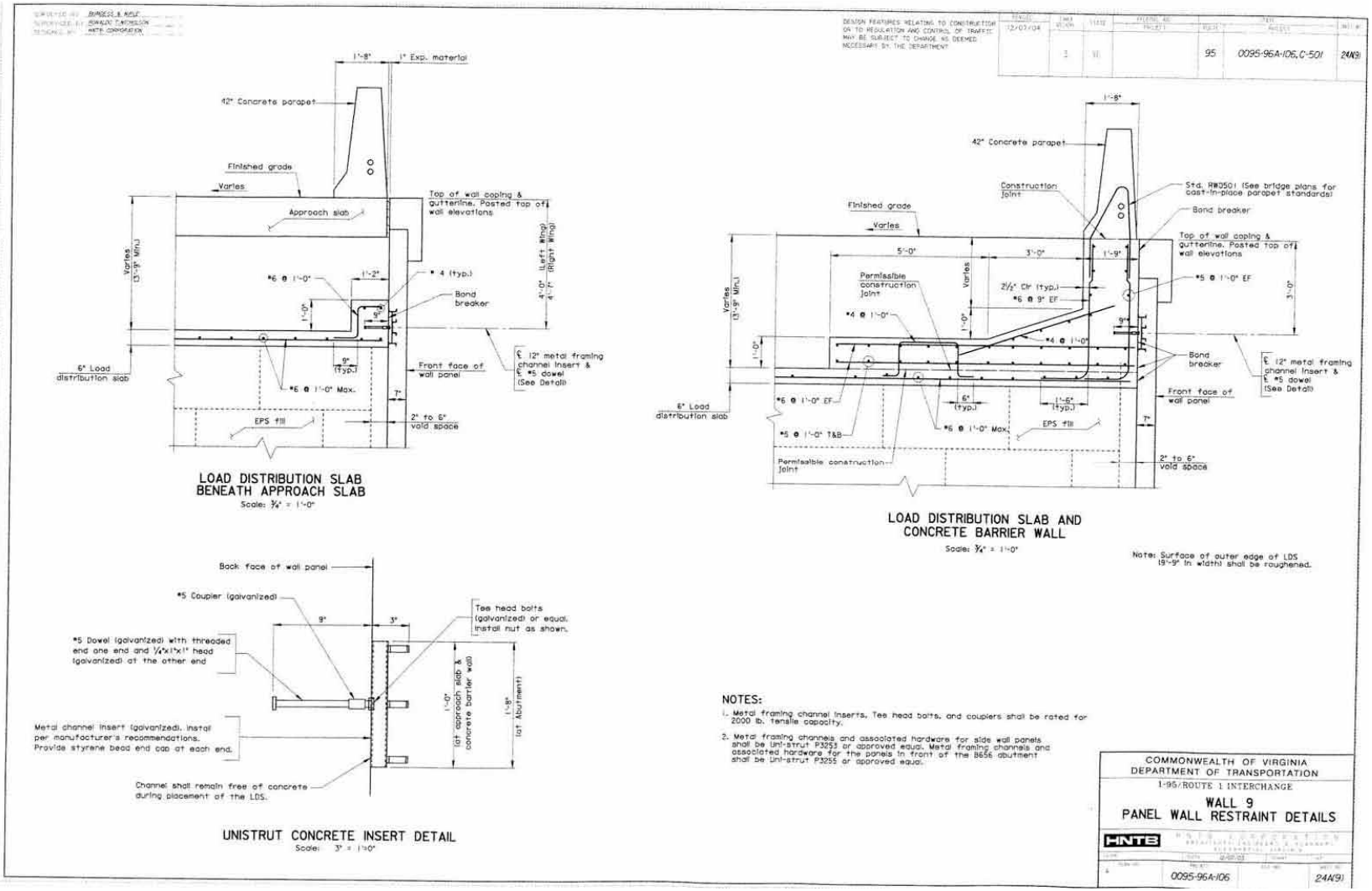
DEPARTMENT OF TRANSPORTATION

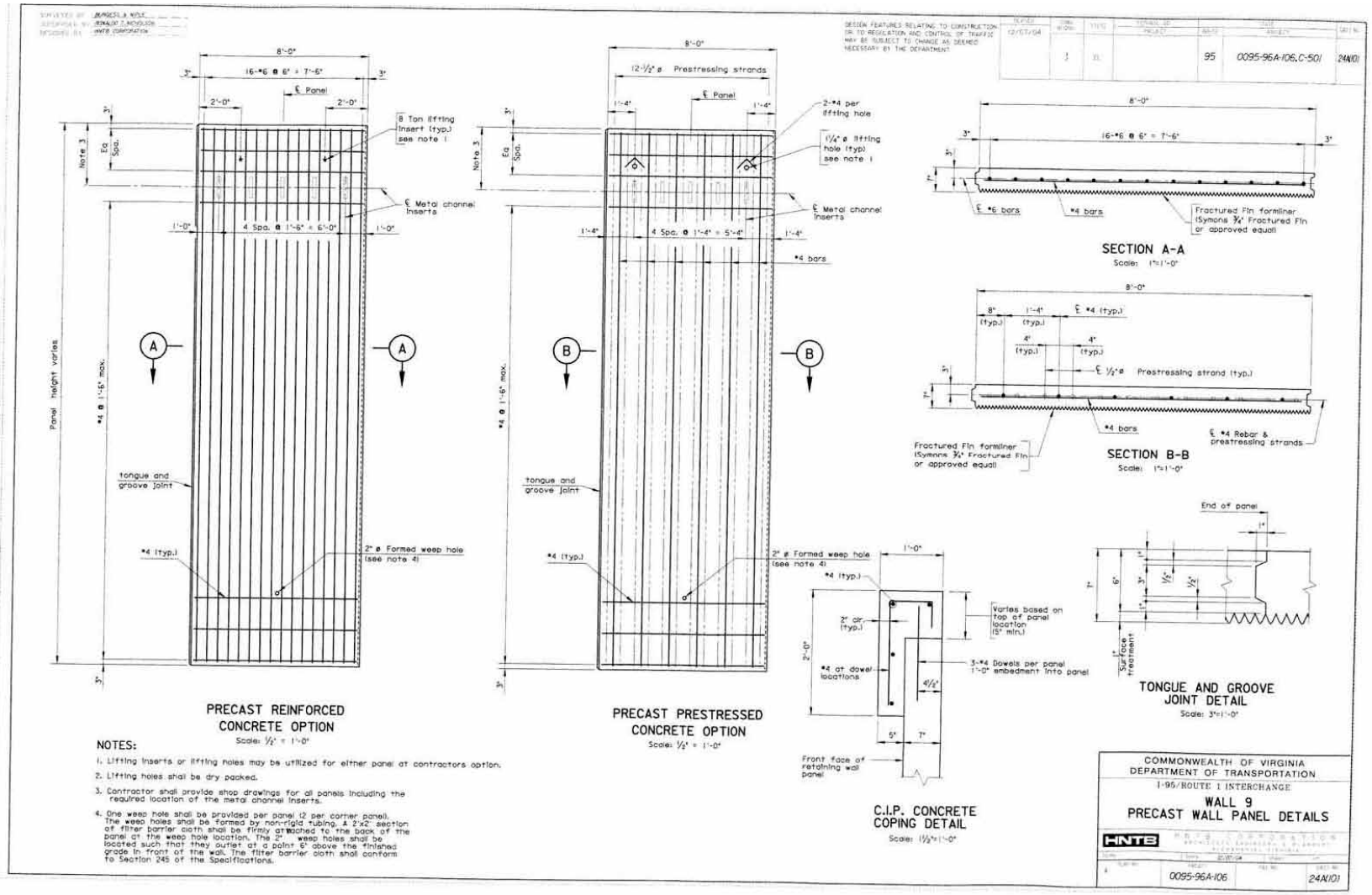
I-95/ROUTE 1 INTERCHANGE

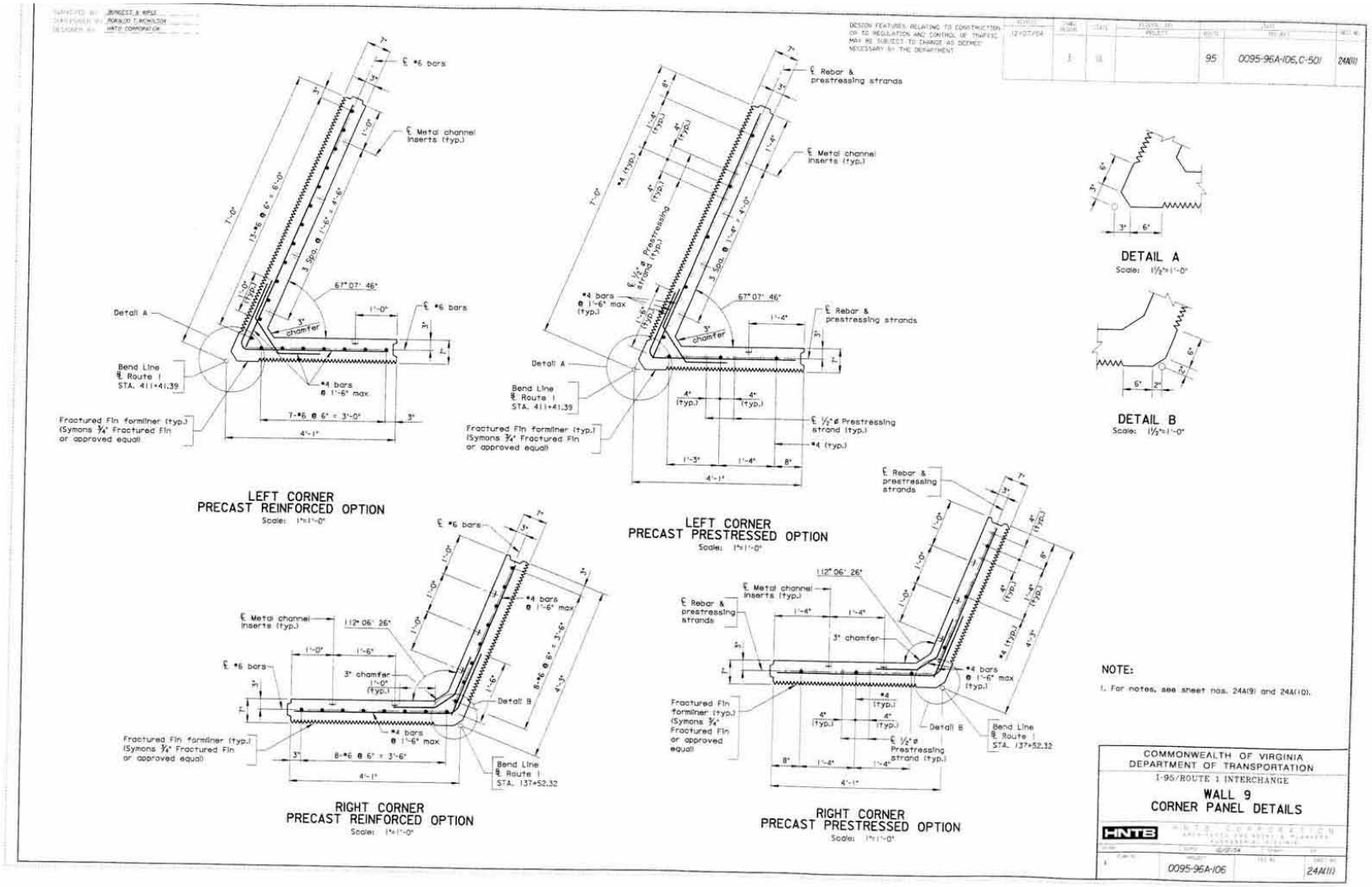
WALL 9 PLAN AND ELEVATION

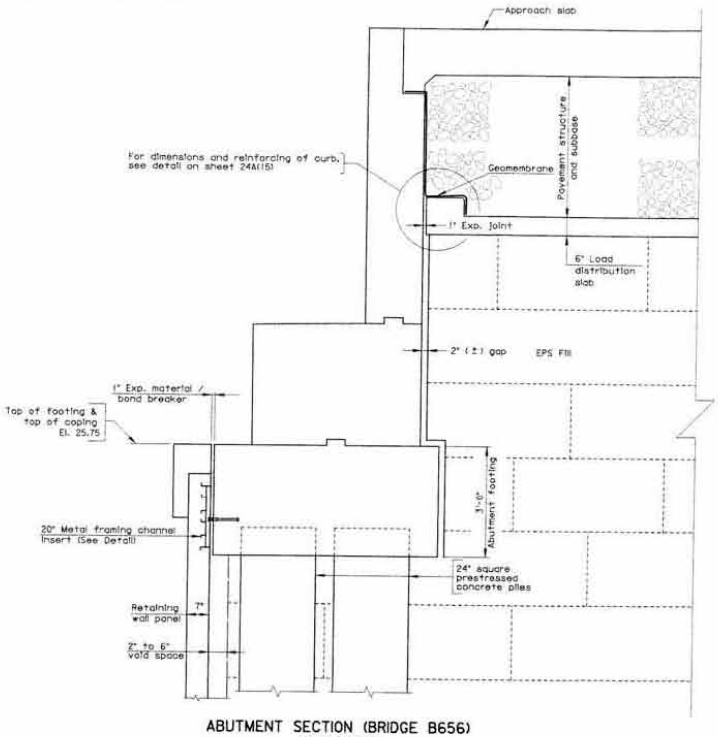
HNTB

0095-96A-106 | 24A(8)

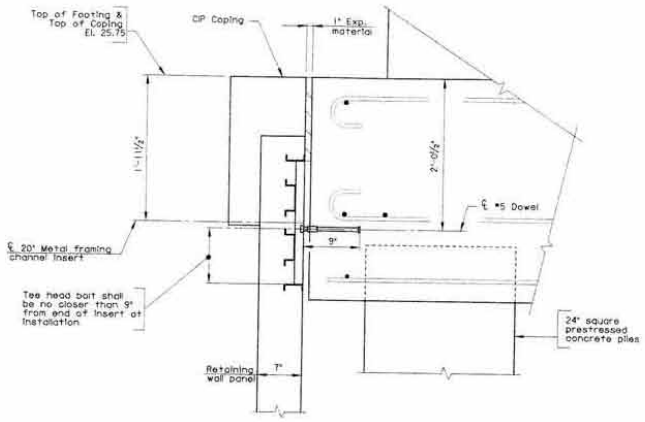

LOAD DISTRIBUTION SLAB BENEATH APPROACH SLAB
Scale: 3/4" = 1'-0"
LOAD DISTRIBUTION SLAB AND CONCRETE BARRIER WALL
Scale: 3/4" = 1'-0"
UNISTRUT CONCRETE INSERT DETAIL
Scale: 3" = 1'-0"
Note: Surface of outer edge of LDS (9'-9" in width) shall be roughened.
NOTES:
1. Metal framing channel inserts, Tee head bolts, and couplers shall be rated for 2000 lb. tensile capacity.
2. Metal framing channels and associated hardware for side wall panels shall be Uni-strut P3253 or approved equal. Metal framing channels and associated hardware for the panels in front of the B656 abutment shall be Uni-strut P3255 or approved equal.
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
I-95/ROUTE 1 INTERCHANGE
WALL 9
PANEL WALL RESTRAINT DETAILS
0095-96A-106
24A(9)


PRECAST REINFORCED CONCRETE OPTION
Scale: 1/2" = 1'-0"
PRECAST PRESTRESSED CONCRETE OPTION
Scale: 1/2" = 1'-0"
SECTION A-A
Scale: 1"=1'-0"
SECTION B-B
Scale: 1"=1'-0"
TONGUE AND GROOVE JOINT DETAIL
Scale: 3"=1'-0"
C.I.P. CONCRETE COPING DETAIL
Scale: 1 1/2"=1'-0"
NOTES:
1. Lifting inserts or lifting holes may be utilized for either panel at contractors option.
2. Lifting holes shall be dry packed.
3. Contractor shall provide shop drawings for all panels including the required location of the metal channel inserts.
4. One weep hole shall be provided per panel (2 per corner panel). The weep holes shall be formed by non-rigid tubing. A 2"x2" section of filter barrier cloth shall be firmly attached to the back of the panel at the weep hole location. The 2" weep holes shall be located such that they outlet at a point 6" above the finished grade in front of the wall. The filter barrier cloth shall conform to Section 245 of the Specifications.
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
I-95/ROUTE 1 INTERCHANGE
WALL 9
PRECAST WALL PANEL DETAILS
HNTB
0095-96A-106
24A101


LEFT CORNER
PRECAST REINFORCED OPTION
Scale: 1"=1'-0"
LEFT CORNER
PRECAST PRESTRESSED OPTION
Scale: 1"=1'-0"
RIGHT CORNER
PRECAST REINFORCED OPTION
Scale: 1"=1'-0"
RIGHT CORNER
PRECAST PRESTRESSED OPTION
Scale: 1"=1'-0"
DETAIL A
Scale: 1½"=1'-0"
DETAIL B
Scale: 1½"=1'-0"
NOTE:
1. For notes, see sheet nos. 24A(9) and 24A(10).
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
I-95/ROUTE 1 INTERCHANGE
WALL 9
CORNER PANEL DETAILS
HNTB
0095-96A-106
24A(11)



ABUTMENT SECTION (BRIDGE B656)
Scale: 3/8"=1'-0"



CONCRETE INSERT DETAIL
@ ABUTMENT B (B656)
Scale: 1½"=1'-0"

NOTES:

1. For concrete insert details and notes, see sheet no. 24A(9).
2. For typical panel details, see sheet no. 24A(10).
3. For corner panel details, see sheet no. 24A(11).
4. For CIP coping details, see sheet no. 24A(10).
5. For additional details, see the bridge plans.
6. For details of EPS at the 24" prestressed piles, see sheet no. 49(184).

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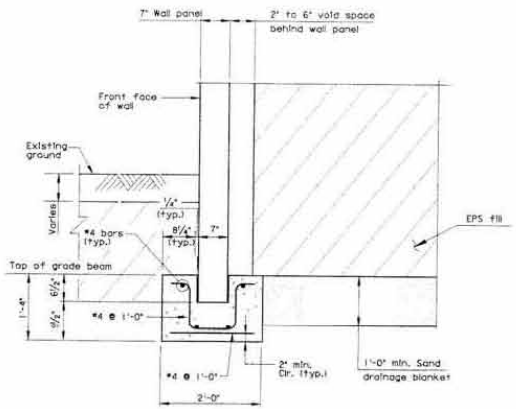
DEPARTMENT OF TRANSPORTATION
I-95/ROUTE 1 INTERCHANGE
PRECAST WALL PANEL CONNECTION
@ ABUTMENT B (B656)

HNTB

0095-96A-106			24A(12)

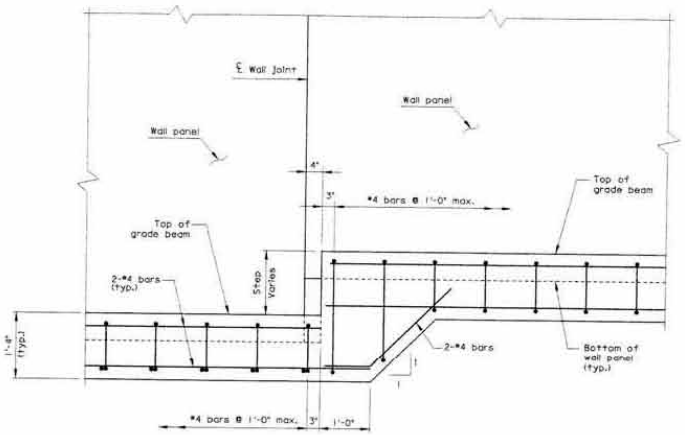
DESIGN FEATURES RELATING TO CONSTRUCTION OR TO REGULATION AND CONTROL OF TRAFFIC MAY BE SUBJECT TO CHANGE AS DEEMED NECESSARY BY THE DEPARTMENT

DATE	SHEET NO.	SCALE	FEDERAL NO. / PROJECT	PROJECT	PROJECT	SHEET NO.
12/07/04	3	NS		95	0095-96A-106,C-501	24A(3)



TYPICAL GRADE BEAM SECTION

Scale: 1"=1'-0"



TYPICAL GRADE BEAM STEP

Scale: 1"=1'-0"

NOTES:

1. Shim bottom of wall panels as required to align vertical wall joints. Shims are only required to level and align panels until LDS is poured. Once panels are set and connected with the LDS the shims are no longer required.
2. Grade beam base course is not shown for clarity.

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION

I-95/ROUTE 1 INTERCHANGE

WALL 9
GRADE BEAM DETAILS

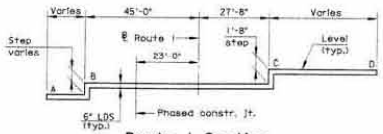
HNTB

DATE	SHEET NO.	FEDERAL NO. / PROJECT	SHEET NO.
12/07/04	0095-96A-106		24A(3)

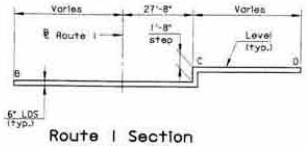
DESIGN FEATURES RELATING TO CONSTRUCTION UP TO REGULATION AND CONTROL OF TRAFFIC MAY BE SUBJECT TO CHANGE AS DEEMED NECESSARY BY THE DEPARTMENT

PROJECT	FUNC. CLASS	YEAR	TOTAL NO. PROJECT	SHEET	PROJECT	REV. NO.
12-0770*	I	14		95	0095-96A-106,C-501	24A(14)

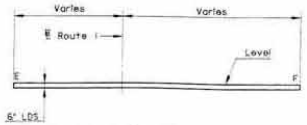
SECTION	BASELINE	STATION	OFFSET (FT.)						ELEVATION (FT.)			
			A	B	C	D	E	F	A-B	B-C	C-D	E-F
1	Route 1	137+10.24	-83.50	-45.00	—	—	—	—	30.55	32.57	—	—
2	Route 1	137+26.51	-84.67	-45.00	-7.36	—	—	—	30.43	32.21	—	—
3	Route 1	137+50.00	-63.00	-45.00	27.67	20.02	—	—	30.25	31.69	33.36	—
4	Route 1	138+00.00	-60.00	-45.00	27.67	70.75	—	—	29.87	30.59	32.26	—
5	Route 1	138+50.00	—	-45.00	27.67	75.00	—	—	—	29.64	31.30	—
6	Route 1	139+00.00	—	-28.00	27.67	65.00	—	—	—	28.99	30.65	—
7	Route 1	139+50.00	—	-1.43	27.67	63.00	—	—	—	28.64	30.30	—
8	Route 1	140+00.00	—	-12.00	27.67	63.00	—	—	—	28.58	30.25	—
9	Route 1	140+46.74	—	-3.00	27.67	61.44	—	—	—	28.81	30.48	—
10	Route 1	141+00.00	—	—	—	—	7.00	66.00	—	—	—	28.98
11	Route 1	141+50.00	—	—	—	—	7.00	58.75	—	—	—	29.88
12	Ramp B	437+50.00	—	—	—	—	2.32	9.00	—	—	—	23.80
13	Ramp B	437+75.00	—	—	—	—	-13.24	9.00	—	—	—	25.08
14	Ramp B	438+00.00	—	—	—	—	-25.00	9.00	—	—	—	26.37
15	Ramp B	438+25.00	—	—	—	—	-25.00	9.00	—	—	—	27.54
16	Ramp B	438+48.26	—	—	—	—	-20.14	9.00	—	—	—	28.43



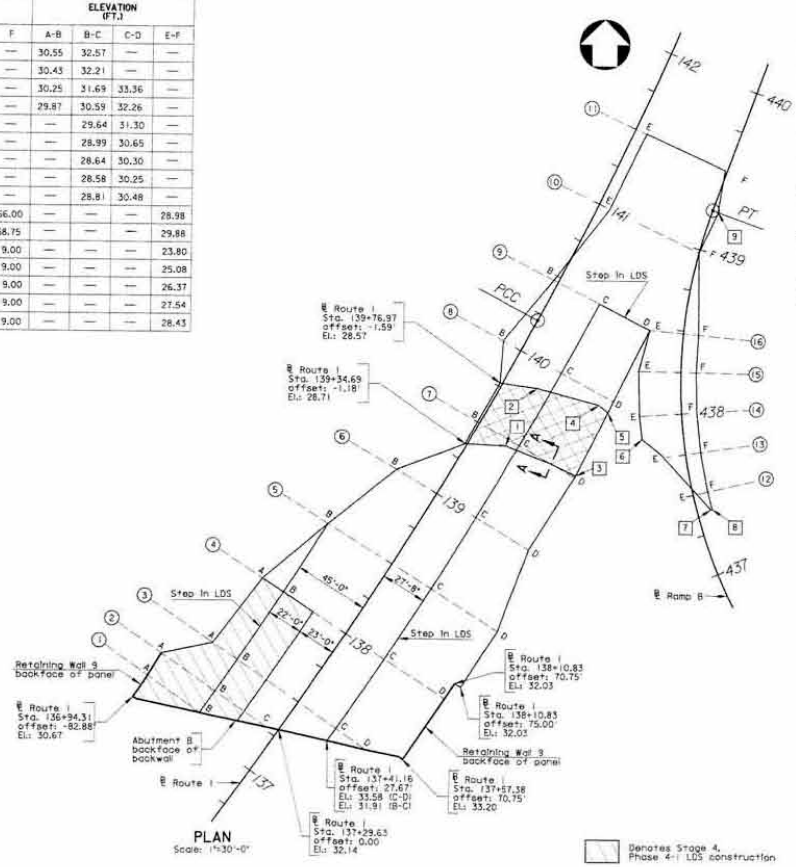
Route 1 Section
(Sta. 137+41.16 to Sta. 138+50)
Scale: NTS



Route 1 Section
(138+50 to Sta. 140+46.74)
Scale: NTS



Route 1 Section
(140+46.74 to Sta. 141+50)
Scale: NTS



PLAN
Scale: 1"=30'-0"

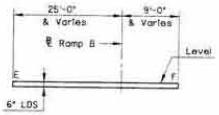
Denotes Stage 4, Phase 4-1 LDS construction

Denotes Stage 3, Phase 3-3 LDS construction

All other LDS construction is in Phase 2.

NOTES:

1. Sections 1 thru 11 are radial to Route 1 Baseline.
2. Sections 12 thru 16 are radial to Ramp B Baseline.
3. Negative offsets are measured to the left of designated Baseline looking upstation.
4. Wheel loads are not permitted within five ft. of free edge of load distribution slab.
5. Transverse construction joints are allowed in the load distribution slab at the option of the Contractor.
6. No construction loads shall be permitted on the load distribution slab until after 7 days of curing.
7. The contractor shall submit a pour sequence for the slab to the Engineer for approval a minimum of twenty working days prior to construction of the LDS.
8. All construction joints and cracks over 0.02" in width shall be sealed with a high molecular weight methyl methacrylate sealant prior to placement of the pavement structure/subbase.
9. For LDS details, see sheet 24A(15).
10. For LDS interface at B656 abutment, see sheet no. 24A(12).
11. For LDS details at walls, see sheet no. 24A(9).
12. For Section A-A, see sheet no. 24A(15).



Ramp B Section
(Sta. 437+38.80 to Sta. 438+48.26)
Scale: NTS

1	℄ Route 1 Sta. 139+45.92 offset: 20.89' El.: 28.65	6	℄ Ramp B Sta. 437+86.72 offset: -24.72' El.: 25.69
2	℄ Route 1 Sta. 139+85.11 offset: 19.60' El.: 28.57	7	℄ Ramp B Sta. 437+38.80 offset: 8.23' El.: 23.22
3	℄ Route 1 Sta. 139+50.82 offset: 62.99' El.: 30.30	8	℄ Ramp B Sta. 437+38.80 offset: 9.25' El.: 23.22
4	℄ Route 1 Sta. 139+94.06 offset: 56.25' El.: 30.24	9	℄ Route 1 Sta. 141+25.15 offset: 65.85' El.: 29.40
5	℄ Route 1 Sta. 139+92.93 offset: 62.94' El.: 30.24		

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION

I-95/ROUTE 1 INTERCHANGE

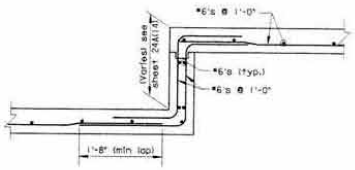
LOAD DISTRIBUTION SLAB (LDS) PLAN

HNTB

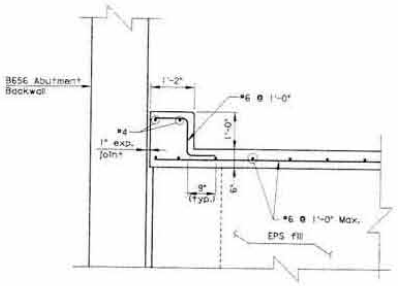
0095-96A-106			24A(14)

DESIGN FEATURES RELATING TO CONSTRUCTION OR TO REGULATION AND CONTROL OF TRAFFIC MAY BE SUBJECT TO CHANGE AS DEEMED NECESSARY BY THE DEPARTMENT

REVISED	FHWA APPROVAL		TOTAL SHEETS	SHEET	PROJECT	SHEET NO.
12/07/04	3	16		95	0095-96A-106,C-501	24A(15)

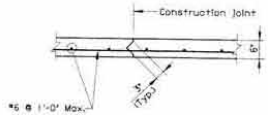


TYPICAL LDS STEP DETAIL
Scale: 1"=1'-0"

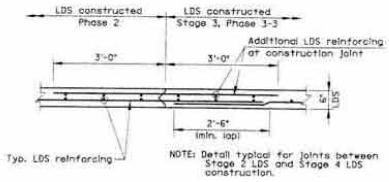


LDS CURB DETAIL AT B656 BACKWALL
Scale: 3/4"=1'-0"

Note: For Abutment Details, see sheet 24A(12) and B656 bridge plans.



TYPICAL LDS CONSTRUCTION JOINT DETAIL
Scale: 3/4"=1'-0"



SECTION A-A
Scale: 3/4"=1'-0"

(Note: Mechanical splices may be used at the construction joints in lieu of lap splices in the reinforcing steel.)

NOTES:

1. Typical reinforcing in the LDS consists of *6's @ 1'-0" o/c in each direction. At edges of slab, splay the reinforcing to follow the edge of slab. The minimum bar spacing shall be 3" and the maximum bar spacing shall be 6".
2. For details of reinforcing at drainage inlets, see sheet 24A(16).
3. The minimum lap length between the *6 bars shall be 2'-6".
4. The minimum lap length between the *4 bars shall be 1'-8".

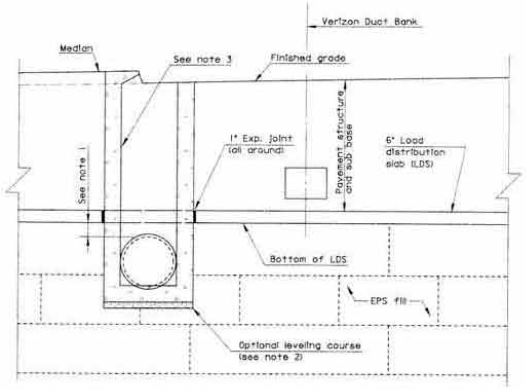
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION

I-95/ROUTE 1 INTERCHANGE

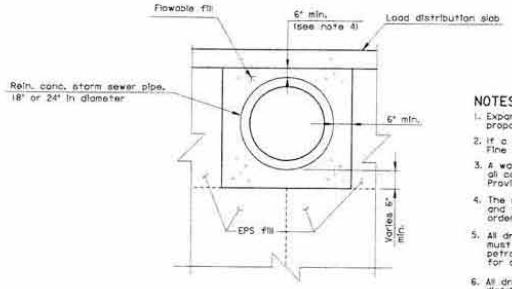
LOAD DISTRIBUTION SLAB (LDS) DETAILS

HNTB

12/07/04			
0095-96A-106			24A(15)



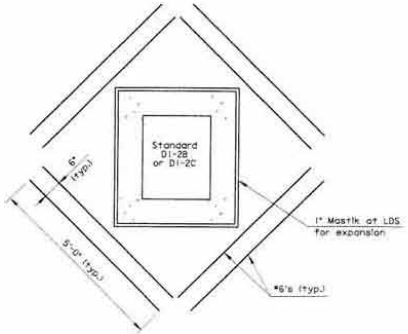
SECTION THROUGH STANDARD DI-2B
Scale: ½" = 1'-0"



TRENCH SECTION IN EPS
Scale: ¾" = 1'-0"

NOTES:

1. Expanded polystyrene foam shall provided between LDS proposed pipes as shown 16" min. thickness).
2. If a precast box is used, a 3" thick leveling course consisting of Fine Aggregate, Grade B Sand is required.
3. A waterproofing system shall be used to coat walls & floors of all catch basins located in the geofoam areas. See Special Provisions for additional details.
4. The minimum clearance between the outside of the RCP and the EPS is 6". The pipes must be supported in order to allow the flowable fill to encase the pipe.
5. All drainage catch basins and pipe adjacent to the geofoam must have two layers of protection in the event of a petroleum or chemical spill. See the Special Provisions for additional details.
6. All drop inlets within the limits of the EPS fill or load distribution slab shall be rectangular in shape.



REINFORCEMENT FOR LOAD DISTRIBUTION SLAB OPENING FOR INLETS
Scale: ¾" = 1'-0"

NOTES:

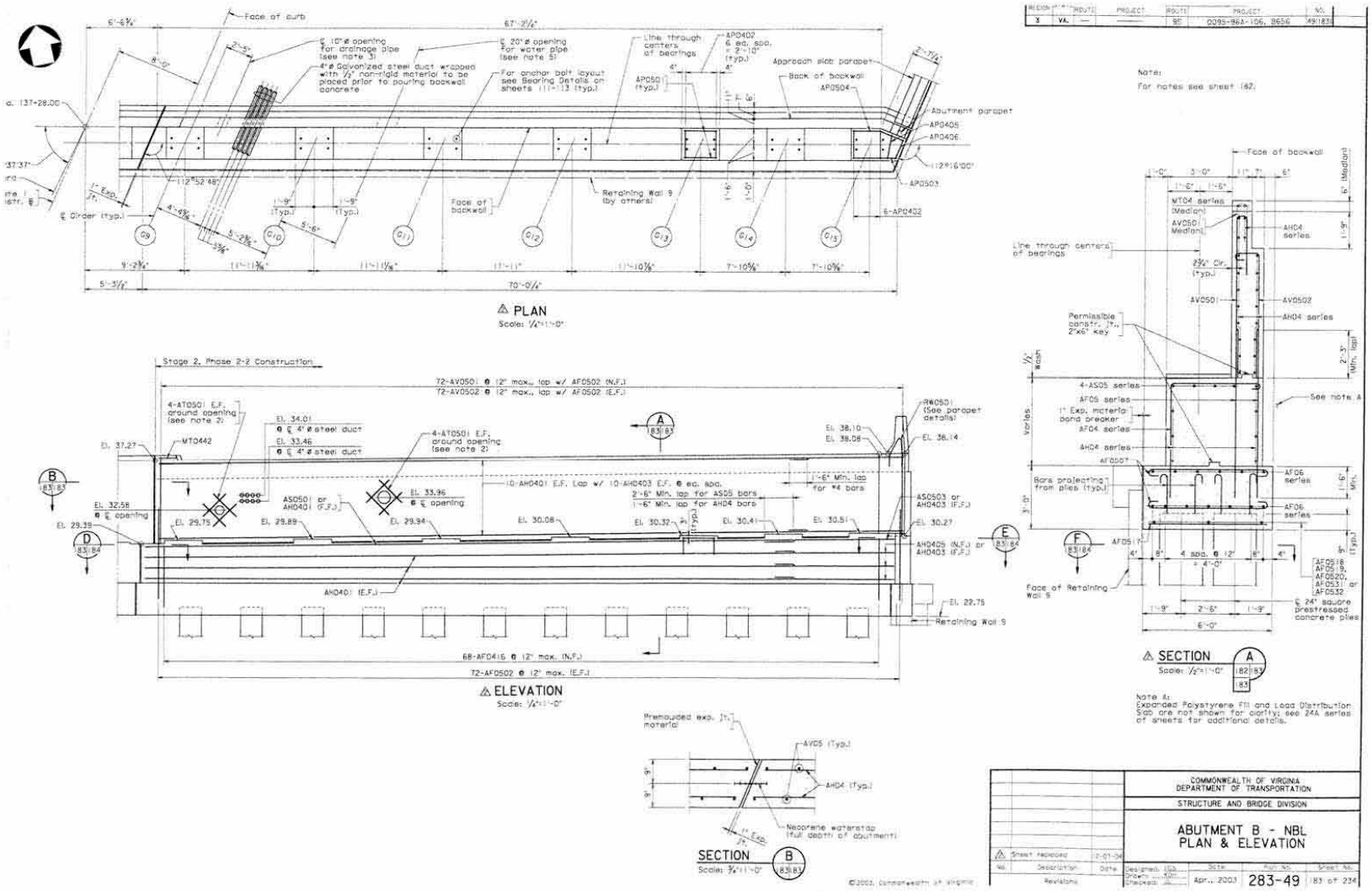
1. 2½" minimum cover over reinforcing.
2. Chamfer all exposed corners ¾".

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
I-95/ROUTE 1 INTERCHANGE

DRAIN INLET / EPS & LDS DETAILS

HNTB

0095-96A-106 | 24A(16)


PLAN
Scale: 1/4"=1'-0"
ELEVATION
Scale: 1/4"=1'-0"
SECTION
Scale: 1/2"=1'-0"
SECTION
Scale: 3/4"=1'-0"
Note:
For notes see sheet 182.
Note A:
Expanded Polystyrene Fill and Load Distribution Slab are not shown for clarity; see 24A series of sheets for additional details.
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
STRUCTURE AND BRIDGE DIVISION
ABUTMENT B - NBL
PLAN & ELEVATION
Apr., 2003
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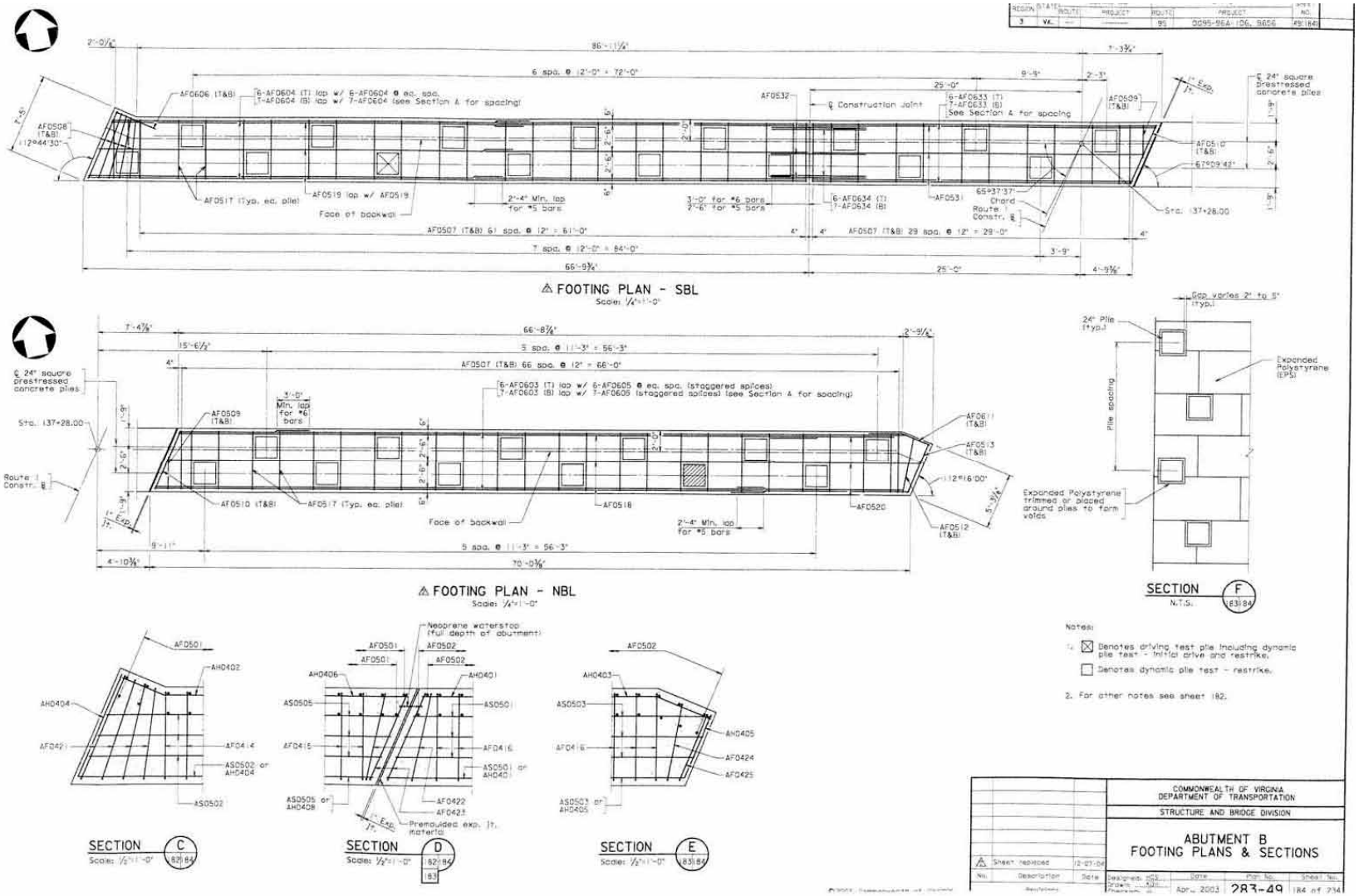
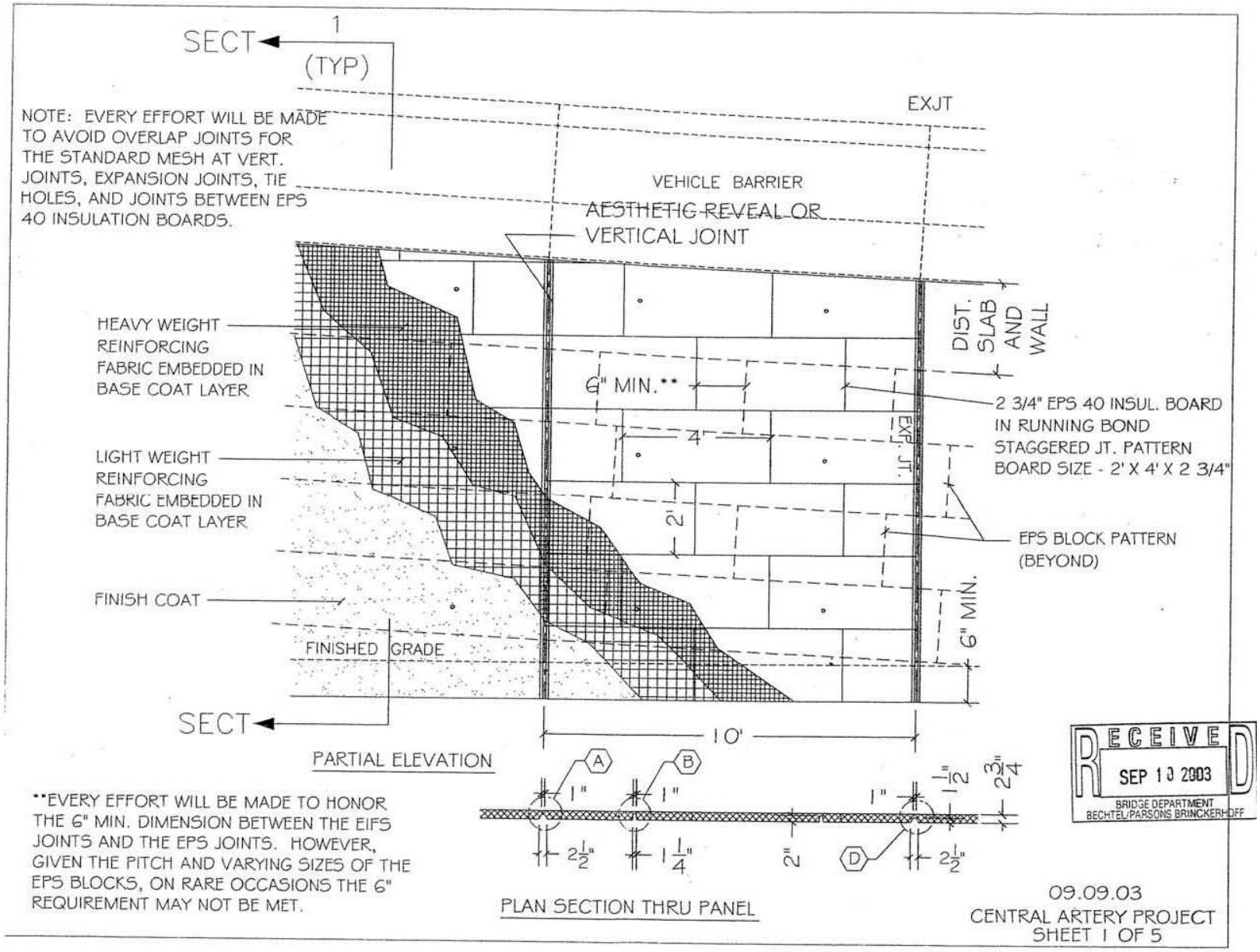
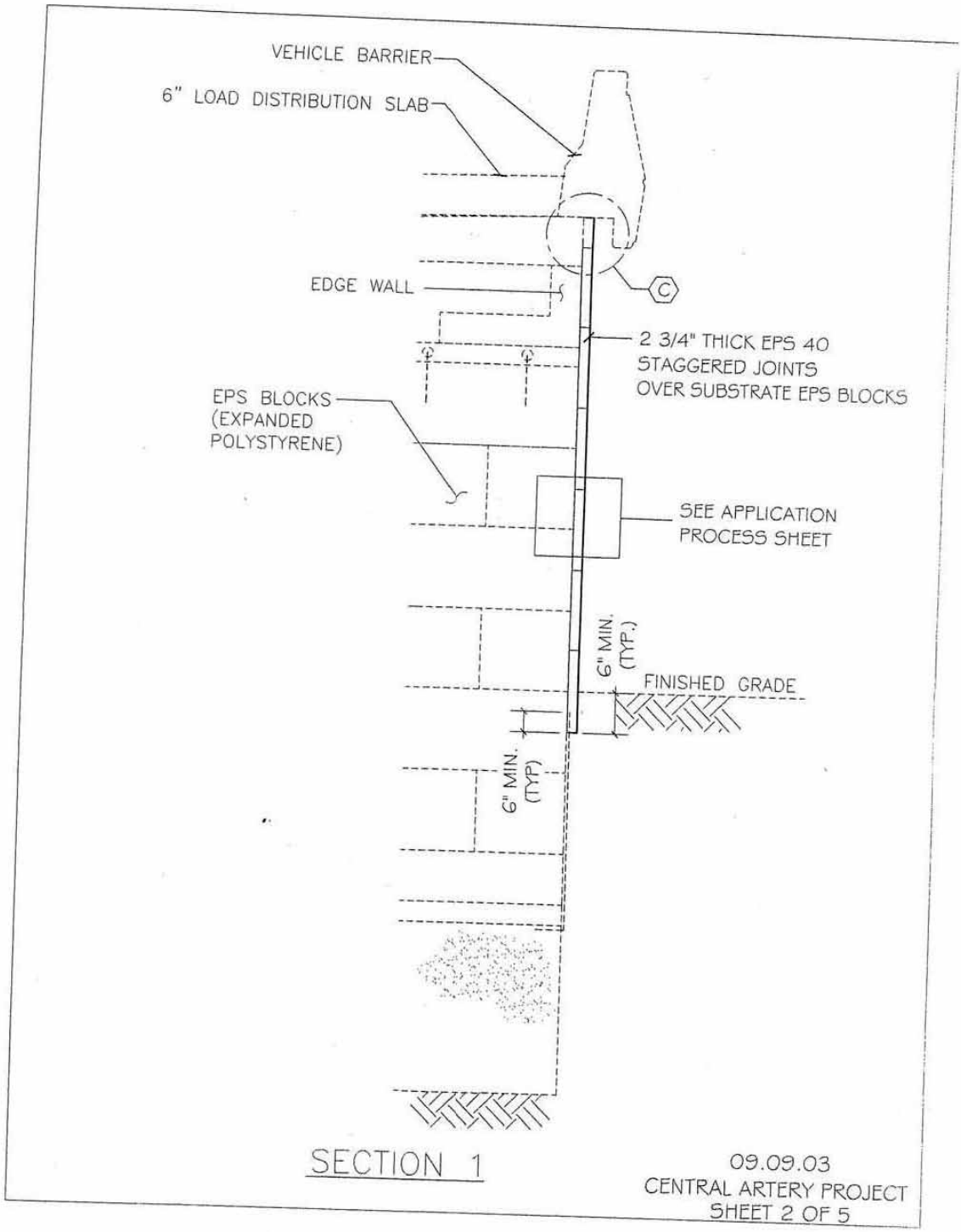
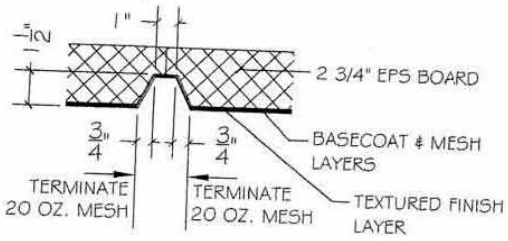

FOOTING PLAN - SBL
Scale: 1/4"=1'-0"
FOOTING PLAN - NBL
Scale: 1/4"=1'-0"
SECTION F
N.T.S.
SECTION C
Scale: 1/2"=1'-0"
SECTION D
Scale: 1/2"=1'-0"
SECTION E
Scale: 1/2"=1'-0"
Notes:
1. Denotes driving test pile including dynamic pile test - initial drive and restrike.
Denotes dynamic pile test - restrike.
2. For other notes see sheet 182.
COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
STRUCTURE AND BRIDGE DIVISION
ABUTMENT B
FOOTING PLANS & SECTIONS
Sheet replaced
Description
Date
Revisions
Apr., 2003
283-49
184 of 234

Project: EPS-Block Geofoam Ramps, Boston Central Artery/Tunnel Project

Source: J.S. Horvath

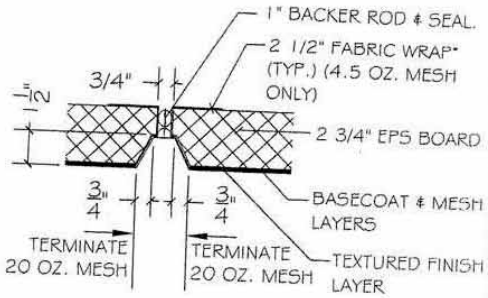

SECT 1 (TYP)
NOTE: EVERY EFFORT WILL BE MADE TO AVOID OVERLAP JOINTS FOR THE STANDARD MESH AT VERT. JOINTS, EXPANSION JOINTS, TIE HOLES, AND JOINTS BETWEEN EPS 40 INSULATION BOARDS.
EXJT
VEHICLE BARRIER
AESTHETIC REVEAL OR VERTICAL JOINT
HEAVY WEIGHT REINFORCING FABRIC EMBEDDED IN BASE COAT LAYER
LIGHT WEIGHT REINFORCING FABRIC EMBEDDED IN BASE COAT LAYER
FINISH COAT
FINISHED GRADE
DIST. SLAB AND WALL
6" MIN.**
4'
2'
EXP. JT.
2 3/4" EPS 40 INSUL. BOARD IN RUNNING BOND STAGGERED JT. PATTERN BOARD SIZE - 2' X 4' X 2 3/4"
EPS BLOCK PATTERN (BEYOND)
6" MIN.
SECT
10'
PARTIAL ELEVATION
A
B
D
1"
1"
1"
1 1/2"
2 3/4"
2 1/2"
1 1/4"
2"
2 1/2"
PLAN SECTION THRU PANEL
**EVERY EFFORT WILL BE MADE TO HONOR THE 6" MIN. DIMENSION BETWEEN THE EIFS JOINTS AND THE EPS JOINTS. HOWEVER, GIVEN THE PITCH AND VARYING SIZES OF THE EPS BLOCKS, ON RARE OCCASIONS THE 6" REQUIREMENT MAY NOT BE MET.
RECEIVED
SEP 10 2003
BRIDGE DEPARTMENT
BECHTEL/PARSONS BRINCKERHOFF
09.09.03
CENTRAL ARTERY PROJECT
SHEET 1 OF 5


VEHICLE BARRIER
6" LOAD DISTRIBUTION SLAB
EDGE WALL
C
2 3/4" THICK EPS 40
STAGGERED JOINTS
OVER SUBSTRATE EPS BLOCKS
EPS BLOCKS
(EXPANDED
POLYSTYRENE)
SEE APPLICATION
PROCESS SHEET
6" MIN.
(TYP.)
FINISHED GRADE
6" MIN.
(TYP)
SECTION 1
09.09.03
CENTRAL ARTERY PROJECT
SHEET 2 OF 5



DETAIL A

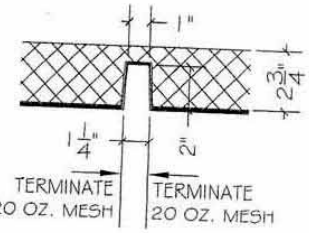
PLAN THRU VERTICAL JOINT



DETAIL D

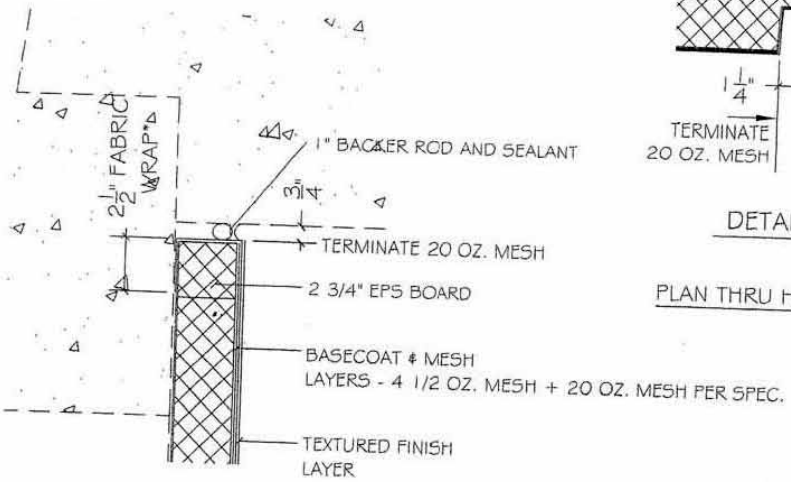
PLAN THRU EXPANSION JOINT

*PROVIDE 2 1/2" FABRIC MESH OVERLAP AT ALL EXPANSION JOINTS AND WHEN ABUTTING NON-EIFS PANEL STRUCTURES. (4.5 OZ. MESH ONLY)



DETAIL B

PLAN THRU HOLE DETAIL

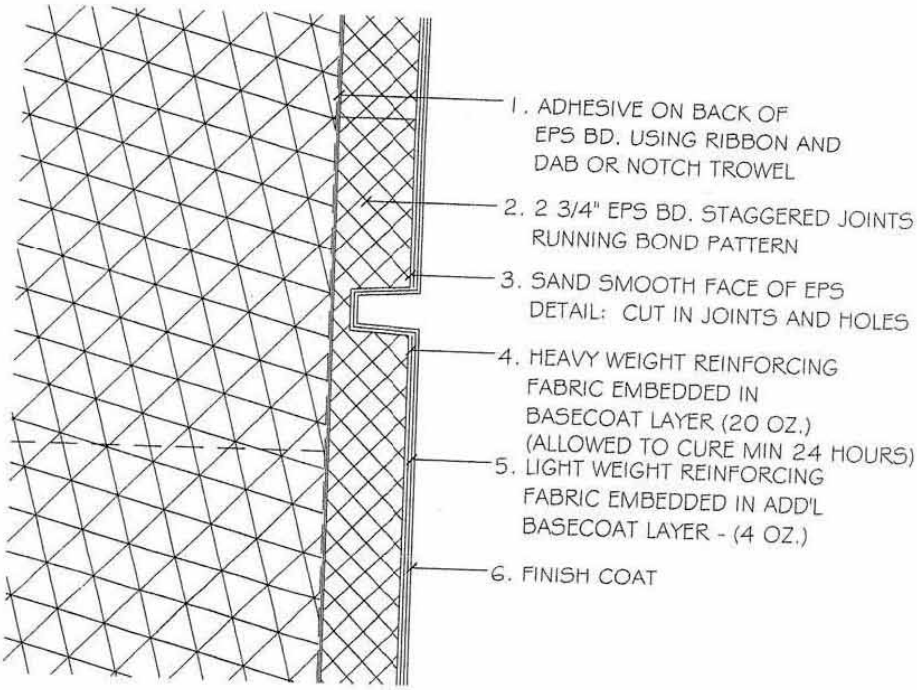


DETAIL C

THRU TOP OF EPS BOARD

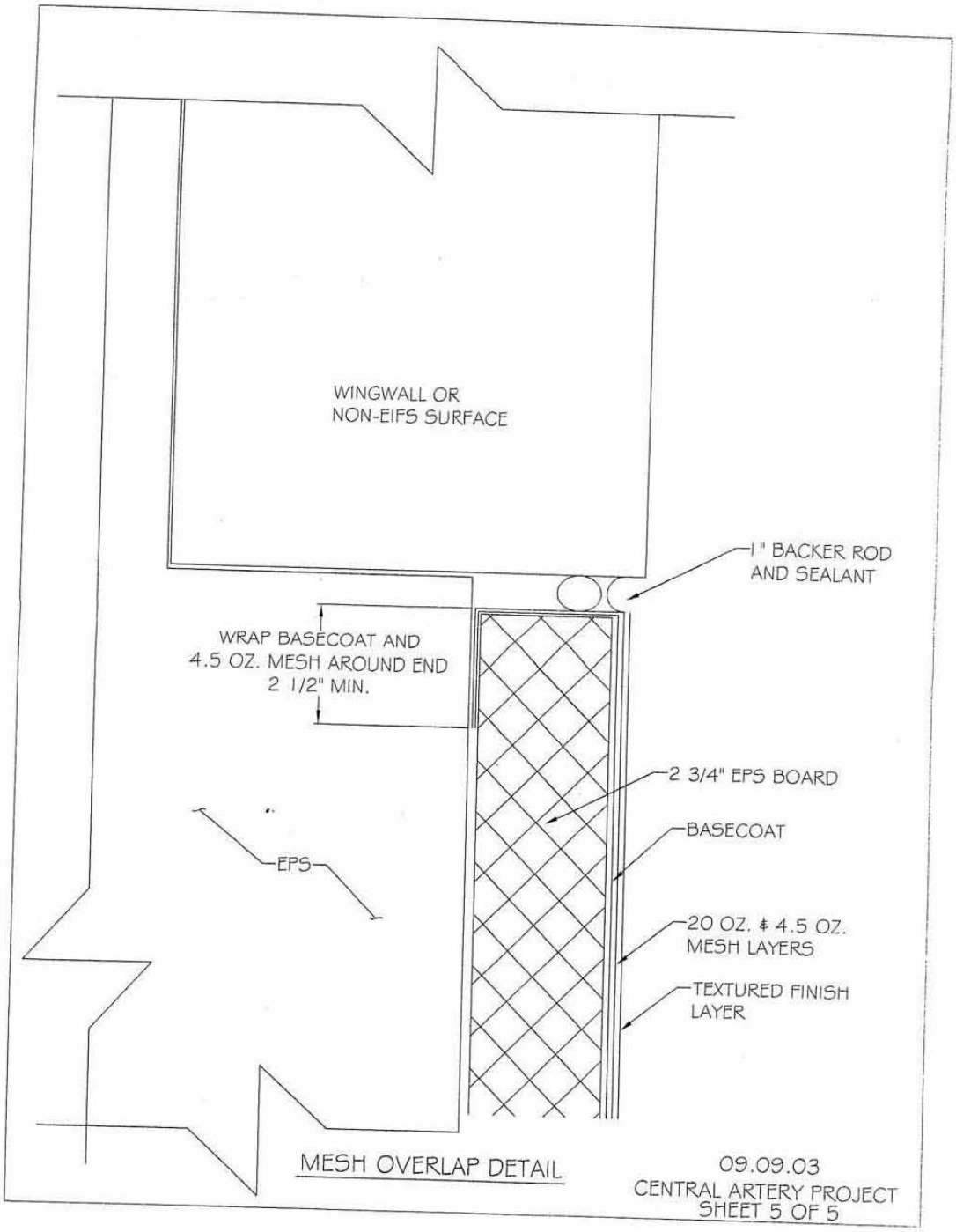
NOTE: THIS DETAIL TO BE USED AT VERTICAL JOINTS AS WELL WHEN ABUTTING NON-EIFS STRUCTURES.

09.09.03
CENTRAL ARTERY PROJECT
SHEET 3 OF 5



APPLICATION PROCESS

09.09.03
CENTRAL ARTERY PROJECT
SHEET 4 OF 5


WINGWALL OR
NON-EIFS SURFACE
1" BACKER ROD
AND SEALANT
WRAP BASECOAT AND
4.5 OZ. MESH AROUND END
2 1/2" MIN.
2 3/4" EPS BOARD
BASECOAT
EPS
20 OZ. & 4.5 OZ.
MESH LAYERS
TEXTURED FINISH
LAYER
MESH OVERLAP DETAIL
09.09.03
CENTRAL ARTERY PROJECT
SHEET 5 OF 5

GENERAL NOTES FOR EPS STRUCTURE

1.0 DESIGN

1.1 THE DESIGN OF EXPANDED POLYSTYRENE (EPS) STRUCTURES IS IN ACCORDANCE WITH THE FOLLOWING CODES, STANDARDS AND SPECIFICATIONS:

"STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES", SIXTEENTH EDITION, AASHTO 1996, WITH CURRENT INTERIM SPECIFICATIONS THROUGH 1999, FOR HS25 LOADING, MODIFIED FOR MILITARY LOADING, SEISMIC ANALYSIS IS IN ACCORDANCE WITH SEISMIC PERFORMANCE CATEGORY "B" (ACCELERATION COEFFICIENT A=0.17) AND SOIL PROFILE TYPE III.

CENTRAL ARTERY (I-93)/TUNNEL (I-90) PROJECT DESIGN EXPANDED POLYSTYRENE (EPS)-BLOCK FILL STRUCTURES DESIGN CRITERIA REVISION 1.

AASHTO "STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINARIES AND TRAFFIC SIGNALS", 1994.

CA/T SUPPLEMENT SPECIFICATIONS FOR EPS - BLOCK FILL MATERIAL.

MASSACHUSETTS DEPARTMENT OF PUBLIC WORKS "BRIDGE MANUAL", PARTS 1 AND 2, 1988, INCLUDING REVISIONS THROUGH 1995.

MASSACHUSETTS DEPARTMENT OF PUBLIC WORKS "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES", 1988, INCLUDING REVISIONS THROUGH 1994.

MASSACHUSETTS DEPARTMENT OF PUBLIC WORKS "CONSTRUCTION STANDARDS", 1977.

1.2 ALL DESIGNS ARE BASED ON THE SERVICE LOAD METHOD (ALLOWABLE STRESS DESIGN) UNLESS NOTED OTHERWISE BELOW:

VEHICLE BARRIERS AND BARRIER FOOTING SLAB (BFS) ARE DESIGNED BY ULTIMATE STRENGTH DESIGN (LOAD FACTOR DESIGN - LFD)

FOUNDATIONS ARE DESIGNED BY ULTIMATE STRENGTH DESIGN. (LOAD FACTOR DESIGN - LFD)

DRILLED SHAFTS ARE DESIGNED BY SERVICE LOAD METHOD AND CHECKED FOR ULTIMATE LOAD CAPACITY USING ULTIMATE STRENGTH DESIGN.

2.0 GENERAL

2.1 DRILLED SHAFTS FOR ABUTMENTS ARE 2'-6" DIAMETER

2.2 ALL DIMENSIONS SHOWN INDICATE THE FINAL CONFIGURATION AFTER CONSTRUCTION IS COMPLETE. ALL DIMENSIONS SHOWN ON THE DRAWINGS ARE THE TRUE PLANE HORIZONTAL PROJECTED DIMENSIONS UNLESS NOTED. ALL DIMENSIONS SHOWN ARE FOR THE STRUCTURES AT 50 DEGREES FAHRENHEIT. THE CONTRACTOR MUST ADJUST ACCORDINGLY FOR FABRICATION AND ERECTION AT OTHER TEMPERATURES.

2.3 SCALES NOTED ON THE DRAWINGS ARE NOT APPLICABLE TO REDUCED SIZE PRINTS. DIVIDE SCALES BY 2 FOR 1/4 SIZE PRINTS (11"x18").

2.4 FINISH GRADE SHOWN ON DRAWINGS REFLECT FINAL GRADES. FOR GRADING REQUIREMENTS AND FINISHED SURFACE MATERIALS, SEE LANDSCAPE DRAWINGS L-243 AND L-283.

3.0 PROJECT DATUM

STANDARD PROJECT DATUM IS USED THROUGHOUT. ELEVATIONS ARE BASED ON THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD). FORMERLY UGS & GS MEAN SEA LEVEL OF 1929. THE PROJECT DATUM PLANE IS ESTABLISHED AT 100.00 FEET BELOW NGVD 1929. NGVD EL. 0.00 EQUALS CA/T PROJECT DATUM EL. 100.00.

4.0 DATE

THE DATE SHALL BE PLACED IN ACCORDANCE WITH THE SPECIFICATIONS.

5.0 SURVEY

THE SURVEY DATA SHOWN ON DRAWINGS INCLUDED HERE ARE BASED ON DRAWINGS NO Y-111 AND Y-113.

6.0 GEOTECHNICAL

6.1 FOR GEOTECHNICAL AND BORING INFORMATION, REFER TO THE GEOTECHNICAL ENGINEERING AND DATA REPORTS DATED FEBRUARY 1993. PREPARED BY GZA GEOENVIRONMENTAL INC. FOR DESIGN SECTION D009C AND THE SUPPLEMENTAL GEOTECHNICAL ENGINEERING AND DATA REPORT BY BLSW DATED MAY 1996.

6.2 GEOTECHNICAL DATA AND DISCUSSION OF SUBSURFACE CONDITIONS INCLUDED IN GEOTECHNICAL REPORTS ARE FACTUAL DATA ONLY AND ARE NOT A WARRANTY OF SUBSURFACE CONDITIONS. BORING LOGS DEPICT DATA (ESPECIALLY GROUNDWATER CONDITIONS) AT THE SPECIFIC LOCATION OF THE BORINGS AT THE TIME MADE AND MAY OR MAY NOT REPRESENT SUBSURFACE CONDITIONS ENCOUNTERED DURING CONSTRUCTION.

6.3 THE ACCURACY AND COMPLETENESS OF SUBSURFACE CONDITIONS CANNOT BE GUARANTEED. THE CONTRACTOR MAY MAKE THE APPROPRIATE FIELD INVESTIGATIONS TO VERIFY THE SUBSURFACE CONDITIONS.

6.4 IF FOUND NECESSARY AND WITH THE APPROVAL OF THE ENGINEER, FOUNDATIONS MAY BE ALTERED TO SUIT CONDITIONS ENCOUNTERED DURING CONSTRUCTION.

6.5 ALL UNSUITABLE MATERIAL WITHIN THE LIMITS OF THE FOUNDATIONS SHALL BE REMOVED AND DISPOSED AS DIRECTED BY THE ENGINEER.

6.6 LOCATION OF ALL UTILITIES SHALL BE VERIFIED IN THE FIELD BEFORE INSTALLATION OF THE DRILLED SHAFT.

7.0 EXPANDED POLYSTYRENE - EPS BLOCKS

7.1 EPS MATERIAL PROPERTIES BELOW ARE MINIMUM ALLOWABLE VALUES REPRESENTING THE DESIGN AND QUALITY CONTROL PARAMETERS FOR INDIVIDUAL TEST SPECIMENS AND NOT FOR THE BLOCK AS A WHOLE.

7.2 COMPRESSIVE AND FLEXURAL STRENGTH, MINIMUM ELASTIC LIMIT STRESS AND INITIAL TANGENT YOUNG'S MODULUS ARE BASED ON MINIMUM DENSITY, AND ARE BOTH USED IN ANALYSES AND DESIGN.

AASHTO MATERIAL TYPE	ELASTIC LIMIT STRESS		INITIAL TANGENT YOUNG'S MODULUS		FLEXURAL STRENGTH		COMPRESSIVE STRENGTH	
	(KPa)	psi	(MN/m^2)	psi	(KPa)	psi	(KPa)	psi
EPS 40	(40)	5.8	(4)	580	(173)	25	(69)	10
EPS 100	(100)	14.5	(10)	1450	(345)	50	(173)	25

7.3 LAYOUT DEPICTED IN DRAWINGS REFLECTS EPS BLOCK WITH 8'x4'x2' (LENGTH x WIDTH x HEIGHT).

7.4 EPS MATERIAL SUPPLIER MAY ELECT TO PROVIDE BLOCKS WITH ALTERNATE DIMENSIONS.

7.5 EPS MATERIAL SUPPLIER SHALL PREPARE EPS SHOP DRAWINGS OF BLOCK PLACEMENT FOR REVIEW AND APPROVAL BY THE ENGINEER PRIOR TO CONSTRUCTION.

8.0 CAST IN PLACE CONCRETE

8.1 CONCRETE MIXES SHALL MEET THE FOLLOWING REQUIREMENTS:

	(1)*	(2)**	(3)***
FOOTINGS: ABUTMENTS, APPROACH SLABS, EDGE WALLS, DRILLED SHAFTS & BARRIERS:	4000	1 1/2	565
ROADWAY SLAB:	4000	3/4	610
	4500	3/4	660

*(1) 28-DAY COMPRESSIVE STRENGTH (PSI)
**(2) MAXIMUM AGGREGATE SIZE (INCHES)
***(3) CEMENT CONTENT (LB(S)/CU YD)

8.2 CEMENT SHALL CONFORM TO THE REQUIREMENTS OF AASHTO M85.

8.3 ALL EXPOSED EDGES OF CONCRETE SHALL BE CHAMFERED 3/4" UNLESS OTHERWISE NOTED.

8.4 ALL EXPOSED EDGES OF CONCRETE SHALL MEET RUSTICATION REQUIREMENTS AS DETAILED ON H-2501, H-2502, H-3481, H-3483, AND H-3488.

8.5 NOT USED.

8.6 FOR DRILLED SHAFTS CONCRETE SHALL BE PLACED BY TREMIE PIPE OR PUMPING.

8.7 THE CASING FOR DRILLED SHAFTS SHALL BE PLACED BEFORE PLACING THE REINFORCING CAGE.

9.0 REINFORCING STEEL

9.1 ALL REINFORCING STEEL SHALL CONFORM THE REQUIREMENTS OF ASTM-A615 (AASHTO-M31) GRADE 60. ALL REINFORCING BARS AND SUPPORTING DEVICES LISTED HEREIN SHALL BE EPOXY COATED IN ACCORDANCE WITH AASHTO-M284 (ASTM-D3963). THIS SHALL INCLUDE BARRIERS, END POSTS, SIGN POSTS SUPPORTS, LIGHT POSTS SUPPORTS, ABUTMENT STEM AND FOOTING-TO-STEM DOWELS. FOOTING REINFORCEMENT TO BE EPOXY COATED AS NOTED. ALL DIMENSIONS RELATING TO THE REINFORCING BARS (I.E. SPACING OF BARS, ETC.) ARE TO THE CENTER OF BARS UNLESS OTHERWISE NOTED.

9.2 ALL VERTICAL AND "BOTTOM" HORIZONTAL BARS SHALL BE LAPPED 24 IN FOR #4 BARS AND 30 IN FOR #5 BARS. FOR HORIZONTAL BARS WITH 12 IN OR MORE OF CONCRETE BELOW THE BAR, THE LAP LENGTH SHALL BE 35 IN FOR #4 BARS AND 44 IN FOR #5 BARS. IF THE ABOVE BARS ARE SPACED 6 IN OR MORE ON CENTER, THE LAP LENGTH SHALL BE 80% OF THE LAP LENGTH GIVEN ABOVE. #6 AND LARGER BARS SHALL OF LAPPED AS SHOWN ON THE PLANS.

10.0 OTHER MATERIALS

FOR MATERIALS AND CONSTRUCTION REQUIREMENTS SEE CONTRACT SPECIFICATIONS.

11.0 CONSTRUCTION

11.1 THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE STABILITY OF THE PRECAST CONCRETE SECTIONS DURING TRANSPORTATION INCLUDING LOADING, UNLOADING, STORAGE AND ERECTION.

11.2 ALL ANCHOR BOLTS SHALL BE SET BY TEMPLATE BEFORE THE CONCRETE IS PLACED. ANCHOR BOLTS SHALL CONFORM TO ASTM F1554.

12.0 STRUCTURAL STEEL

12.1 STRUCTURAL STEEL SHALL CONFORM TO AASHTO M270 (ASTM A709) GRADE 50 AND SHALL BE PAINTED, UNLESS OTHERWISE NOTED.

INTERSTATE I-93/I-90

F.A.H.A. REGION NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1495	

13.0 BOLTED CONNECTIONS

13.1 ALL BOLTS FOR STRUCTURAL STEEL CONNECTIONS SHALL CONFORM TO AASHTO M164 (ASTM A325) TYPE 1 IN CLASS A SLIP CRITICAL CONNECTIONS AND ALL BOLTS SHALL BE 7/8" DIAMETER IN STANDARD HOLES UNLESS OTHERWISE NOTED.

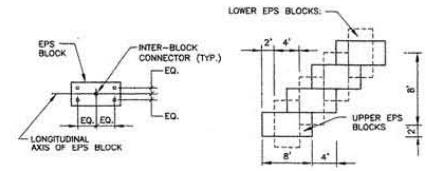
14.0 LAYOUT OF EPS BLOCKS

14.1 LAYOUT OF EPS BLOCKS DEPICTED IN THESE DRAWINGS IS REPRESENTATIVE OF THE FINISHED LAYOUT. THE CONTRACTOR IS RESPONSIBLE TO DEVELOP THE SPECIFIC LAYOUT TO MEET THE REQUIREMENTS OF THE GEOMETRIC SCHEDULE, DRAWING S-5937 AND THE GUIDELINES BELOW.

14.2 NOT USED.

14.3 EPS DESIGN SHALL BE LIMITED ONLY TO THE USE OF EPS MATERIAL TYPE EPS 100 FOR ALL EPS - BLOCKS FILL STRUCTURES, EPS 40, HOWEVER, MAY BE USED OUTSIDE OF CURTAILMENT TO THE STRUCTURAL (LOAD BEARING) EPS.

14.4 THE FOLLOWING GUIDELINES SHALL BE SATISFIED ON ALL EPS - BLOCK:



A) BLOCKS SHALL BE PLACED WITH THEIR SMALLEST DIMENSION ORIENTED PERPENDICULAR TO ROADWAY SURFACE WHEN VIEWED IN ELEVATION.

B) A MINIMUM OF TWO LAYERS OF BLOCKS MUST ALWAYS BE USED IN ALL EPS - BLOCK FILL STRUCTURES.

C) BLOCKS MUST BE PLACED IN A PATTERN THAT RESULTS IN MINIMAL OR NO CONTINUITY OF THE VERTICAL JOINTS BETWEEN BLOCKS.

D) LONGITUDINAL AXES OF EPS BLOCKS WITHIN ANY LAYER SHALL BE PERPENDICULAR TO THE LONGITUDINAL AXES OF THE EPS BLOCKS IN THE LAYERS ABOVE AND BELOW SUCH LAYER AS APPLICABLE BLOCK PLACEMENT DETAIL USING EPS BLOCK DIMENSIONS (8'x4'x2') AS SHOWN IN FIGURE ABOVE.

E) THE UPPERMOST LAYER OF EPS BLOCKS SHALL BE ORIENTED SUCH THAT ITS LONG (MAXIMUM) DIMENSION IS TRANSVERSE TO TRAFFIC (I.E. PERPENDICULAR TO THE ROADWAY ALIGNMENT WHEN VIEWED IN PLAN.)

F) THE TOP SURFACE OF THE ASSEMBLAGE OF EPS BLOCKS SHOULD ALWAYS BE PARALLEL TO THE FINAL PAVEMENT SURFACE. THUS ANY DESIRED CHANGE IN ELEVATION (GRADE) ALONG THE ROADWAY ALIGNMENT MUST BE ACCOMMODATED BY SLOPING THE SUBGRADE SURFACE AS NECESSARY PRIOR TO PLACEMENT OF THE FIRST LAYER OF EPS BLOCKS.

G) ALL BLOCKS MUST BE PLACED IN CLOSE DIRECT CONTACT AGAINST ADJACENT BLOCKS ON ALL SIDES.

H) MECHANICAL INTER-BLOCK CONNECTORS (SUCH AS GALVANIZED STEEL BARBED PLATES) SHALL BE PLACED ON ALL HORIZONTAL SURFACES BETWEEN LAYERS OF EPS BLOCKS TO ENHANCE THE INTER-BLOCK FRICTION RESISTANCE TO LOADS BETWEEN BLOCKS. A SUGGESTED LAYOUT OF INTER-BLOCK CONNECTORS FOR AN 8'x4'x2' EPS BLOCK IS SHOWN IN FIGURE ABOVE.

I) BELOW FINISHED GRADE, ONE (1) STEEL INTER-BLOCK CONNECTOR PLATE SHALL BE PLACED ON THE TOP AND BOTTOM FACES OF EACH EPS BLOCK CENTERED AT THE BLOCK HORIZONTAL SURFACE.

REV	DATE	BY	SUB	APP	DESCRIPTION
4	28JAN02	DW	EJ	EJ	REMOVED BID ONLY NOTE
3	14DEC01				INCORP ADD 5 NARRATIVE
2	21NOV01				REVISED NOTE
1	24OCT01				UPDATED NOTE

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

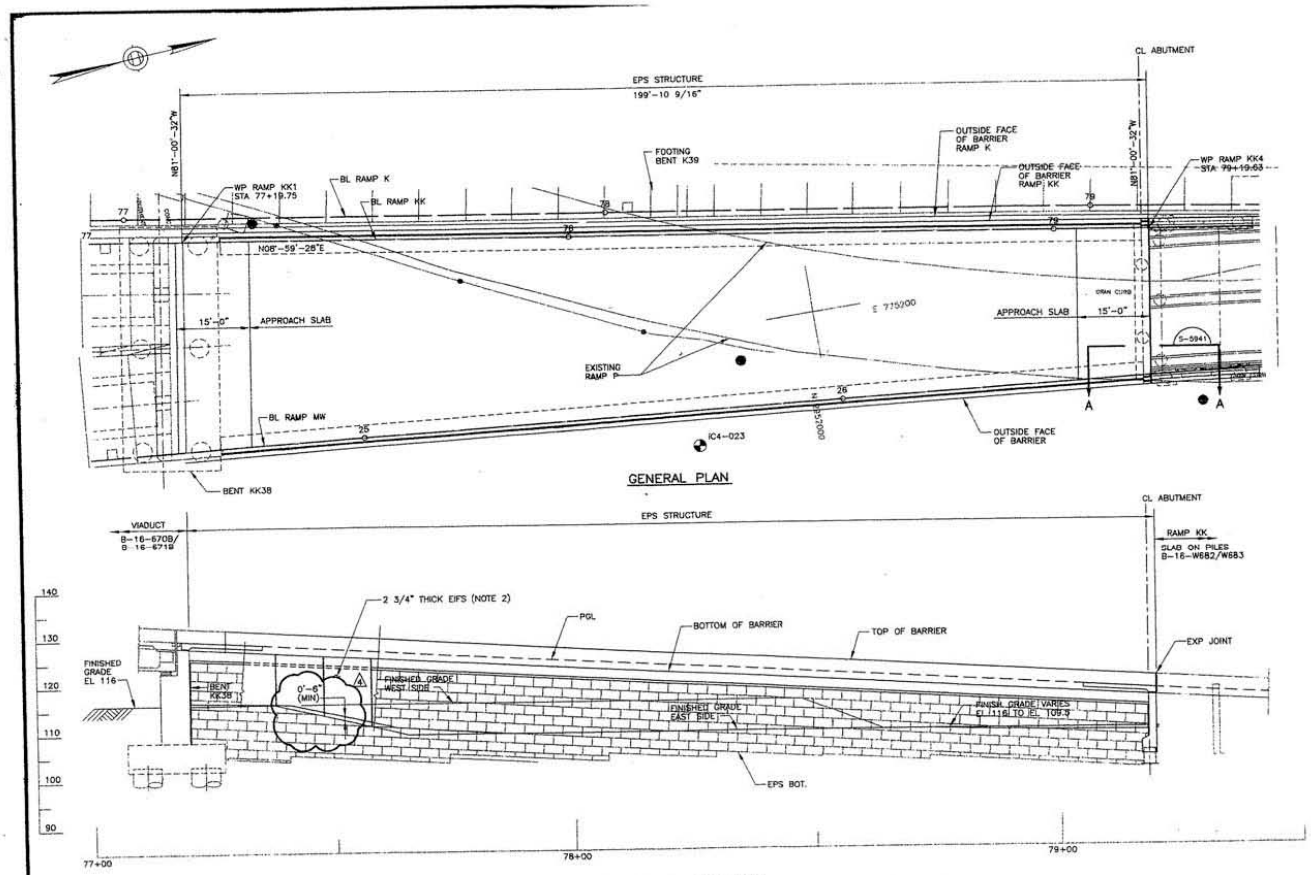
BERGER/LOCHNER/STONE & WEBSTER
TECHNICAL DESIGNER
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
GENERAL NOTES

SCALE: NO SCALE
CONTRACT NO. C09C2
DRAWING NO. S-5926
REV. 4

STRUCTURE NO B-16-506



INTERSTATE I-93/I-90

FAHWA WORK NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1496	

NOTES:

1. FOR BORING LOCATIONS AND IDENTIFICATIONS SEE DRAWINGS K-102 THROUGH K-107.
2. DRAWING S-5940 SHOWS NUMBER AND SPACING OF PANEL SECTIONS IN THE EXTERIOR INSULATION AND FINISH SYSTEM (EIFS).
3. FOR CONSTRUCTION STAGING SEE DRAWING C-1502 THROUGH C-1516.
4. ALL EXPOSED FACES OF CONCRETE AND EIFS SHALL MEET THE RUSTICATION REQUIREMENTS OF DRAWINGS H-2501 AND H-2502.
5. FOR ELECTRICAL ROUTING REFER TO DRAWING E-345.
6. THE SURVEY DATA SHOWN ON DRAWINGS INCLUDED HERE IS BASED ON DRAWINGS Y-111 AND Y-113.
7. FINISHED GRADE SHOWN ON DRAWINGS REFLECT FINAL GRADES. FOR GRADING REQUIREMENTS AND FINISH SURFACE OF MATERIALS, SEE LANDSCAPE DRAWINGS L-243 AND L-283.

WORK POINT COORDINATES		
WP	NORTHING	EASTING
RAMPKK1	2951873.490	775162.877
RAMPKK4	2952070.909	775194.114

SEE STRUCTURE B-16-W682/W663 FOR WPS KK5-KK8. WORK POINTS KK2 AND KK3 NOT USED

REFERENCE DRAWINGS

B-16-W682/W683, S-6021 THROUGH S-6027
B-16-6708/6718 TRANSITION PIER, C09C4-S-4054 THROUGH C09C4-S-4058

0 10 20 50 FEET

1" = 10'-0" HORIZONTAL

REV.	DATE	BY	SUB	APP	DESCRIPTION
4	28JAN02	FS	RJ	EJ	INCORP ADD 9 NARRATIVE REMOVED BID ONLY NOTE
3	14DEC01				INCORP ADD 5 NARRATIVE
2	21NOV0				DEL PILES & GRADING BEAM REV NOTE, ADD EDGEWALL
1	24OCT01				UPD ABUT, APP SLAB & NOTES

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

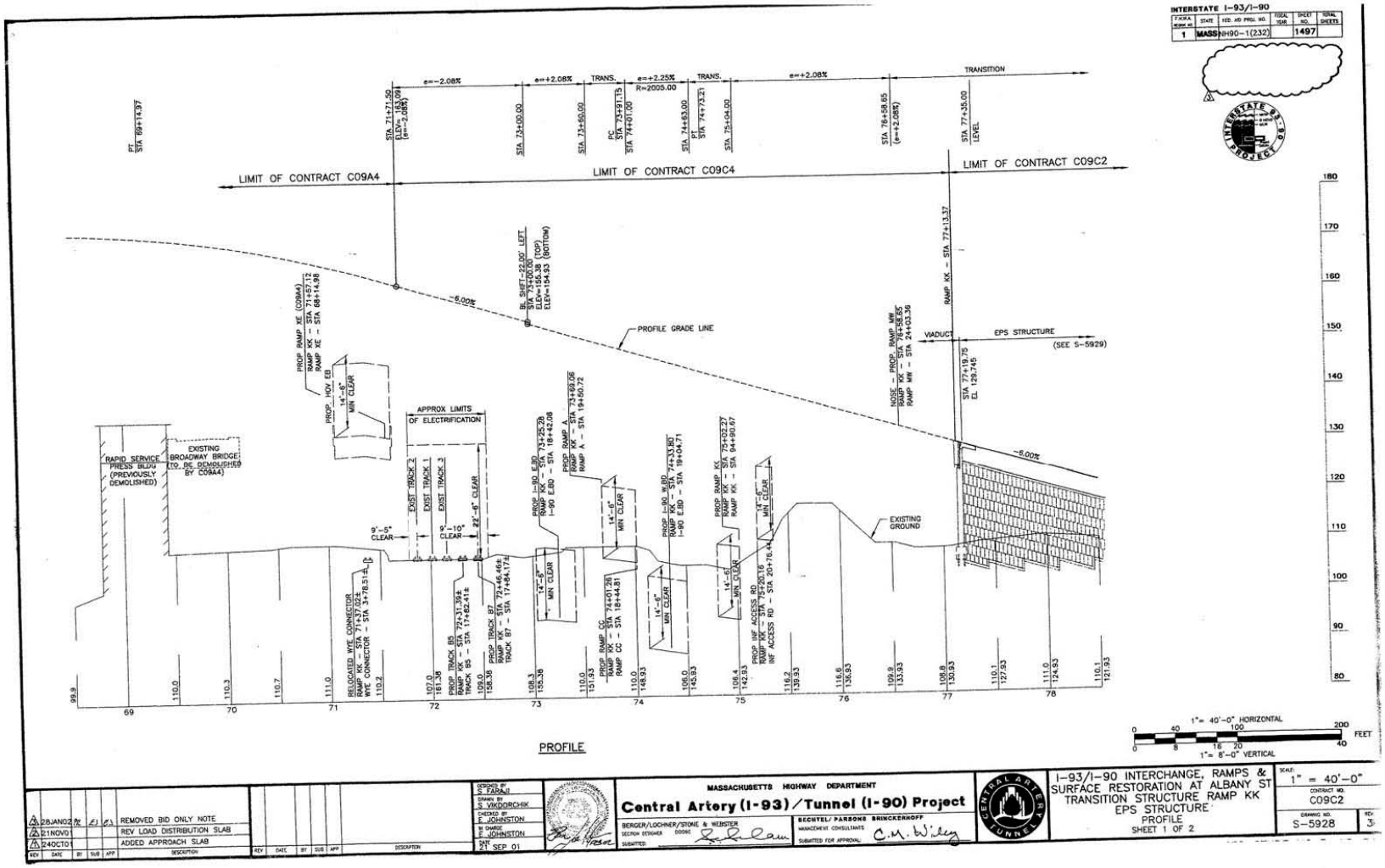
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER C09C2
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST TRANSITION STRUCTURE RAMP KK EPS STRUCTURE
GENERAL PLAN AND LONGITUDINAL ELEV

SCALE: 1" = 10'-0"
CONTRACT NO. C09C2
DRAWING NO. S-5927

STRUCTURE NO B-16-5



REV	DATE	BY	SUB	APP	DESCRIPTION
3	28JAN02				REMOVED BID ONLY NOTE
2	21NOV01				REV LOAD DISTRIBUTION SLAB
1	24OCT01				ADDED APPROACH SLAB

DESIGNED BY S. FARNJI
DRAWN BY S. VIKDORCHIK
CHECKED BY F. JOHNSTON
IN CHARGE F. JOHNSTON
DATE SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

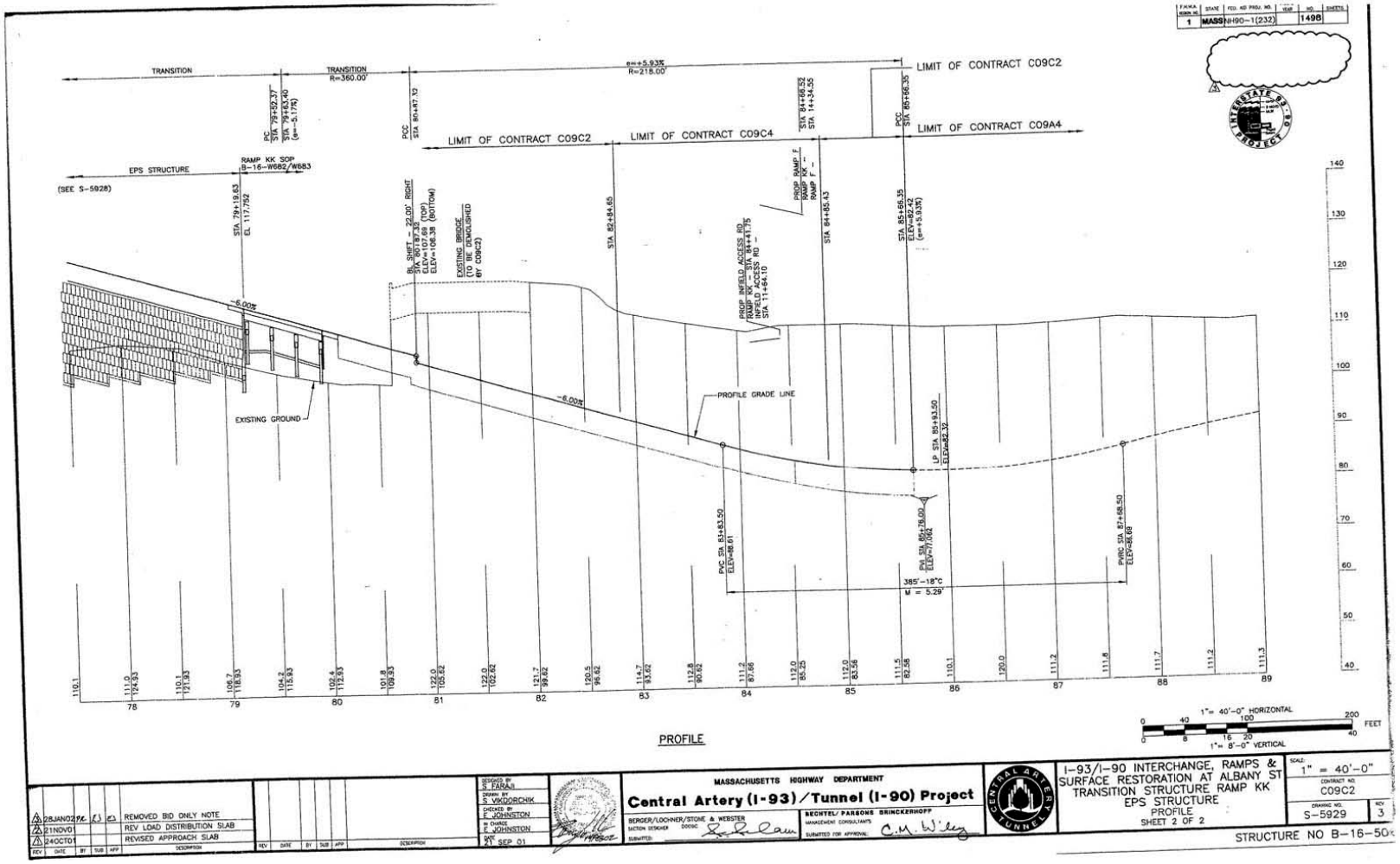
Central Artery (I-93) / Tunnel (I-90) Project

BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOD96C
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST TRANSITION STRUCTURE RAMP KK EPS STRUCTURE
PROFILE
SHEET 1 OF 2

SCALE: 1" = 40'-0"
CONTRACT NO. C09C2
DRAWING NO. S-5928
REV. 3



F.H.W.A. REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)	1498	

REV	DATE	BY	SUB	APP	DESCRIPTION
3	28JAN02	PK	RJ	RS	REMOVED BID ONLY NOTE
2	21NOV01				REV LOAD DISTRIBUTION SLAB
1	24OCT01				REVISED APPROACH SLAB

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY F. JOHNSTON
IN CHARGE F. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93) / Tunnel (I-90) Project

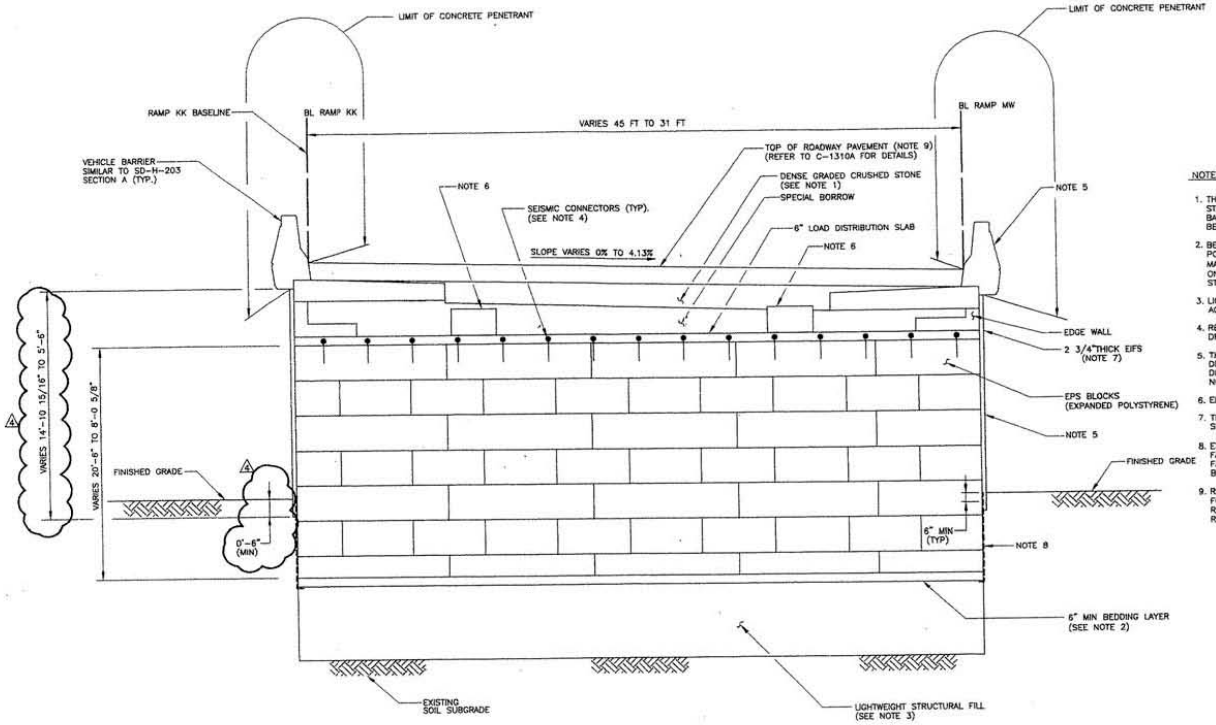
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOD3C
SUBMITTED:

BECHTEL/PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
PROFILE
SHEET 2 OF 2

SCALE: 1" = 40'-0"
CONTRACT NO. C09C2
DRAWING NO. S-5929
REV. 3

STRUCTURE NO B-16-50



TYPICAL SECTION

SCALE: 3/8" = 1'-0"

FEDERAL REGION NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1499	

NOTES:

1. THE BOTTOM LIMIT OF DENSE GRADED CRUSHED STONE SHALL BE AT THE BOTTOM OF THE VEHICLE BARRIER FOOTING BUT NOT LESS THAN 24 INCHES BELOW THE ROADBED SURFACE.
2. BEDDING LAYER CONSISTS OF NON-WOVEN POLYPROPYLENE GEOTEXTILE FABRIC WITH A MINIMUM MASS OF 4.0 OZ/SY AND 6 INCHES OF SAND PLACED ON SMOOTHED SOIL SUBGRADE/LIGHTWEIGHT STRUCTURAL FILL.
3. LIGHTWEIGHT STRUCTURAL FILL SHALL BE IN ACCORDANCE WITH THE SPECIFICATIONS.
4. REFER TO THE SEISMIC CONNECTOR DETAIL ON DRAWING S-5937.
5. THE NOTED VEHICLE BARRIER AND EIFS SHALL BE DELAYED UNTIL AFTER TEMPORARY RAMP KK HAS BEEN DISMANTLED. THE VEHICLE BARRIER FOOTING SHALL NOT BE DELAYED.
6. ELECTRICAL DUCTBANK, REFER TO DRAWING E-345.
7. THE EXTERIOR INSULATION AND FINISH SYSTEM (EIFS) SHALL BE IN ACCORDANCE WITH SPECIFICATION 909.300.
8. EXTEND NON-WOVEN POLYPROPYLENE GEOTEXTILE FABRIC (PART OF BEDDING LAYER) ALONG VERTICAL FACE OF EPS BLOCKS TO A MINIMUM OF 6 INCHES BEYOND END OF EIFS.
9. REFER TO DRAWINGS S-6408, S-6410 AND S-6413 FOR ADDITIONAL REINFORCEMENT AND PAVEMENT REQUIREMENTS FOR THE SUPPORT OF TEMPORARY RAMP KK.

0 2 4 6 8 16

3/8" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
4	28JAN02	FK	EJ	EJ	INCORP ADD 9 NARRATIVE REMOVED BID ONLY NOTE
3	14DEC01				INCORP ADD 5 NARRATIVE
2	21NOV01				ADD EDGE WALL DEL SHPTS & GRADING BEAM
1	24OCT01				REVISED NOTE

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOD9C
SUBMITTED:

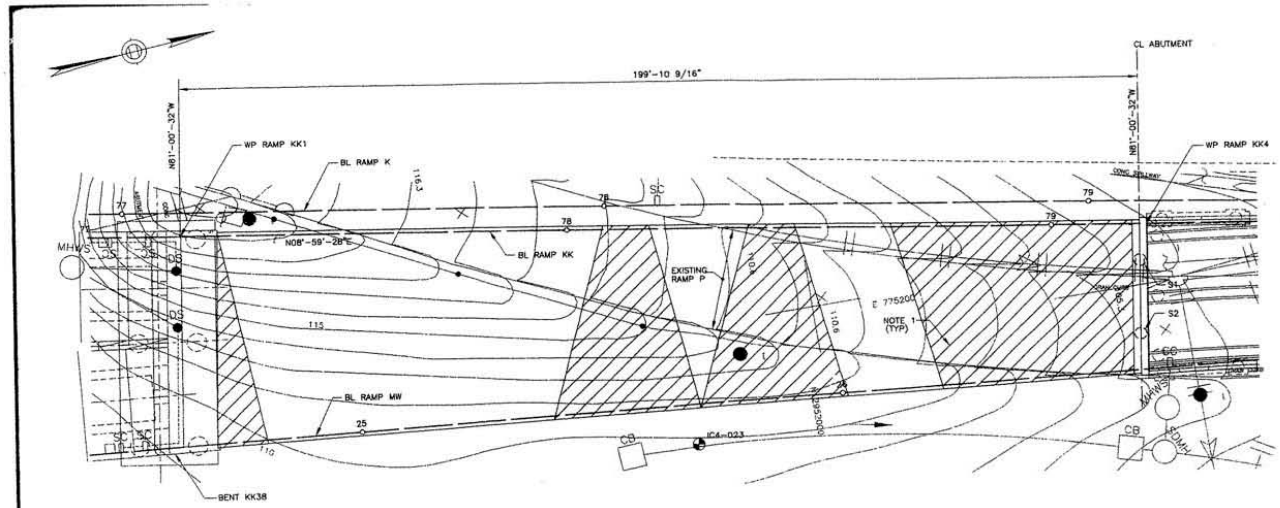
BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
TYPICAL SECTION

SCALE: 3/8" = 1'-0"
CONTRACT NO. C09C2
DRAWING NO. S-5930
REV. 4

STRUCTURE NO B-16-50

F.H.W.A. REGN. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1500	



PLAN

FOR BIDDING PURPOSES ONLY

NOTES:

1. LIGHTWEIGHT STRUCTURAL FILL REQUIRED HERE. REFER TO DRAWING S-5937 FOR REQUIRED LENGTH, WIDTH AND DEPTH OF LIGHTWEIGHT STRUCTURAL FILL.
2. LIGHTWEIGHT STRUCTURAL FILL SHALL BE IN ACCORDANCE WITH THE SPECIFICATIONS.
3. FOR BORING LOCATIONS AND IDENTIFICATIONS SEE DRAWINGS K-102 THROUGH K-107.

0 10 20 50 FEET

1" = 10'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
4	14DEC01				ADDED NOTE
3	21NOV01				DEL SHFTS & GRADING BEAM
1	12OCT01				REVISED FILL REQMNTS

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOD6C
SUBMITTED:

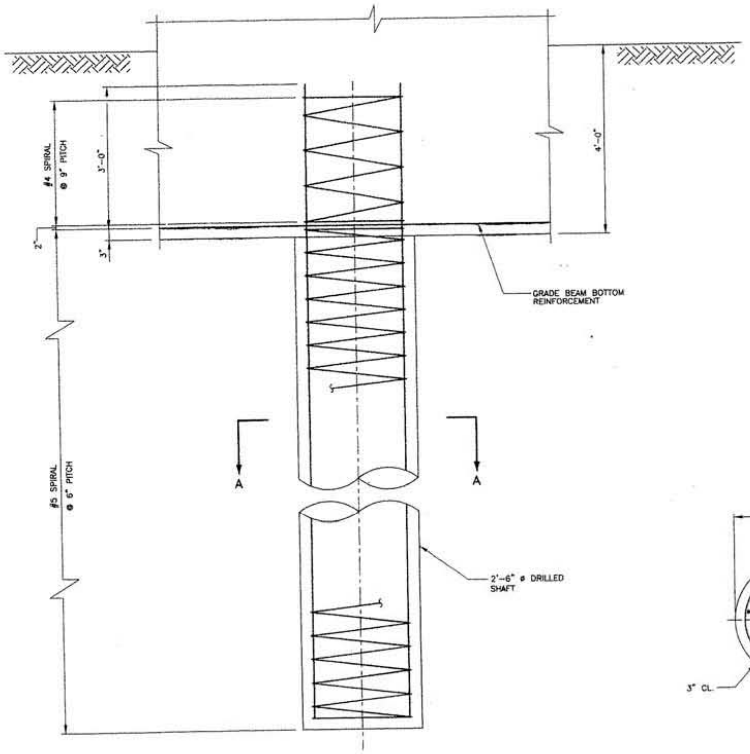
BECHTEL/PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL:

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
FOUNDATION PLAN

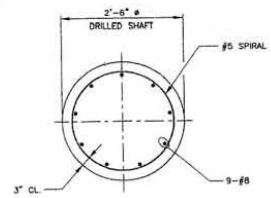
SCALE: 1" = 10'-0"
CONTRACT NO. C09C2
DRAWING NO. S-5931
REV. 3

STRUCTURE NO B-16-50

F.H.W.A. REGION NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1501	



2'-6" DIAMETER DRILLED SHAFT DETAILS
SCALE: 1" = 1'-0"



SECTION A
SCALE: 1" = 1'-0"

NOTES:
1. REFER TO DRAWING S-5935 FOR GRADE BEAM REINFORCEMENT.

0 1 2 3 6
1" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
1	28JAN02	FB	ES	ES	REMOVED BID ONLY NOTE

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOBSC
SUBMITTED:
BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST TRANSITION STRUCTURE RAMP KK EPS STRUCTURE
DRILLED SHAFT DETAILS

SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-5932
REV. 1

STRUCTURE NO B-16-50E

BOSTON
INTERSTATE I-93/I-90

FHWA REGION NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1502	

DRILLED SHAFT SCHEDULE

ID	STATION	OFFSET	DRILLED SHAFT DIAMETER (FT)	SOCKET DIAMETER (FT)	CAPACITY OF COMPRESSION/ UPLIFT (TONS)	ESTIMATED TOP OF SHAFT ELEVATION	MINIMUM TIP ELEVATION	MINIMUM LENGTH (FT)	MINIMUM SOCKET LENGTH (FT)	DESIGNATED END BEARING STRATUM
S1	79+18.13	7.75	2.5	2.5	57	104.00	-28.0	132.0	3	B1/B3
S2	79+18.13	23.39	2.5	2.5	57	104.00	-28.0	132.0	3	B1/B3

NOTES:

1. BEARING STRENGTH DESIGNATIONS IN ORDER OF INCREASING QUALITY:
   - GT: GLACIAL TILL/GLACIAL OUTWASH
   - B1: ARGILLITE, FELSITE, OR DACITE, VERY SOFT, VERY SEVERELY OR COMPLETELY WEATHERED
   - B2: ARGILLITE, FELSITE, OR DACITE, SOFT TO VERY SOFT, SEVERELY OR MODERATELY TO SEVERELY WEATHERED
   - B3: ARGILLITE, FELSITE, OR DACITE, HARD TO MEDIUM HARD, SLIGHTLY WEATHERED OR MODERATELY WEATHERED
   - B4: DIABASE, MODERATELY HARD TO SOFT, SLIGHTLY TO SEVERELY WEATHERED, SLIGHTLY TO EXTREMELY FRACTURED, CLOSE TO VERY CLOSE JOINTS
2. DRILLED SHAFT DESIGN PARAMETERS SHALL BE AS FOLLOWS:

STRATUM DESIGNATION	COMPRESSION (KSF)	
	SKIN FRICTION	END BEARING
GT	3.5	30
B1	3.5	30
B2	6	80
B3	14	200
B4	14	200
MARINE CLAY	0	0

3. DRILLED SHAFT CAPACITY FOR COMPRESSION LOADS IS OBTAINED FROM A COMBINATION OF SKIN FRICTION AND END BEARING WITHIN THE DESIGNATED END BEARING STRATUM. UPLIFT CAPACITY IS OBTAINED ONLY FROM SKIN FRICTION IN THE DESIGNATED BEARING STRATA AND MARINE CLAY.
4. AT ANY INDIVIDUAL PIER IN THE DRILLED SHAFT SCHEDULE WHERE THE SOCKET LENGTH EXTENDS THROUGH MORE THAN ONE BEARING STRATUM, THE SPECIFIED CAPACITY WILL BE OBTAINED BY THE SUM OF CAPACITIES OF THE SOCKET LENGTHS IN THE BEARING STRATA SPECIFIED AND THE DESIGNATED END BEARING STRATUM. THE ORDER OF THE SPECIFIED SOCKET LENGTHS IS FROM THE LOWER QUALITY TO HIGHER QUALITY BEARING STRATUM.
5. WHERE BEDROCK AT THE MINIMUM TIP ELEVATION IN THE SCHEDULE DIFFERS FROM THE DESIGNATED END BEARING STRATUM, CONTRACTOR SHALL MODIFY THE DRILLED SHAFT SOCKET LENGTH TO PROVIDE THE REQUIRED CAPACITY.
6. REINFORCING STEEL SHALL BE FIELD CUT TO MAINTAIN THE COVER AT THE TOP OF THE FOOTING.
7. NEGATIVE OFFSET MEANS TO THE LEFT.
8. ULTIMATE CAPACITY OF DRILLED SHAFT IS 114 TONS. THE DESIGN AXIAL CAPACITY IS 57 TONS (SERVICE LOAD)
9. CROSS HOLE SONIC LOGGING (CSL) ACCESS TUBES SHALL BE PLACED IN ALL DRILLED SHAFTS.

REV	DATE	BY	SUB	APP	DESCRIPTION
4	28JAN02	PE	ES	ES	INCORP. ADD 8 NARRATIVE REMOVED BID ONLY NOTE
3	14DEC01				REVISED TITLE
2	21NOV01				REVISED TABLE, ADD NOTE 9
1	12OCT01				REVISED SHAFT REQMNTS

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER CODOC
SUBMITTED:

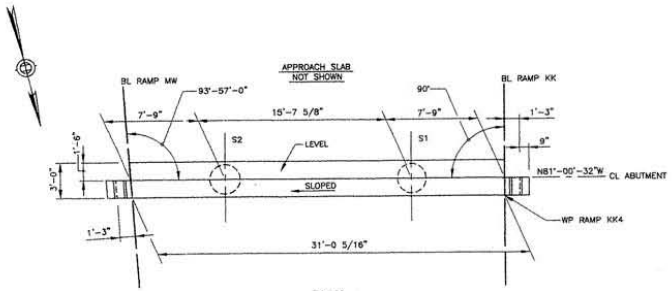
BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
DRILLED SHAFT SCHEDULE

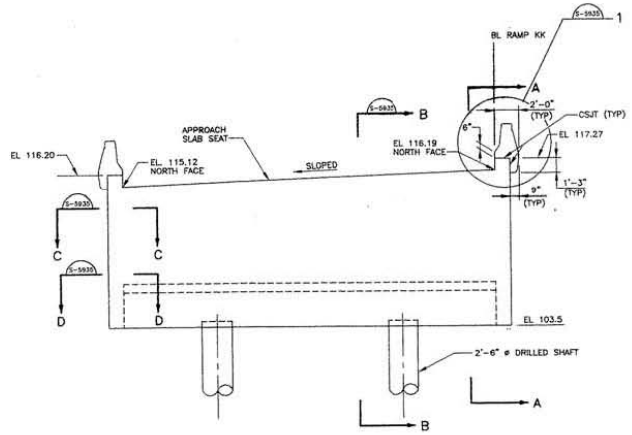
SCALE: 1" = 1'-0"
CONTRACT NO. C09C2
DRAWING NO. S-5933
REV. 4

BOSTON INTERSTATE I-93/I-90

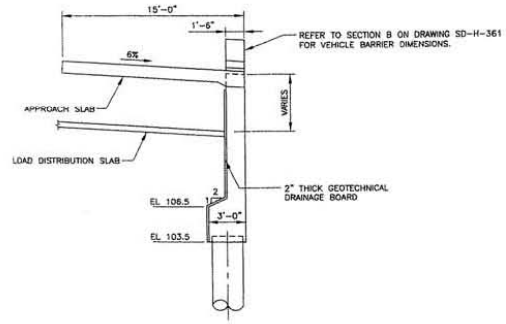
F.H.W.A. REGN. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1503	



PLAN
SCALE: 1/4"=1'-0"



ELEVATION
SCALE: 1/4"=1'-0"



SECTION A
SCALE: 1/4"=1'-0"

NOTES:

1. REFER TO S-5935 FOR APPROACH SLAB DETAILS.

0 4 8 12 24
1/4" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
4	26JAN02	JZ	ES	ES	REMOVED BID ONLY NOTE
3	14DEC01				ADD DRAINAGE BOARD
2	21NOV01				REV NOTE/ABUTMENT THICK & EL
1	12OCT01				DESIGN DEVELOPMENT

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

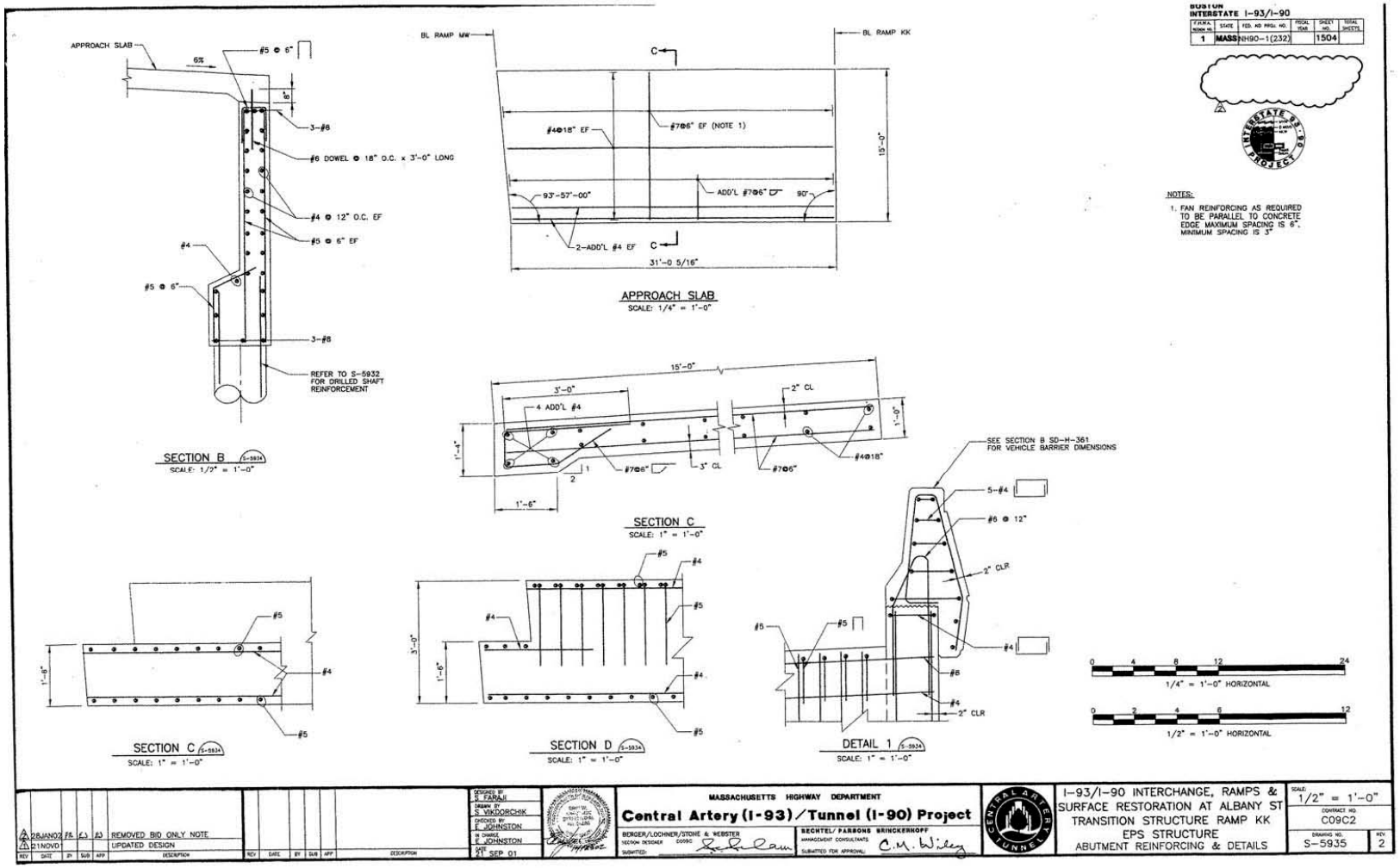
Central Artery (I-93)/Tunnel (I-90) Project

BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOORC
SUBMITTED

RECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

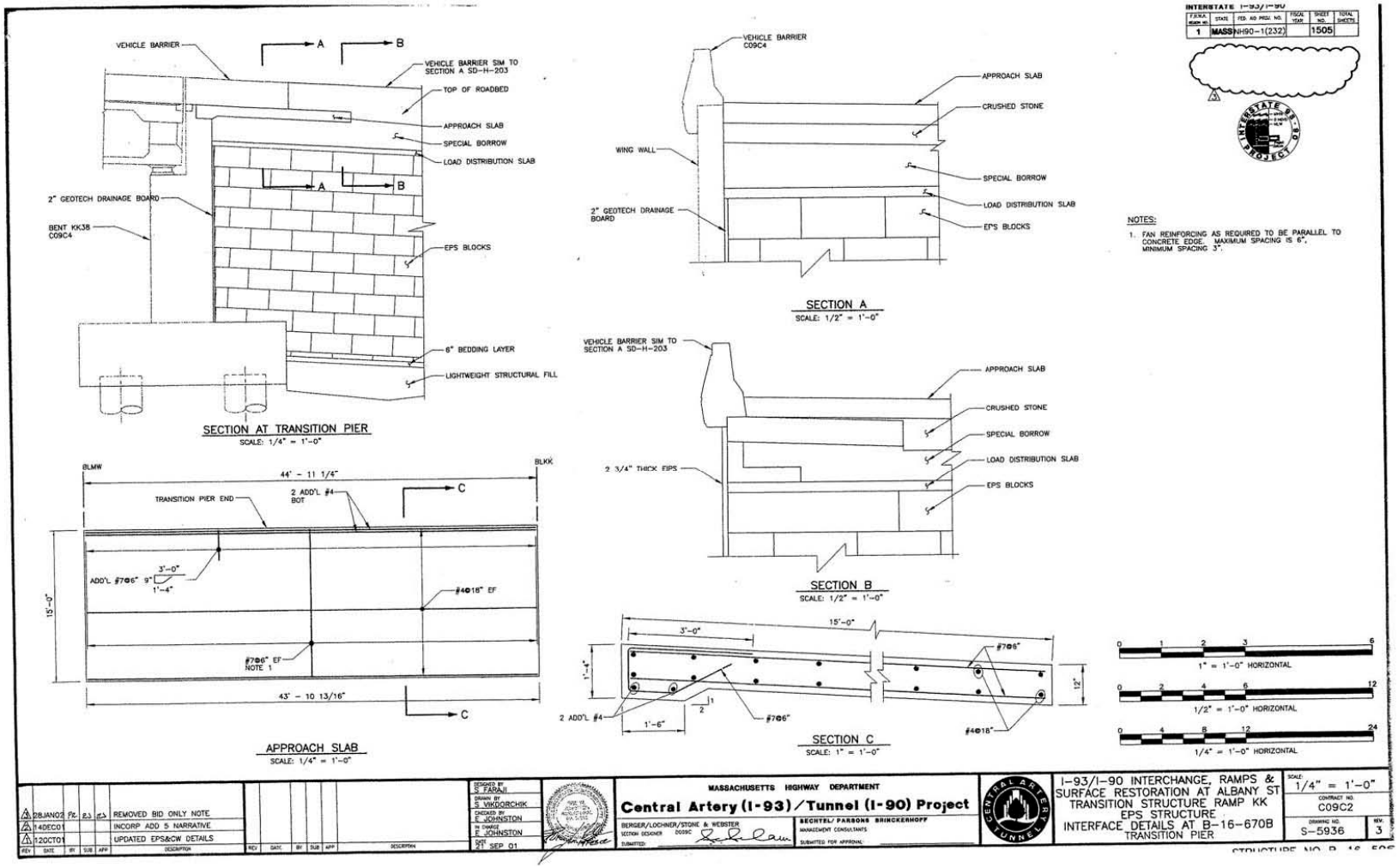
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
ABUTMENT PLAN & ELEVATION

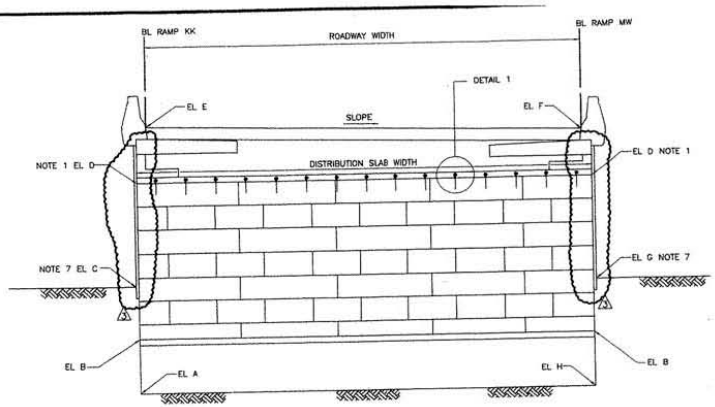
SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-5934
REV. 4



NOTES:

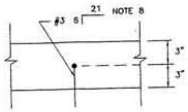
1. FAN REINFORCING AS REQUIRED TO BE PARALLEL TO CONCRETE EDGE MAXIMUM SPACING IS 6". MINIMUM SPACING IS 3"


INTERSTATE I-93/I-90
FEDERAL REGION NO. 1
STATE MASS
FED. AID PROJ. NO. NH90-1(232)
FISCAL YEAR
SHEET NO. 1505
TOTAL SHEETS
VEHICLE BARRIER
VEHICLE BARRIER SIM TO SECTION A SD-H-203
TOP OF ROADBED
APPROACH SLAB
SPECIAL BORROW
LOAD DISTRIBUTION SLAB
2" GEOTECH DRAINAGE BOARD
BENT KK38 C09C4
EPS BLOCKS
6" BEDDING LAYER
LIGHTWEIGHT STRUCTURAL FILL
SECTION AT TRANSITION PIER
SCALE: 1/4" = 1'-0"
VEHICLE BARRIER C09C4
WING WALL
2" GEOTECH DRAINAGE BOARD
SECTION A
SCALE: 1/2" = 1'-0"
2 3/4" THICK EIPS
SECTION B
SCALE: 1/2" = 1'-0"
NOTES:
1. FAN REINFORCING AS REQUIRED TO BE PARALLEL TO CONCRETE EDGE. MAXIMUM SPACING IS 6", MINIMUM SPACING 3".
BLMW
BLKK
44' - 11 1/4"
TRANSITION PIER END
2 ADD'L #4 BOT
ADD'L #7@6" 9"
3'-0"
1'-4"
#4@18" EF
#7@6" EF NOTE 1
15'-0"
43' - 10 13/16"
APPROACH SLAB
SCALE: 1/4" = 1'-0"
3'-0"
15'-0"
#7@6"
2 ADD'L #4
1'-6"
#4@18"
12"
SECTION C
SCALE: 1" = 1'-0"
1" = 1'-0" HORIZONTAL
1/2" = 1'-0" HORIZONTAL
1/4" = 1'-0" HORIZONTAL
REV DATE BY SUB APP DESCRIPTION
2 28JAN02 REMOVED BID ONLY NOTE
1 14DEC01 INCORP ADD 5 NARRATIVE
0 12OCT01 UPDATED EPS&CW DETAILS
DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01
MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER C09C
SUBMITTED:
BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL:
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
INTERFACE DETAILS AT B-16-670B
TRANSITION PIER
SCALE: 1/4" = 1'-0"
CONTRACT NO. C09C2
DRAWING NO. S-5936
REV. 3



TYPICAL SECTION & GEOMETRIC SCHEDULE

SCALE: 1/4" = 1'-0"



DETAIL 1

SCALE: 1 1/2" = 1'-0"

STATION	ELEVATION									WIDTH (FT)		SLOPE	ESTIMATED DEAD LOAD DEFLECTION (NOTE 2)
	A	B	C	D	E	F	G	H	REMARKS	ROADWAY	LOAD DIST. SLAB		
77+10.75	NOTE 4	108.25	116.2	125.75	129.75	130.17	116.00	NOTE 4	NOTE 3	44.83	44.83	NOTE 6	0.84
77+26.75	NOTE 4	108.25	116.2	125.33	129.33	129.33	116.00	NOTE 4	----	44.35	44.35	NOTE 6	0.83
77+27.17	NOTE 4	104.80	116.2	125.30	129.30	129.30	116.00	103.21	----	44.32	44.32	NOTE 6	0.83
77+37.17	NOTE 4	104.20	116.2	124.68	128.70	128.68	116.00	NOTE 4	----	43.63	44.88	0.05%	0.81
77+47.17	NOTE 4	103.60	116.2	123.98	128.10	127.98	114.81	NOTE 4	----	42.95	44.20	0.28%	0.78
77+57.17	NOTE 4	104.60	116.2	123.29	127.50	127.29	112.50	NOTE 4	----	42.26	43.51	0.50%	0.76
77+67.17	NOTE 4	104.00	116.2	122.60	126.90	126.60	110.91	NOTE 4	----	41.57	42.82	0.73%	0.74
77+77.17	NOTE 4	104.80	116.2	121.91	126.30	125.91	110.00	NOTE 4	----	40.88	42.13	0.95%	0.72
77+87.17	NOTE 4	104.20	116.2	121.23	125.70	125.23	109.75	NOTE 4	----	40.19	41.44	1.18%	0.69
77+97.17	NOTE 4	103.60	116.2	120.54	125.10	124.54	109.25	NOTE 4	----	39.50	40.75	1.41%	0.67
78+07.17	NOTE 4	106.20	116.2	119.87	124.50	123.87	109.00	102.06	----	38.81	40.06	1.63%	0.65
78+17.17	NOTE 4	105.60	116.2	119.19	123.90	123.19	109.00	103.33	----	38.12	39.37	1.86%	0.63
78+27.17	NOTE 4	105.00	116.2	118.52	123.30	122.52	109.00	NOTE 4	----	37.44	38.69	2.09%	0.61
78+37.17	NOTE 4	105.90	116.2	117.85	122.70	121.85	109.00	102.91	----	36.75	38.00	2.31%	0.60
78+47.17	NOTE 4	105.30	114.36	117.18	122.10	121.18	109.00	104.05	----	36.06	37.31	2.54%	0.59
78+57.17	NOTE 4	104.70	111.38	116.52	121.50	120.52	109.00	NOTE 4	----	35.37	36.62	2.77%	0.57
78+67.17	NOTE 4	104.10	109.50	115.86	120.90	119.86	109.00	NOTE 4	----	34.68	35.93	2.99%	0.56
78+77.17	98.47	103.50	109.50	115.21	120.30	119.21	109.00	NOTE 4	----	33.99	35.24	3.22%	0.55
78+87.17	97.33	105.37	109.50	114.55	119.70	118.55	109.00	101.02	----	33.30	34.55	3.44%	0.53
78+97.17	96.07	104.77	109.50	113.90	119.10	117.90	109.00	99.95	----	32.61	33.86	3.67%	0.52
79+07.17	95.11	104.16	109.50	113.26	118.50	117.26	108.50	98.56	----	31.93	33.18	3.90%	0.50
79+16.63	95.47	104.60	109.50	112.65	117.93	116.65	108.50	98.53	----	31.27	32.52	4.11%	0.48

FORM NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1506	

FOR BIDDING PURPOSES ONLY

NOTES:

1. ELEVATION D IS THE FINAL ELEVATION AFTER CONSTRUCTION IS COMPLETE. THE EPS BLOCKS WILL DEFLECT DUE TO THE WEIGHT OF THE ELEMENTS ABOVE THE BLOCKS. THE CONTRACTOR IS RESPONSIBLE TO ENSURE THAT ELEVATION D IS OBTAINED AT CONSTRUCTION COMPLETION AND SHALL TAKE APPROPRIATE MEASURES WHEN INSTALLING THE EPS TO AN UNLOADED HEIGHT TO ACCOUNT FOR THIS DEFLECTION.
2. AN ESTIMATED DEAD LOAD DEFLECTION (IN) IS PROVIDED IN THE TABLE. THIS VALUE IS THE RESPONSIBILITY OF, AND SHALL BE DETERMINED BY, THE CONTRACTOR.
3. ELEVATIONS A, B AND H, FOR STATIONS NOTED, ARE AT THE TOP OF THE TRANSITION PIER FOUNDATION. REFER TO DRAWING C09C4-S-3951.
4. ZERO LIGHTWEIGHT STRUCTURAL FILL NEEDED AT THIS LOCATION.
5. THICKNESS OF LIGHTWEIGHT STRUCTURAL FILL SHALL VARY LINEAR BETWEEN ELEVATION POINTS A AND H, IN BOTH THE LONGITUDINAL AND TRANSVERSE DIRECTIONS.
6. REFER TO DRAWING C-1165.
7. THESE ARE APPROXIMATE FINISH GRADES. FOR ACTUAL FINISHED GRADES REFER TO DRAWING L-283.
8. SEISMIC CONNECTORS SHALL BE PLACED NO CLOSER THAN 1'-0" TO THE EDGE OF AN EPS BLOCK AND SHALL NOT EXCEED 2'-0" O.C. EW.

0 1 2 4

1 1/2"=1'-0" HORIZONTAL

0 4 8 12 24

1/4" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
4	4DEC01				INCORP ADD 5 NARRATIVE
3	21NOV01				ADD EDGE WALL
					DEL SHFTS & GRADING BEAM
1	12OCT01				ADD SEISMIC CONN. REQMNTS

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKOORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

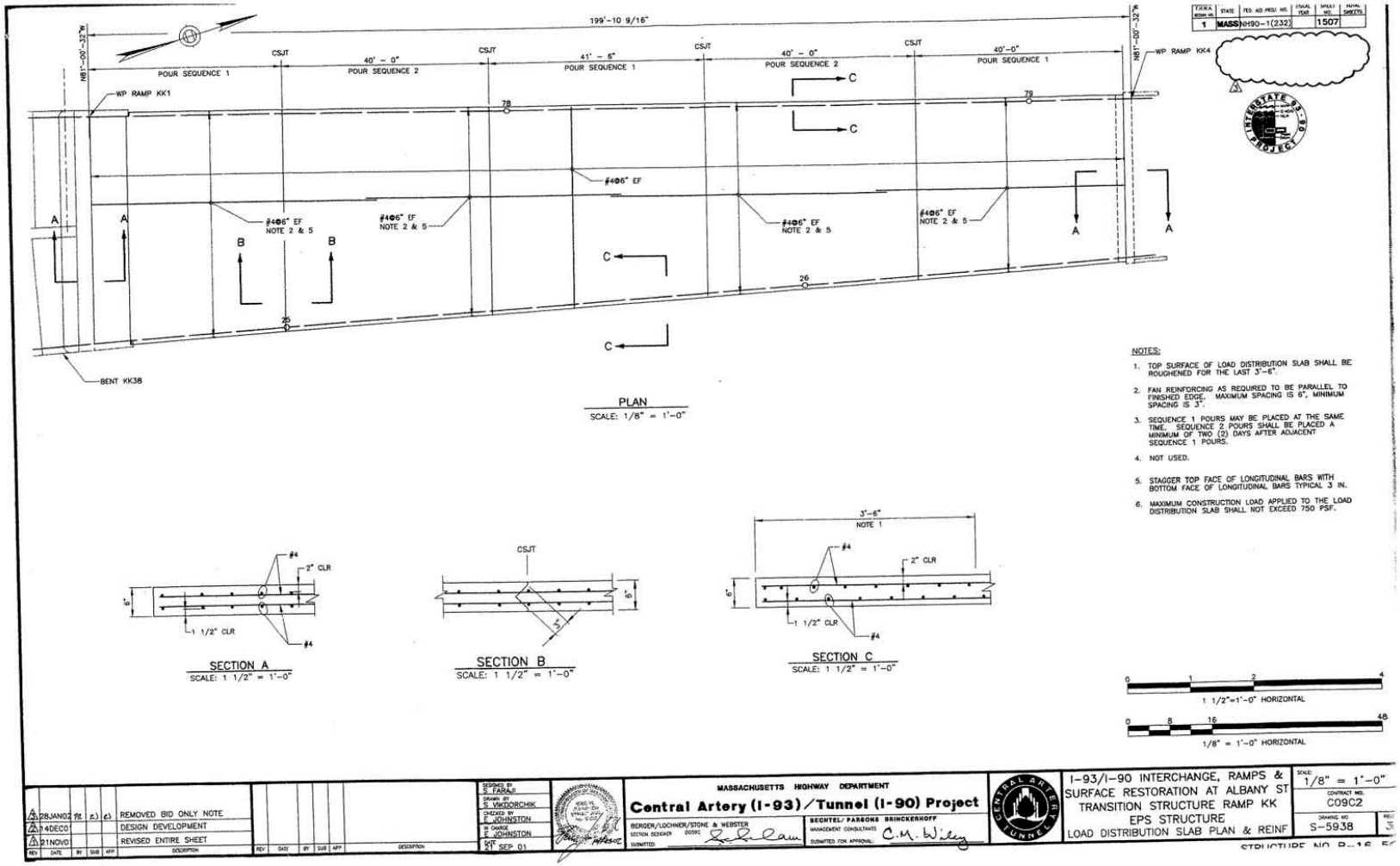
Central Artery (I-93)/Tunnel (I-90) Project

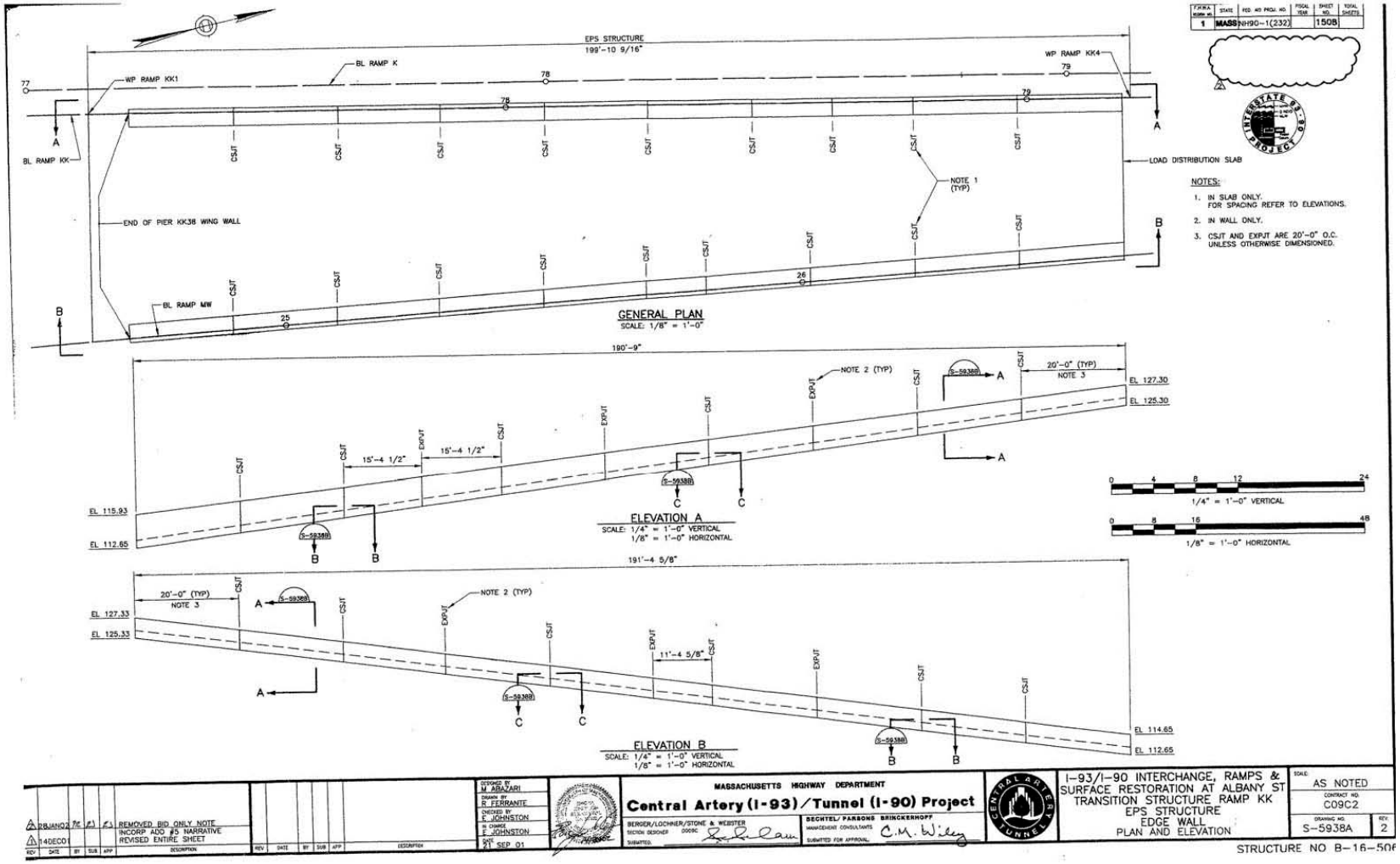
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOERNC
SUBMITTED

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
TYPICAL SECTION & GEOMTERIC SCHEDULE

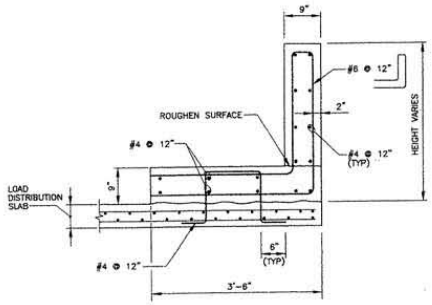
SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-5937


199'-10 9/16"
N81°-00'-32"W
POUR SEQUENCE 1
40' - 0"
POUR SEQUENCE 2
41' - 6"
POUR SEQUENCE 1
40' - 0"
POUR SEQUENCE 2
40'-0"
POUR SEQUENCE 1
CSJT
WP RAMP KK1
WP RAMP KK4
BENT KK38
#4@6" EF
#4@6" EF NOTE 2 & 5
PLAN
SCALE: 1/8" = 1'-0"
NOTES:
1. TOP SURFACE OF LOAD DISTRIBUTION SLAB SHALL BE ROUGHENED FOR THE LAST 3"-6".
2. FAN REINFORCING AS REQUIRED TO BE PARALLEL TO FINISHED EDGE. MAXIMUM SPACING IS 6", MINIMUM SPACING IS 3".
3. SEQUENCE 1 POURS MAY BE PLACED AT THE SAME TIME. SEQUENCE 2 POURS SHALL BE PLACED A MINIMUM OF TWO (2) DAYS AFTER ADJACENT SEQUENCE 1 POURS.
4. NOT USED.
5. STAGGER TOP FACE OF LONGITUDINAL BARS WITH BOTTOM FACE OF LONGITUDINAL BARS TYPICAL 3 IN.
6. MAXIMUM CONSTRUCTION LOAD APPLIED TO THE LOAD DISTRIBUTION SLAB SHALL NOT EXCEED 750 PSF.
3"-6"
NOTE 1
#4
2" CLR
1 1/2" CLR
SECTION A
SCALE: 1 1/2" = 1'-0"
SECTION B
SCALE: 1 1/2" = 1'-0"
SECTION C
SCALE: 1 1/2" = 1'-0"
1 1/2"=1'-0" HORIZONTAL
1/8" = 1'-0" HORIZONTAL
MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
BECHTEL/PARSONS BRINCKERHOFF
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST TRANSITION STRUCTURE RAMP KK EPS STRUCTURE
LOAD DISTRIBUTION SLAB PLAN & REINF
1/8" = 1'-0"
C09C2
S-5938
REMOVED BID ONLY NOTE
DESIGN DEVELOPMENT
REVISED ENTIRE SHEET
S. VIKDORCHIK
E. JOHNSTON
21 SEP 01

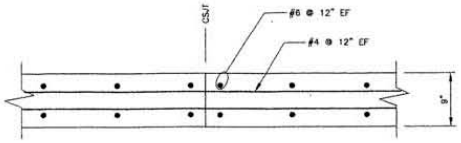

EPS STRUCTURE
199'-10 9/16"
WP RAMP KK1
BL RAMP K
WP RAMP KK4
BL RAMP KK
END OF PIER KK38 WING WALL
BL RAMP MW
LOAD DISTRIBUTION SLAB
NOTE 1 (TYP)
CSJT
GENERAL PLAN
SCALE: 1/8" = 1'-0"
190'-9"
NOTE 2 (TYP)
20'-0" (TYP)
NOTE 3
EXPJT
15'-4 1/2"
EL 127.30
EL 125.30
EL 115.93
EL 112.65
ELEVATION A
SCALE: 1/4" = 1'-0" VERTICAL
1/8" = 1'-0" HORIZONTAL
191'-4 5/8"
11'-4 5/8"
EL 127.33
EL 125.33
EL 114.65
ELEVATION B
1/4" = 1'-0" VERTICAL
1/8" = 1'-0" HORIZONTAL
NOTES:
1. IN SLAB ONLY. FOR SPACING REFER TO ELEVATIONS.
2. IN WALL ONLY.
3. CSJT AND EXPJT ARE 20'-0" O.C. UNLESS OTHERWISE DIMENSIONED.
MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
EDGE WALL
PLAN AND ELEVATION
AS NOTED
C09C2
S-5938A
STRUCTURE NO B-16-50

INTERSTATE I-93/I-90

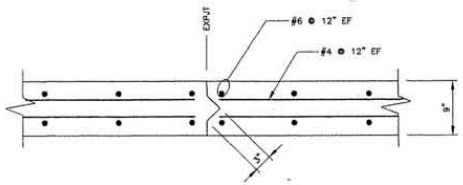
F.A.N.A. REGION NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1509	



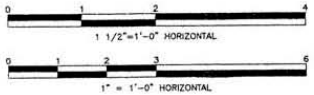
SECTION A (S-5938A)
SCALE: 1" = 1'-0"



SECTION B (S-5938A)
SCALE: 1 1/2" = 1'-0"



SECTION C (S-5938A)
SCALE: 1 1/2" = 1'-0"



REV	DATE	BY	SUB	APP	DESCRIPTION
2	28JAN02	FE	EJ	ES	REMOVED BID ONLY NOTE
1	14DEC01				REVISED REBAR REQMNTS

DESIGNED BY M ABAZARI
DRAWN BY R FERRANTE
CHECKED BY E JOHNSTON
IN CHARGE E JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

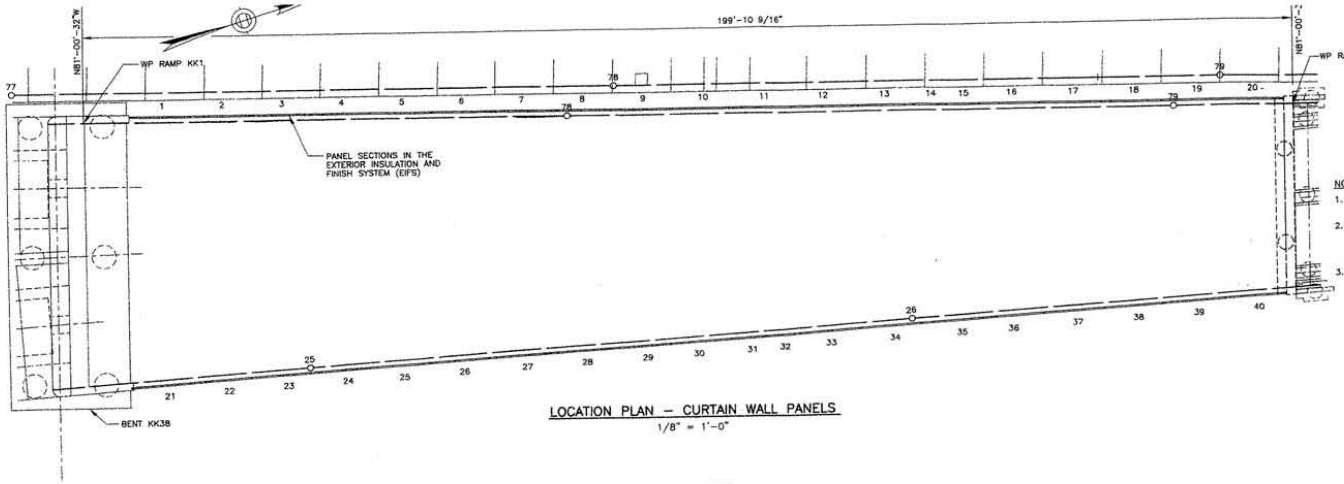
Central Artery (I-93)/Tunnel (I-90) Project

BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER C09C
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
EDGE WALL
SECTIONS AND DETAILS

SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-5938B
REV. 2



LOCATION PLAN – CURTAIN WALL PANELS

1/8" = 1'-0"

1	MASS	NH90-1(232)	1510

NOTES:

1. FOR WORK POINT COORDINATES REFER TO DRAWING S-5927.
2. EIFS SHALL EXTEND FROM THE BOTTOM OF THE VEHICLE BARRIER TO A MINIMUM OF 12 INCHES BELOW FINISHED GRADE. REFER TO DRAWING L-283 FOR FINISH GRADE ELEVATIONS.
3. EXPOSED FACES OF EIFS SHALL MEET RUSTICATION REQUIREMENTS OF DRAWINGS H-2501 AND H-2502.

PANEL SCHEDULE

PANEL NUMBER	PANEL WIDTH	PANEL NUMBER	PANEL WIDTH
1	9.917	21	9.917
2	9.917	22	9.917
3	9.917	23	9.917
4	9.917	24	9.917
5	9.917	25	9.917
6	9.917	26	9.917
7	9.917	27	9.917
8	9.917	28	9.917
9	9.917	29	9.917
10	9.917	30	9.917
11	9.917	31	5.692
12	9.917	32	5.692
13	9.917	33	9.917
14	5.291	34	9.917
15	5.291	35	9.917
16	9.917	36	9.917
17	9.917	37	9.917
18	9.917	38	9.917
19	9.917	39	9.917
20	9.917	40	9.917

0 8 16 48

1/8" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
3	28JAN02	RK	KS	KS	REMOVED BID ONLY NOTE
2	14DEC01				INCORP ADD 5 NARRATIVE
1	21NOV01				REVISED ENTIRE SHEET

DESIGNED BY S. PARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

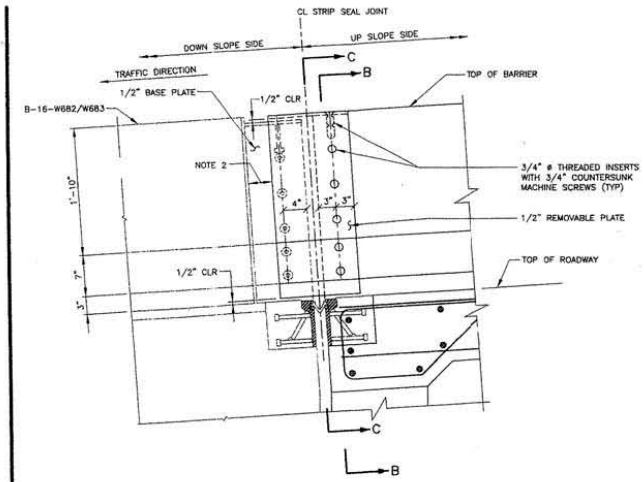
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOD9C
SUBMITTED

BECHTEL/PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL

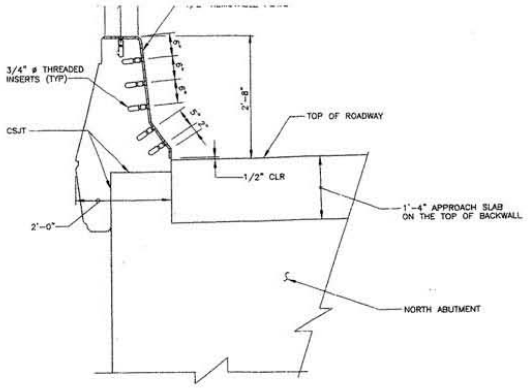
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
EIFS CURTAIN WALL

SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-5940
REV. 3

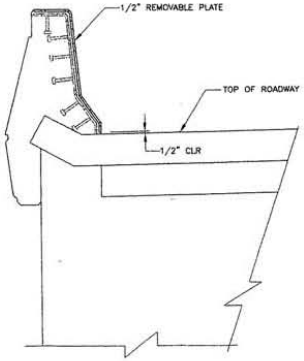
STRUCTURE NO B-16-50



SECTION A (S-5927)
SCALE: 1 1/2" = 1'-0"
ELEVATION – BARRIER PLATE



REMOVABLE PLATE
SECTION B
SCALE: 1" = 1'-0"



SECTION C
SCALE: 1" = 1'-0"

FEDERAL ROUTE NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1511	

NOTES:

1. EXPANSION JOINT IS PART OF B-16-W682/W683 AND IS TO BE PLACED IN ACCORDANCE SD-S-172.
2. SET THIS GAP TO BE THE SAME AS THAT OF THE STRIPSEAL.
3. NO REINFORCING STEEL SHALL BE CUT TO CLEAR THE BRIDGE EXPANSION DEVICE WITHOUT PRIOR APPROVAL OF THE ENGINEER.
4. GALVANIZE ALL STEEL COMPONENTS OF THE JOINT ASSEMBLY INCLUDING STUDS.
5. ALL STEEL PLATES FOR THE VEHICLE BARRIER SHALL BE GALVANIZED.
6. THE JOINT DETAILS SHOWN ARE REPRESENTATIVE OF THE TYPE OF JOINT ACCEPTABLE FOR USE. EXACT JOINT CONFIGURATION WILL DEPEND ON MANUFACTURER'S DETAILS. THE CONTRACTOR SHALL SUBMIT JOINT SHOP DETAILS AND INSTALLATION DRAWINGS FOR ENGINEERS APPROVAL.

0 1 2 3 6
1" = 1'-0" HORIZONTAL

0 1 2 4
1 1/2" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
1	28JAN02	PE	PJ	PJ	REMOVED BID ONLY NOTE

REV	DATE	BY	SUB	APP	DESCRIPTION

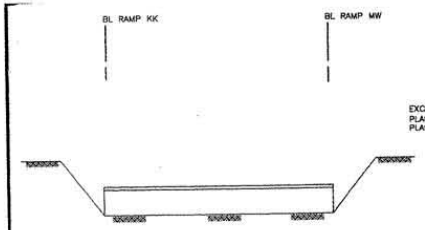
DESIGNED BY S. TARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 12 OCT 01

MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER — SECTION DESIGNER — DOORO — SUBMITTED
BECHTEL/ PARSONS BRINCKERHOFF — MANAGEMENT CONSULTANTS — SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
EXPANSION JOINT DETAILS

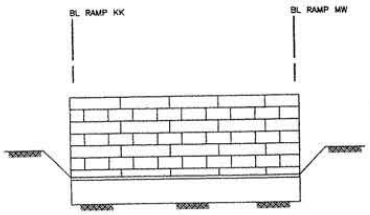
SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-5941
REV. 1

STRUCTURE NO B-16-50E



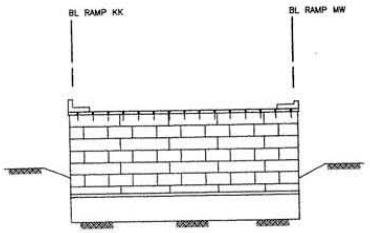
EXCAVATE TO REQUIRED ELEVATIONS
PLACE LIGHTWEIGHT STRUCTURAL FILL
PLACE BEDDING LAYER

STAGE 1
NTS



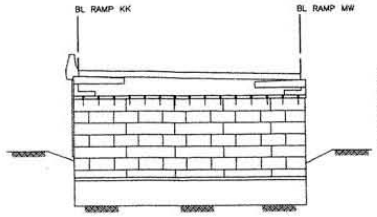
INSTALL EPS BLOCKS

STAGE 2
NTS



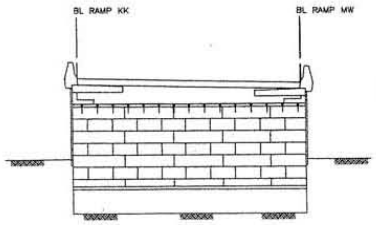
CONSTRUCT LOAD DISTRIBUTION SLAB
CONSTRUCT EDGE WALLS

STAGE 3
NTS



PLACE SPECIAL BORROW
CONSTRUCT EAST VEHICLE BARRIER
CONSTRUCT WEST VEHICLE BARRIER FOOTING
CONSTRUCT APPROACH SLABS
PLACE ROADWAY PAVEMENT
INSTALL EXPANSION JOINTS
CONSTRUCT EAST EIFS CURTAIN WALL
CONSTRUCT AND OPEN TEMPORARY RAMP KK
CLOSE AND DISMANTLE TEMPORARY RAMP KK

STAGE 4
NTS



CONSTRUCT WEST VEHICLE BARRIER
CONSTRUCT WEST EIFS CURTAIN WALL

STAGE 5
NTS

F.H.W.A. DOCKET NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1512	

NOTES:

1. THE CONSTRUCTION SEQUENCE DEPICTED HERE IS A SUGGESTED SEQUENCE. THE CONTRACTOR SHALL DEVELOP AND SUBMIT A CONSTRUCTION SEQUENCE PLAN TO THE ENGINEER FOR APPROVAL.
2. MAXIMUM CONSTRUCTION LOAD APPLIED DIRECTLY TO THE LOAD DISTRIBUTION SLAB SHALL NOT EXCEED 750 PSF. USE OF WOOD CRIBBING/MATS TO SPREAD THE LOAD IS ACCEPTABLE.

REV	DATE	BY	SUB	APP	DESCRIPTION
2	28JAN02	RR	KS	KS	REMOVED BID ONLY NOTE
1	14DEC01				INCORP ADD 5 NARRATIVE
1	21NOV01				ADD EDGE WALL, REV NOTES DEL SHIFTS & GRADING BEAM
1	24OCT01				ADDED NOTE 2

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

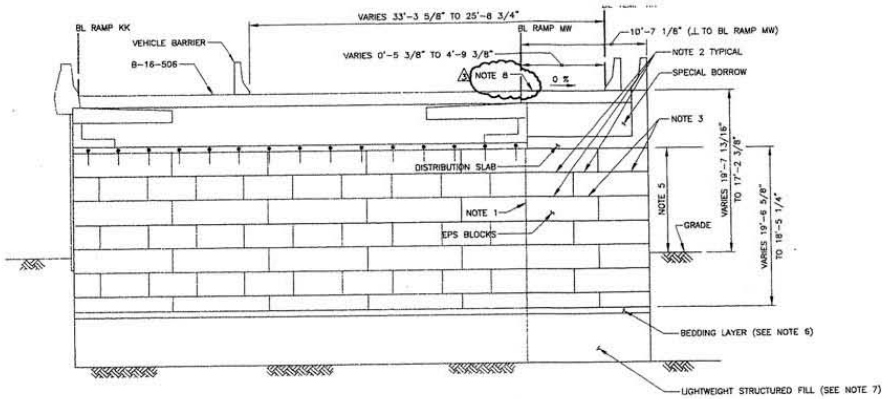
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER CC09C
SUBMITTED: [signature]

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST TRANSITION STRUCTURE RAMP KK
EPS STRUCTURE
CONSTRUCTION SEQUENCING

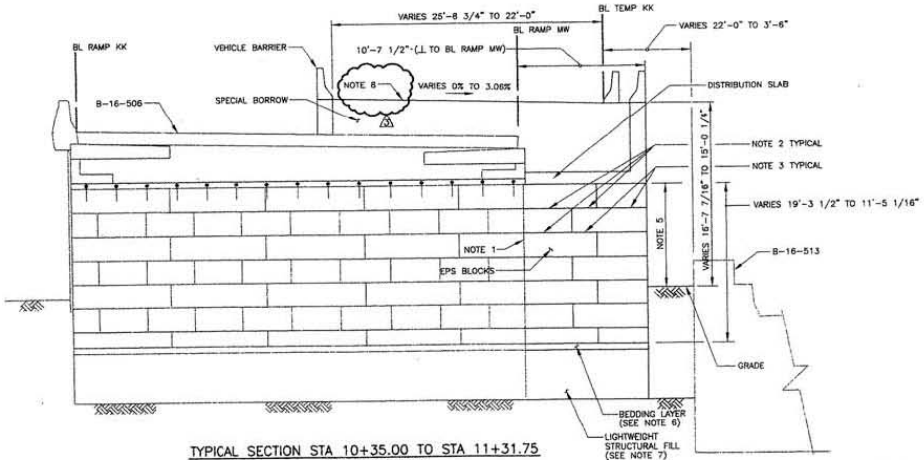
SCALE: NO SCALE
CONTRACT NO. C09C2
DRAWING NO. S-5942
REV. 4

STRUCTURE NO. B-16-506



TYPICAL SECTION STA 09+94.11 TO STA 10+35.00

SCALE: 1/4" = 1'-0"



TYPICAL SECTION STA 10+35.00 TO STA 11+31.75

SCALE: 1/4" = 1'-0"

F.H.W.A. SERIAL NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1605	

NOTES:

1. INTER-BLOCK CONNECTOR PLATES SHALL BE PLACED ALONG THE VERTICAL INTERFACE BETWEEN B-16-506 AND TEMPORARY RAMP KK EPS STRUCTURE.
2. INTER-BLOCK CONNECTOR PLATES SHALL NOT BE PLACED BETWEEN HORIZONTAL SURFACES OF ANY EPS BLOCK THAT HAS A VERTICAL INTER-BLOCK CONNECTOR PLATE PLACED PER NOTE 1.
3. PLACE INTER-BLOCK CONNECTOR PLATES ON THESE HORIZONTAL SURFACES.
4. ALL DIMENSIONS ARE PERPENDICULAR TO TEMP KK BASELINE UNLESS OTHERWISE NOTED.
5. APPLY SHOTCRETE TO THE EXPOSED SURFACE OF EPS BLOCKS. REFER TO S-6411 FOR DETAILS.
6. BEDDING LAYER CONSISTS OF NON-WOVEN POLYPROPYLENE GEOTEXTILE FABRIC WITH A MINIMUM MASS OF 4.0 OZ/SY AND 6 INCHES OF SAND PLACED ON SMOOTHED SOIL SUBGRADE/LIGHTWEIGHT STRUCTURAL FILL.
7. LIGHTWEIGHT STRUCTURAL FILL SHALL BE IN ACCORDANCE WITH THE SPECIFICATIONS.
8. WEARING SURFACE CONSISTS OF GRAVEL BORROW OVERLAID BY 3 INCHES OF BITUMINOUS CONCRETE PAVEMENT TYPE I-1 AND 3 INCHES OF BITUMINOUS CONCRETE BASE COURSE.

0 4 8 12 24

1/4" = 1'-0" HORIZONTAL

DATE	BY	SUB	APP	DESCRIPTION
28JAN02	PE	ES	ES	INCORP ADD 9 NARRATIVE REMOVED BID ONLY NOTE
14DEC01				DESIGN DEVELOPMENT & INCORP ADD 5 NARRATIVE
21NOV01				REV RAMP KK

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY F. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

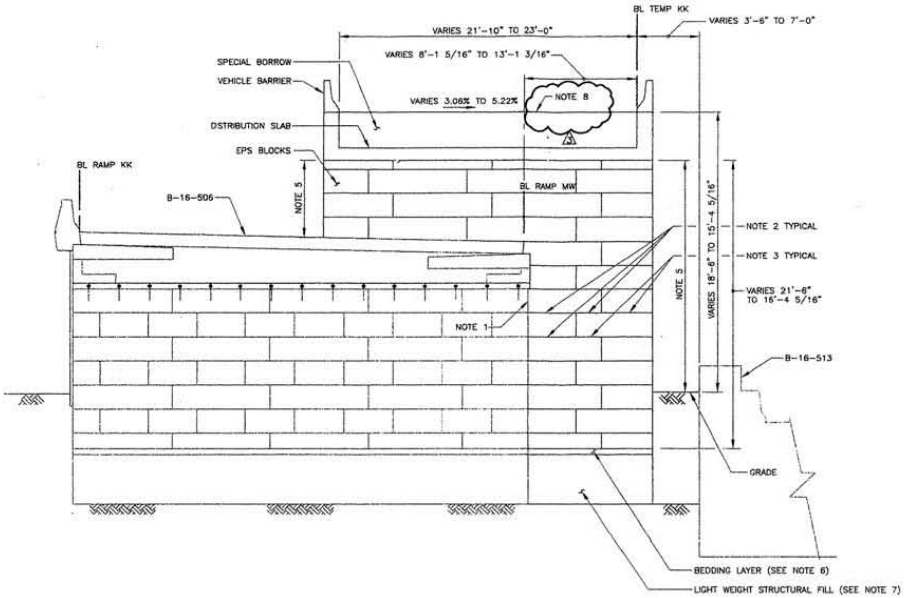
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOCOC
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
TYPICAL SECTION
SHEET 1 OF 2

SCALE: 1/4" = 1'-0"
CONTRACT NO. C09C2
DRAWING NO. S-6404
REV. 3

F.A.H.A. REGION NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1606	



TYPICAL SECTION STA 11+31.75 TO STA 12+00.31

SCALE: 1/4" = 1'-0"

NOTES:

1. REFER TO DRAWING S-6404 FOR NOTES.

0 4 8 12 24

1/4" = 1'-0" HORIZONTAL

DATE	BY	SUB	APP	DESCRIPTION
JAN02	PZ	KJ	EJ	INCORP ADD 9 NARRATIVE REMOVED BID ONLY NOTE
DEC01				DESIGN DEVELOPMENT & INCORP ADD 5 NARRATIVE
NOV01				REV RAMP KK

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKBORCHIK
CHECKED BY F. JOHNSTON
IN CHARGE F. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

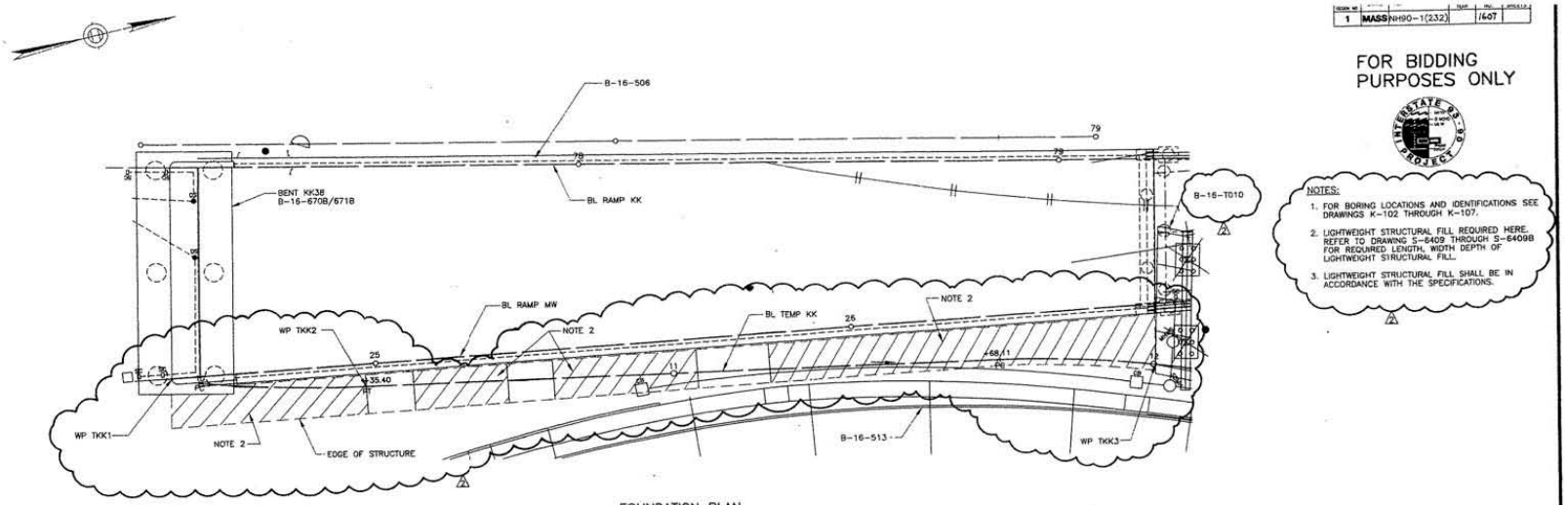
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOORC
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
TYPICAL SECTION
SHEET 2 OF 2

SCALE: 1/4" = 1'-0"
CONTRACT NO. C09C2
DRAWING NO. S-6404A
REV. 3

STRUCTURE NO. B-16-TXX



FOUNDATION PLAN

DESIGN NO				YEAR	PKG	SHEET
1	MASS	NH90-1(232)			1607	

FOR BIDDING PURPOSES ONLY

NOTES:

1. FOR BORING LOCATIONS AND IDENTIFICATIONS SEE DRAWINGS K-102 THROUGH K-107.
2. LIGHTWEIGHT STRUCTURAL FILL REQUIRED HERE. REFER TO DRAWING S-6408 THROUGH S-6409B FOR REQUIRED LENGTH, WIDTH DEPTH OF LIGHTWEIGHT STRUCTURAL FILL.
3. LIGHTWEIGHT STRUCTURAL FILL SHALL BE IN ACCORDANCE WITH THE SPECIFICATIONS.

0 10 20 50 FEET

1" = 10'-0" HORIZONTAL

DATE	BY	SUB	APP	DESCRIPTION
4DEC01				DESIGN DEVELOPMENT
1NOV01				REV RAMP KK

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

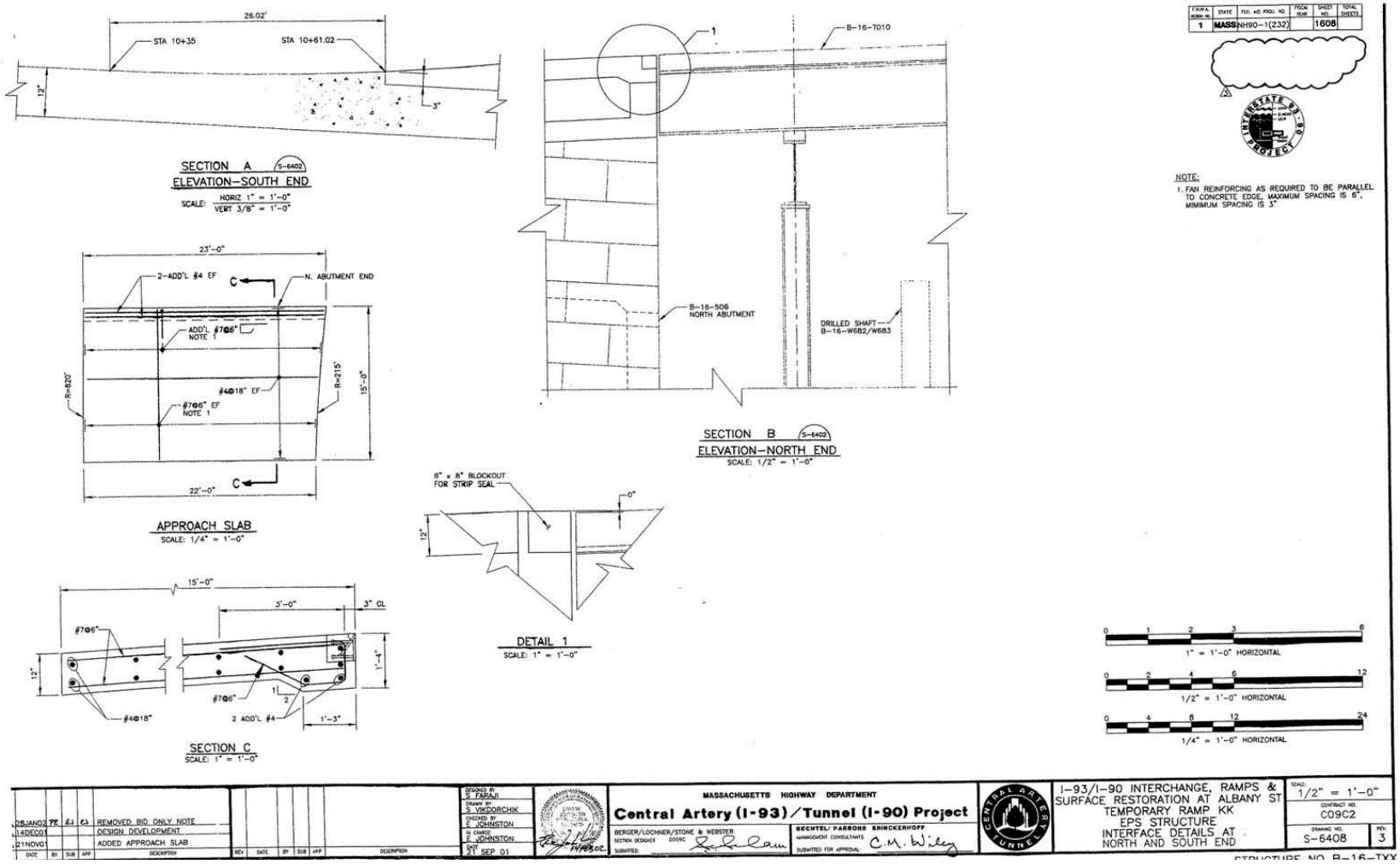
BERGER/LOCHNER/STONE & WEUSTER
SECTION DESIGNER DOGBC
SUBMITTED

BECHTEL/PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
FOUNDATION PLAN

SCALE: 1" = 10'-0"
CONTRACT NO C09C2
DRAWING NO S-6405
REV 2

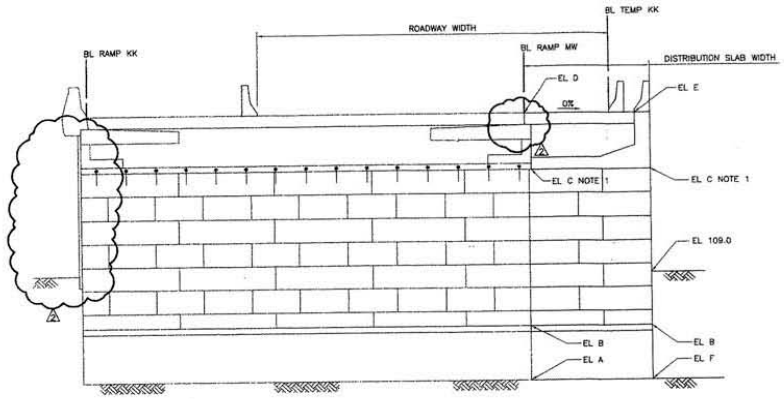
STRUCTURE NO B-16-TX)


26.02'
STA 10+35
STA 10+61.02
12"
3"
SECTION A (S-6402)
ELEVATION-SOUTH END
SCALE: HORIZ 1" = 1'-0" VERT 3/8" = 1'-0"
23'-0"
2-ADD'L #4 EF
N. ABUTMENT END
ADD'L #7@6" NOTE 1
#4@18" EF
#7@6" EF NOTE 1
R=820'
R=215'
15'-0"
22'-0"
APPROACH SLAB
SCALE: 1/4" = 1'-0"
B-16-T010
B-16-506 NORTH ABUTMENT
DRILLED SHAFT B-16-W682/W683
SECTION B (S-6402)
ELEVATION-NORTH END
SCALE: 1/2" = 1'-0"
8" x 8" BLOCKOUT FOR STRIP SEAL
0"
12"
DETAIL 1
SCALE: 1" = 1'-0"
15'-0"
3'-0"
3" CL
#7@6"
#4@18"
2 ADD'L #4
1'-3"
1'-4"
SECTION C
SCALE: 1" = 1'-0"
1" = 1'-0" HORIZONTAL
1/2" = 1'-0" HORIZONTAL
1/4" = 1'-0" HORIZONTAL
NOTE:
1. FAN REINFORCING AS REQUIRED TO BE PARALLEL TO CONCRETE EDGE. MAXIMUM SPACING IS 6". MIMIMUM SPACING IS 3"
FEDERAL AID PROJECT NO. 1
STATE MASS
FED. AID PROJ. NO. NH90-1(232)
SHEET NO. 1608
25JAN02 REMOVED BID ONLY NOTE
14DEC01 DESIGN DEVELOPMENT
21NOV01 ADDED APPROACH SLAB
DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01
MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER C09C
SUBMITTED:
BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL:
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
INTERFACE DETAILS AT NORTH AND SOUTH END
SCALE: 1/2" = 1'-0"
CONTRACT NO. C09C2
DRAWING NO. S-6408
REV. 3
STRUCTURE NO. B-16-TXX

INTERSTATE I-93/I-90

F.A.H.A. REGION NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		1609	

FOR BIDDING PURPOSES ONLY



TYPICAL SECTION & GEOMETRIC SCHEDULE STA 09+94.11 TO STA 10+35.00

SCALE: 1/4" = 1'-0"

STATION	ELEVATION							WIDTH (FT)		ESTIMATED DEAD LOAD DEFLECTION (NOTE 2)
	A	B	C	D	E	F	REMARKS	ROADWAY	DISTRIBUTION SLAB	
09+94.11	NOTE 4	106.56	125.00	130.15	130.15	98.76	NOTE 3 & 6	33.30	7.33	0.74
10+07.16	101.11	105.78	124.51	129.37	129.37	101.11	----	31.19	9.93	0.75
10+17.00	102.18	105.18	124.73	128.78	128.78	102.18	----	29.15	9.88	0.78
10+27.00	NOTE 4	104.58	124.03	128.18	128.18	NOTE 4	----	27.25	9.85	0.78
10+35.00	NOTE 4	104.10	123.48	127.70	127.70	NOTE 4	----	25.73	9.84	0.79

NOTES:

1. ELEVATION C IS THE FINAL ELEVATION AFTER CONSTRUCTION IS COMPLETE. THE EPS BLOCKS WILL DEFLECT DUE TO THE WEIGHT OF THE ELEMENTS ABOVE THE BLOCKS. THE CONTRACTOR IS RESPONSIBLE TO ENSURE THAT ELEVATION C IS OBTAINED AT CONSTRUCTION COMPLETION AND SHALL TAKE APPROPRIATE MEASURES WHEN INSTALLING THE EPS TO AN UNLOADED HEIGHT TO ACCOUNT FOR THIS DEFLECTION.
2. AN ESTIMATED DEAD LOAD DEFLECTION (IN) IS PROVIDED IN THE TABLE. THIS VALUE IS THE RESPONSIBILITY OF, AND SHALL BE CONFIGURED BY, THE CONTRACTOR.
3. ELEVATIONS A, B AND G, FOR STATIONS NOTED, ARE AT THE TOP OF THE TRANSITION PIER FOUNDATION. REFER TO DRAWING C09C4-S-3951.
4. ZERO LIGHTWEIGHT STRUCTURAL FILL NEEDED AT THIS LOCATION.
5. THICKNESS OF LIGHTWEIGHT STRUCTURAL FILL SHALL VARY LINEAR BETWEEN ELEVATION POINTS A AND F, IN BOTH THE LONGITUDINAL AND TRANSVERSE DIRECTIONS.
6. TABLE INFORMATION ON THIS LINE IS AT A SKEW (I.E., NOT PERPENDICULAR TO THE BASELINE).

0 4 8 12 24

1/4" = 1'-0" HORIZONTAL

ADDENDUM 7 OF CONTRACT C09C2

DATE	BY	SUB	APP	DESCRIPTION
14DEC01				INCORP ADD 5 NARRATIVE
21NOV01				REVISED ENTIRE DRAWING

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

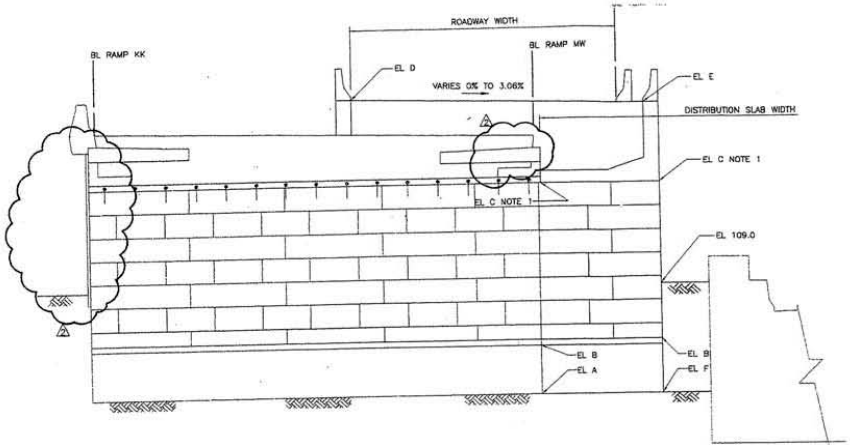
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DRAWING
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL:

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
TYPICAL SECTION & GEOMTERIC SCHEDULE
SHEET 1 OF 3

SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-6409
REV. 2

STRUCTURE NO. B-15-TMV



TYPICAL SECTION & GEOMETRIC SCHEDULE STA 10+35.00 TO STA 11+31.75

SCALE: 1/4" = 1'-0"

FOR BIDDING PURPOSES ONLY

NOTES:

1. REFER TO DRAWING S-6409 FOR NOTES.

STATION	ELEVATION							WIDTH (FT)		SLOPE	ESTIMATED DEAD LOAD DEFLECTION (NOTE 2)
	A	B	C	D	E	F	REMARKS	ROADWAY	DISTRIBUTION SLAB		
10+35.00	NOTE 4	104.10	123.48	127.70	127.70	NOTE 4	----	25.73	9.84	0.00%	0.79
10+45.00	101.20	103.50	122.79	127.14	127.12	101.20	----	24.03	9.84	0.32%	0.78
10+55.00	105.00	105.30	122.10	126.67	126.62	105.00	----	22.85	9.84	0.63%	0.70
10+65.00	NOTE 4	104.70	121.41	126.29	126.22	NOTE 4	----	22.18	9.84	0.95%	0.70
10+75.00	NOTE 4	104.10	120.73	126.00	125.91	NOTE 4	----	22.00	9.84	1.26%	0.72
10+85.00	101.39	103.50	120.05	125.80	125.69	101.39	----	22.00	9.84	1.58%	0.73
10+95.00	104.94	105.30	119.38	125.70	125.56	104.94	----	22.00	9.84	1.89%	0.66
11+05.00	NOTE 4	104.70	118.70	125.68	125.52	NOTE 4	----	22.00	9.84	2.21%	0.68
11+15.00	NOTE 4	104.10	118.03	125.76	125.57	NOTE 4	----	22.00	9.84	2.53%	0.70
11+25.00	100.10	103.50	117.37	125.92	125.71	100.10	----	22.00	9.84	2.84%	0.73
11+31.75	103.75	105.50	116.92	126.59	125.86	103.75	----	22.00	26.52	3.06%	0.66

0 4 8 12 24

1/4" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
1	14DEC01				INCORP ADD 5 NARRATIVE
2	21NOV01				REVISED ENTIRE DRAWING

DESIGNED BY S. FARAJI
DRAWN BY S. VIKOORCHIK
CHECKED BY F. JOHNSTON
IN CHARGE F. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER 0008C
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL:

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST TEMPORARY RAMP KK
EPS STRUCTURE
TYPICAL SECTION & GEOMTERIC SCHEDULE
SHEET 2 OF 3

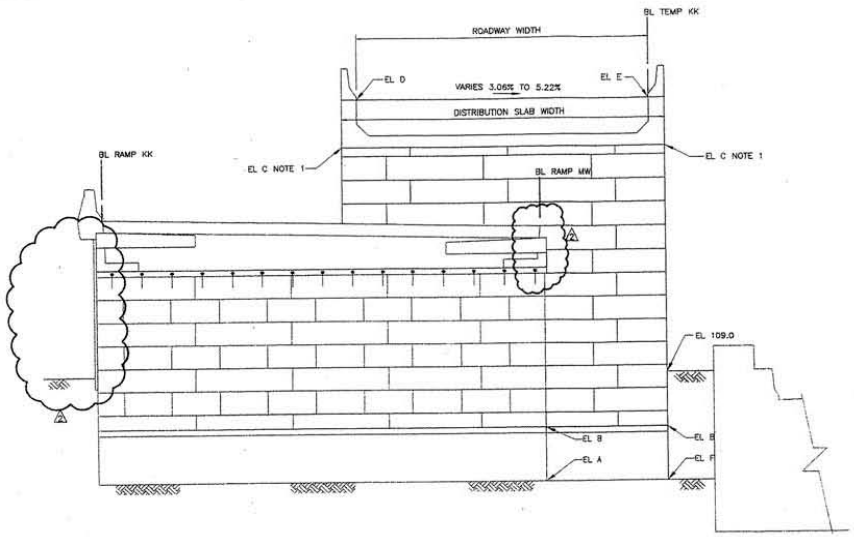
SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-6409A
REV. 2

STRUCTURE NO B-16-TXX

ADDENDUM 7 OF CONTRACT C09C2

INTERSTATE I-93/I-90					
FUNCTIONAL DESIGN NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	NH90-1(232)		16/1	

FOR BIDDING PURPOSES ONLY



TYPICAL SECTION & GEOMETRIC SCHEDULE STA 11+31.75 TO STA 12+00.31

SCALE: 1/4" = 1'-0"

NOTES:

1. REFER TO DRAWING S-6409 FOR NOTES.

STATION	ELEVATION						REMARKS	WIDTH (FT)		SLOPE	ESTIMATED DEAD LOAD DEFLECTION (NOTE 2)
	A	B	C	D	E	F		ROADWAY	DISTRIBUTION SLAB		
11+31.75	103.75	105.50	121.86	126.59	125.86	103.75	----	22.00	26.52	3.06%	0.66
11+35.00	103.86	105.30	121.95	126.70	125.95	103.86	----	22.00	26.39	3.16%	0.67
11+45.00	104.30	104.70	122.27	127.39	126.27	104.30	----	22.00	26.05	3.47%	0.71
11+55.00	99.95	104.10	122.67	127.56	126.67	99.95	----	22.00	25.99	3.79%	0.75
11+65.00	99.27	103.50	123.16	128.13	127.16	99.27	----	21.90	25.90	4.11%	0.79
11+75.00	100.32	105.00	123.74	128.77	127.74	100.32	----	21.80	25.80	4.42%	0.75
11+85.00	99.24	104.40	124.32	129.43	128.32	99.24	----	22.03	26.03	4.74%	0.80
11+95.00	97.80	103.88	124.79	130.01	128.79	97.80	----	22.62	26.63	5.05%	0.84
12+00.31	96.44	103.50	125.00	130.12	129.00	96.44	NOTE 6	23.00	27.01	5.22%	0.86

1/4" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
4	14DEC01				INCORP. ADD. 5 NARRATIVE
3	21NOV01				REVISED ENTIRE DRAWING

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY F. JOHNSTON
IN CHARGE F. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

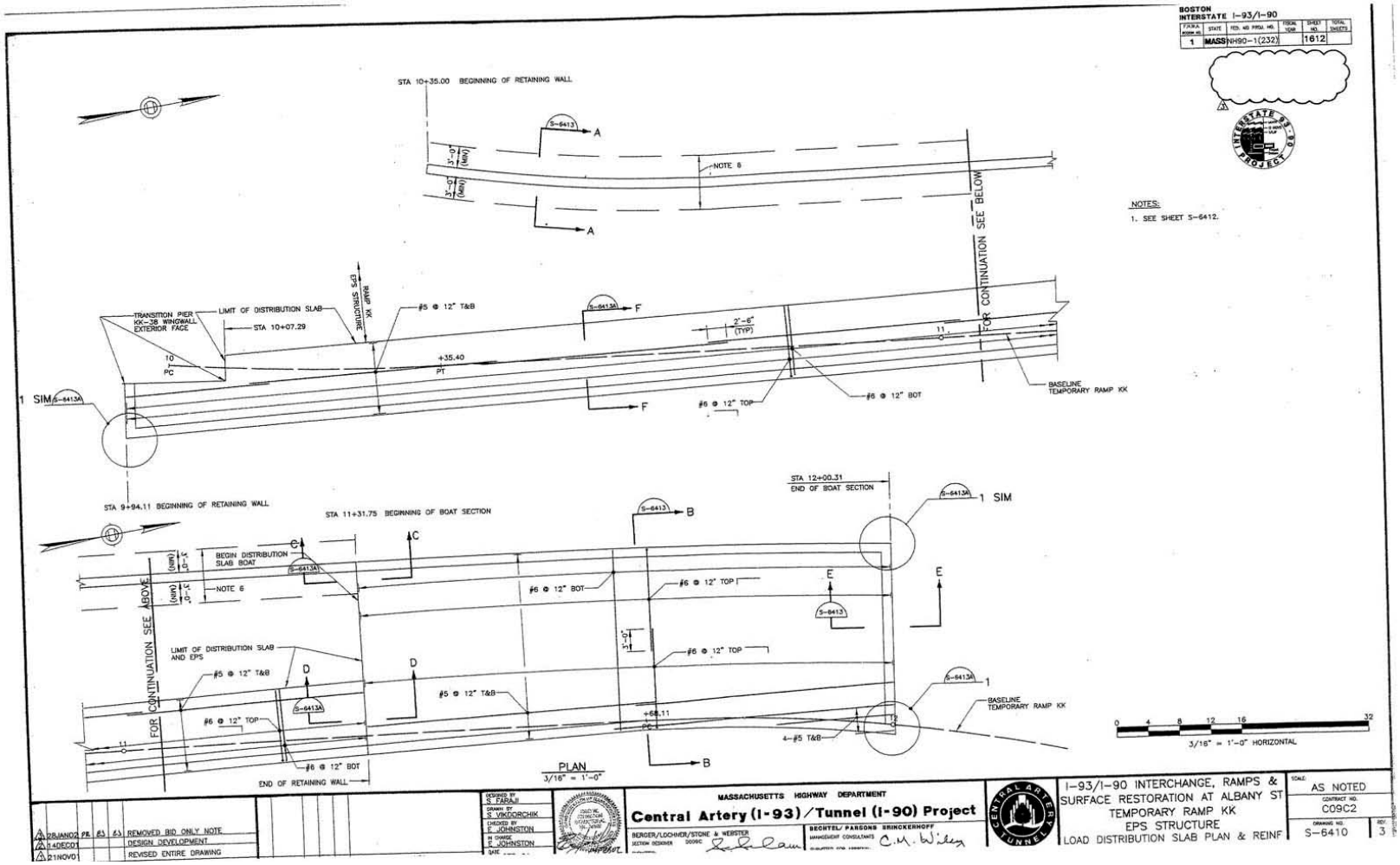
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER
SUBMITTED:

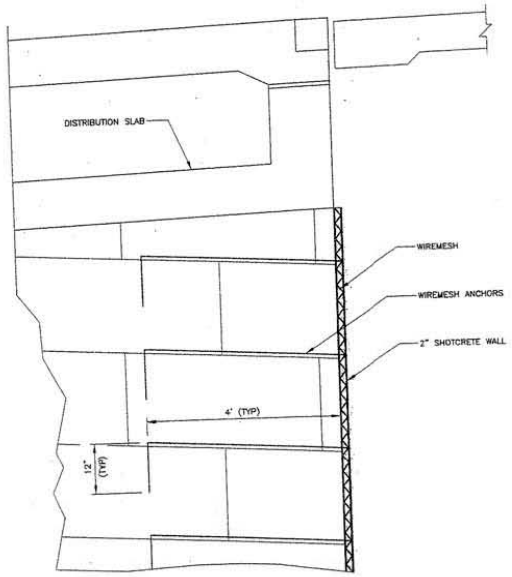
BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL:

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
TYPICAL SECTION & GEOMTERIC SCHEDULE
SHEET 3 OF 3

SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-6409B
REV. 2

STRUCTURE NO. R-16-TXX


BOSTON INTERSTATE I-93/I-90
1 MASS NH90-1(232) 1612
STA 10+35.00 BEGINNING OF RETAINING WALL
NOTE 6
FOR CONTINUATION SEE BELOW
NOTES:
1. SEE SHEET S-6412.
TRANSITION PIER KK-38 WINGWALL EXTERIOR FACE
LIMIT OF DISTRIBUTION SLAB
STA 10+07.29
RAMP KK EPS STRUCTURE
#5 @ 12" T&B
2'-6" (TYP)
+35.40 PT
PC
#6 @ 12" TOP
#6 @ 12" BOT
BASELINE TEMPORARY RAMP KK
1 SIM
STA 9+94.11 BEGINNING OF RETAINING WALL
STA 11+31.75 BEGINNING OF BOAT SECTION
STA 12+00.31 END OF BOAT SECTION
BEGIN DISTRIBUTION SLAB BOAT
FOR CONTINUATION SEE ABOVE
LIMIT OF DISTRIBUTION SLAB AND EPS
+68.11 PC
END OF RETAINING WALL
PLAN
3/16" = 1'-0"
3/16" = 1'-0" HORIZONTAL
MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
BECHTEL/ PARSONS BRINCKERHOFF
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
LOAD DISTRIBUTION SLAB PLAN & REINF
AS NOTED
C09C2
S-6410
3
REMOVED BID ONLY NOTE
DESIGN DEVELOPMENT
REVISED ENTIRE DRAWING



SHOTCRETE WALL DETAIL
SCALE: 1" = 1'-0"

F.H.W.A. REGION NO.	STATE	FED. AID PROJ. NO.	CONTROL YEAR	SHEET NO.	SHEETS
1	MASS	NH90-1(232)		1613	

NOTE:
1. THE DETAIL SHOWN HERE IS REPRESENTATIVE OF HOW TO APPLY SHOTCRETE TO THE EPS BLOCKS. THE CONTRACTOR SHALL DEVELOP MEANS AND SUBMIT TO THE ENGINEER FOR APPROVAL.

0 1 2 3 6
1" = 1'-0" HORIZONTAL

0 2 4 6 12
1/2" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
2	28JAN02	ZP	JD	JD	REMOVED BID ONLY NOTE
1	14DEC01				DESIGN DEVELOPMENT

REV	DATE	BY	SUB	APP	DESCRIPTION

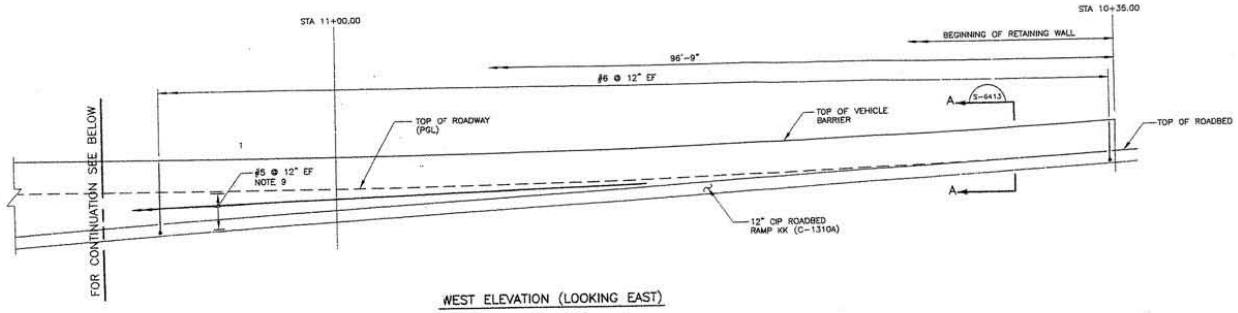
DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER CODING
SUBMITTED:
BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
SHOTCRETE WALL & DETAILS

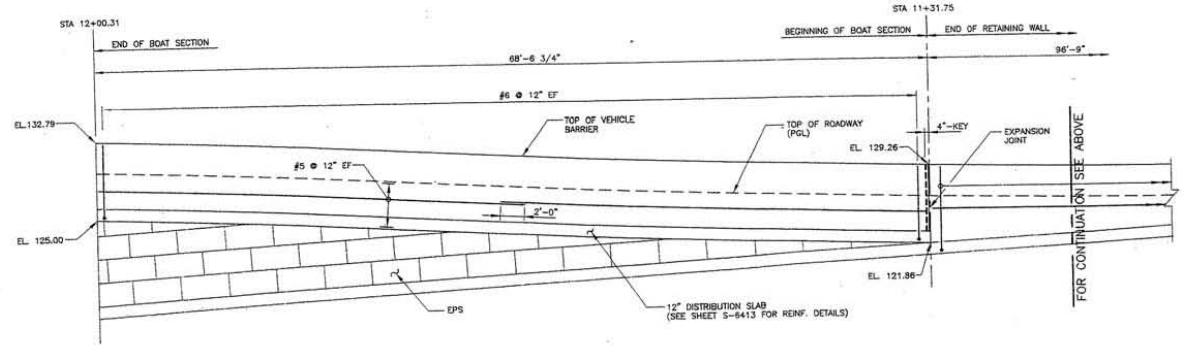
SCALE 1/8" = 1'-0"
CONTRACT NO. C09C2
DRAWING NO. S-6411
REV. 2

STRUCTURE NO B-16-TXX



WEST ELEVATION (LOOKING EAST)

1/4" = 1'-0"



WEST ELEVATION (LOOKING EAST)

1/4" = 1'-0"

NOTES:

1. FOR ADDITIONAL REINFORCEMENT IN RAMP KK ROADBED SEE SHEET S-6413.
2. SEE SD-S-116 FOR TYPICAL VEHICLE BARRIER REINFORCEMENT DETAILS.
3. FOR ELEVATIONS ALONG WALLS SEE SHEET S-6409, S-6409A, S-6409B.
4. CONSTRUCTION JOINTS NOT TO EXCEED 20 FEET INTERVALS.
5. REINFORCEMENT SHALL BE CONTINUOUS THROUGH CONSTRUCTION JOINTS AND END 2" CLEAR OF EXPANSION JOINTS.
6. THIS IS THE LIMITS OF THE REINFORCEMENT ADDED TO THE RAMP KK ROADBED TO SUPPORT THE TEMPORARY RAMP KK RETAINING WALL.
7. FOR APPROACH SLAB DETAILS SEE SHEET S-6408.
8. 1 INCH CLOSED CELL ELASTOMERIC BONDED TO TOP OF WALL WITH PRESSURE SENSITIVE ADHESIVE.
9. HORIZONTAL BARS TO START WHEN THE DISTANCE BETWEEN THE TOP OF ROADBED (RAMP KK) AND THE TOP OF ROADWAY (TEMPORARY RAMP KK) IS 12 INCHES.

0 4 8 12 24

1/4" = 1'-0" HORIZONTAL

REV	DATE	BY	SUB	APP	DESCRIPTION
2	28JAN02	PE	CS	CS	REMOVED BID ONLY NOTE
1	14DEC01				DESIGN DEVELOPMENT

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY F. JOHNSTON
H. CHARGE F. JOHNSTON
DATE 21 NOV 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93) / Tunnel (I-90) Project

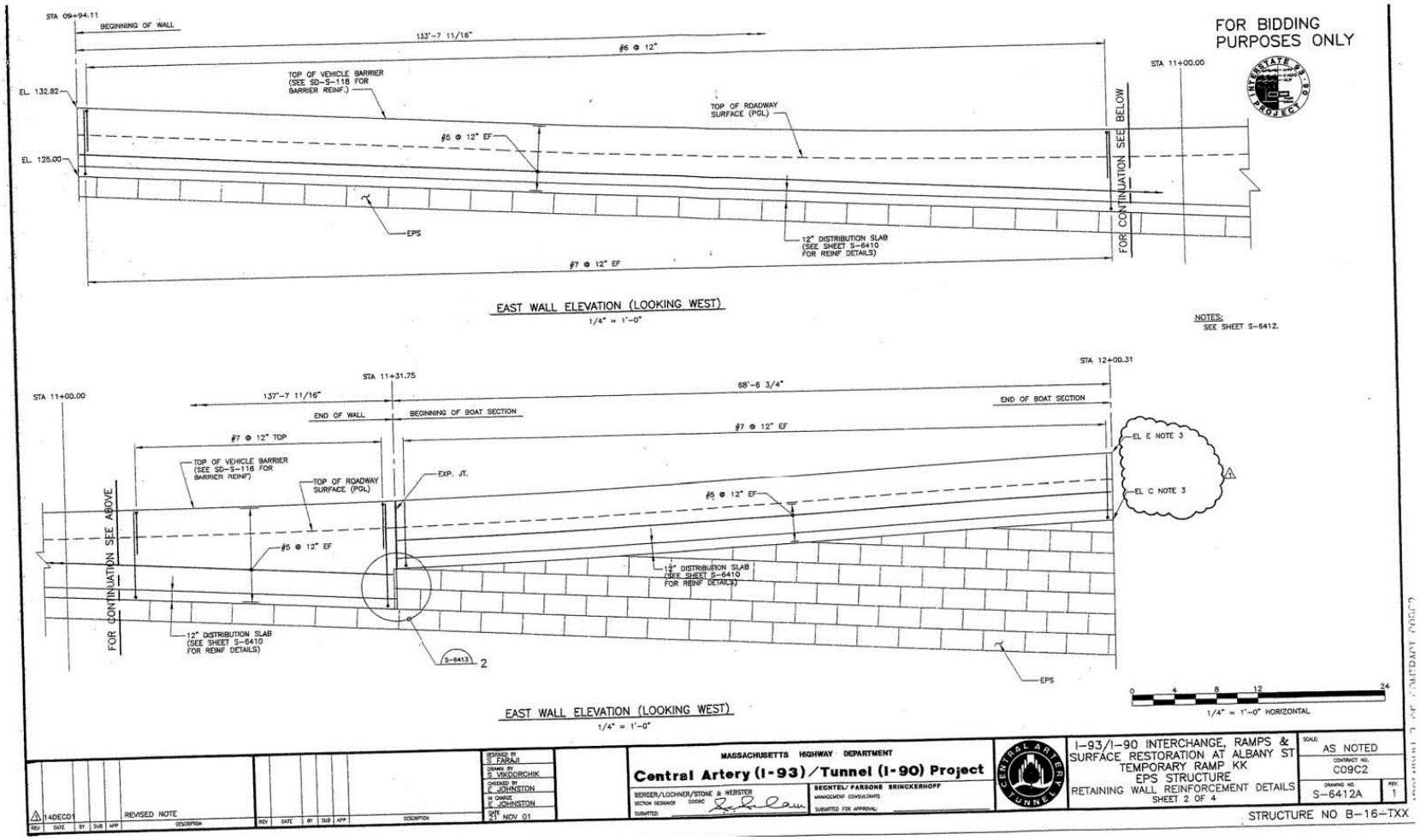
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER 0009G
SUBMITTED

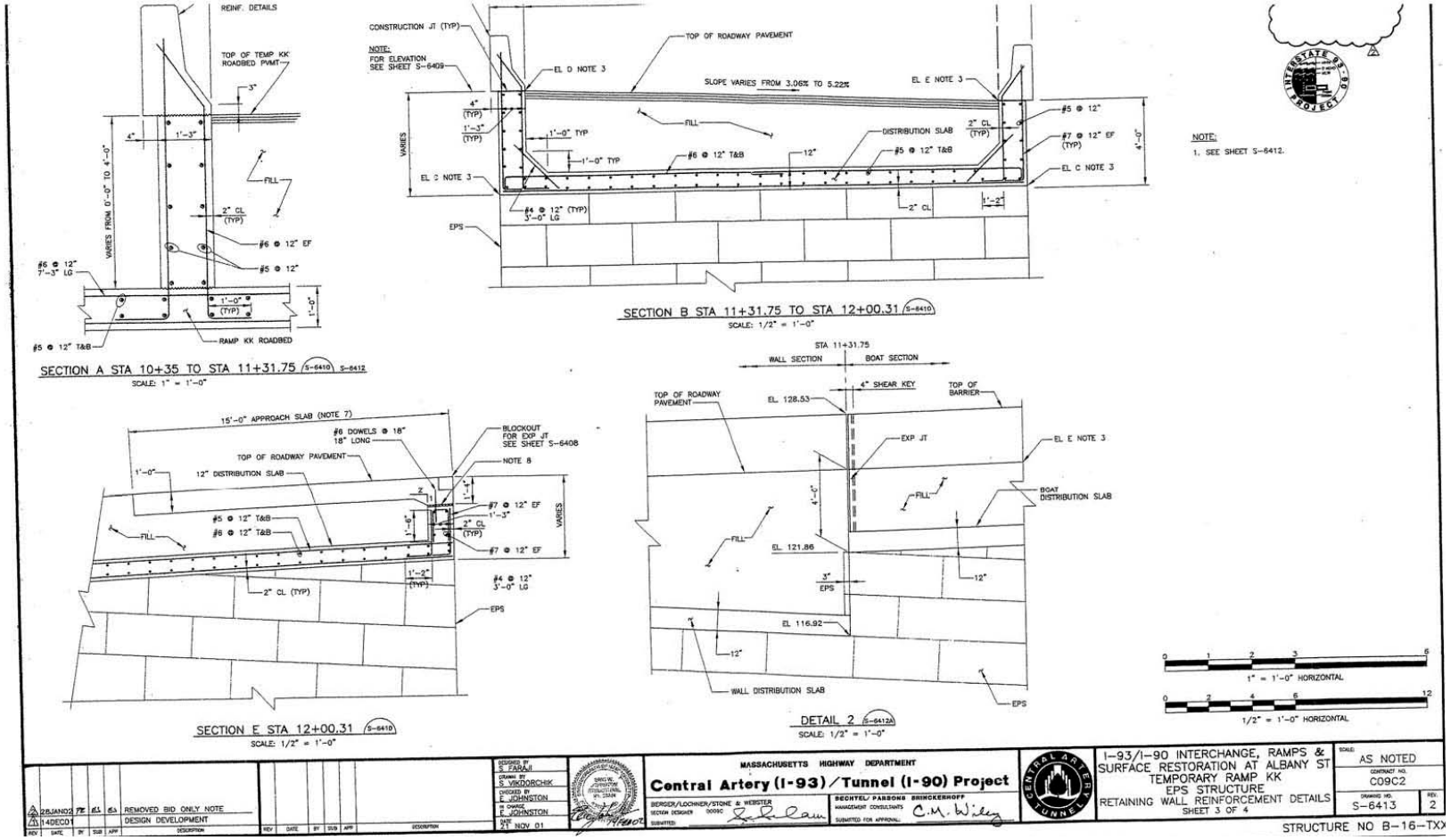
BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL

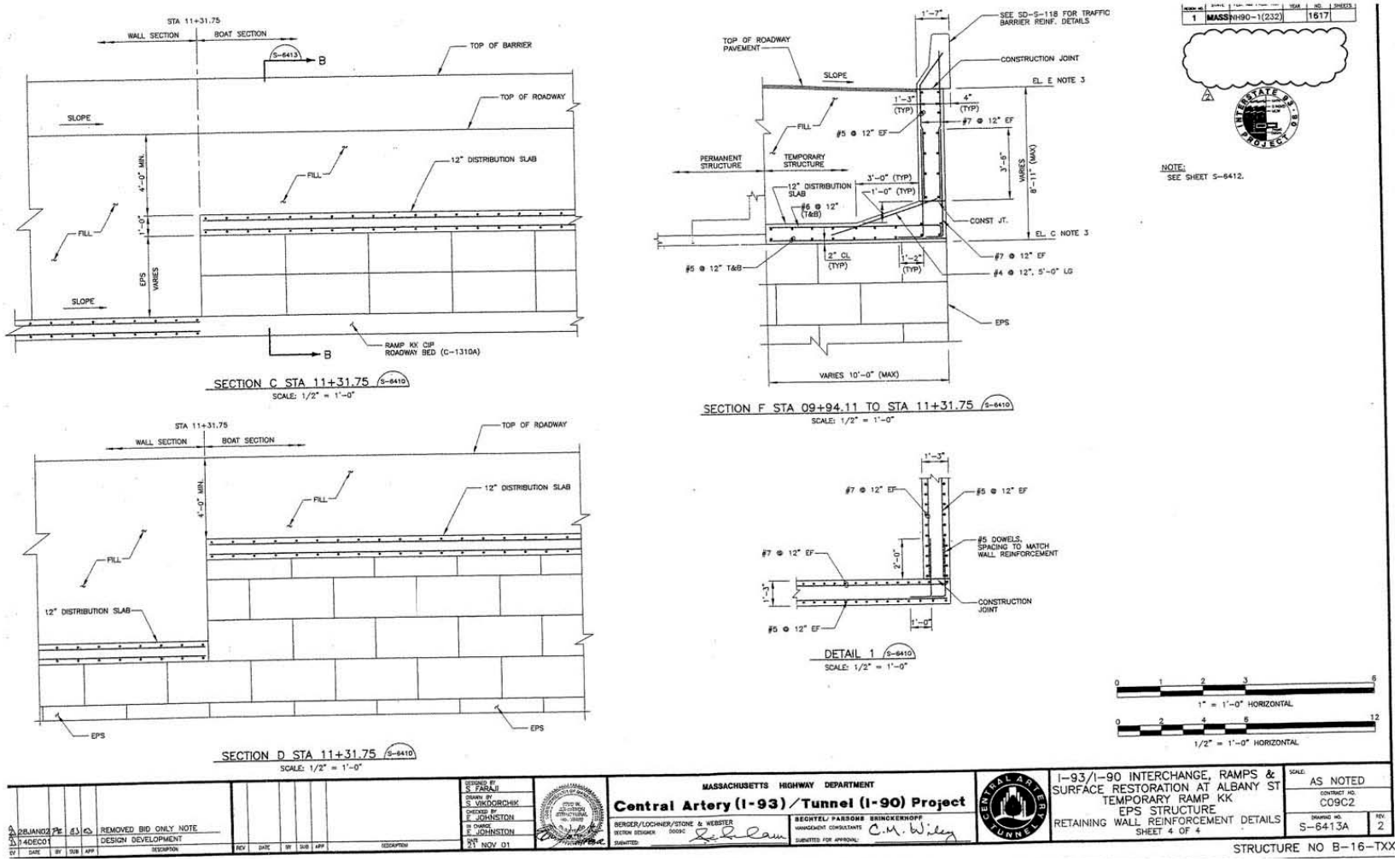
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
RETAINING WALL REINFORCEMENT DETAILS
SHEET 1 OF 4

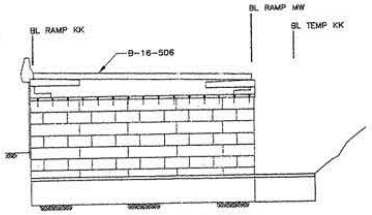
SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-6412
REV. 2

STRUCTURE NO B-16-TXX

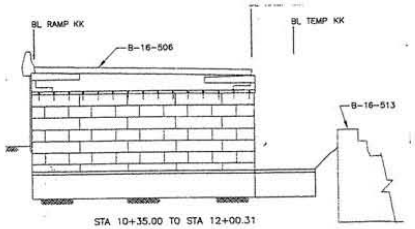

FOR BIDDING PURPOSES ONLY
STA 09+94.11
BEGINNING OF WALL
133'-7 11/16"
#6 @ 12"
STA 11+00.00
EL. 132.82
EL. 125.00
TOP OF VEHICLE BARRIER (SEE SD-S-118 FOR BARRIER REINF.)
TOP OF ROADWAY SURFACE (PGL)
#5 @ 12" EF
EPS
12" DISTRIBUTION SLAB (SEE SHEET S-6410 FOR REINF DETAILS)
#7 @ 12" EF
FOR CONTINUATION SEE BELOW
EAST WALL ELEVATION (LOOKING WEST)
1/4" = 1'-0"
NOTES:
SEE SHEET S-6412.
STA 11+00.00
STA 11+31.75
STA 12+00.31
137'-7 11/16"
88'-6 3/4"
END OF WALL
BEGINNING OF BOAT SECTION
END OF BOAT SECTION
#7 @ 12" TOP
#7 @ 12" EF
EL E NOTE 3
EL C NOTE 3
TOP OF VEHICLE BARRIER (SEE SD-S-118 FOR BARRIER REINF)
TOP OF ROADWAY SURFACE (PGL)
EXP. JT.
#5 @ 12" EF
FOR CONTINUATION SEE ABOVE
12" DISTRIBUTION SLAB (SEE SHEET S-6410 FOR REINF DETAILS)
S-6413
2
EPS
EAST WALL ELEVATION (LOOKING WEST)
1/4" = 1'-0"
0 4 8 12 24
1/4" = 1'-0" HORIZONTAL
14DEC01 REVISED NOTE
DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 NOV 01
MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
BECHTEL/PARSONS BRINCKERHOFF
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
RETAINING WALL REINFORCEMENT DETAILS
SHEET 2 OF 4
SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-6412A
REV. 1
STRUCTURE NO B-16-TXX


REINF. DETAILS
TOP OF TEMP KK ROADBED PVMT
VARIES FROM 0'-0" TO 4'-0"
FILL
#6 @ 12" EF
#5 @ 12"
#6 @ 12" 7'-3" LG
RAMP KK ROADBED
#5 @ 12" T&B
SECTION A STA 10+35 TO STA 11+31.75 S-6410 S-6412
SCALE: 1" = 1'-0"
CONSTRUCTION JT (TYP)
NOTE: FOR ELEVATION SEE SHEET S-6409
TOP OF ROADWAY PAVEMENT
EL D NOTE 3
SLOPE VARIES FROM 3.06% TO 5.22%
EL E NOTE 3
DISTRIBUTION SLAB
#6 @ 12" T&B
#5 @ 12" T&B
EL D NOTE 3
EL C NOTE 3
#4 @ 12" (TYP) 3'-0" LG
EPS
SECTION B STA 11+31.75 TO STA 12+00.31 S-6410
SCALE: 1/2" = 1'-0"
NOTE:
1. SEE SHEET S-6412.
15'-0" APPROACH SLAB (NOTE 7)
#6 DOWELS @ 18" 18" LONG
BLOCKOUT FOR EXP JT SEE SHEET S-6408
NOTE 8
TOP OF ROADWAY PAVEMENT
12" DISTRIBUTION SLAB
#7 @ 12" EF
#5 @ 12" T&B
#6 @ 12" T&B
FILL
2" CL (TYP)
#4 @ 12" 3'-0" LG
EPS
SECTION E STA 12+00.31 S-6410
SCALE: 1/2" = 1'-0"
STA 11+31.75
WALL SECTION
BOAT SECTION
4" SHEAR KEY
TOP OF ROADWAY PAVEMENT
EL. 128.53
TOP OF BARRIER
EXP JT
EL E NOTE 3
BOAT DISTRIBUTION SLAB
FILL
EL. 121.86
3" EPS
EL. 116.92
WALL DISTRIBUTION SLAB
EPS
DETAIL 2 S-6412A
SCALE: 1/2" = 1'-0"
1" = 1'-0" HORIZONTAL
1/2" = 1'-0" HORIZONTAL
REMOVED BID ONLY NOTE
DESIGN DEVELOPMENT
DESIGNED BY S. TARAJI
DRAWN BY S. VIKTOROHCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE NOV. 01
MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
BECHTEL/PARSONS BRINCKERHOFF
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
RETAINING WALL REINFORCEMENT DETAILS
SHEET 3 OF 4
SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-6413
REV. 2
STRUCTURE NO B-16-TXX


STA 11+31.75
WALL SECTION
BOAT SECTION
S-6413
B
TOP OF BARRIER
TOP OF ROADWAY
SLOPE
4'-0" MIN.
1'-0"
FILL
12" DISTRIBUTION SLAB
EPS
VARIES
RAMP KK CIP ROADWAY BED (C-1310A)
SECTION C STA 11+31.75 S-6410
SCALE: 1/2" = 1'-0"
TOP OF ROADWAY PAVEMENT
SEE SD-S-118 FOR TRAFFIC BARRIER REINF. DETAILS
CONSTRUCTION JOINT
EL. E NOTE 3
1'-3" (TYP)
4" (TYP)
#5 @ 12" EF
#7 @ 12" EF
PERMANENT STRUCTURE
TEMPORARY STRUCTURE
3'-0" (TYP)
1'-0" (TYP)
12" DISTRIBUTION SLAB
#6 @ 12" (T&B)
3'-0"
VARIES
8'-11" (MAX)
CONST JT.
EL. C NOTE 3
2" CL (TYP)
1'-2" (TYP)
#5 @ 12" T&B
#4 @ 12", 5'-0" LG
EPS
VARIES 10'-0" (MAX)
SECTION F STA 09+94.11 TO STA 11+31.75 S-6410
SCALE: 1/2" = 1'-0"
NOTE:
SEE SHEET S-6412.
12" DISTRIBUTION SLAB
SECTION D STA 11+31.75 S-6410
SCALE: 1/2" = 1'-0"
1'-3"
#7 @ 12" EF
#5 @ 12" EF
#5 DOWELS, SPACING TO MATCH WALL REINFORCEMENT
2'-0"
CONSTRUCTION JOINT
1'-0"
DETAIL 1 S-6410
SCALE: 1/2" = 1'-0"
1" = 1'-0" HORIZONTAL
1/2" = 1'-0" HORIZONTAL
REMOVED BID ONLY NOTE
DESIGN DEVELOPMENT
MASSACHUSETTS HIGHWAY DEPARTMENT
Central Artery (I-93)/Tunnel (I-90) Project
BERGER/LOCHNER/STONE & WEBSTER
BECHTEL/ PARSONS BRINCKERHOFF
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
RETAINING WALL REINFORCEMENT DETAILS
SHEET 4 OF 4
AS NOTED
C09C2
S-6413A
2
STRUCTURE NO B-16-TXX



STA 09+94.11 TO STA 10+35.00

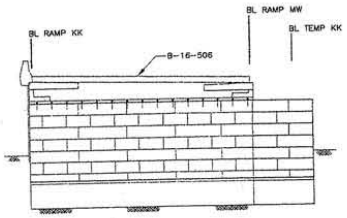


STA 10+35.00 TO STA 12+00.31

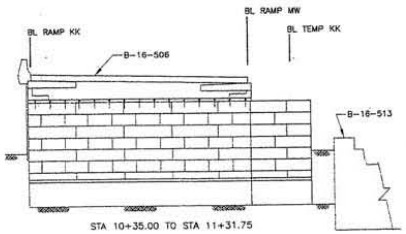
STAGE 1

NTS

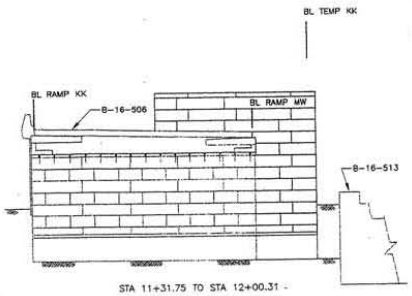
EXCAVATE TO ELEVATION
PLACE LIGHTWEIGHT STRUCTURAL FILL
PLACE BEDDING LAYER



STA 09+94.11 TO STA 10+35.00



STA 10+35.00 TO STA 11+31.75

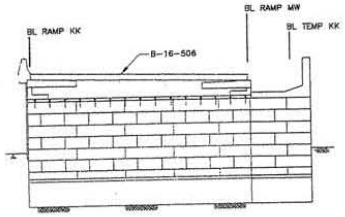


STA 11+31.75 TO STA 12+00.31

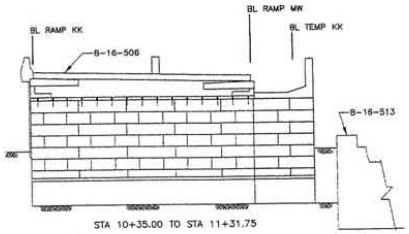
STAGE 2

NTS

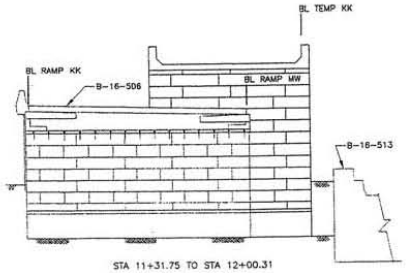
INSTALL EPS BLOCKS



STA 09+94.11 TO STA 10+35.00



STA 10+35.00 TO STA 11+31.75



STA 11+31.75 TO STA 12+00.31

STAGE 3

NTS

CONSTRUCT DISTRIBUTION SLAB

FEDERAL AID PROJ. NO.	
MASS NHS0-1(232)	1618

NOTES:

1. THE CONSTRUCTION SEQUENCE DEPICTED HERE IS A SUGGESTED SEQUENCE. THE CONTRACTOR SHALL DEVELOP AND SUBMIT A CONSTRUCTION SEQUENCE PLAN TO THE ENGINEER FOR APPROVAL.
2. MAXIMUM CONSTRUCTION LOAD APPLIED DIRECTLY TO THE LOAD DISTRIBUTION SLAB SHALL NOT EXCEED 750 PSF. USE OF WOOD CRIBBING/MATS TO SPREAD THE LOAD IS ACCEPTABLE.

REV	DATE	BY	SUB	APP	DESCRIPTION
2	28JAN02				REMOVED BID ONLY NOTE
1	14DEC01				INCORP ADD 5 NARRATIVE
1	21NOV01				REVISED ENTIRE DRAWING

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

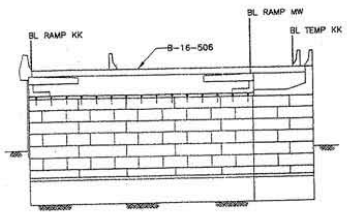
BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER C09C
SUBMITTED:

BECHTEL/ PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS
SUBMITTED FOR APPROVAL: C.M. Wiley

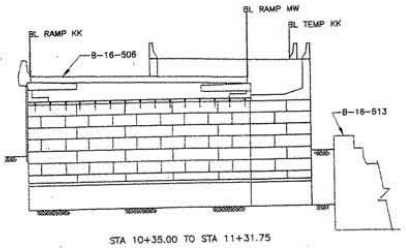
I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
CONSTRUCTION SEQUENCING
SHEET 1 OF 2

SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-6414
REV. 3

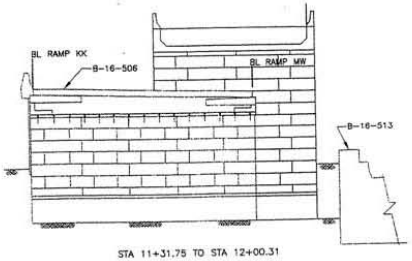
STRUCTURE NO B-16-TXX



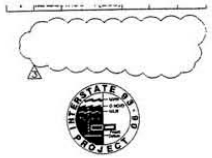
STA 09+94.11 TO STA 10+35.00



STA 10+35.00 TO STA 11+31.75



STA 11+31.75 TO STA 12+00.31



NOTES:

1. SEE S-6414 FOR NOTES.

STAGE 4

NTS

PLACE SPECIAL BORROW
CONSTRUCT VEHICLE BARRIER
PLACE ROADWAY PAVEMENT
CONSTRUCT SHOTCRETE WALL

REV	DATE	BY	SUB	APP	DESCRIPTION
3	28JAN02	TE	CS	CS	REMODED BID ONLY NOTE
2	14DEC01				INCORP ADD 5 NARRATIVE
1	21NOV01				REVISED ENTIRE DRAWING

REV	DATE	BY	SUB	APP	DESCRIPTION

DESIGNED BY S. FARAJI
DRAWN BY S. VIKDORCHIK
CHECKED BY E. JOHNSTON
IN CHARGE E. JOHNSTON
DATE 21 SEP 01

MASSACHUSETTS HIGHWAY DEPARTMENT

Central Artery (I-93)/Tunnel (I-90) Project

BERGER/LOCHNER/STONE & WEBSTER
SECTION DESIGNER DOONG
SUBMITTED:

BECHTEL/PARSONS BRINCKERHOFF
MANAGEMENT CONSULTANTS C.M. Wiley
SUBMITTED FOR APPROVAL:

CENTRAL ARTERY TUNNEL

I-93/I-90 INTERCHANGE, RAMPS & SURFACE RESTORATION AT ALBANY ST
TEMPORARY RAMP KK
EPS STRUCTURE
CONSTRUCTION SEQUENCING
SHEET 2 OF 2

SCALE: AS NOTED
CONTRACT NO. C09C2
DRAWING NO. S-6415
REV. 3

STRUCTURE NO B-16-TX>

Appendix H

Example Specifications

Project: North Abutment of the Route 1 Bridge over I-95 as part of the Woodrow Wilson Bridge Replacement Project, Alexandria, VA

Source: Virginia Department of Transportation

VIRGINIA DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION FOR
BLOCK-MOLDED EXPANDED POLYSTYRENE LIGHWEIGHT FILL (EPS-BLOCK FILL)

December 13, 2004

I. DESCRIPTION

1.01 GENERAL

This Section specifies the furnishing and installation of block-molded expanded polystyrene lightweight fill for use in bridge approaches and highway embankments referred to herein after as "EPS-Block Fill" structures on soft ground, as indicated on Contract Drawings. Provide all labor, materials and equipment necessary to complete the work of this Section.

1.02 DEFINITIONS

For the purposes of this specification, the following definitions are used for the parties indicated below, involved with EPS-Block Fill applications on this contract:

EPS-Block Fill Structure:
An assemblage of EPS blocks forming an embankment structure that constitutes part of this contract.

Molder:
Is the company actually manufacturing the EPS blocks used for the proposed EPS-Block Fill structures. In the event several EPS molders are supplying EPS blocks to this contract, one Molder shall be designated as the Primary Molder and assume the responsibility for all EPS block material supplied to this contract and its compliance with the requirements of this specification. All references herein after to the "Molder" shall be taken to mean the Primary Molder if there are multiple molders.

Supplier:
Is the company having the contractual relationship with the Contractor for the supply of the EPS blocks. This may be the Molder (or Primary Molder in case of multiple molders) directly or an intermediary company (typically a distributor of construction and / or geosynthetic products manufactured by others). Where appropriate, the Supplier may delegate certain tasks of this Specification to the Molder or Primary Molder.

EPS-block grade:
Refers to one of the AASHTO material designation types indicated in Table 1 of this specification.

1.03 REFERENCES

A. GENERAL:

Comply with the provisions of the following codes, specifications and standards except as otherwise indicated.

1. ASTM C203 - Breaking Load and Flexural Properties of Block-type Thermal Insulation.
2. ASTM C578 - Standard Specification for Rigid, Cellular Polystyrene Thermal Insulation.

1.04 SUBMITTALS

A. PRODUCT DATA

Molder's product data, installation instructions, use limitations and recommendations for each material specified herein shall all be submitted for review and approval by the Engineer. Certifications shall be provided stating that materials comply with requirements.

B. SHIPPING PLAN

As part of Phase I Manufacturing Quality Assurance (MQA) Supplier Pre-certification requirements under Section 1.05.B, the Contractor shall submit its proposed shipping procedure for the EPS block to the Engineer for review. Such procedure shall include protective measures during shipping to avoid any damage to the blocks including crushing to the edges, sides and corners of blocks. Timber cribbing with straps, tarp attached to the truck or other effective means may be proposed to secure the blocks to the transporting vehicle. Alternatively, closed container trucks may be used to transport the blocks to the Project construction site.

C. SHOP DRAWINGS

1. As described in Section 1.05.B, Contractor shall submit full-size shop drawings to scale for each EPS-Block Fill structure on this contract, indicating the proposed location and layout of all EPS blocks, inter-block connectors, and all accessory items to be used. The submitted drawings shall include, but not limited to, plans, elevations, cross-sections showing profiles and cross-slopes, location of connectors between EPS blocks, connections and accessory items as necessary.

2. Contractor shall include on the submitted shop drawings a step-by-step description of the installation and construction procedure proposed for the EPS-Block Fill structure on this contract. Installation and construction sequence, supplemented by drawings as necessary, of the EPS blocks, pavement system and utilities shall all be included. EPS block sizes and laying pattern as well methods of temporarily ballasting and stabilizing EPS blocks to prevent movement during construction, prior to placement of the Load Distribution Slab, as applicable, shall all be identified.

3. Shop drawings, working drawings, installation and construction procedures and supporting calculations shall all be stamped by a professional engineer registered in the Commonwealth of Virginia.

4. All details different than those depicted on the contract drawings, required in support of the construction procedures shall be engineered by the Contractor. All Contractor engineered items shall be stamped by a professional engineer registered in the Commonwealth of Virginia. All such details shall be reviewed and approved by the Engineer prior to construction.

5. The layout of all EPS blocks and attachments shall be in conformance with the design details shown on the contract documents.

D. MANUFACTURING QUALITY CONTROL (MQC) SUBMITTALS

Submit the following manufacturing quality control submittals:

1. Test Compliance:

a) The Contractor shall supply a summary of test compliance with specified performance characteristics and physical properties for the EPS blocks and galvanized steel connector plates for review and approval by the Engineer.

2. Certificates:

a) The Contractor shall supply for review and approval by the Engineer, hard copy product certificates showing compliance to the Material Properties stated in this specification. Product certificates shall be signed by the Supplier to certify material compliance with the specified performance characteristics, criteria and physical requirements outlined in this specification.

b) The Contractor shall indicate in writing to the Engineer whether or not the Supplier has a Third Party Certification. If Third Party Certification is offered, this notification shall be accompanied by documentation that indicates the business entity providing the Third-Party Certification and describes in detail the steps to be taken by this agency to verify the Molder's compliance with the specific requirements of this specification. Acceptance of the Molder's Third Party Certification by the Engineer will not waive the need for pre-construction product submittal and testing as specified in Sections 1.05.B.2 and 1.05.B3.

c) The Contractor shall submit, for the Department's acceptance, Supplier's standard warranty document or certificate executed by an authorized company official. Supplier's warranty is in addition to, and not a limitation of, other rights the Department may have under Contract Documents.

1.05 MANUFACTURING QUALITY ASSURANCE (MQA)

A. GENERAL REQUIREMENTS

1. Manufacturing Quality Assurance (MQA) of the EPS-block product will be conducted to verify the Molder's Quality Control (MQC) procedures. The Engineer will have primary responsibility for all MQA unless the Department notifies the Contractor otherwise. The Engineer will communicate directly only with the Contractor in matters and questions of MQA unless all parties agree otherwise.

2. MQA of the EPS-block will consist of two phases:
   - Phase I MQA – Supplier Pre-certification, which consists of pre-certification of the Supplier and shall be conducted prior to shipment of any EPS blocks to the Project construction site. Phase I MQA is covered in Section 1.05.B of this specification.
   - Phase II MQA – Block Verification, which is conducted as the EPS blocks are delivered to the Project construction site. Phase II MQA is covered in Section 1.05.D of this specification.

3. Regulatory Requirements: Installation must comply with the requirements of all applicable local, state and national jurisdictions.

4. Pre-installation Meeting: A pre-installation meeting shall be held to verify Project requirements, substrate conditions, and details relative to the manufacturing, shipping and placement of the EPS-block. The meeting shall involve the Contractor, Engineer, Section Design Consultant (SDC) and the EPS Supplier.

5. Construction Quality Control: Contractor shall be directly responsible for all Construction Quality Control (CQC). Items covered by CQC include all earthwork and related activities other than manufacturing and shipment of the EPS blocks.

6. Previously used EPS blocks are not allowed in part or in full on any and all EPS-Block Fill structures on this contract.

B. PHASE I MQA – SUPPLIER PRECERTIFICATION

No EPS blocks shall be shipped to the Project construction site until such time as all parts of Phase I MQA – Supplier Precertification, as specified in this section, have been completed in the order listed below:

1. The Contractor shall supply a scale with sufficient capacity and precision for weighing of the EPS blocks. This scale shall be delivered to the Project construction site or to an alternate location specified by the Engineer. This scale shall be recently calibrated and certification of such calibration made available to the Engineer.

2. The Contractor shall deliver a minimum of three full-size EPS blocks for each AASHTO EPS-block grade to be used on this contract to a location specified by the Engineer. These blocks shall in all respects be the same as the blocks to be supplied to this contract, including required seasoning as described in Section 1.06.C.

3. The Engineer will weigh and measure each of the three blocks of each grade of EPS supplied. The Engineer will sample and test at least one of the three blocks of each grade, selected randomly, to evaluate the ability of the supplier to deliver EPS blocks of quality as specified herein. The sampling and testing protocol will be the same as for Phase II MQA as discussed in Section 1.05.D. Any EPS blocks not used for testing may be utilized for construction provided they satisfy all the requirements outlined in this specification.

4. The Contractor has submitted shop drawings as required by Section 1.04.C of this specification, and such drawings have been reviewed and approved for construction in conformance with Project standards.

5. Prior to delivery of any EPS blocks to the Project construction site, a meeting shall be held, as a minimum, between the Engineer and Contractor. The Supplier and/or Molder of the EPS-blocks may also attend at the Contractor's discretion to facilitate answering any questions first hand. The purpose of this meeting shall be to review the Phase I MQA results and discuss Phase II MQA as well as other aspects of construction to ensure that all parties are familiar with the requirements of this specification. At the satisfactory conclusion of this meeting, the Contractor shall be allowed to begin on-site receipt, storage (if desired) of the EPS-blocks in accordance with Section 1.05.C of this specification.

C. PRODUCT DELIVERY, STORAGE AND HANDLING

Prior to delivery of the EPS block to the Project construction site; Contractor shall review, plan and implement, with the assistance of the Supplier, a material handling procedure that shall include the following as a minimum:

1. Care should be exercised during shipping to prevent any damage to the EPS blocks. Particular attention should be paid to avoid crushing to the edges, sides and corners of the blocks during shipping from the Molder to the Project construction site. All shipment shall conform to shipping plan as indicated in Article 1.04.B.

2. The Contractor shall prevent any damage to the EPS blocks during delivery, handling, storage, and construction. EPS blocks with cracks of any size are considered not acceptable and shall be rejected and replaced by the Contractor with undamaged equal EPS blocks at no additional cost to the Department. Holes shall not be created in the blocks at any stage of manufacturing or construction to facilitate shipping or handling of the blocks during placement.

3. Each EPS block shall be labeled to indicate the name of the Molder (if there is more than one for a given EPS-Block Fill structure), the date the block was molded, the mass of the entire block in pounds as measured after a satisfactory period of seasoning as specified in Section 1.06.C, the dimensions of the block in inches and the actual dry unit weight in pounds per cubic foot.

   Additional identification markings using alphanumeric characters and/or symbols, applied as necessary by the Supplier, to indicate the location of placement of each block relative to the shop drawing shall also be provided. Stripes of different paint, color can be utilized to identify blocks of higher grade of EPS if multiple grades of EPS are to be supplied for a given EPS-Block fill structure. If two grades of EPS blocks are to be supplied, the use of no marking shall be considered an acceptable marking for one of the material grades as long as it is used for the lower (lowest) grade EPS blocks supplied for that structure. Any paint, etc. used to mark EPS blocks shall be chemically compatible with EPS and not cause any dissolution of the EPS during application of the paint, etc.

4. If the EPS blocks are to be stockpiled at the construction site until placement, a secure storage area shall be identified and designated by the Contractor for this purpose. The storage area shall be away from any heat source or construction activity that produces heat or flame or would expose the blocks to hydrocarbon fuels (diesel, kerosene, gasoline). In addition, personal tobacco smoking shall not be allowed in the storage area. EPS blocks in temporary on-site storage shall be secured with sandbags and similar "soft" weights to prevent their being dislodged by wind. The blocks shall not be covered in any manner that might allow the build up of heat beneath the cover. The blocks shall not be trafficked by any vehicle or equipment. In addition, foot traffic by persons shall be kept to a minimum.

5. EPS is not an inherently dangerous or toxic material so there are no particular safety issues to be observed other than normal construction safety and protection against heat and flame and hydrocarbon fuels (diesel, kerosene, gasoline) which can cause EPS to melt. However, extra caution is required during wet or cold weather since surfaces of the EPS blocks tend to be more slippery wet than dry. In addition, when air temperatures approach or go below freezing, a thin layer of hoarfrost (ice) can readily develop on the exposed surfaces of EPS blocks if the dew-point is sufficiently high. Thus the surfaces of the EPS blocks can pose particular slip hazards in this condition. The maximum amount of time during which EPS blocks can be stored at the Project construction site shall be limited to a maximum of 30 calendar days.

D. PHASE II MQA – BLOCK VERIFICATION

The Engineer shall assume the primary responsibility for conducting this phase of the work. Contractor shall cooperate with and assist the Engineer in implementing Phase II MQA – Block Verification.

No EPS blocks shall be placed on any structure of this contract until such time as all activities of Phase II MQA – Block Verification, as specified in this section, have been completed successfully in the order listed below:

1. Each block of every grade of EPS delivered on any truck to the construction site shall be inspected on-site visually to check for damage as well as for verification of the labeled information included on each block. Any blocks with damage or not meeting the requirements of the specifications will be rejected on the spot.

2. At least one block of every grade EPS delivered on every truck to the construction site shall be checked to verify its compliance with the requirements of the specifications for the minimum block dry unit weight specified in 2.01.A, as well as the physical

tolerances, specified in sections 2.01.D through 2.01.F, inclusive. Weighing of blocks shall conducted onsite using a scale with sufficient capacity and precision for weighing of EPS blocks to be supplied by the Contractor in conformance with Section 1.06.B.1.

3. Should verification of the parameters of item 2 above indicates lack of compliance, at least three additional blocks of every grade EPS delivered from the same truck-load will be individually checked. The entire shipment of the grade of EPS in question shall be rejected should any one of the three additional blocks fails to meet the requirements of the specifications for minimum block dry unit weight and physical tolerances outlined above.

4. At least one block of every grade of EPS delivered on the first truck to the construction site for use on any one EPS-Block Fill structure, shall be selected for sampling and testing. Sampling will be at locations A, B and C shown in Figure 1. The samples should be approximately square in cross-section and of sufficient width to enable preparing the test specimen required by this specification. The Contractor shall cooperate with and assist the Engineer with obtaining the necessary samples. Testing will be performed by or under the direction of the Engineer.

   Laboratory tests will check for compliance with the Material Properties shown on Table 2 of this specification. Additional blocks of each grade of EPS shall be selected by the Engineer for sampling during the course of construction at a rate of sampling not to exceed one block for every 500 cubic yards of EPS delivered.

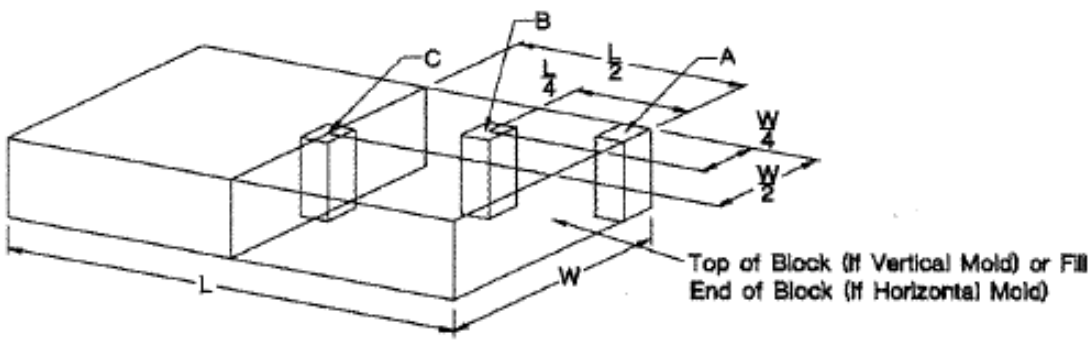


Figure 1. Locations of required EPS block sampling

5. Portions of sampled blocks that are used for testing are not acceptable for construction. Portions of sampled blocks that are otherwise acceptable can be used as desired by the Contractor provided that they comply with all other requirements of this specification.

6. The Contractor shall allow for early delivery of the EPS blocks to the construction site to allow for conducting laboratory testing of the blocks. For those truckloads where EPS blocks will be selected for sampling and testing per section 1.05.D.4, a minimum of 3 business days are required prior to their scheduled installation, to allow for samples to be taken and laboratory testing conducted. Any shipment of EPS blocks, for which the representative samples fail to meet the parameters outlined in this specification, are considered defective and shall be replaced by the Contractor with non-defective EPS block at no additional cost or time to the Department.

1.06 MANUFACTURING QUALITY CONTROL (MQC)

A. Manufacturing Quality Control (MQC) of EPS blocks product is the primary responsibility of the Molder. The purpose of this section is to define the parameters for use by a Molder in developing an MQC plan. These parameters will also be those measured as part of the Manufacturing Quality Assurance (MQA) to be conducted by the Engineer. MQA requirements are detailed in Section 1.05 of this specification.

B. All EPS blocks shall consist entirely of expanded polystyrene. All EPS-block shall consist of virgin raw material (expandable polystyrene bead or resin). Previously used EPS blocks are not allowed in part or in full on any and all EPS-Block Fill structures on this contract.

C. All EPS blocks shall be adequately seasoned prior to shipment to the Project construction site. For the purposes of this specification, seasoning is defined as storage in an area suitable for the intended purpose as subsequently defined herein for a minimum of 72 hours after an EPS block is released from the mold. Seasoning shall be done within a building or other structure that protects the EPS blocks from moisture as well as UV radiation. The area in which EPS blocks are stored for seasoning shall also be such that adequate space is allowed between blocks and positive air circulation and venting of the structure provided so as to foster the out-gassing of blowing agent and trapped condensate from within the blocks. The Engineer shall be allowed to inspect the structure to be used for seasoning upon request and during normal business days and hours. The supplier may request a shortened seasoning period if the EPS blocks are seasoned within an appropriate heated storage space and the molder demonstrates to the satisfaction of the Engineer that the alternative seasoning treatment produces blocks that equal or exceed the quality of blocks subjected to the normal 72-hour seasoning period.

D. All EPS blocks shall satisfy the product flammability requirements specified in ASTM C 578.

E. All EPS blocks shall satisfy the product the Material Property requirements outlined in Section 2.01 of this specification.

F. All galvanized steel inter-block connectors shall satisfy the requirements outlined in Section 2.02 of this specification.

1.07 PROJECT CONDITIONS

Provide inserts and anchorages which must be built into other work at the time they are needed.

II. MATERIALS

2.01 EXPANDED POLYSTYRENE BLOCK

A. Only AASHTO material type designations shall be used on all correspondence and communication related to this contract. Table 1 indicates the AASHTO material type designations used for the different unit weights of EPS blocks that are covered by this specification. For a given material type, the dry unit weight of each EPS block (as measured for the overall block as a whole) after the period of seasoning as defined in Section 1.06.C of this specification shall equal or exceed that shown in Table 1. The dry unit weight shall be determined by measuring the mass of the entire block by weighing on a scale and dividing the mass by the volume of the block.

AASHTO Material Designation	Minimum Allowable Dry Unit Weight of entire EPS block (Lbs/ft^3)
EPS70	1.50

Table 1. AASHTO Material Type Designations for EPS-Block

B. Table 2 gives the minimum allowable values for the Material Properties corresponding to each AASHTO material type shown in Table 1. These material properties shall be obtained by testing specimens prepared from samples taken from actual blocks produced for the Project covered by this specification for either MQC by the Molder or MQA by the Engineer as described in Sections 1.06.D and 1.05.D, respectively, of this specification. Testing for material properties shall comply with the following:

- All test specimens shall be seasoned as specified in ASTM C 578.
- Dry density, compressive strength and flexural strength shall all be measured as specified in ASTM C 578.
- The specimens used for compressive testing shall be cubic in shape with a two (2) inch side length.
- A strain rate of 10% per minute shall be used for the compressive strength tests.
- Both the elastic-limit stress and initial tangent Young's modulus shall be determined in the same test used to measure compressive strength.
  The elastic-limit stress is defined as the measured compressive normal stress at a compressive normal strain of 1%. The initial tangent Young's modulus is defined as the average slope of the compressive stress versus compressive strain curve between 0% and 1% strain.

AASHTO Material Designation	Dry Density (Lbs/ft^3)	Compressive Strength (Psi)	Flexural Strength (Psi)	Elastic Limit Stress (Psi)	Initial Tangent Young's Modulus (Psi)
EPS70	1.35	15	40	10.2	1015

Table 2. Minimum Allowable Values of MQC/MQA Material Properties for Individual Test Specimens

C. Each EPS block shall meet dimensional tolerances as determined in three distinct areas:
- Variations in linear dimensions as defined in Section 2.01.D.
- Deviation from perpendicularity of block faces as defined in Section 2.01.E.
- Overall warp of block faces as defined in Section 2.01.F.

D. The thickness, width and length dimensions of an EPS block are defined herein as the minimum, intermediate and maximum overall dimensions of the block, respectively, as measured along a block face. These dimensions of each block shall not deviate from the theoretical dimensions by more than ± 0.5%.

E. The corner or edge formed by any two faces of an EPS block shall be perpendicular, i.e. form an angle of 90°. The deviation of any face of the block from a theoretical perpendicular plane shall not exceed one eighth (1/8) inch over a distance of twenty (20) inches.

F. Any one face of a block shall not deviate from planarity by more than one quarter (1/4) inch when measured using a straightedge with a length of ten (10) feet.

G. The EPS shall contain a flame retardant additive and shall have UL Certification of Classification as to External Fire Exposure and Surface Burning Characteristics so as to comply with the Oxygen Index requirements of ASTM C 578.

2.02 INTER-BLOCK CONNECTORS

A. Inter-block Connectors shall be used to restrain EPS blocks from moving laterally in layer over layer applications. Contractor shall supply samples and test data for the proposed inter-block connectors to comply with Section 2.02.B of this specification for review and approval by the Engineer prior to construction.

B. Inter-block Connectors shall be made of 20 gauge (minimum) galvanized steel with two-sided multi-barbed design, or approved equal, capable of piercing the EPS up to three quarters (3/4) inch. Each plate shall be capable of a lateral holding strength of at least 60 lbs. Refer to Contract Drawings for suggested layout and number of Inter-block Connectors required.

C. Should the size of supplied EPS block be different than that shown on the Contract Drawings, Contractor shall propose a suggested layout for the inter-block connectors for review and approval by the Engineer prior to the start of construction. The revised layout shall be incorporated on the shop drawings in conformance with Section 1.04.C of this specification.

III. CONSTRUCTION METHODS

3.01 CONSTRUCTION QUALITY CONTROL (CQC)

A. The Contractor shall be directly responsible for all Construction Quality Control (CQC). Items covered by CQC include all earthwork and related activities other than manufacturing and shipment of the EPS-blocks. Items of particular relevance to the placement of EPS-block fill structure are given in sections 3.02 through 3.05, inclusive, of this specification.

B. The Engineer will be responsible for providing Construction Quality Assurance (CQA) of the contractor's construction activities

3.02 SITE PREPARATION

A. The natural soil subgrade shall be cleared of vegetation, any large or sharp-edged soil particles, any kind of debris and be reasonably planar prior to placing sand bedding layer. Reasonably planar is defined as ± one (1) inch over ten (10) feet.

B. The sand bedding layer shall consist of sand conforming to the Specifications Section 202-Fine Aggregate, Grading B. The required smoothness of the sand bedding layer prior to placement of the first layer of EPS blocks shall be no more than ± three-eighth (3/8) of one inch over any ten (10) ft. distance.

C. There shall be no debris of any kind on the sand bedding surface at the time EPS blocks are placed.

D. Unless directed otherwise by the Engineer, there shall be no standing water or accumulated snow or ice on the sand bedding layer within the area where EPS blocks are placed at the time of block placement.

E. EPS blocks shall not be placed on a frozen subgrade nor de-icing salts be used except as directed by the Engineer

3.03 EPS BLOCK PLACEMENT

A. EPS blocks shall be placed at the locations shown on approved shop drawings submitted by the Contractor. Particular care is required if EPS blocks of different density are to be used on EPS-Block Fill structures of this contract.

B. There shall be no debris of any kind between adjacent surfaces of EPS blocks at the time adjacent EPS blocks are placed.

C. There shall be no standing water or accumulated snow or ice on the previously placed EPS block layer within the area where subsequent EPS blocks are to be placed at the time of block placement.

D. EPS blocks shall be placed so that all vertical and horizontal joints between blocks are tight.

E. While placing successive layers of EPS blocks, Contractor should exercise care to guarantee that all placed blocks are supported over their entire bearing area. In the event the top constructed surface of an assembly of blocks becomes uneven or where rocking of the blocks is observed, Contractor shall notify the Engineer and propose a remedial procedure for corrective action. Such procedure shall be submitted for review and approval by the Engineer prior to resuming construction.

F. Blocks shall be placed such that the resulting exterior surfaces on the sides of the EPS-Block Fill structures are vertical and planar within a tolerance of ± one-eighth (1/8) inch between blocks. Block faces not satisfying this criterion shall be field trimmed using a hot wire cutting apparatus to achieve the desired evenness within the above tolerance.

G. The inter-block connectors shall be placed at the locations shown on the shop drawings and shall be set into the EPS block such that the inter-block connectors do not cause a gap to exist between adjacent layers of EPS blocks.

H. The final surface of the EPS blocks shall be covered as shown on the Contract Drawings. Care shall be exercised during placement of the cover material so as not to cause any damage to the EPS blocks.

3.04 FIELD QUALITY REQUIREMENTS

A. Field cutting of EPS blocks shall be permitted, but limited to the use of the following devices:

1. Hot wire cutters
2. Wire saws
3. Chain saws

B. With the exception of sand bags or similar "soft" weights used to temporarily restrain EPS blocks against wind, no construction material other than that shown on the contract drawings shall be placed or stockpiled on the EPS blocks.

C. At no time shall heat, open flame or motor vehicle fuels be used in proximity to the EPS blocks so as to cause combustion or melting of the EPS.

D. The final surface of the EPS blocks, beneath paved areas, shall be covered with a cast-in-place concrete Load Distribution Slab as shown on the Contract Drawings. Care shall be exercised during placement of the cover material so as not to cause any damage to the EPS blocks.

E. The surfaces of the EPS blocks shall not be directly traversed by any vehicle or construction equipment during or after placement of the blocks.

3.05 PAVEMENT CONSTRUCTION

A. The pavement system is defined for the purposes of this specification as all material placed above the EPS blocks within the limits of the roadway, including any shoulders.

B. The pavement system shall be constructed above the EPS blocks as shown on the contract drawings. Specifications covering construction of the pavement system are given elsewhere in the contract documents.

C. Concrete and reinforcement steel for the Load Distribution Slab shall be installed over the EPS blocks or separation layer using appropriate labor and equipment that will not damage the EPS blocks.

3.06 LOAD DISTRIBUTION SLAB (LDS) CONSTRUCTION

A. The LDS system is defined for the purposes of this specification as the reinforced concrete slab placed above the EPS blocks within the limits indicated in the plans.

B. The LDS system shall be constructed above the EPS blocks as shown on the contract drawings. Specifications covering construction of the LDS system are given elsewhere in the contract documents.

C. Concrete and reinforcement steel for the Load Distribution Slab shall be installed over the EPS blocks or separation layer using appropriate labor and equipment that will not damage the EPS blocks.

3.07 PRECAST WALL SYSTEM

A. The precast wall system is defined for the purposes of this specification is the precast wall facing and footing adjacent to portions of the EPS.

B. The precast wall system shall be constructed as shown on the contract drawings. Specifications covering construction of the precast wall system are given elsewhere in the contract documents.

3.08 PROTECTION

Protect partial installation and installed product and finish surfaces from damage during construction in accordance with Section 1.05.C.

3.09 EPS DISPOSAL

Contractor shall assume total responsibility for disposal of EPS block material or portions of unused blocks resulting from testing or construction by returning it to the Supplier / Molder for recycling. Such process shall be conducted on a regular basis or as instructed by the Engineer.

3.10 GEOMEMBRANE

Where shown in the drawings, shall be as specified in the Geomembrane Special Provision.

3.11 SOIL COVER

Soil cover over geomembrane on embankment slopes shall meet requirements of Section 303 of the Specifications.

IV. METHOD OF MEASUREMENT AND BASIS OF PAYMENT

4.01 EXPANDED POLYSTYRENE FILL (EPS)

EPS will be measured and paid for per cubic yard of blocks satisfactorily installed and accepted by the Engineer. The EPS volume will be measured from the top of the sand layer to the bottom of the load distribution slab and will include the volume occupied by drainage structures, utilities, etc.

The price bid shall also include furnishing, installation, any required cutting, providing modifications to the block dimensions/size to comply with utility, conduit and drainage pipe placement, connector plates, shop drawing and all quality control submittals, all required supplier pre-certifications, product storage, handling and protection provisions, site preparation for sand bedding and any required off-site EPS disposal.

Payment will be made under:

Pay Item	**Pay Unit**
Expanded Polystyrene Fill (EPS)	Cubic Yard

Project: Slide repair on AL 44 near Guin, AL

Source: Alabama Department of Transportation

ALABAMA DEPARTMENT OF TRANSPORTATION

DATE: August 24, 2004

Special Provision No. 02-0985

SUBJECT: Geofoam Blocks, Project Number ST-047-044-001, Marion County.

Alabama Standard Specifications, 2002 Edition, are hereby amended by the addition of a NEW SECTION 247 as follows:

SECTION 247
GEOFOAM BLOCKS

247.01 Description.

This section shall cover work of furnishing and installing geofoam blocks as lightweight fill for the construction of embankments on soft ground and the correction of landslides.

247.02 Materials.

(a) REQUIRED PHYSICAL PROPERTIES.

Geofoam blocks shall consist entirely of expanded polystyrene {EPS}. The nominal size of each block shall be 8 feet long by 4 feet wide by 3 feet deep. The minimum required physical properties of the geofoam material (measured in laboratory testing) are given in the following table.

MINIMUM REQUIRED PHYSICAL PROPERTIES OF GEOFOAM MATERIAL					
Material Designation	Dry Density lb/ft^3 $\{kg/m^3\}$	Compressive Strength lb/in^2 {kPa}	Flexural Strength lb/in^2 {kPa}	Elastic-Limit Stress lb/in^2 {kPa}	Initial Tangent Young's Modulus lb/in^2 $\{MN/m^2\}$
EPS40	0.90 {15}	10 {69}	25 {173}	5.80 {40}	580 {4}
EPS50	1.15 {18}	13 {90}	30 {208}	7.25 {50}	725 {5}
EPS70	1.35 {22}	15 {104}	40 {276}	10.15 {70}	1015 {7}
EPS100	1.80 {29}	25 {173}	50 {345}	14.50 {100}	1450 {10}

(b) TESTING OF GEOFOAM MATERIAL.

1. TESTING LABORATORY AND CERTIFIED TEST REPORTS.

Testing shall be done by an independent testing laboratory acceptable to the Engineer. The Contractor shall submit certified test results from the testing laboratory that the geofoam material meets the required physical properties.

2. SAMPLING AND SEASONING OF TEST SPECIMENS.

The actual physical properties of the geofoam blocks shall be obtained by testing specimens prepared from samples taken from actual blocks produced for this project. All test specimens shall be seasoned in accordance with the requirements given in ASTM C 578 "Standard Specification for Rigid Cellular Polystyrene Thermal Insulation".

3. MEASUREMENT OF DRY DENSITY, COMPRESSIVE STRENGTH AND FLEXURAL STRENGTH.

Dry density, compressive strength, and flexural strength shall be measured in accordance with the requirements given in ASTM C 578. The specimens used for compressive testing shall be cubic in shape with a 2 inch {50 mm} face width. A strain rate of 10 % per minute shall be used for the compressive strength tests. Both the elastic-limit stress and initial tangent Young's modulus shall be determined in the same test used to measure compressive strength. The elastic-limit stress shall be defined as the measured compressive normal stress at the compressive normal strain of 1%. The initial tangent Young's modulus shall be defined as the average slope of the compressive stress versus compressive strain curve between 0 % and 1 % strain.

(c) SEASONING.

Geofoam blocks shall be adequately seasoned prior to shipment to the project site. Seasoning shall be storage in a suitable area for a minimum of 72 hours after the blocks are released from the mold. Seasoning shall be done within a building or other structure that protects the blocks from moisture and UV radiation. The blocks shall be stored for seasoning with adequate space between blocks for ventilation. Positive air circulation and venting of the structure shall be provided to foster the out gassing of blowing agent and trapped condensate from within the blocks.

(d) FLAMABILITY.

Geofoam blocks shall have limited flammability in accordance with the product flammability limitations given in ASTM C 578.

(e) DRY DENSITY.

The minimum dry density of each geofoam block (as measured for the overall block as a whole) after seasoning is given in the following table. The actual dry density shall be determined by measuring the weight {mass} of the entire block by weighing the block on a scale and dividing the mass by the volume of the block.

MINIMUM ALLOWABLE DRY DENSITY OF EACH GEOFOAM BLOCK	
Material Designation	Minimum Allowable Dry Density Of Entire Geofoam Block, lb/ft^3 $\{kg/m^3\}$
EPS40	1.0 {16}
EPS50	1.25 {20}
EPS70	1.5 {24}
EPS100	2.0 {32}

(f) DIMENSIONAL TOLERANCES.

The thickness, width and length of a geofoam block shall be the minimum, intermediate and maximum overall dimensions of the block, respectively, as measured along a block face. The dimensions of each block shall not deviate from the nominal dimensions by more than plus or minus 0.5 %.

The corner or edge formed by any two faces of a block shall be perpendicular (i.e. form an angle of 90 °). The deviation of any face of the block from a theoretical perpendicular plane shall not exceed 0.125 inches over a distance of 20 inches {3mm in 500mm}.

Any one face of a block shall not deviate from a flat plane by more than 0.25 inches when measured using a straightedge with a length of 10 feet {5 mm in 3 m}.

(g) FASTENERS.

Galvanized, barbed metal fasteners shall be supplied by the Geofoam Block manufacturer. The details of the fasteners shall be submitted to the Engineer for review and approval prior to the use of the fasteners.

(h) BLOCK PRODUCTION.

The geofoam blocks shall be labeled with the manufacturer's name, product type, material lot number, density and date of manufacture. The blocks shall be produced by a manufacturer with an in place, independent laboratory certified quality control program which is monitored by the independent testing organization. The contractor shall furnish the Engineer with two copies of the independent testing agency certified test reports showing that the geofoam blocks meet the required physical properties and production tolerances.

247.03 Construction Requirements.

(a) PREPARATION OF SITE FOR INSTALLATION.

All areas where the geofoam blocks are required shall be properly prepared as shown on the plans, or directed by the Engineer.

(b) HANDLING AND STORAGE OF BLOCKS.

At all stages of manufacturing, shipment, and construction, the blocks shall be handled in a manner that will not damage the blocks.

Secure storage areas shall be provided for stockpiling the blocks if blocks are stored at the project site. The storage area shall be away from sources of heat and construction activity that produces heat or flame. Blocks in temporary storage shall be secured with sandbags and similar "soft" weights to prevent their being dislodged by wind.

(c) VERIFICATION OF THE QUALITY OF THE GEOFOAM BLOCKS.

The Engineer will perform a visual inspection of each block delivered to the project site to check for damage. The Engineer will also check to make sure that each block is labeled with the required information. Blocks with damage or not meeting labeling and materials requirements will be rejected. The size and shape of the block will also be checked.

(d) VERIFICATION OF THE REQUIRED MINIMUM DRY DENSITY.

The Engineer will inspect the geofoam blocks when they are delivered to make sure that the blocks have the required minimum dry density (whole block). The Engineer will perform on-site density tests by weighing one block randomly chosen from each truck load or each approximately 100 cubic yards of geofoam block delivered to the project site, with additional blocks checked if initial measurements indicate lack of compliance. The contractor shall supply a scale on site with sufficient capacity and precision for weighing the geofoam blocks. This scale shall be calibrated within 4 months and a copy of the certification of the calibration shall be furnished to the Engineer.

(e) INSTALLATION OF THE BLOCKS.

The blocks shall be installed in accordance with the requirements shown on the plans.

Blocks shall be placed so that all vertical and horizontal joints between blocks are tight. To avoid continuous joints, blocks shall be laid in a bond pattern with each successive layer turned with the long axis of blocks at 90 ° to previous layer.

Fasteners shall be installed in accordance with the manufacturer's recommendations to lock the blocks in place.

Construction traffic (vehicles, equipment, personnel) shall not be moved on or be placed on the surface of the blocks.

With the exception of sand bags or similar "soft" weights used to temporarily restrain the blocks against wind, construction material other than that shown on the contract drawings shall not be placed or stockpiled on the EPS blocks.

At no time shall heat or open flame be used near the blocks.

(f) FILL OVER THE BLOCKS.

Vehicles and construction equipment shall not move on the blocks until a minimum of 12 inches {300 mm} of earth fill is placed over the blocks.

247.04 Method of Measurement.

Geofoam block will be measured for payment by the number of cubic yards {cubic meters} of block material installed.

247.05 Basis of Payment.

(a) UNIT PRICE COVERAGE.

The quantity of geofoam block installed and measured as noted above will be paid for at the contract unit price per cubic yard {cubic meter}, which shall be full compensation for furnishing all materials, testing, labor, equipment, tools, and incidentals necessary to complete this item of work.

(b) PAYMENT WILL BE MADE UNDER ITEM NO.:

247-A Geofoam Blocks Type __*__ - per cubic yard {cubic meter}

* Type of required block: EPS40, EPS50, EPS70 or EPS100.

Project: EPS-Block Geofoam Ramps, Boston Central Artery/Tunnel Project

Source: J.S. Horvath

SECTION 909.300

EXTERIOR INSULATION AND FINISH SYSTEM (EIFS)

DESCRIPTION

1.01 GENERAL

A. This document contains the general requirements for use, and installation of a Polymer Based (PB), Class PB, Exterior Insulation and Finish System (EIFS) for use on the EPS-Block Fill Structures of this contract. Provide all labor, materials and equipment necessary to complete work in this section in accordance with the requirements of this section and the EIFS manufacturer's product installation requirements.

1.02 RELATED WORK SPECIFIED ELSEWHERE

A. Carefully examine all of the Contract Documents for the requirements, which affect the work of this section.

B. Other sections of the CA/T Supplemental Specifications and Special Provisions, as applicable, which directly relate to the work of this section include, but are not limited to the following:

1. Section 120.010 - Excavation and Backfill
2. Section 140.140 - Ground Water Control
3. Section 150.020 - Lightweight Aggregate Fill
4. Section 170 - Grading
5. Section 901.310 - Reinforced Concrete Structures
6. Section 909.101 - Block-Molded Expanded Polystyrene Lightweight Fill (EPS-Block Fill)

1.03 DEFINITIONS

Adhesive:
A material used to attach the insulation board to the substrate.

Aesthetic Reveal:
A groove cut into the insulation board, which serves the function of decoration and/or provides a starting or stopping point for application of the finish coat.

Applicator:
The company or business entity having the contractual relationship with the Contractor to install and affix the System to the noted Substrate.

Backer Rod:
Closed-cell, flexible polyethylene foam rod. It is sized for the specific joint width(s) plus 25-30% and is compressed and inserted into a joint cavity to a specific depth to form the face of a joint. The rod limits the depth of the sealant joint and helps to produce an hourglass sealant shape that helps to distribute stresses to the sealant.

Base Coat:
A material applied to the face of the insulation board that is used to encapsulate the reinforcing mesh(es).

Class PB:
EIFS where the base coat varies in thickness depending upon the number of layers, or thickness of reinforcing mesh(es). The reinforcing mesh is glass fiber mesh that is encapsulated by the base coat per EIFS manufacturer recommendations and with no mesh color visible. Protective finish coats, of various thicknesses in a variety of textures and colors, are applied over the base coat.

Cold Joint:
The visible aesthetic junction in a finish coat.

Cure:
A chemical process through which the properties of a material are developed.

Dry:
A process of volatile evaporation through which the properties of a material are developed.

Durability:
The capability of the system to maintain serviceability over a specific period of time.

Edge wrap:
A method used to protect the exposed edges of the insulation board with reinforcing mesh(es) and base coat.

EIFS:
A non-load bearing, exterior finish system generally consisting of the following components:

A. Adhesive that attaches the insulation board to the substrate
B. Insulation board
C. Base coat on the face of the insulation board
D. Glass fiber reinforcing mesh(es)
E. A textured protective finish coat
F. Elastomeric coating
G. Sealer

Elastomeric Coating:
A 100% acrylic-based non-textured smooth copolymer Elastomeric resin with dirt pickup Resistance chemistry.

Encapsulated:
When the reinforcing mesh color is not visible in the dried or cured base coat.

Expansion Joint:
A joint through an entire structure or structural element designed to accommodate structural movement.

Finish Coat:
A decorative and protective textured coating, which includes Elastomeric Coating and Sealer, applied over the dry reinforced base coat.

Flash Set (Quick Set):
The early hardening or stiffening in the working characteristics of a Portland-cement paste, mortar or concrete, usually with the evolution of considerable heat. Stiffness cannot be dispelled nor the plasticity regained by further mixing without the addition of water.

Initial Grab:
The ability of a wet state material to remain in place initially after it has been applied.

Initial Set:
A time-related set caused by the hydration process.

Insulation Board:
Expanded Polystyrene (EPS) of a specific AASHTO material type designation, material properties and dry unit weight, which is affixed to the substrate.

Lamina:
The layer comprised of the base coat, reinforcing mesh(es) and the finish coat.

Pot Life:
The duration of time that the wet state material remains workable after it has been mixed.

Primer:
A material used to prepare a surface for application of EIFS to the substrate or of the finish coat to the base coat.

Reinforced Base Coat:
A base coat in which an open-weave glass-fiber fabric mesh(es) has been encapsulated to provide reinforcement.

Reinforcing Mesh(es):
Glass-fiber fabric mesh(es) treated for compatibility with other materials of the system and functions to reinforce the base coat, and provide impact resistance.

Running Bond:
A pattern used when installing the insulation board, to offset the vertical insulation board joints from joints in previous rows of insulation.

Sealant:
A material that has the adhesive and cohesive properties to form a seal. (ASTM C717)

Sealer:
An acrylic based transparent coating resistant to mildew, algae and dirt.

Substrate:
The Block-Molded Expanded Polystyrene lightweight fill (EPS-Block Fill) and other concrete elements to which the System is affixed.

System:
A Polymer Based Exterior Insulation and Finish System (EIFS), Class PB, consisting of an adhesive, insulation board, base coat with reinforcing mesh(es), and finish coat that includes elastomeric coating and sealer.

System Manufacturer:
A company or business entity having the contractual relationship with the Contractor to supply the System and associated products utilized on this contract.

Temper:
To mix additional water into a material in order to bring it to a workable state.

Wet Edge:
The leading edge of a continuously applied wet state material.

Weather-Resistive Barrier:
A membrane or coating used to protect the structure from moisture intrusion.

Wet State Materials:
The adhesive, base coat and finish coat components applied in their liquid or semi-liquid states.

1.04 REFERENCES

A. Comply with the provisions of the following codes, specifications and standards, except as otherwise indicated:

1. ASTM B 117 - Test Method of Salt Spray (Fog) Testing.
2. ASTM C 67 - Method of Sampling and Testing Brick and Structural Clay Tile.
3. ASTM C 150 - Specification for Portland Cement.
4. ASTM C 203 - Breaking Load and Flexural Properties of Block-type Thermal Insulation.
5. ASTM C 237 - Test Method for Shear Properties in Flat wise Plane of Flat Sandwich Construction or Sandwich cores.
6. ASTM C 297 - Test Method for Tensile Strength of Flat Sandwich Constructions in Flat wise Plane.
7. ASTM C 578 - Standard Specification for Rigid, Cellular Polystyrene Thermal Insulation.
8. ASTM C 717 - Terminology of Building Seals and Sealants.
9. ASTM C 1063 - Specification for Installation of Lathing and Furring for Portland Cement-Based
10. ASTM C 1135 - Test Method for Determining Tensile Adhesion Properties of Structural Sealants
11. ASTM C 1186 - Specification for Flat Non-Asbestos Fiber Cement Sheets
12. ASTM C 1382 -Test Method for Determining the Tensile Adhesion Properties of Sealants When Used in Exterior Insulation and Finish Systems (EIFS)
13. ASTM C 1397 - Standard Practice for Application of Class PB Exterior Insulation and Finish Systems (EIFS)
14. ASTM D 968 - Test Method for Abrasion Resistance of Organic Coatings by Falling Abrasive.
15. ASTM D 2247 - Practice for Testing Water Resistance of Coatings in 100% Relative Humidity.
16. ANSI/ASTM D2863-00 -Test Method for Measuring the Minimum Oxygen Concentration to Support Candle-Like Combustion of Plastics (Oxygen Index)
17. ASTM D 3273, Test Method for Resistance to Growth of Mold on the Surface of Interior Coatings in an Environmental Chamber
18. ASTM E 84 - Test Method for Surface Burning Characteristics of Building Materials.
19. ASTM E 96 - Test Methods for Water Vapor Transmission of Materials.
20. ASTM E 108 - (Modified) Method for Fire Tests of Roof Coverings.
21. ASTM E 119 - Method for Fire Tests of Building Construction and Materials.
22. ASTM E 330 - Test Method for Structural Performance of Exterior Windows, Curtain Walls, and Doors by Uniform Static Air Pressure Difference.
23. ASTM E 331 - Test Method For Water Penetration of Exterior Windows, Curtain Walls, and Doors by Uniform Static Air Pressure Difference.
24. ANSI/ASTM E695-79(85) E1- Method for Measuring Relative Resistance of Wall, Floor, and Roof Constructions to Impact Loading.
25. ANSI/ASTM E2098-00 - Standard Test Method for Determining the Tensile Breaking Strength of Glass Fiber Reinforcing Mesh for Use in Class PB EIFS After Exposure to Sodium Hydroxide Solutions.
26. ASTM G153/G152-00, Practice for Operating Light Exposure Apparatus (Carbon-Arc Type) With and Without Water for Exposure of Nonmetallic Materials
27. ASTM G154-00, Practice for Operating Light-and Water-Exposure Apparatus (Fluorescent UV-Condensation Type) for Exposure of Nonmetallic Materials
28. Manufacturers System Installation Details.
29. Manufacturers Cleaning and Recoating.
30. Manufacturers Expansion Joints and Sealants.
31. Manufacturers System Application Instructions.

32. UBC Standard 26-4, Method of Test for the Evaluation of Flammability Characteristics of Exterior, Non-Load Bearing Wall Panel Assemblies Using Foam Plastic Insulation, 1997 Uniform Building Code; International Conference of Building Officials (ICBO).
33. UBC Standard 26-9, Method of Test for the Evaluation of Flammability Characteristics of Exterior, Non-Load Bearing Wall Assemblies Containing Combustible Components using the Intermediate-Scale, Multistory Test Apparatus, 1997 Uniform Building Code; International Conference of Building Officials (ICBO).
34. ANSI/NFPA 268-1998, Standard Test Method for Determining Ignitability of Exterior Wall Assemblies Using a Radiant Heat Source.
35. ANSI/NFPA 285-1998, Standard Method of Test for the Evaluation of Flammability Characteristics of Exterior Non-load Bearing Wall Assemblies Containing Combustible Components Using the Intermediate Scale Multi-Story Test Apparatus.
36. EIMA 101.01, Standard Test Method for Freeze-Thaw Resistance of Exterior Insulation and Finish Systems (EIFS), Class PB (Modified ASTM C67)
37. EIMA 101.86 Standard Test Method for Resistance of Exterior Insulation Finish Systems (EIFS), Class PB to the Effects of Rapid Deformation (Impact).
38. EIMA 105.01, Standard Test Method for Alkali Resistance of Glass Fiber Reinforcing Mesh for Use in Exterior Insulation and Finish Systems (EIFS), Class PB.
39. EIMA 200.01, Standard Specification for Performance of Exterior Insulation and Finish Systems (EIFS), Class PB.
40. EIMA 200.02, Standard Test Method for Determining the Drainage Performance of Exterior Insulation and Finish Systems (EIFS), Class PB.
41. EIMA 200.03, Standard Test Method for Drying Potential for Exterior Insulation and Finish Systems
42. (EIFS), Class PB.
43. MILITARY STANDARD 810F Environmental Engineering Considerations and Laboratory Tests.

1.05 SUBMITTALS

A. PRODUCT DATA

Submit for review in accordance with CA/T Project Specifications Divisions I Subsections 5.02 and 6.01 the following for the review by the Engineer:

1. System Manufacturer's literature including, product data sheets including products literature, specifications, application instructions and details which will be used on this contract.
2. System Manufacturer report certifying that the supplied materials are in conformance with product and materials standards and contract documents.
3. System Manufacturer self-certification of compliance with this Specification.

B. SAMPLES

At least thirty (30) days prior to the start of construction, Contractor shall submit for the review and approval by the Engineer two samples of the System for each finish, texture, and color to be used on the EPS-Block Fill structures of this contract. The same tools and techniques proposed for the actual construction and installation shall be used to produce such samples. The Samples shall be 4 ft. x 4 ft. in size to accurately represent each color and texture to be utilized on this contract.

C. TEST REPORTS

Contractor shall submit in accordance with CA/T Project Specifications Divisions I Subsections 5.02 and 6.01 for the review by the Engineer, copies of the test reports verifying the performance of the System in compliance with the requirements of section 1.06.D of this Specification.

D. MOCK-UP

Contractor shall provide in accordance with CA/T Project Specifications Divisions I Subsections 5.02 and 6.01, a mock-up for the System for evaluation and approval by the in compliance with the requirements of section 1.07.C of this Specification.

1.06 SYSTEM DESCRIPTION

A. GENERAL

The System is a Polymer Based Exterior Insulation and Finish System (EIFS), Class PB, consisting of an adhesive, insulation board, base coat with reinforcing mesh(es), and protective finish coat.

B. METHOD OF INSTALLATION

Field Applied: The System is affixed to the substrate system in place through application of an adhesive.

C. DESIGN REQUIREMENTS

1. The EIFS shall be designed in accordance with this Specification. System details shall also conform to the System Manufacturer's specification.

2. All EIFS areas on this contract shall require an impact resistance Level 4, as defined by EIMA Test Method and Standard 101.86, and as indicated on the contract drawings and contract documents.

3. Wind load resistance shall be determined in accordance with Section 1.06.D.2(c) of this specification and meet the applicable AASHTO Standard Specification for Highway Bridges requirements for the applied wind loads.

4. Substrate for the System shall be primarily the Block-Molded Expanded Polystyrene lightweight fill material forming the EPS-Block fill structures of this contract. Substrate shall also include concrete surfaces representing the vertical faces of concrete elements confining regular fill at the top of the EPS-Block fill structures.

5. Expansion Joints

   Expansion joints shall be provided in the system as follows:

   a) In continuous elevations at intervals not exceeding seventy five (75) feet as shown on the Contract Drawings.

6. Terminations

   a) At all System terminations, the insulation board shall be treated by wrapping with reinforcing mesh(es) and base coat in one of the following methods:

      1) Reinforcing mesh(es) of sufficient width shall be applied to the substrate so it will encapsulate the edge of the insulation board and extend a minimum of two and half (2-1/2) inch onto the substrate behind the insulation board and a minimum of two and half (2-1/2) inch onto the face of the insulation board.

2) The encapsulation of the edge of the insulation board with the base coat and the reinforcing mesh(es) shall be completed after the installation and prior to the application of the mesh(es) to the field of the insulation board.

b) All terminations shall be performed using System Manufacturer's approved accessories and shall be applied in accordance with the System Manufacturer's requirements.

c) The System shall be held back from adjoining materials a minimum of three quarters (3/4) inch for backer rod sealant application.

d) The EPS insulation board shall be terminated a minimum of six (6) inch below finished grade.

e) The base coat shall terminate vertically at the base of the System below finished grade.

f) The finish coat shall terminate vertically maximum two (2) inch below finished grade.

g) Sealants shall satisfy the following requirements:
   1. Shall be provided and installed by the Contractor in accordance with System Manufacturer requirements.
   2. Shall be compatible with the System materials. Refer to current System Manufacturer's listing of sealants tested by sealant manufacturer for compatibility.
   3. The sealant backer rod shall be of closed cell type.

D. PERFORMANCE REQUIREMENTS

1. Durability: The System shall have been tested for durability as follows:

   a) Abrasion Resistance:
   Test Method – ASTM D 968 (Federal Test Standard 141A Method 6191)
   Acceptance Criteria – No cracking, checking or loss of film integrity at 528 quarts of sand.

   b) Absorption-Freeze-Thaw:
   Test Method – EIMA 101.10 (Modified ASTM C67) Air Dry at 120 °F minimum eight (8) hours, total immersion in water at 70-80 °F for eight (8) hours then exposure to 20 °F for sixteen (16) hours.
   Acceptance Criteria – 60 cycles, No cracking, checking, crazing, erosion, rusting, blistering, peeling or delamination when under 5x magnification.

   c) Accelerated Weathering:
   Test Method – ASTM G 153/G152 or G154.
   Acceptance Criteria – No cracking, checking, crazing, erosion, rusting, blistering, peeling or delamination when under 5x magnification.

   d) Mildew Resistance:
   Test Method – ASTM D3273, MIL- STD 810F.
   Acceptance Criteria – No growth supported during 28-day exposure period.

   e) Moisture Resistance:
   Test Method – ASTM D 2247
   Acceptance Criteria – No cracking, checking, crazing, erosion, rusting, blistering, peeling or delamination after fourteen (14) day exposure.

   f) Salt Spray Resistance:
   Test Method – ASTM B 117

Acceptance Criteria – No cracking, checking, crazing, erosion, rusting, blistering, peeling or delamination three hundred (300) hour exposure period..

g) Water Penetration:
Test Method – ANSI/ASTM E 331
Acceptance Criteria – No water penetration beyond the plane of base coat/insulation board interface after fifteen (15) minutes at 6.24 psf or 20% of positive design wind pressure, whichever is greater.

h) Water Vapor Transmission:
Test Method – ASTM E 96 Water Method, Procedure B;
Acceptance Criteria – Standard lamina fourteen (14) gr/hr•ft^2.

i) Alkali Resistance of Reinforcing Mesh:
Test Method – ANSI/ASTM E 2098
Acceptance Criteria – Retained tensile strength of 120 pli.

2. Structural Performance: The System shall have been tested for structural performance as follows:

a) EPS Material and Physical Properties:
Test Method – ASTM C 578
Acceptance Criteria – Meets dry density, compressive strength and flexural strength as specified for AASHTO Material Designation EPS40.

b) Bond Strength:
Test Method – ASTM C 297
Acceptance Criteria – No failure in the adhesive base coat or finish coat. The insulation board shall fail cohesively except that 25% adhesive failure is acceptable. For tested values of nineteen (19) psi or greater, adhesive failure up to 100% is acceptable.

c) Wind Load:
Test Method – ASTM E 330
Acceptance Criteria – Minimum failure load under positive or suction force of 90 psf unless otherwise specified; substrate failure.

d) Impact Resistance:
Test Method – EIMA Standard 101.86
Acceptance Criteria – EIFS shall satisfy the Impact Performance Characteristics requirements corresponding to EIMA Classification, as shown in Table 1 below :

EIMA Classification	Impact Range J (in-lb.s)	Minimum Value Required J (in-lbs)
Level 1	2.83 – 5.54 (25 – 49)	3 (30)
Level 2	5.65 – 10.1 (50 – 89)	6 (56)
Level 3	10.2 – 17 (90 – 150)	12 (108)
Level 4	>17 (>150)	20 (175)

Table 1 – Impact Characteristics of EIFS

3. Fire Performance: The System shall have been tested for fire performance as follows:

   a) Flame Spread
      Test Method – ASTM E 84
      Acceptance Criteria – Insulation board and reinforced coating system shall each, separately, satisfy the following:

      1- The EPS insulation board shall have a Flame Spread index not exceeding 25 and a Smoke Developed index not exceeding 450.
      2- The adhesives and coatings shall have a Flame Spread index not exceeding 20 and a Smoke Developed index not exceeding 10.

   b) Full Scale Diversified Fire Test:
      Test Method – ASTM E 108
      Acceptance Criteria – No significant contribution to vertical or horizontal flame spread:

   c) Fire Endurance:
      Test Method – ASTM E 119
      Acceptance Criteria – Maintains fire resistance of walls rated at One-Hour Assembly.

   d) Intermediate Scale Fire Test:
      Test Method – ANSI/NFPA 285 (UBC Standard 26-9)
      Acceptance Criteria – Maintains fire resistance o-f walls rated at Two-Hour Assembly.

   e) Full Scale Fire Test:
      Test Method – UBC Standard 26-4
      Acceptance Criteria – Maintains fire resistance of walls rated at Two-Hour Assembly.

   f) Radiant Heat Exposure:
      Test Method – ANSI/NFPA 268
      Acceptance Criteria – No surface ignition when exposed to 12.5 kW/m^2.

1.07 QUALITY ASSURANCE

A. QUALIFICATIONS

1. System Manufacturer:
   Shall have, as a minimum, five (5) years experience in manufacturing and supplying the materials required for the System and shall have supplied at least two hundred thousand (200,000) square feet of such materials on at least one construction project.

2. Materials:
   All EIFS component materials shall be in conformance with the requirements of this Specification. All such materials shall be supplied by the selected System Manufacturer or its recognized system component supplier.

3. Applicator:
   Shall be knowledgeable in the proper installation of the System and shall be experienced and competent in the installation of Exterior Insulation and Finish Systems. Additionally the Applicator shall possess a current trained Applicator Certificate from the System Manufacturer. Applicator shall have, as a minimum, five (5) years experience in the installation and construction of the materials required for the System and shall have installed at least two hundred thousand (200,000) square feet of such system on at least one construction project. The applicator shall provide the proper manpower and supervision at the construction site to install the System in compliance with Project plans and Specifications.

4. Insulation Board Manufacturer:
   Shall be the EPS Supplier or Molder capable of producing the Expanded Polystyrene (EPS) insulation board in compliance with the Manufacturing Quality control (MQC) and Material Property requirements outlined in Sections 1.07 and 2.01, respectively, of CA/T Specification 909.101 Division II – Special Provision for Block-Molded Expanded Polystyrene Lightweight Fill (EPS-Block Fill).

B. CERTIFICATIONS

1. All Expanded Polystyrene (EPS) Material utilized in the System shall satisfy the Product flammability requirements specified in ASTM C 578. The surface burning characteristics of the EPS shall be classified by Underwriters Laboratories and be listed in the U.L. Building Materials Directory as having a Flame Spread and Smoke Development rating of not greater than 25 and 450 respectively.

C. MOCK-UP

1. The mock-up shall be full size, representing actual segment of the finished product, with a minimum width of ten (10) feet and a height of at least ten (10) feet. The mock-up shall accurately represent the products being installed, including vertical rustication (aesthetic reveal), expansion joints as well as each color and texture to be utilized on the EPS-Block fill structures of this contract.

2. The mock-up shall be prepared with the same products, tools, equipment, and techniques required for actual construction. The finish used shall be from the same batch as that used for actual construction on this contract.

3. The approved mock-up shall be available and maintained at the Project construction site.

1.08 DELIVERY, STORAGE AND HANDLING

A. All materials shall be delivered to the Project construction site in the original, unopened and undamaged, regulatory compliant packaging with legible manufacturer's identification labels intact.

B. Upon arrival to the construction site, all materials shall be inspected by the Engineer for physical damage, freezing or overheating. Defective materials shall be rejected on the spot and shall be replaced with non-defective materials at no additional cost or time to the Department.

C. All products supplied by the System Manufacturer shall be stored at the Project construction site on a level platform in a cool, dry location, protected from direct sunlight, weather and other damaging exposure. Minimum storage temperature shall be 40°F for all wet products at all times.

D. Portland Cement shall be stored in a dry protected area until ready to use.

E. Material Safety Data Sheets (MSDS) shall be supplied for the components of the System and be available at the Project construction site.

1.09 PROJECT CONDITIONS

A. ENVIRONMENTAL REQUIREMENTS

1. Application of wet materials shall not take place during inclement weather unless appropriate protection is provided. Protect materials from inclement weather until they are dry. Contractor shall submit for review by the Engineer the method proposed to provide protection for the System.

2. Application of wet materials shall not be allowed when the surface and ambient temperatures are leass than 40°F unless appropriate protection and supplemental heat are provided to maintain material temperature at a minimum of 40°F during application. Such temperature shall be maintained for a minimum of 24 hours after application, or until completely dry.

3. Immediately following the application of EIFS, the tops of the finished system shall be temporarily protected to prevent water infiltration behind the system.

4. Open joints shall be protected from water intrusion during construction with closed cell backer rod, or temporary covering, until permanently sealed.

1.10 SEQUENCING AND SCHEDULING

A. Installation of the System shall be coordinated with other construction trades.

B. Sufficient labor force and equipment shall be provided by the Applicator to ensure a continuous operation, free of cold joints, scaffold lines, texture variations, etc.

C. The Contractor shall provide electric power supply, clean water, and a clean work area at the location where the materials are to be applied.

1.11 LIMITED MATERIALS WARRANTY

A. The System Manufacturer shall provide a written, standard five (5) year limited warranty against defective materials as part of the product data submittals, as indicated in Section 1.05.A of this Specification.

1.13 MAINTENANCE AND REPAIR

A. Should maintenance or repair to the System be required during the course of construction, maintenance and repair shall follow the procedures noted in System Manufacturer's Application Instructions.

B. All of the specified products are designed to minimize maintenance. However, as with all exterior finishes products, depending on location; some cleaning may be required. See System Manufacturer's publication on Cleaning & Recoating.

PRODUCTS

2.01 MANUFACTURING

A. All components of the System shall be obtained from the System Manufacturer or its authorized distributors.

2.02 MATERIALS

A. Portland Cement: Shall be Type I or II, meeting ASTM C 150, white or gray in color, fresh and free of lumps.
B. Water: Provide clean potable water complying with Standard Specification M4.02.04

2.03 COMPONENTS

A. ADHESIVE

Adhesives shall be compatible with the substrate, insulation board and reinforcing mesh(es). Adhesive shall be a high-performance, dry mixed Polymer-Based, Fiber-Reinforced material.

B. INSULATION BOARD

Expanded Polystyrene (EPS) insulation board shall correspond to AASHTO material type designation EPS40, in conformance with the Manufacturing Quality control (MQC) and Material Property requirements outlined in Sections 1.07 and 2.01, respectively, of CA/T Specification 909.101 Division II – Special Provision for Block-Molded Expanded Polystyrene Lightweight Fill (EPS-Block Fill). The supplied insulation board shall meet the System Manufacturers specification for Insulation Board and supplied by an EPS board supplier in compliance with the qualifications requirements outlined in section 1.07A.1 of this Specification.

The dimensions of the supplied EPS insulation boards shall be as follows:

Maximum width	24 inch
Maximum length	48 inch
Minimum thickness	2.75 inch

C. BASE COAT

Shall be compatible with the EPS insulation board and reinforcing mesh(es).

1. Ready mixed: A High-performance, dry mixed Polymer-Based, Fiber-Reinforced Base Coat.

D. REINFORCING MESHES

Shall be a balanced open weave, glass fiber fabric treated for compatibility with other System materials. A Level 4 reinforcing mesh shall be used in the first layer of the base coat. A second layer of Level 1 mesh shall be applied after the first layer has dried. Reinforcing mesh(es)es shall be embedded in the base coat and available in the following weights, and shall provide the indicated minimum tensile strengths:

1. Level 1: 4 – 6 oz/dy^2; 150 lbs/in. warp, 200 lbs/in. weft.
2. Level 2: > 6 – 11 oz/dy^2; 330 lbs/in. warp, 460 lbs/in. weft.
3. Level 3: > 11 – 15 oz/dy^2; 525 lbs/in. warp, 540 lbs/in. weft.
4. Level 4: > 20 oz/dy^2; 875 lbs/in. warp, 855 lbs/in. weft.

E. FINISH COAT

Shall be the type, color, and texture as selected by the Department and shall be one or more of the following:

1. Standard formulated dirt resistant pickup water-based, 100% acrylic coatings with integral color and texture, and formulated with the special chemistry: The aggregate shall be either quartz, calcium carbonate or approved equal.

F. ELASTOMERIC COATING

Shall be a non-textured smooth coating with Dirt Pickup Resistance chemistry and is based upon a 100% acrylic, copolymer Elastometric resin. The coating must also provide an integral mildewcide chemistry to inhibit the development of mold and mildew.

EXECUTION

3.01 EXAMINATION

A. Prior to installation of the System, the Contractor shall ensure that the substrate:

1. Satisfies the requirements listed in Section 2.03.B of this Specification.
2. Has a resulting exterior surface that is vertical and planar within a tolerance of ± one eighth (1/8) inch between EPS blocks.
3. Satisfies the requirements of CA/T Specification 909.101 Division II – Special Provision for Block-Molded Expanded Polystyrene Lightweight Fill (EPS-Block Fill).
4. Is sound, dry, connections are tight, has no surface voids, projections or other conditions that may interfere with the System installation.

B. Prior to the installation of the System, the Contractor shall insure that all needed drainage attachments, conduits, and other waterproofing details have been completed, if such completion is required prior to application of the System.

C. The Contractor shall notify the Engineer of all discrepancies in the substrate and propose a repair procedure for corrective action prior to the start of construction.

3.02 SUBSTRATE SURFACE PREPARATION

A. Prior to application of the insulation board of the EIFS system, the substrate shall be prepared as to be free of any UV degraded EPS that may have developed, any surface deposits of foreign materials such as oil, dust, dirt, form-release agents, efflorescence, paint, wax, water repellants, moisture, frost, soil and any other materials that inhibit adhesion.

B. Substrate temperature shall be a minimum of 40°F during application and for 24 hours following the installation of the EIFS. Protection and supplemental heat shall be provided as necessary on the substrate to maintain this condition.

C. Concrete elements in substrate, to which EPS insulation board is affixed, shall be in place for at least 28 days prior to first installation of any component of the System.

3.03 MIXING

A. All materials requiring preparation in the field shall be accompanied by the System Manufacturer's complete mixing instructions. The Applicator shall comply with all instructions. Materials supplied by any party other than the System Manufacturer are not allowed and shall be rejected and replaced at no additional cost or time to the Department.

3.04 INSTALLATION OF INSULATION BOARD

A. The System shall be installed strictly in accordance with this Specification, contract drawings and the System Manufacturer's current installation instructions.

B. Insulation board shall be affixed to the substrate using one of the adhesive methods shown below, as outlined by the System Manufacturer:

1) Notched Trowel Method: The adhesive shall be applied to the entire surface of the insulation board using a notched trowel as recommended by the System Manufacturer.
2) Ribbon and Dab Method: The adhesive shall be applied to the entire perimeter of the insulation board in a ribbon fashion two (2) inch wide by three eighth (3/8) inch thick. Dabs of approximately four (4) inch in diameter, in the same thickness, are then applied eight (8) inch on center over the remainder of the board.

C. Joints between adjacent insulation boards shall be staggered with the horizontal and vertical joints formed between the surfaces of any two adjacent EPS blocks in the substrate in both the longitudinal direction and along the height of the embankment.

D. Installation of the insulation board shall start at the top of the substrate down to the base located below finished grade.

E. After the adhesive has been applied to the back of an insulation board, such board shall be installed by sliding it into place until it abuts the adjoining insulation board. Pressure shall be applied over the entire surface of the insulation board to achieve uniform contact and high initial grab. If an installed insulation board shows sign of planar irregularities or lack of fit, such insulation board shall be checked for proper contact to the substrate. Proper contact has been achieved when a piece of insulation board is removed and a similar amount of adhesive is adhered to both the substrate and the insulation board or the insulation board cannot be removed without breaking.

F. All insulation boards shall be butted tightly. Any gaps greater than one sixteenth (1/16) inch between the insulation boards shall be filled with small pieces of insulation board cut to size. Gaps shall not be filled with adhesive or any other non-insulating material.

G. Aesthetic reveals shall be cut into the insulation board prior to application of the reinforcing mesh(es).

H. Insulation board thickness shall not be less than two (2) inch everywhere as well as in the bottom of an aesthetic reveal.

I. The adhesively applied insulation board shall remain undisturbed for 24 hours prior to proceeding with the installation of the base coat/reinforcing mesh(es), or as necessary for the adhesive to dry.

J. Prior to base coat/reinforcing mesh(es) application, all insulation board irregularities shall be rasped flat, depending on the type of insulation board used. Insulation board shall be free of deleterious material prior to base coating application. Insulation board exposed to sunlight may develop a yellowing, powdery film on the surface. This film must be entirely removed by sanding or dusting prior to base coat application.

3.05 APPLICATION OF BASE COATING AND REINFORCING MESH

A. Application of the base coating and reinforcing mesh(es) shall be strictly in accordance with this specification, contract drawings and the System Manufacturer's current application instructions.

B. The surface shall be inspected to ensure that it is free of surface irregularities such as voids, projections and trowel marks.

C. Backwrapping at all system terminations shall be completed by encapsulating the backwrap reinforcing mesh(es) in the base coat or using EIFS manufacturer recommended trim accessories.

D. The field reinforcing mesh(es) shall be encapsulated in the wet base coat, toweling from the center to the edge of the reinforcing mesh(es), to avoid wrinkles. The reinforcing mesh(es) shall be continuous at all corners, overlapped a minimum two and half (2-1/2) inch at adjoining reinforcing mesh(es) edges, including previously backwrapped mesh(es). The reinforcing mesh(es) shall be encapsulated in the base coat in accordance with the EIFS manufacturer's recommendation and with no mesh color visible.

E. A layer of Level 4 reinforcing mesh underlying a Level 1 reinforcing mesh, shall both be encapsulated to develop the high impact performance required for all EIFS System applications constructed on this contract. Incorporating heavier weight reinforcing mesh provides higher impact performance resistance.

F. Reinforcing mesh(es) shall be doubled within eight (8) inch of outside corners.

G. EPS shapes installed with EIFS shall have reinforcing mesh(es) encapsulated into the base coat as described in item C of this section.

H. All edges of the insulation board at expansion joints shall be wrapped with the base coat and reinforcing mesh(es) as specified by the EIFS manufacturer for the particular EIFS use. The reinforcing mesh(es) shall be fully encapsulated such that mesh color is not visible.

I. At all aesthetic reveals, Level 4 mesh shall terminate at the edge of the reveal. Level 1 mesh, however, shall be continuous and backwrapped around the insulation board. Care shall be taken to ensure that the Level 1 reinforcing mesh is fully encapsulated into the reveal and that such reinforcing mesh shall not be cut during application of the base coat.

J. The reinforcing mesh(es) shall be fully encapsulated in the base coat throughout the field of the wall, at corners, edges and joints to conceal color. The surface shall be free of voids, projections, trowel marks and other surface irregularities.

K. Primer shall be installed in accordance with the EIFS manufacturer's recommendations, as required.

L. The base coat/reinforcing mesh(es) application shall be cured prior to the application of the finish coat.

3.06 APPLYING FINISH COAT

A. The finish coat shall be mixed according to the specifications of the EIFS manufacturer. Only clean, potable water shall be added to achieve the desired workability. The same amount of water shall be added to each container of finish within a given lot.

B. Prior to application of the finish coat, surface irregularities in the base coat, such as trowel marks, board lines, and reinforcing mesh(es) laps shall be corrected.

C. Application of the finish shall be with a stainless steel trowel or equipment as specified by the EIFS manufacturer. Tools and equipment shall be kept clean at all times.

D. Finish coat shall be applied to the cured base coat maintaining a wet edge at all times to obtain a uniform appearance. Finish shall be applied continuously at a natural break such as corners, joints or tape line. Sufficient labor force, equipment, material and scaffolding shall be provided to continuously

finish a distinct wall area. Scaffolding shall be placed a distance from the wall, consistent with safety standards, which will allow uniform texturing of the finish. The thickness of the finish coat shall be in accordance with the EIFS manufacturer's current published instructions.

E. Finish materials shall not be applied onto joint surfaces that will be the substrate for sealants unless otherwise specified by the EIFS manufacturer.

F. The texture and color of the finish shall be as specified and in accordance with the approved sample. All mechanics applying and texturing the finish shall utilize the same tools, equipment and techniques to achieve uniformity.

G. All finish work shall be protected from damage until fully cured.

3.07 CONSTRUCTION QUALITY CONTROL (CQC)

A. The Contractor shall assume primary responsibility for the proper application and installation of the System and associated material components.

B. The Contractor shall provide a written Certification to the Engineer that the quality of work performed relative to the substrate system, details, installation procedures, workmanship are all in compliance with System Manufacturer's requirements.

C. The Contractor shall supply for review and approval by the Engineer a Certification in writing indicating compliance of the supplied EPS insulation board to the Manufacturers Quality Control requirements outlined in CA/T Specification 909.101 Sections 1.05.D and 1.07 as well as the Material Properties of Table 2 of the same specification.

D. The Contractor shall supply for review and approval by the Engineer a Certification in writing showing compliance of the sealant application as installed with the sealant manufacturers and System Manufacturer's recommendations.

E. Contractor shall test all materials previously installed with the System Manufacturer's recommended moisture monitoring meter to determine it is suitable to begin further installation of components.

3.08 PROJECT CONSTRUCTION SITE CLEAN-UP

A. All excess System materials shall be removed from the Project construction site by the Contractor and disposed of in accordance with all applicable legal requirements.

B. All surrounding areas, where the System has been installed, shall be left free of debris and foreign substances resulting from the Contractor's work.

3.09 PROTECTION

A. Contractor shall provide protection for the System and the finished product from weather and other damage until such time permanent protection in the form of flashings, sealants, etc. are installed. Contractor shall submit for review by the Engineer the method proposed to provide protection for the System.

COMPENSATION

4.01 METHOD OF MEASUREMENT

A. Supply and installation of Exterior Insulation and Finish System (EIFS) willnot be measured separately.

4.02 BASIS OF PAYMENT

A. Payment for work in this section will be included as part of the Contract price for the structure of which it forms a part.

Project: UDOT I-15 Core

Source: Utah Department of Transportation

GEOMEMBRANE

A. Geomembrane (gasoline-resistant) shall consist of a separate, puncture-free geomembrane. The geomembrane shall be flexible and, by its own weight, shall cover and conform closely to 90 degree edges and corners of Geofoam blocks at ambient temperatures above 45 degrees Fahrenheit, without additional heating of the geomembrane.

B. The geomembrane shall be reinforced or unreinforced geomembrane. It shall be manufactured from a tripolymer consisting of polyvinyl chloride, ethylene interpolymer alloy, and polyurethane or a comparable polymer combination. It shall meet the following physical and chemical requirements (specified as minimum or maximum, not average roll properties):

1. ASTM D 751: Thickness, mils: Minimum = 28
2. ASTM D 814: Unleaded Gasoline Vapor:: Maximum = 0.40 transmission rate, ounces per square foot per 24 hours
3. ASTM D 751: Grab tensile strength, pounds: Minimum = 600 both machine and cross direction (1-inch grip, 4-inch 8-inch sample)
4. ASTM D 751: Elongation at break, percent: Minimum = 20; Toughness: Minimum = 14,000 grab tensile times percent elongation (for example: 620 pounds x 23% = 14,260); puncture resistance, pounds: Minimum = 800
5. ASTM D 751 (ball tip)
6. ASTM D 2136: Cold crack, pass degrees Fahrenheit -30 (1-inch mandrel, 4 hours) factory seams bonded width, inches each seam: Minimum = 2
7. ASTM D 751: Shear, pounds (modified per minimum = 320 National Sanitation Foundation Std. No. 54) Fail in base geomembrane material

C. Furnish a Certificate of Compliance stating that the selected geomembrane has been tested, meets the above-mentioned requirements, and is:

1. Free from pinholes, tears, and other defects that would cause leakage of liquids through the geomembrane.

2. Acceptable for spill containment of hydrocarbons, including automobile gasoline, aviation gas, diesel fuel, kerosene, hydraulic fluid, methanol, ethanol, mineral spirits, and naphtha.

Project: Idaho Bridge Project

Source: J.S. Horvath

SP-XXA SHOTCRETE PROTECTIVE FACING FOR EPS BLOCK FILL
SP-XXB SHOTCRETE PROTECTIVE FACING FOR WELDED WIRE MSE CULVERT EXTENSION HEADWALLS

Description. This work shall consist of designing, providing all materials and labor for shotcrete protective facing for exposed faces of EPS Block Fills and for Welded Wire MSE headwalls of the three 120 inch diameter culvert extensions. Design by the Contractor shall include the method and materials for attaching the protective facing to the exposed faces of the EPS Block Fill and the headwalls. The Contractor designing and installing the EPS Block fill will have the choice of constructing the protective finish of either Exterior Insulation and Finish System (EIFS) or shotcrete. If EIFS is the selected finish, then the work of designing and installing shotcrete as the protective facing for the EPS Block fill will not be performed.

Materials. Materials shall conform to the following requirements of the Standard Specifications supplemented and modified as follows:

Portland Cement (Type II with silica fume)	701.01
Air-Entraining and other Chemical Admixtures	709.02 - 709.04
Curing Materials	709.01
Water	711.01
Fine Aggregate	703.02
Coarse Aggregate	703.03
Welded Wire Fabric	708.02
Bar Reinforcement	708.02

Shotcrete: Shotcrete shall comply with the requirements of ACI 506.2-95, "Specification for Shotcrete", except as otherwise noted. These Specifications refer to premixed cement and aggregate pneumatically applied by suitable equipment and competent operators. Shotcrete shall be composed of Portland Cement, fine and coarse aggregate and water. Either wet-mix or dry-mix shotcrete may be used. The shotcrete shall be reinforced with welded wire mesh, and shall be fiber reinforced. The Contractor shall be solely responsible for the design of shotcrete mixes and for the quality of shotcrete placed in the work. Premixed and packaged concrete product, specifically manufactured as a shotcrete product, may be provided for on-site mixed shotcrete, if approved by the Engineer. The premixed packages shall contain Portland cement, aggregate and steel fibers, all conforming to the requirements of this specification.

The shotcrete mix design shall be designed by the Contractor, and shall as a minimum attain compressive strengths of 1,800 psi at 3 days, 2,500 psi at 7 days, and 4,000 psi at 28 days, as determined by AASHTO T 22 (ASTM C39) testing of compression test cylinders.

Water: In addition to the requirements set forth in Subsection 711.01, the water used in the shotcrete mix shall also be free of elements which could cause staining.

Fly Ash: Fly ash shall consist of Type F and shall meet the requirements of section 502.02 and Section 714 of the ITD Standard Specifications.

Silica Fume: Silica fume meeting the requirements of ASTM C 1240 shall be added to the cement at the rate of 10 to 14 percent by weight of Portland cement.

Admixtures: Admixtures shall not be used without permission of the Engineer. If admixtures are used to entrain air, reduce water-cement ratio, retard or accelerate setting time or accelerate the development of strength, they shall be used at the rate specified by the manufacturer and must be compatible with the cement used. Use of calcium chloride accelerating agent will not be permitted. When used, admixtures shall be dissolved in water before introduction into the mixture.

Welded Wire Mesh: Unless shown otherwise on the plans, welded wire fabric shall be galvanized 4 inch X 4 inch mesh meeting the requirements of AASHTO M 55. The welded wire fabric shall be clean and free from loose mill scale, rust, oil or other coatings that may interfere with bond.

Steel Fiber Reinforcement: Steel fibers shall meet the requirements of ACI 5061R-98. Steel fibers shall have a length between 3/4 and 1 1/4 inches, have blunt or hooked ends, have a length to diameter ratio of less than 80, and shall be cold drawn carbon steel with a minimum tensile strength of 160,000 psi. Only steel fibers manufactured specifically for use in shotcrete applications shall be allowed. The steel fiber content shall be not less than 100 pounds for each cubic yard of shotcrete. The steel fibers shall be premixed with the cement.

Anchors and Fasteners: Anchors and fasteners for attaching the protective facing to either the EPS Block or the welded with walls shall be designed by the Contractor, subject to approval of the Engineer.

Finish Coat: The finish coat for shotcrete facing over EPS Block shall consist of two layers of polymer modified cementitious paint.

Equipment:

Pump system: The pump system utilized to convey premixed shotcrete ingredients shall deliver a uniform and uninterrupted flow of material, without segregation or loss of the ingredients. The mixing equipment shall be capable of thoroughly mixing the specified materials in sufficient quantity to maintain continuous placing.

Air compressor: The air compressor shall be capable of maintaining a supply of clean air adequate for maintaining sufficient nozzle velocity for all parts of the work and for the simultaneous operation of a blow pipe for clearing away rebound. The compressor shall be capable of providing a minimum of 9 cubic yards per minute per operating nozzle. The Contractor shall not use air supply systems that deliver oil-contaminated air or are incapable of maintaining constant pressure.

Water Supply System. For dry mix, provide a water storage tank at the job site. Provide a positive displacement pump with a regulating valve that is accurately controlled to provide water in the pressures and volumes recommended by the delivery machine manufacturer.

Dry-mix process:

Batching and mixing equipment: The mixing equipment shall be capable of thoroughly mixing the materials containing the specified maximum size aggregate and admixtures in sufficient quantity to maintain continuous application. When prepackaged material is used, predampening, also referred to as premoisturizing, equipment shall be used.

Delivery equipment: The equipment shall be capable of discharging the aggregate-cement mixture into the delivery hose and delivering a continuous stream of uniformly mixed material to the discharge nozzle at a uniform rate. The discharge nozzle shall be equipped with a manually operated water injection system (water ring) for directing an even distribution of water through the aggregate-cement mixture. The water valve shall be capable of ready adjustment to vary the quantity of water, and shall be convenient to the nozzleman. The water pressure at the discharge nozzle shall be sufficiently greater than the operating air pressure to assure that the water is thoroughly mixed with the other materials. The water pressure shall be steady (nonpulsating). Equipment parts, especially the nozzle liner and water ring, shall be regularly inspected and replaced as required. Written recommendations of the equipment manufacturer shall be followed on the type and size of the nozzle to be used and on cleaning, inspection and maintenance of the equipment.

Wet-mix process:

Batching and mixing equipment: The mixing equipment shall be capable of thoroughly mixing the specified materials in sufficient quantity to maintain continuous application.

Delivery equipment: The equipment shall be capable of discharging the premixed materials into the delivery hose and delivering a continuous stream of uniformly mixed materials to the discharge nozzle. Recommendations of the equipment manufacturer shall be followed on the type and size of nozzle to be used, and on cleaning, inspection and maintenance of the equipment.

Construction Requirements.

Qualifications:

Designer: The designer of the shotcrete protective facings shall be a Professional Engineer licensed in the State of Idaho, and shall have at least 5 years experience in the design and construction of shotcrete coverings, and shall have performed at least three projects of similar size and complexity in the last 5 years.

Contractor: The Contractor's field staff for the work shall include a full time shotcrete supervisor having not less than 5 years experience as a shotcrete nozzle operator. The shotcrete nozzle operator shall have at least 2,000 hours of experience performing shotcrete applications and shall be certified by ACI as a shotcrete nozzle operator. The nozzle operator's experience shall include at least 3 projects of similar size and complexity in the last 5 years. The delivery equipment operator shall have at least 2

years experience on similar use of equipment and applications. Prior to the start of shotcreting for this job, nozzlemen shall, in the presence of the Engineer, demonstrate their ability to apply shotcrete of the required quality on test panels. Two satisfactory test panels (one plain and the other reinforced), shot in a vertical position for each mix used during the course of the work, shall be the minimum qualification test for nozzlemen before they shall be permitted to place shotcrete in permanent construction.

Design: Design of the protective facings shall be performed by a Professional Engineer licensed in the State of Idaho. The design shall include design of the shotcrete mix, detailed analysis and design of all components of the facings, including systems for attaching or adhering the protective facing to the EPS Block Fill or the Welded Wire MSE wall of the culvert headwalls, including but not limited to anchors, fasteners, inclusions in the fill or EPS Block, etc. The facings shall be designed for a life of 75 years without spalling or deteriorating during the life of the structure. The designer shall coordinate design efforts and proposed applications with the designers/Contractors of the EPS Block Fill and the culvert headwalls. The design shall include structural calculations for analysis of the facings, designing all reinforcement and attachment components, and accounting for the weight of the facing being supported by the substrate to which it is attached. Design of protective wall facings shall be subject to approval of the Engineer. The design shall provide sufficient expansion joints to allow expansion and contraction of the shotcrete without damaging the surface.

The shotcrete protective surface for EPS Block fills shall be 2 inches thick, plus or minus ¼ inch, and shall include a welded wire mesh. If for any reason or at any location, the thickness of the shotcrete on the EPS Block fill is greater than this thickness, the designer and Contractor shall demonstrated that the facing will be adequately supported by adherence to the EPS block without sagging or inducing excessive creep in the EPS block material. The shotcrete protective facing for Welded Wire MSE headwalls shall be a minimum of 4 inches thick. The edges of the shotcrete, particularly the top edge, shall be protected by flashing or other covering, as designed by the Contractor, subject to approval of the Engineer.

Pre-production Testing: The Contractor shall conduct preconstruction shotcrete field trials before starting shotcrete production. The field trials shall be performed in the presence of the Engineer.

1. Field Trials: Construct rigid wood forms at least 6-in. thick by 3 ft by 3 ft. in size. The material in the forms shall be non-absorbant and non reactive with the cement. The field trials shall be performed using the same shotcrete mix, air and water pressure and nozzle tip as will be used for the actual placement of the shotcrete on production surfaces. Have each proposed nozzle operator make test panels on two vertical wood forms. The test panels shall be left undisturbed and protected at the point of placement for at least 24 hours or until the final set has taken place, and shall be cured using the proper curing compound in a manner similar to the anticipated field conditions. Cure the test panels according to AASHTO T 23, without immersing the panels. At least two panels shall be made for each shotcrete mix design to be used on the project.

2. Coring: Drill six 3-in. diameter cores from each test panel according to AASHTO T 24. Trim the ends of the cores according to AASHTO T 24 to make cylinders at least 3-in. long. All compressive test cylinders shall have a minimum length to width ratio of 2:1.

3. Compressive Strength Testing: Soak the cylinders in water for 40 hours immediately before testing. Test three cylinders from each test panel four days after field trial and test

the remaining three cylinders 28 days after the field trial. Perform tests according to AASHTO T 23. The Engineer shall receive a copy of the compressive strength test results at least five days prior to starting any production work. Production shotcrete work shall not begin until satisfactory test results are obtained. All specified strength requirements shall be satisfied before the shotcrete mix design will be considered for acceptance.

4. Mix Design Acceptance: The Engineer will accept or reject the shotcrete mix design based on the results of the preconstruction field trials and testing. Before approving any changes to a previously accepted mix design, the Engineer may require additional preconstruction testing at no additional cost to the State.

Production Testing: The Contractor shall prepare in the presence of the Engineer at least one test panel shot daily for each shotcrete mix deisgn used, plus one test panel whenever the nozzle operator or equipment is changed during the daily work period. The shotcrete panels shall be allowed to cure using the proper curing compound in the field under the same conditions as the production shotcrete.

Compressive test cylinders shall be prepared by the Contractor by coring 3 inch outside diameter cores (3 inch inside diameter core bit) from the cured shotcrete test panels. Six cores shall be taken from each panel. The cores shall be transported to the testing laboratory within three days of being shot in a manner to prevent being damaged. Should any shotcrete section be deficient in any of the specified criteria, that section shall be remedied to the Engineer's satisfaction at no expense to the State. Such remedies may include but not be limited to removal and replacement of the substandard section.

The surface shall be sounded with a hammer for voids beneath the shotcrete resulting from rebound pockets or lack of bond. Voids, sags, or other defects shall be carefully cut out and replaced with a succeeding layer, at no expense to the State. When welded wire mesh reinforcement is used and is damaged or destroyed by such repairs, the damaged area shall be replaced by properly lapped and tied additional welded wire mesh.

Batching and Mixing Shotcrete:

Dry-mix process: The cement and aggregate shall be batched by weight. Predampening shall be carried out prior to flow into the main hopper and immediately after flow out of the packaging in order to ensure that the premix will flow at a uniform rate (without slugs) through the main hopper, delivery hose and nozzle to form uniform shotcrete, free of dry pockets. No predampened cement/aggregate mix shall be used if allowed to stand for more than 90 minutes.

Wet-mix process: Batching and mixing shall be done according to the applicable provisions of ASTM C 94.

Surface Preparation: Clean loose material, mud, rebound, and other foreign matter from all surfaces to receive shotcrete. Remove curing compound on previously placed shotcrete surfaces by sandblasting. All surfaces to receive shotcrete shall be moistened before application.

Shotcrete Application: Shotcrete shall be pneumatically applied according to these specifications and the American Concrete Institute's "Guide to Shotcrete" (ACI 506R-05). Shotcrete shall not be placed on any surface which is frozen, spongy, or where there is free water. The surface shall be dampened just before applying shotcrete. The thickness of the shotcrete blanket shall be controlled by installing noncorrosive pins, nails, or other gauging devices normal to the face, such that they protrude the required shotcrete thickness outside the face. These pins shall be placed on a maximum 5 feet spacing in a square pattern, with no less than 2 pins per increment of surface area to receive shotcrete. A minimum 1 inch cover of shotcrete shall be placed over the welded wire fabric.

The shotcrete shall be applied from the lower portion of the area upwards so that rebound does not accumulate of the portion on the surface that still has to be covered. The nozzle shall be held at a distance and at an angle approximately perpendicular to the working face so that rebound material will be minimal and compaction will be maximized. Shotcrete shall be applied in a circular motion to build up the required layer thickness. Shotcrete shall emerge from the nozzle in a steady uninterrupted flow. When, for any reason, the flow becomes intermittent, the nozzle shall be diverted from the work until steady flow resumes.

Shotcrete shall be applied within 45 minutes of adding cement to the mixture, and at a temperature between 50°F and 85 °F. The surface of each shotcrete layer shall be uniform and free of sags, drips or runs. Layer thickness of each shotcrete layer shall be limited to 2 inches. If additional thickness is required, broom or scarify that applied surface and allow the layer to harden. The initial layer shall be cleaned of all loose material prior to placing succeeding layers. Dampen the surface before applying an additional layer. Thicker layers may be approved by the Engineer if the Contractor can demonstrate that no sloughing or sagging is occurring.

A nozzleman's helper equipped with an air blowout jet shall attend the nozzleman at all times during the placement of shotcrete, to keep the working area free from rebound. Rebound material shall not be worked into the finished product. Rebound is defined as the shotcrete constituents which fail to adhere to the surface to which shotcrete is being applied. It shall not be salvaged or included in later batches.

Construction joints shall be tapered to a thin edge over a distance of at least 1 foot, and the surface of such joints shall be thoroughtly wetted before any adjacent section of shotcrete is placed. Square construction joints shall not be permitted.

Weather Limitations: Place shotcrete when the ambient temperature is 50 degrees F or higher. Shotcrete shall not be placed when night time temperatures are expected to be below 40 degrees F, or shotcrete shall be covered and heated or otherwise protected. Do not perform shotcrete operations during high winds and heavy rains.

Shooting shall be suspended if:

- High wind prevents the nozzleman from proper application of the material.
- The temperature is below 50 degrees F, and steady or falling.
- The fresh shotcrete temperature is below 50 degrees F.
- External factors such as rain wash cement out of the freshly placed material or cause sloughs in the work.

Curing: Air placed shotcrete shall be cured by applying a white pigmented, liquid membrane-forming curing compound as specified in Subsection 709.01 of the Standard Specifications. The curing compound shall be applied immediately after gunning. The air in contact with shotcrete surfaces shall be maintained at temperatures above 40 degrees F for a minimum of 7 days. Curing compounds shall not be used on any surfaces against which additional shotcrete or other cementitious finishing materials are to be bonded unless positive measures, such as sandblasting, are taken to completely remove curing compounds prior to the application of such additional materials.

Submittals: Preproduction: The following shall be submitted to the Engineer for review and approval at least 30 days before production shotcreting:

1. Design of the shotcrete facing including supporting structural calculations. This shall include the method of attaching the facing to the substrate (either the EPS Block or the Welded Wire of the headwalls).

2. Shotcrete mix design long with results of preproduction testing to verify the adequacy of the mix.

3. Shotcrete test panels as described in this special provision.

4. Other information necessary to verify compliance with ACI 506.2.

5. List of all equipment to be used in the hauling, storage, mixing and application of shotcrete, along with capacities, calibrations, and/or manufacturer's certifications, instructions or descriptions, as appropriate.

6. Qualifications of the facing designer, shotcrete superintendent, all nozzlemen to be used on the project, delivery equipment operators and nozzlemen helpers. Qualifications shall include the list of projects performed by the various individuals, along with compete descriptions, and the names and contact information of persons who can verify the person's experience and qualifications.

7. A written plan describing the sequence of operations, methods and procedures for mixing, applying and testing the shotcrete.

Submittals during Production: In addition to the above, the Contractor shall submit the following to the Engineer during production:

1. Results of production testing of shotcrete within 1 day of completion of tests. Submittals shall include sample identification, mix design, test panel number, name of nozzleman, times and dates of preparation and testing, description of curing, compressive test results.

2. A Production Summary that summarizes quantity and location of placement for each work shift. The summary shall include the name of the nozzleman placing the shotcrete during that shift, observation reports verifying placement of all hardware for attachment of the facing, including, as appropriate, welded wire, anchors, fasteners, etc.

3. A summary of all repairs made to deficient shotcrete.

Method of Measurement.

Measurement of the shotcrete protective facings shall be by the square foot, complete in-place. Payment shall include all labor, materials, equipment necessary to complete the design, testing, providing materials on site, mixing, placing, and all finishing coats.

Basis of Payment.

Pay Item	**Pay Unit**
Shotcrete Protective Facing for EPS Block Fill	Square Yard
Shotcrete Protective Facing for Welded Wire MSE Culvert Extension Headwalls	Square Yard

Title: Spec-Data Sheet

Source: AFM® Corporation, February 1994

SPEC DATA®

This Spec-Data sheet conforms to editorial style prescribed by The Construction Specifications Institute. The manufacturer is responsible for technical accuracy.

1. PRODUCT NAME

Perform® EPS lightweight fill and ground stabilization Geofoam
Perform™ Guard EPS lightweight fill and ground stabilization Geofoam with insect preventative.

2. MANUFACTURER

AFM Corporation
PO Box 246
Excelsior, MN 55331
Phone: (612) 474-0809 (in MN)
(800) 255-0176

3. PRODUCT DESCRIPTION

Basic Use: Perform and Perform Guard EPS Geofoam are used in ground fill applications where a lightweight fill material is required to reduce stresses on underlying soils. Projects involving roads, bridge approach fills, embankments, levees, berms, foundations, landscaping, etc., can benefit from the use of Perform EPS Geofoam.

EPS Geofoam has been used in like applications worldwide for over 30 years. Using Perform EPS Geofoam maximizes onsite installation efficiency: material arrives ready to place, no weather delays, material can be prefabricated, no staging required, material can be inventoried, production efficiency improved, easy to handle.

Traditional earth materials used as fill are heavy and can cause undesirable settlement or instability of underlying soils. Other fill materials such as foamed concrete, waste tires, woodchips, wood fiber, etc., have higher densities, are variable in their make-up and are not engineered due to field execution variables. They also have limitations in handling and can be weather sensitive. Both earth and these fill materials may require staged construction of preloading and surcharging, draining, etc.

Composition and Materials: Perform EPS Geofoam is a cellular plastic material that is strong, but has very low density (1% of traditional earth materials). It is a manufactured block material meeting the engineered product specification standards of ASTM C 578 and CGSB 51.20. Standard densities range from 11 kg/m³ (.7 lb/ft³) to 32 kg/m³ (2 lb/ft³) which have typical compressive strengths of 35 kPa (5 psi) to 17 kPa (25 psi) (at 10% deformation) under short-term loading conditions. Other strength materials are available. Design values can be found on page 3.

Perform EPS Geofoam is unaffected by normally occuring weather at time of installation and will retain its physical properties under pre-engineered conditions of use. Perform EPS Geofoam is made under a Quality Assured manufacturing process monitored by a third party laboratory.

Size and Shape: Perform EPS Geofoam is produced in block form and is easily positioned at the work site [508 mm (20") to 762 mm (30") thicknesses, 1220 mm (4') widths, and 2440 mm (8') up to 4880 mm (16') lengths are standard]. Other sizes and fabrication can be provided by the manufacturer.

Environmentally Safe: Perform EPS Geofoam contains no CFC's, HCFC's, HFC's, or formaldehyde. It is inert, non-nutritive and highly stable, and therefore will not decompose, decay, or produce undesirable gases or leachates. Perform EPS is recyclable and safe for WTE Systems and landfills.

Limitations and Cautions: Perform EPS Geofoam stands up well to normal short-term weather conditions encountered during installation. Long-term (6 months or greater) exposure to UV will cause discoloration. Material should be covered as soon as practical. Perform EPS Geofoam is unaffected by freeze thaw cycling, moisture, or road salts. Protect Perform EPS Geofoam from exposure to hydrocarbons, highly solvent extended mastics and coal tar pitch. Perform EPS Geofoam contains a flame retardant additive; however, it should be considered combustible and should not be exposed to open flame or any source of ignition.

Applicable Standards: Perform EPS Geofoam can be manufactured to the following standards: ASTM C 578 (superseding FS HH-I-524 c), CAN CGSB 51.20, UL 723 (ASTM E 84), ULC S102.2.

4. TECHNICAL DATA

See Table 1 for typical physical properties of Perform EPS Geofoam:
- ASTM C 578
Types XI, I, VIII, II, IX
- CGSB 51.20*
Types 1, 2, 3

*CGSB 51.20 Types 1, 2, and 3 are comparable to ASTM I, II, and IX respectively on Table 1.

See Graph 1 for short-term loading stress strain curves for various densities of Perform EPS Geofoam. Values in Graph 1 are for AFM EPS tested in accordance with ASTM D 1621.

Technical Design Notes:
- Reference Density Section on Table 1. Perform EPS Geofoam should be designed with the following density modifications when

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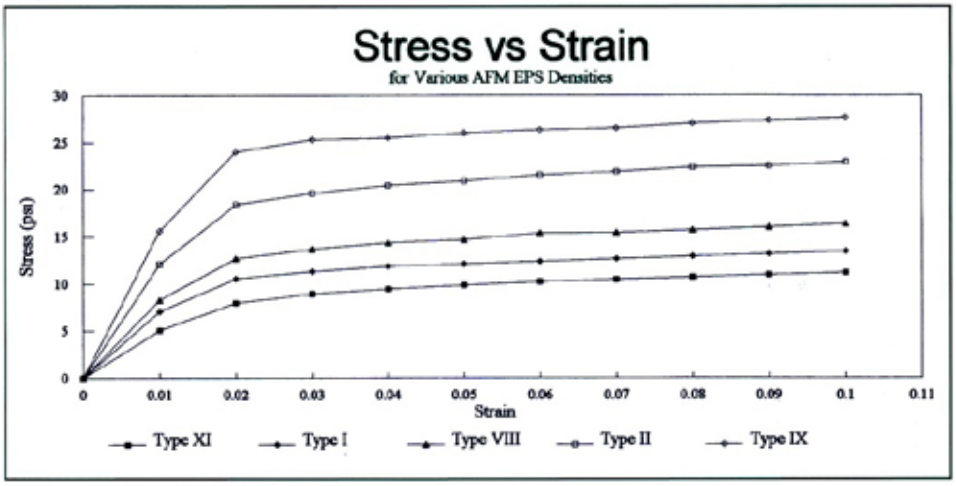
Table 1

Physical Properties of Perform EPS Geofoam ASTM C 578 Classification						
Property	**ASTM Test**	**Type XI**	**Type I**	**Type VIII**	**Type II**	**Type IX**
Density kg/m³ (lbs/ft³) Nominal Minimum	C 303/D 1622	12 (0.75) 11 (0.70)	16 (1.00) 15 (0.90)	20 (1.25) 18 (1.15)	24 (1.50) 22 (1.35)	32 (2.00) 29 (1.80)
Thermal Resistance 25.4 mm (1.00 in) thickness minimum k·m²/W (F·ft²·h/BTU) @ 4.4°C (40°F) @ 23.9°C (75°F)	C 177/C 518	 0.58 (3.3) 0.55 (3.1)	 0.70 (4.0) 0.63 (3.6)	 0.74 (4.2) 0.68 (3.8)	 0.77 (4.4) 0.70 (4.0)	 0.81 (4.6) 0.74 (4.2)
Compressive Resistance at yield or 10% deformation Min. kPa (psi)	C 165/D 1621	35 (5.0)	69 (10)	90 (13)	104 (15)	173 (25)
Flexural Strength Min. kPa (psi)	C 203	70 (10)	173 (25)	208 (30)	276 (40)	345 (50)
Water Absorption by total immersion Max. Vol. %	C 272	4.0	4.0	3.0	3.0	2.0
Dimensional Stability (change in directions) Max. %	D 2126	2.0	2.0	2.0	2.0	2.0
Buoyancy Force kg/m³ (lbs/ft³)	—	961 (60)	961 (60)	961 (60)	961 (60)	961 (60)
Modulus of Elasticity (Young's Modulus) kPa (psi)	D 1621	3103 (450)	4655 (675)	5862 (850)	7935 (1150)	10344 (1500)
Stress kPa (psi) @ .5% Strain @ 1% Strain	D 1621	17 (2.5) 35 (5.0)	24 (3.5) 48 (7.0)	29 (4.3) 58 (8.5)	41 (6) 82 (12)	55 (8) 110 (16)
Poisson's Ratio	—	.05	.05	.05	.05	.05

Graph 1



bulk water will be present in the insitu condition:

— In conditions where Perform EPS Geofoam is periodically subjected to submergence from fluctuating ground water, add 30 kg/m³ (1.87 lb/ft³).

— In conditions where Perform EPS Geofoam is continually below ground water, add 80 kg/m³ (5.00 lb/ft³).

These design recommendations are based on potential water absorption and the effects on density when analyzing cases involving downward loading. For analysis cases involving uplift loading, the nominal dry density given in Table 1 should be utilized. Perform EPS Geofoam physical properties are unaffected by water.

- Short-term load stress strain curves are provided in Graph 1. Long-term design loads should not exceed the linearly elastic range of Perform EPS Geofoam. Design load stresses should not exceed 1% strain of combined live and dead loads.
- In general earth work applications such as levees, dikes, berms, etc., uplift buoyancy forces must be considered. Perform EPS has a buoyancy of 961 kg/m³ (60 lbs/ft³). The buoyancy force must be counteracted with overburden or restraint devices, such as geogrids or geomembranes, etc.

5. INSTALLATION

Perform EPS Geofoam is commonly used in the following applications. Other engineered applications may also be appropriate.

Transportation Earth Works
- Embankments
- Side-hill fill
- Approach fill (bridge abutments)
- General fill (roadways, parking, etc.)
- Median and sound barriers

Architectural
- Landscape
- Plaza decks
- Bermed structures

Structural
- Structural fill (foundations, etc.)
- Earth retaining structures

General Earth Works
- Flood Control Levees
- Dikes/Berms

For most applications utilizing solid subgrades the following guidelines apply. Additional guidelines for specific applications are shown on application sections, page 4.

A. Subgrade Preparation

1. Clear and grub site.
2. Excavate existing soil if required.
3. At design engineer's discretion, place geotextile over graded surface, i.e., soft soils, etc.
4. Dewater site as required.
5. Place a sand pad/leveling course over the prepared surface, 50 mm (2") thickness minimum. Level to ±10 mm over 3 meters (½" per 10') horizontal. Sand pad surface should be above ground water level at time of Perform EPS Geofoam placement.

B. Placement of Perform EPS Geofoam

1. At time of material delivery, verify Quality Assurance and identification marks on face of the product. Labels on material must comply with manufacturer's data shown on its hard copy Project Certificate Form. Use material of proper type only and as specified.

Field sampling and testing of the Perform EPS Geofoam will be as specified by the Engineer. Properties of density, compressive strength, and dimensional tolerances shall be verified in accordance with Table 1 of this document.

2. Place material as required by the engineer and as shown on the drawings.
3. Blocks of Perform EPS Geofoam should be placed tightly on the prepared sand pad/leveling course (sand must not be frozen). If multiple layers of Perform EPS Geofoam are required, orient successive layers with long axis of blocks at 90° to previous layer. Offset block joints between layers.
4. In order to facilitate construction during precipitation or when frost or icing is encountered, horizontal restraint between layers of Perform EPS Geofoam may be desired. Use of AFM Gripper Plates placed between horizontal layers of blocks should occur. Consult manufacturer for plate specifications.
5. Perform EPS Geofoam should be ballasted in windy conditions both in storage and as placed. Activities involving high heat or open flame should not occur near the material. Heavy equipment should not operate directly on the material surface.
6. Commence with the placement of overlying materials as quickly as practical.
7. In pavement design for cold regions where differential icing may occur, provide an adequate thickness of a well graded (must contain a high degree of fines) subbase mix which will retain moisture. Most designs are adequate with subbase thicknesses of 500 mm to 800 mm (20" to 32") placed over the Perform EPS Geofoam.

6. AVAILABILITY AND COST

Availability: Perform EPS Geofoam is available throughout the U.S., Canada and Puerto Rico from AFM Partner Plants.

Insect Protection: If insect protected material is required, Perform Guard EPS Geofoam may be specified. Perform Guard EPS has been tested against termites and carpenter ants. Manufacturer has specific species on file.

Cost: Prebid budget counseling and job pricing are done through the local AFM representative. Contact may also be made to the AFM Office by calling (612) 474-0809 or (800) 255-0176.

7. WARRANTY

AFM can offer a warranty covering the long-term properties of Perform EPS Geofoam and Perform Guard EPS Geofoam. Contact AFM for sample warranty forms.

AFM provides certified material under its Third Party Certification and Quality Assurance Program. The program meets recognized national and international standards for EPS.

8. MAINTENANCE

Under normal condition of use, Perform EPS Geofoam and Perform Guard EPS Geofoam require no maintenance for the life of the fill system.

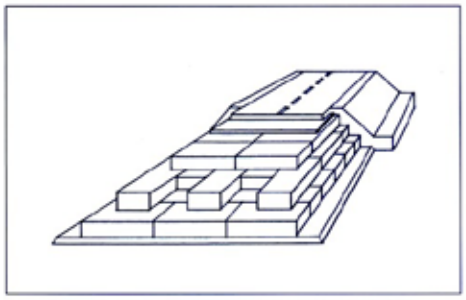
9. TECHNICAL SERVICES

AFM Regional Partner Plants have personalized representation throughout the country. The local AFM manufacturing plant is able to assist in appropriate aspects of the design and construction phases. Contact can also be made to the AFM Corporate Office for technical support: (612) 474-0809 or (800) 255-0176.

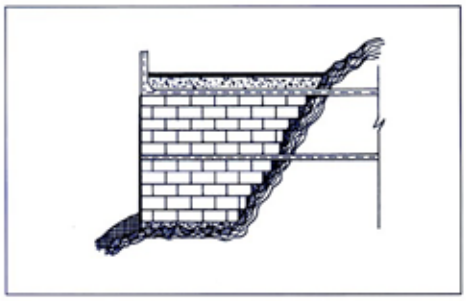
10. FILING SYSTEMS

- SPEC-DATA® II
- Sweet's General Building Directory

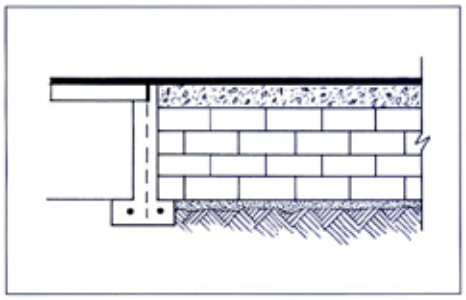
TYPICAL APPLICATION SECTIONS



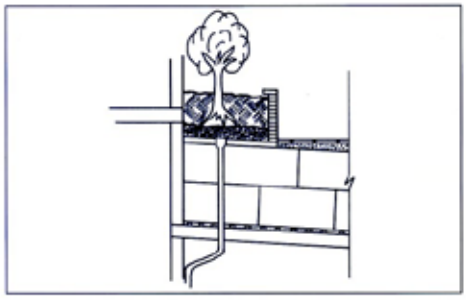
Road construction with sloping embankments.



Embankment with tie backs.

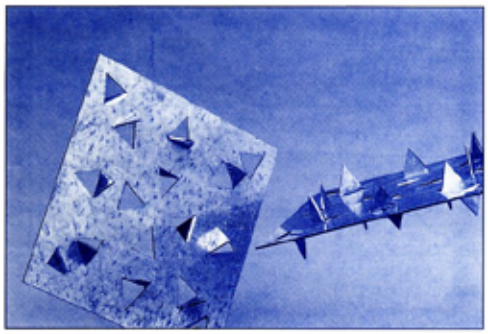


Approach embankments to bridge abutments.



Landscape voids.

NOTE: *Schematics for illustration purposes only. Design professional must provide design for specific applications.*



AFM® **Gripper**™ Plate

Made of galvanized steel, the AFM Gripper Plate is used in field applications between layers of Perform EPS Geofoam to provide additional horizontal restraint.

2-94-3288

Appendix I

Draft Contract Special Provision for Price Adjustment for EPS-Block Geofoam

S T A T E OF XXXX

SPECIAL PROVISION
REGARDING

PRICE ADJUSTMENT FOR EXPANDED POLYSTYRENE (EPS)-BLOCK GEOFOAM

This Special Provision covers the method of price adjustment for expanded polystyrene (EPS) block geofoam.

The normal bid items in the contract covering the EPS-block geofoam material shall remain the same, but the contract unit bid prices for these items will be adjusted to compensate for increases and decreases in the contractor's EPS-block geofoam material cost in the following manner:

A "Basic EPS-Block Geofoam Material Index" will be established by the xxxxxx Department of Transportation prior to the time the bids are opened. This "Basic EPS-Block Geofoam Material Index" is the average of the current quotations on polystyrene solid resin beads or expanded polystyrene beads (pre-puff) from suppliers furnishing polystyrene solid resin beads or expanded polystyrene beads (pre-puff) to EPS-block geofoam molders in the State of xxxxxx. These quotations are the cost per cubic yard f.o.b. supplier's terminal.

The "Basic EPS-Block Geofoam Material Index" for this project is $xxxxxxx per cubic yard.

The "Monthly EPS-Block Geofoam Material Index" is also established on the first day of each month by the same method. The "Monthly EPS-Block Geofoam Adjustment Factor" is the difference (+/-) between the "Basic EPS-Block Geofoam Material Index" and the "Monthly EPS-Block Geofoam Index."

The "Monthly EPS-Block Geofoam Adjustment Factor" shall be applied to the contract unit price bid provided the increase or decrease differs 5 percent or more from the Basic EPS-Block Geofoam Material Index." The Engineer reserves the right to alter the quantities of material or modify the design if the change in prices warrants material or design substitution. If adjustments are made in quantities or design, the contractor shall accept the unit price bid or the applicable monthly adjusted unit prices as full compensation for all work performed according to the provisions of Subsection xxxx of the Standard Specifications.

The unit price for EPS-block geofoam material used after the expiration of the allocated working time as set forth in the contract, or as extended by Supplemental Agreement, will revert to the original contract unit bid price or the adjusted unit price as set forth herein, whichever is less.

The adjustment will be calculated in accordance with the following formula only when the "Monthly EPS-Block Geofoam Adjustment Factor" is 5 percent or greater than the "Basic EPS-Block Geofoam Material Index."

$$\text{If } \frac{Fm}{Ib} \times 100 \geq \pm 5\% \text{ then}$$

$$PA = [Im - Ib] \times CY$$

where

Fm=Monthly EPS-Block Geofoam Adjustment Factor=Im - Ib

PA = Price Adjustment for Adjustment Month

Ib = Basic EPS-Block Geofoam Material Index

Im = Monthly EPS-Block Geofoam Material Index

CY = Cubic yards EPS-Block Geofoam for Adjustment Month

Price adjustment will be applied to all EPS-block geofoam material used for lightweight fill on this project.

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Appendix J

Project Phase I Work Plan

PHASE I WORK PLAN

to the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM (NCHRP)

on Project 24-11 (02)

Guidelines for Geofoam Applications in Slope Stability Projects

LIMITED USE DOCUMENT

This Work Plan is furnished only
for review by members of the NCHRP project
panel and is regarded as fully privileged.
Dissemination of information included herein
must be approved by the NCHRP.

November 1, 2006

from

David Arellano, Ph.D., P.E.
Timothy D. Stark, Ph.D., P.E.
Dov Leshchinsky, Ph.D.
John S. Horvath, Ph.D., P.E.

WORK PLAN

PROJECT OBJECTIVE

The objective of this research is to develop a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design guidance to engineers for the use of EPS-block geofoam for the function of lightweight fill in slope stability applications. This document will include a design guideline as well as an appropriate material and construction standard.

The document will facilitate the use of geofoam in highway projects by providing engineers with the five primary research products required to ensure successful technology transfer, i.e., summary of relevant engineering properties, a comprehensive design guideline, a material and construction standard, economic data, and a detailed numerical example.

OVERVIEW OF RESEARCH APPROACH

The work plan proposed herein is based on the detailed research plan the proposed research team prepared and executed during NCHRP Project 24-11. Dr. Horvath's knowledge of the lessons learned from applying the NCHRP Project 24-11 results into practice during the Boston Central Artery/ Tunnel project as well as a preliminary reassessment of the change in the state of knowledge since NCHRP Project 24-11 was completed has also been considered in developing the work tasks.

The work plan included herein is similar to the research plan included in the project proposal. However, the project panel general comments to the proposal and the relevant responses to these comments dated April 21, 2006 were incorporated in the work plan.

To accomplish the research objective and to facilitate implementation of the research results into practice, the research will consist of two phases and ten tasks. The details of the research are described subsequently.

PHASE I

The objective of Phase I is to review, document, and synthesize the worldwide experience of using EPS-block geofoam as lightweight fill in new and existing slope stability applications and to develop an interim design guideline and a material, product, and construction standard. The first phase is to consist primarily of a literature review and a geofoam usage survey which is to be conducted via a questionnaire to obtain case history information, cost data, design details, and other geofoam related information. The Phase I report will serve as an interim design document pending completion of the final report. Phase I will consist of the following six tasks.

Task 1. Perform Literature Search.

Objective: Review and evaluate the results of case histories and research projects on the use of geofoam in new and existing slope stability projects.

Approach: This task covers the collection, review, and evaluation of published case histories and research projects related to using EPS-block geofoam in slope stability applications.

Both the Syracuse University report (Negussey, 2002) and Dr. Horvath's monograph, *Geofoam Geosynthetic*, will be used as initial resources for this research phase. Additional material and references pertaining to slope stability applications that Dr. Horvath has collected during preparation of the monograph as well as his Manhattan College research report (Horvath, 2001) that is a bibliography of all geofoam publications since publication of his monograph in 1995 will also be included in the assessment. Additional resources that will be assessed include the TRB manual on landslides (1996) and recently published textbooks on slope stability and stabilization methods (Abramson, et al., 2002, Duncan and Wright, 2005).

This task will also include a supplemental review of design guidelines and manuals already published and in use in various countries to determine if recommendations for the use of geofoam in slope stability applications are included. These countries include France (Laboratoire Central Ponts et Chaussées/SETRA, 1990), Germany (Arbeitsgruppe Erd- und Grundbau, 1995, BASF AG, 1995), Japan (Public Works Research Institute, 1992), Norway (Norwegian Road Research Laboratory, 1980, Norwegian Road Research Laboratory, 1992a, Norwegian Road Research Laboratory, 1992b, Norwegian Road Research Laboratory, 1992c), the United Kingdom (Sanders and Seedhouse, 1994), and The Netherlands (Duskov and Houben, 2000). In particular, the Japanese literature will be revisited because most of their EPS-block geofoam applications have been in slope stability applications.

The design procedures that designers have used in the past for other types of lightweight fills to reduce the driving forces to stabilize slopes may be useful in developing the design guideline for geofoam. Therefore, the literature search will be extended to include the evaluation of published case histories involving stabilization of slopes using lightweight fills. The international document from the Permanent International Association of Road Congresses (PIARC) (Permanent International Association of Road Congresses, 1997), which describes the use of various lightweight fill materials for different applications in road construction, will be evaluated.

Several geofoam slope stability case histories such as New York State Route 23A and Wisconsin Bayfield County Trunk Highway A are summarized in Chapter 11 of the NCHRP Project 24-11 report (Stark, et al., 2004) because these case histories include useful cost or construction data that were applicable to traditional stand-alone embankment geometries. The slope stabilization design aspects of these case histories will be re-evaluated during this literature search. The supplemental design aspects of New York State Route 23A included in the Syracuse University study (Negussey, 2002) will be assessed.

As part of the literature search task, another geofoam usage survey will be conducted to obtain case history information, cost data, design details, and other geofoam information related to slope stability. Comments about the extent of both static and seismic slope stability analyses that DOTs perform will be solicited from State DOTs as part of the geofoam usage survey. The survey will not be limited to only geofoam but will also inquire about the use of other types of lightweight fills in slope stability projects because these case histories can be used to develop or confirm the new design procedure that is to be developed during Phase II of this project. This survey will also be used to obtain

feedback on the NCHRP Project 24-11 recommended material and construction standard because it will be the basis for the one that will be developed for slope stabilization in Task 4.

Based on the experience from NCHRP Project 24-11, information needed to develop design guidelines and standards and to perform an economic analysis is not readily available in the literature. Therefore, contacts obtained during the previous NCHRP Project 24-11 and participants that replied to the previous project survey that indicated had utilized or was planning to utilize geofoam in slope stabilization will be contacted. These contacts will include those obtained from unsolicited inquiries made to the research team about obtaining copies of the NCHRP Project 24-11 report. Additionally, DOTs, companies, manufacturers, designers, and individuals that are referenced in relevant geofoam technical literature that have been involved in slope stabilization repairs that have incorporated lightweight fills will also be contacted. Dr. Stark's experience with the use of geofoam to stabilize a single-family residence impacted by a landslide near Seattle, Washington will also be useful (Stark and Mann 2006).

The literature search performed as part of NCHRP Project 24-11 was based primarily on literature reviewed prior to April 2000. Therefore, the literature search will also include a review of publications published after April 2000 and that focus on the engineering properties of block-molded EPS. New engineering property findings will be used in evaluating the applicability of the NCHRP Project 24-11 recommended standard to slope stability applications (Task 4) as well as in the development of the new design guideline (Phase II). Based on the new information obtained, the NCHRP Project 24-11 EPS-block geofoam standard will also be updated.

Task 2. Summarize Design Methods.

Objective: Evaluate and summarize methods of design and analysis of new and existing slope stabilization projects using geofoam. Consider methods that account for both external and internal design factors as well as static and dynamic forces. The evaluation will address key material and engineering properties used for slope stabilization design and the analysis of slope performance under dynamic conditions including traffic and seismic loads.

Approach: It is envisioned that the design procedure for slope stability applications will require the interaction between the three primary components of a geofoam slope, i.e., foundation soil, which may include the displaced soil mass of the slope; fill mass; and pavement system. Therefore, it is anticipated that the design process can be divided into the same three phases used in the design of traditional stand-alone roadway embankments used in NCHRP Project 24-11, i.e., design for external (global) stability of the overall embankment, design for internal stability within the embankment, and design of an appropriate pavement system for the subgrade provided by the underlying EPS blocks.

Based on the NCHRP Project 24-11 experience, the procedure to develop a comprehensive design procedure for the use of geofoam in slope stabilization will consist of nine steps. Step 1 will consist of identifying the potential failure mechanisms that need to be considered during design of an EPS-block geofoam slope. Identification of these failure mechanisms will require consideration of the interaction between the three primary components of an EPS-block

geofoam slope. Step 2 will consist of identifying the engineering properties that influence the failure mechanisms and are, therefore, relevant for design. Step 3 will consist of categorizing the failure mechanisms identified in Step 1 into an external stability, internal stability, or pavement system design issue. Step 4 will consist of identifying loads that need to be considered in the analysis of each failure mechanism. Step 5 will consist of evaluating existing design methods typically used to analyze each failure mechanism, if any. Step 6 will consist of developing a design method for analyzing those failure mechanisms for which existing design methods are not available or suitable. Step 7 will consist of determining tolerable design criteria for each failure mechanism such as minimum factor of safety and maximum settlement. If the design procedure indicates that a particular failure mechanism does not meet the tolerable design criteria, the designer can modify the engineering properties of one of the embankment components, modify the embankment geometry, or select an alternative slope stabilization procedure. Therefore, Step 8 will consist of recommending remedial treatment procedures for each failure mechanism that a designer can utilize if the design does not meet tolerable criteria. Step 9 will consist of developing a design algorithm that optimizes both technical performance and cost. After developing the design procedure, design examples will be established to facilitate EPS-block geofoam slope design (Step 10). Step 10 will also focus on establishing recommended EPS-block geofoam properties that can be utilized in typical commercial slope stability analysis software programs.

Task 2 will consist of a synthesis of information reviewed under Task 1 and completion of Steps 1 through 5 of the design development procedure. Steps 6 through 10 of the design development procedure will be considered during Phase II.

Although the design considerations of the Syracuse University report (Negussey, 2002) will be reassessed, the design philosophy that is suggested in the report is different than the one recommended and included in the NCHRP Project 24-11 design guideline. Specifically, the design implied in the Syracuse University report is based on compressive strength while the recommended design for highway applications incorporated in NCHRP Project 24-11 is based on elastic limit stress to keep long-term compressive strains within acceptable levels and to limit both creep and plastic deformations. The benefits of utilizing the elastic limit stress in design is summarized on Pages 6-30 and 6-31 of the NCHRP Project 24-11 report (Stark, et al., 2004). The recommended material and construction standard incorporated in NCHRP Project 24-11 is also based on the elastic limit stress. To minimize confusion in highway practice, it is anticipated that the material and construction standard for slope stability applications will also be based on the elastic limit stress instead of ultimate compressive strength.

The primary difference between geofoam design involving slopes, i.e., side-hill fills, from traditional stand-alone embankments is the possibility that the geofoam within a side-hill fill system may need to resist unbalanced earth loads under gravity conditions from the earth materials adjacent to the geofoam fill as shown in Figure 1. It is typically not feasible for the geofoam fill to resist the applied earth force because the mass of the geofoam fill is usually very small. Techniques that have been used in practice to design geofoam fills to resist unbalanced earth loads within

slopes include the use of a conventional retaining wall and the use of anchor systems either only through the earth material adjacent to the geofoam or through both the geofoam fill and the earth material. Thus, anchoring and retaining systems utilized in practice to support unbalanced earth loads in geofoam side-hill embankments will be evaluated and summarized. Dr. Leshchinsky's experience with reinforced slopes will be invaluable during this evaluation. Relevant FHWA documents will be reviewed to relate appropriate reinforced soil slope methods in these documents for use in slope stability applications involving geofoam.

Another significant difference between stand-alone embankments and side-hill fill systems is the need to consider the potential for ground water seepage through the soil adjacent to the geofoam fill. Methods that have been used in practice to address this issue will be evaluated during this task.

The results of seismic stability analyses performed on model stand-alone embankments using pseudo-static slope-stability analysis during NCHRP Project 24-11 revealed that the use of EPS block may be a benefit for external seismic stability design because of the low unit weight and thus small inertial force generated by the seismicity. However, the NCHRP Project 24-11 study identified several areas related to seismic stability where further research would enhance the current state of knowledge of geofoam.

One issue related to geofoam material properties that requires further study is quantifying interface friction angles for geofoam/soil interfaces at large displacements and displacement reversals for use in seismic internal stability analyses. If the literature search that will be performed in Task 1 does not provide typical interface strength values at large displacements that can be used in preliminary seismic analysis, interface shear tests will be conducted on geofoam/soil interfaces during Task 7 of Phase II.

Several seismic stability issues requiring further research include developing a better understanding of the seismic behavior of EPS-block geofoam fills, particularly their interaction with the adjacent soil or rock material. Seismic design utilized in the various case histories obtained in Task 1 will be evaluated to determine what the state-of-practice is for addressing this interaction between geofoam and the adjacent earth materials during seismic stability analysis.

Additionally, investigation of the seismic behavior of relatively tall and slender EPS-block geofoam fills is needed to assess the rocking mode of behavior. This mode of behavior has been observed in recent full-scale shake-table tests performed in Japan and was considered in the design of the Boston Central Artery/Tunnel vertical embankments. Dr. Horvath's experience with the seismic analysis performed on the Boston artery vertical embankments will be useful in assessing the rocking mode of behavior in slope stability applications.

If the calculated resistance forces along the horizontal planes between EPS blocks is insufficient to resist the horizontal driving forces during internal stability analysis such as an unbalanced water head, wind, or seismic shaking, additional resistance between EPS blocks is generally provided by adding mechanical inter-block connectors (typically prefabricated barbed metal plates) along the horizontal interfaces between the EPS blocks. The cost analysis performed as part of NCHRP Project 24-11 revealed that the connectors added a significant cost to the project. A rational methodology for determining when mechanical connectors are required between EPS blocks is included

in the NCHRP Project 24-11 design guideline. However, further research is required to develop a methodology for selecting the number and placement location of mechanical connectors to ensure seismic stability. The current literature will be reviewed to determine what the current state-of-practice is for selecting the number and placement location of mechanical connectors.

Because the connector plates are typically propriety, the cost of installing the plates may be significant. To overcome the proprietary nature of the common metal connector plates, new mechanical connectors such as barbed timber fasteners, special barbed geofoam connector plates, and sections of steel reinforcing bars are being developed. The literature will be reviewed to determine if any new mechanical connector designs have been developed and utilized in practice.

Recent Japanese research indicates that the effectiveness of barbed-plate connectors is limited especially under seismic loading because it involves strain reversals and accumulated cyclic strains. The effectiveness of mechanical connectors, especially under reverse loading conditions is disputed in (Bartlett, et al., 2000, Sanders and Seedhouse, 1994). The current literature will be reviewed to assess the current state-of-practice of incorporating mechanical connectors to resist seismic loads.

Task 3. Summarize Geofoam Construction Practices.

Objective: Evaluate and summarize available information on construction practices and post-construction performance monitoring.

Approach: NCHRP Project 24-11 revealed that various aspects of both design and manufacturing of EPS-block geofoam for lightweight fill applications interact with and impact construction. Two construction issues that directly impact the design of an EPS-block geofoam embankment is placement of blocks and mechanical connectors. Both of these issues will be assessed in Task 2.

Three manufacturing issues that impact construction and constructability include the flammability of the EPS blocks, dimensional tolerances of the EPS blocks, and the broad aspect of manufacturing quality control (MQC) and manufacturing quality assurance (MQA). All of these manufacturing issues are addressed in the EPS-block geofoam standard developed during NCHRP Project 24-11. The standard will be reassessed for slope stability applications as well as based on lessons learned from project experience in Task 4.

Items covered by construction quality control and construction quality assurance (CQC/CQA) include all earthwork and related activities other than manufacturing and shipment of the EPS-block geofoam. Items of particular relevance include site preparation, block handling and construction site storage, block placement, and pavement construction. These CQC/CQA items will be reassessed during this task by evaluating case histories that have incorporated geofoam in slope stability applications. Dr. Horvath's experience with the Boston Central Artery/Tunnel project will be useful in assessing the lessons learned from the Boston project with respect these CQC/CQA items.

An objective of this project is to facilitate the use of EPS-block geofoam in slope stability applications. To accomplish this objective, in addition to the recommended design guideline and the material and construction standard that will be developed, construction drawing details

need to be presented so the design engineer can distribute the EPS-block geofoam design for bidding and construction. Therefore, to facilitate the use of geofoam in slope stability applications, typical construction drawings and details obtained during Task 1 from actual geofoam construction drawings from projects throughout the United States will be evaluated and organized during this task. These design details and drawings will be included in the project report so the designer can use these details as a guide for developing site-specific drawings or details and aid in the preparation of bid and construction documents.

An important aspect of constructing a geofoam embankment is preparation of the foundation soil prior to block placement to facilitate placement and alignment of the blocks. Methods utilized to prepare a suitable level working platform in slope stability projects to include the use of any geosynthetics will be summarized from the case histories.

As noted in the NCHRP Project 24-11 report, vertical-faced geofoam embankments, which typically require the use of a facing system, are extremely cost-effective. Design details for facing systems are included in the NCHRP Project 24-11 report (Stark, et al., 2004). Side-hill fills tend to require a vertical-sided embankment more often than with stand-alone embankment applications. Therefore, any supplemental facing system details that are obtained during the literature search of Task 1 will be included in this report. For example, permission will be sought to include the design details for the facing systems used in the Boston Central Artery/ Tunnel project vertical embankments. In particular, the new developments during the Boston project of incorporating lightweight Exterior Insulation Finish System (EIFS) technology for the facing system will be considered.

Post-construction activities that have been utilized from the case histories that will be obtained as part of Task 1 will be summarized. Additionally, post-construction activities, including monitoring, typically utilized in slope stability projects will be assessed. The TRB landslide manual (Transportation Research Board, 1996) will be used as a source.

Task 4. Review and Modify NCHRP Project 24-11 Recommended EPS-Block Geofoam Standard.

Objective: Evaluate the recommended EPS-block geofoam material, product, and construction standard developed in NCHRP Project 24-11 for stand-alone embankments and modify it for slope stability applications.

Approach: As part of NCHRP Project 24-11, this research team developed a recommended combined material, product, and construction standard covering block-molded EPS for use as lightweight fill in stand-alone road embankments and related bridge approach fills on soft ground. The recommended standard is intended to be used to create a project-specific specification. The specification that was used to construct the geofoam embankments as part of the Boston Central Artery/Tunnel project was based on the NCHRP Project 24-11 standard. Thus, it is the first major project to utilize the standard. Dr. Horvath was the primary author of the NCHRP Project 24-11 standard and was also involved in the development of the Boston project specification. Therefore, Dr. Horvath will be the primary investigator for Task 4.

This task will also include an evaluation of the lessons learned from case histories that have utilized the NCHRP Project 24-11

standard that are identified during the literature search and geofoam usage survey as part of Task 1. For example, the lessons learned from the ongoing Woodrow Wilson Bridge project, which also utilized the NCHRP Project 24-11 standard, will be evaluated. Thus, the lessons learned from the use of the NCHRP Project 24-11 standard based on the Boston Central Artery/Tunnel and Woodrow Wilson Bridge project experiences and any other case histories will be summarized and appropriate modifications will be made to the 24-11 standard. It should be noted that a comprehensive evaluation of the lessons learned from the implementation of the NCHRP 24-11 material and construction standard will require the availability of the manufacturing processes and procedures utilized during the project.

Based on preliminary feedback from the FHWA, the following four issues will be evaluated: (1) the availability of a statistical analysis procedure that relates the compressive strength results from 2 in. specimens to the overall compressive strength of a full-size block, (2) the availability of a testing protocol to obtain reliable elastic limit stress values from 2 in. specimens, (3) the availability of a density range that can be used in the acceptance of blocks as part of QA/QC, and (4) the availability of data that may support a reduction in the currently recommended 72 hour seasoning time.

Task 4 will also include a review of engineering property information from current literature that has been published since the NCHRP Project 24-11 literature search was completed such as from papers presented at the EPS Geofoam 2001 3rd International Conference held on 10-12 December 2001 in Salt Lake City, Utah; the International Workshop on Lightweight Geo-Materials held on 26-27 March 2002 in Tokyo, Japan; the North American Geosynthetics Society Past President's Seminar on Geofoam in Highway and Bridge Applications held on 15 May 2002; and the North American Geosynthetics Society Past President's Seminar on Lightening the Load held on 11 February 2003. New engineering property findings will be used in evaluating the applicability of the NCHRP Project 24-11 recommended standard to slope stability applications.

The recommended standard was approved by the NCHRP project technical review panel in June 2000. However, since the recommended standard was drafted and approved by the NCHRP panel, a new ASTM standard for geofoam titled "Standard Specification for Rigid Cellular Polystyrene Geofoam" was developed and approved in June 2002 and a draft European standard titled "Factory made products of expanded polystyrene (EPS)- Specification" was developed. Dr. Arellano was a member of ASTM and participated in the review process by providing comments (Arellano, 2000). Thus, Dr. Arellano is familiar with the philosophy incorporated in the ASTM standard. Material and construction standards must complement the design methodology so the necessary engineering properties are present in the geofoam. The design methodology incorporated in the NCHRP Project 24-11 recommended design guideline for stand-alone embankments is based on maintaining the long-term compressive stresses below the elastic-limit stress (within the elastic range) to keep long-term compressive strains within acceptable levels and to limit both creep and plastic deformations. Therefore, the NCHRP Project 24-11 recommended standard is based on and emphasizes the elastic-limit stress. The ASTM and European standards will be assessed to determine their applicability to the design methodology that

will be developed for geofoam usage in slope stability applications.

In summary, this task will consist of modifying the existing standard to make it specific to geofoam usage in slope stability applications. The standard will also be modified based on a review of the lessons learned from the Boston Central Artery/Tunnel and Woodrow Wilson Bridge projects` and from other case histories that have utilized the recommended standard. New engineering property findings will also be used in evaluating the applicability of the NCHRP Project 24-11 recommended standard to slope stability applications. The applicability of the ASTM and European standards to the design methodology that will be developed for geofoam usage in slope stability applications will be evaluated.

Task 5. Perform an Economic Analysis.

Objective: Summarize cost data related to projects that have utilized or considered EPS-block geofoam in slope stability applications. Tabulate cost-benefit data for geofoam and alternate designs where available.

Approach:

The experience with NCHRP Project 24-11 showed that limited published cost data for geofoam embankments was available in the literature. Therefore,

Cost-benefit data will be tabulated from the cost data obtained for geofoam and alternative designs, where available, for each case history that will be evaluated. A summary of the manufacturing, design, and construction issues that impact cost will be prepared. The cost of retaining systems and anchoring systems that are sometimes utilized to support the adjacent earth will be searched.

The cost savings that the use of geofoam can generate by minimizing the amount of soil that may require removal to stabilize a slope will also be considered. Any supplemental cost data available for facing systems that are used with vertical embankments will be included in the summary. A summary of the advantages of using geofoam as a design alternative compared to other slope stabilization techniques will also be prepared.

DOTs are particularly interested in the benefit of accelerated construction that EPS-block geofoam can provide when constructing embankments. In June 2002, the FHWA in a joint effort with AASHTO organized a geotechnical engineering scanning tour of Europe (American Association of State Highway and Transportation Officials and the United States Department of Transportation Federal Highway Administration, 2002). The purpose of the European scanning tour was to identify and evaluate innovative European technology for accelerated construction and rehabilitation of bridge and embankment foundations. Lightweight fills was one of the technologies that was evaluated. One of the preliminary findings of the scanning project is that lightweight fills such as geofoam is an attractive alternative because construction can be accelerated. The benefits of accelerated construction that EPS-block geofoam can provide when constructing embankments using geofoam in slope stabilization projects will be summarized. Therefore, the aspects of how the use of geofoam can accelerate construction will be evaluated and summarized.

One cost issue that was identified during NCHRP Project 24-11 is that the cost of EPS-block geofoam is highly dependent on the cost of oil, which may fluctuate during the duration of a long-term project. Since the cost of asphalt is also dependent on the cost

of oil, the current trend of how state DOTs address the fluctuation of the cost of asphalt on long-term projects will be investigated.

The NCHRP Project 24-11 study revealed that the benefits of stand-alone EPS-block geofoam embankments include (1) ease and speed of construction, i.e., accelerated construction, (2) placement in adverse weather conditions, (3) possible elimination of the need for preloading, surcharging, and staged construction, (4) decreased maintenance costs as a result of less settlement from the low density of EPS-block geofoam, (5) alleviation of the need to acquire additional right-of-way to construct flatter slopes because of the low density of EPS-block and/or the use of a vertical embankment because of the block shape of EPS, (6) reduction of lateral stress on bridge approach abutments, (7) use over existing utilities which reduces or eliminates utility relocation, and (8) excellent durability. It is anticipated that some of these benefits will also be applicable to slope stability applications and will be evaluated during this task.

Task 6. Prepare an Interim Geofoam Report.

Objective: Submit an interim report summarizing the results and conclusions of Tasks 1 through 5. In the report, present a provisional method for design and analysis of slopes incorporating geofoam as a lightweight fill and a draft standard in AASHTO format for their construction. Include a detailed work plan for Phase II.

Approach: This task will consist two major components:

1. Preparation of an interim report. This report will not only document the results of Tasks 1 through 5, inclusive, but will contain the synthesized results in a format suitable for interim implementation into practice. Based on the experience with NCHRP Project 24-11, it is anticipated that the interim report will consist of an introduction and research approach (Chapter 1), a summary of findings of the literature search to include a summary of case histories, and the results of the geofoam usage survey (Chapter 2), a summary of design methods and a preliminary recommended design procedure (Chapter 3), a summary of geofoam construction practices (Chapter 4), a summary of the results of the evaluation of the recommended NCHRP 24-11 standard to include a summary of updated engineering properties and recommended modifications (Chapter 5), a summary of cost data (Chapter 6), conclusions and suggested research approach for Phase II (Chapter 7). The appendices will contain the bibliography of references reviewed during Phase I, the geofoam usage survey with replies, provisional recommended design guideline, provisional recommended standard, and the Phase II work plan.

The interim report will be primarily written by Drs. Arellano and Horvath with peer review by Drs. Stark and Leshchinsky.

A plan for implementing the results of the Phase I research, such as making the interim report accessible on the NCHRP web-site after the NCHRP panel has reviewed it and the research team has responded to any comments that the panel may have, will be developed.

2. Preparation of a detailed work plan for Phase II of the project. An updated Phase II work plan will be developed based on the Phase I findings. A plan to integrate the design guideline and the material and construction standard for the use of EPS-block geofoam in slope stability applications with the NCHRP Project 24-11 guideline and standard for stand-alone embankments will be prepared. Dr. Arellano, and possibly Drs. Stark, Horvath and Leshchinsky, will meet with the NCHRP panel approximately 1 month after submission of the Phase I report to obtain approval of the work plan for the Phase II research.

PHASE II

The objective of Phase II is to develop a comprehensive design methodology that optimizes both technical performance and cost for geofoam as lightweight fill in new and existing slope stability applications for highway projects. The exact scope of the Phase II research will be determined at the end of the Phase I research in cooperation with the NCHRP technical review panel. However, based on the NCHRP Project 24-11 experience, it is anticipated that Phase II will consist of the following four tasks.

Task 7. Perform Applicable Geofoam Analyses.

Objective: Perform detailed analyses to establish the range of applicability and sensitivity of the provisional design and analysis methods developed in Phase I. Validate the results of the analyses with field observations and with available results from physical modeling, as appropriate.

Approach: This task will consist of Steps 6 through 8 of the design development procedure outlined in Task 2. Step 6 will consist of developing a design method for analyzing the failure mechanisms for which existing design methods are not available or suitable. Step 7 will consist of determining tolerable design criteria for each failure mechanism such as minimum factor of safety. Step 8 will consist of recommending remedial treatment procedures for each failure mechanism that a designer can utilize if the design does not meet tolerable criteria.

It is anticipated that the analyses and sensitivity studies will be conducted using limit equilibrium procedures. The slope stability computer program ReSSA from ADAMA Engineering, Inc. will be evaluated for use during this task. Dr. Leshchinsky is the developer of ReSSA, an interactive program for assessing the stability of mechanically reinforced earth slopes. Therefore, Dr. Leshchinsky can make modifications to the software program to perform any required applicable analyses that will be required during this task. The suitability of using the finite difference computer program FLAC (Itasca Consulting Group Inc., 2000) and FLAC/Slope will also be considered.

The implementation of the various AASHTO group loading combinations included in the current AASHTO Standard Specifications for Highway Bridges will be evaluated. This evaluation will include the applicability of the recommended factors of safety for various AASHTO group loading combinations in geofoam slopes. The Manual for Design and Construction of Soil Nail Walls (Byrne, et al., 1998) addresses

the issue of factor safety for various AASHTO group loadings. These recommended factors of safety will be evaluated to determine the applicability of these factors for geofoam slopes. Additionally, the factors of safety and AASHTO group loadings used for the geofoam structures as part of the Central Artery/Tunnel (CA/T) project will be evaluated to determine the applicability of these factors for geofoam slopes.

The issue of slope design using allowable stress design (ASD) versus load resistance factor design (LRFD) will be addressed. State DOTs are currently planning to transition from the traditional ASD to the LRFD by the mandated date of October 1, 2007. The Manual for Design and Construction of Soil Nail Walls (Byrne, et al., 1998) incorporates both design methods. Additionally, the recently completed report sponsored by the Missouri DOT (Loehr, et al., 2005) addresses the use of LRFD in the design of earth slopes. Therefore, both ASD and LRFD will be addressed in this research.

An anticipated key difference between the design of EPS-block geofoam embankments over soft ground and the design of EPS-block geofoam embankments in slope stability applications is that the former is primarily a serviceability problem, i.e., settlement, while the later is likely to be a failure-based problem, i.e., shear strength. Therefore, the issue of strain compatibility between the EPS blocks and the foundation soil is more critical in slope stability applications. Therefore, the issue of strain compatibility between the EPS-block geofoam and the adjacent and underlying soil will be investigated. Dr. Stark's experience with strain compatibility between geosynthetics and soil will be invaluable during this portion of the research.

The foundation soil for the proposed embankment fill may be located above the failure surface or anticipated critical failure surface. Consequently, in addition to the weight or volume of soil to remove and replace with geofoam, the location of geofoam fill placement along the soil mass above the failure surface is important in the design of lightweight fill to stabilize slopes (Transportation Research Board, 1996). Therefore, the issue of the proper placement of lightweight fill materials and the required strength of the foundation soil to ensure slope stability will be evaluated.

Another variable to be considered is the overall geometry of the embankment. As noted in the NCHRP Project 24-11 report, vertical-faced geofoam embankments are extremely cost-effective. Thus, both slope-sided and vertical-faced embankment geometries will be considered.

The applicability of current wind design procedures for buildings and other structures to EPS-block geofoam embankments will be evaluated and, if necessary, the recommended wind load design procedure outlined in Project 24-11 will be reassessed and revised.

As noted in Task 2, one seismic stability issue requiring further research includes developing a better understanding of the seismic behavior of EPS-block geofoam fills, particularly their interaction with the adjacent earth. Seismic design utilized in the various case histories obtained in Task 1 will be evaluated during Task 2 to determine what the state-of-practice is for addressing this interaction between geofoam and the adjacent earth during seismic stability analysis. If the findings of Task 2 do not yield a suitable analysis procedure to evaluate the interaction between geofoam and the adjacent earth during seismic stability analysis, detailed numerical analyses using FLAC or FLAC/Slope will be performed to study this interaction. The applicability of current AASHTO seismic

standards will be evaluated. Specifically, the improved damping properties of a geofoam versus traditional soil fill will be evaluated. Dr. Stark's extensive experience in analyzing slopes under seismic loads and using FLAC will be valuable for this analysis. Dr. Leshchinsky's extensive experience in analyzing reinforced-earth slopes and walls will also be valuable for this analysis.

One issue related to geofoam material properties that requires further study is quantifying interface friction angles for geofoam/geofoam and geofoam/soil interfaces at large displacements and displacement reversals for use in seismic stability analyses. If the literature search that will be performed in Task 1 does not provide typical interface values at large displacements that can be used in preliminary design, interface shear tests will be conducted on geofoam/geofoam and geofoam/soil interfaces.

Geosynthetic interface testing was conducted during the NCHRP Project 24-11 study to evaluate the interface shear resistance between an EPS-block geofoam and a nonwoven geotextile interface and a geofoam and gasoline containment geomembrane interface. The purpose of these tests was to obtain interface friction values that can be used in preliminary design.

Interface tests involving geosynthetics are typically performed in accordance with ASTM Standard Test Method D 5321 (2001). ASTM D 5321 allows other shear devices to be used for geosynthetic shear testing if they yield similar results as the large-scale direct shear box. To investigate this substitution, both large-scale direct shear and torsional ring shear tests are typically conducted on the same geofoam interfaces.

The main difference between the ring shear and direct shear test methods is in the values obtained for the residual friction angle and shear displacement at the residual strength. The direct shear test terminates at a shear displacement of approximately 100 mm (3.9 in.) and, thus, the resulting friction angle does not correspond to a residual friction angle whereas the ring shear test is conducted until a constant minimum, i.e., residual, strength is reached. Therefore, the torsional ring shear apparatus is preferred to measure residual strength because it allows for unlimited continuous shear displacement to occur in one direction, resulting in the development of a true residual strength condition and a constant cross-sectional area during shear.

The tests performed as part of NCHRP Project 24-11 yielded ring shear device results similar to the large-scale direct shear apparatus for the interfaces tested and normal stresses tested. Therefore, the ring shear device can be substituted for the direct shear apparatus as suggested in ASTM D 5321. The successful use of a torsional ring shear apparatus to measure the shear strength of geosynthetic/geosynthetic and geosynthetic/soil interfaces is further described in Eid and Stark (1997), Stark and Poeppel (1994), and Stark et al. (1996).

In summary, if interface friction angle values at large displacements are not provided in the literature for geofoam/geofoam and geofoam/soil interfaces, interface tests will be tested using the large direct shear and torsional ring shear apparatus at the University of Illinois under the supervision of Dr. Stark.

Task 8. Develop a Design Algorithm.

Objective: Develop a design algorithm that optimizes both technical performance and cost.

Approach: The design of an EPS – block geofoam roadway embankment requires consideration of the interaction between the three major components of the embankment, i.e., foundation soil (slope material), fill mass, and pavement system. As shown in the NCHRP Project 24-11 study, because of this interaction, the design procedure must involve interconnected analyses between the three components. For example, some issues of pavement system design act opposite to some of the design issues involving internal and external stability of a geofoam embankment, i.e., the thickness of the pavement system will affect both external and internal stability of the embankment because the dead load imposed by the pavement system may decrease the factor of safety of some failure mechanisms, e.g., slope stability, while increasing it in others e.g., uplift due to water. Therefore, some compromise is required during design.

As demonstrated by the NCHRP Project 24-11 study, it is possible to optimize the final design of both the pavement system and the overall embankment considering both performance and cost so that a technically effective and cost efficient embankment is obtained. Because of the inherent interaction between the three embankment components, overall design optimization of a roadway embankment incorporating EPS-block geofoam requires an iterative analysis procedure to achieve a technically acceptable design at the lowest overall cost.

A comprehensive design algorithm for the design of new and existing slopes utilizing EPS-block geofoam as a lightweight fill that considers all potential failure mechanisms and the interaction between the three primary components of a geofoam slope is currently unavailable. Therefore, this task will include Step 9 of the design development procedure outlined in Task 2, which consists of developing a design algorithm of the entire pavement-geofoam-foundation soil (slope material) system to minimize the iterative analysis procedure required during design as well as minimize cost. Thus, the overall objective of the design algorithm is to develop a technically optimal and cost efficient slope design. The design algorithm will be summarized as a flow chart as in NCHRP Project 24-11.

Dr. Leshchinsky is particularly knowledgeable in the area of geotechnical software in general and geosynthetics software in particular. He has developed ReSSA (a program for assessing the stability of mechanically reinforced slopes), MSEW (a program for design and analysis of mechanically stabilized earth walls), and ReSlope (a program for analyzing geosynthetic-reinforced slopes). Therefore, he has the essential knowledge required to develop a design algorithm for geofoam in slope stability applications.

An anticipated difference between the design of EPS-block geofoam stand-alone embankments over soft ground and the design of EPS-block geofoam embankments in slope stability applications is that the former is likely to be a failure-based problem, i.e., shear strength. Therefore, the design algorithm for slope stability applications will most likely be different than the one developed for stand-alone embankments over soft ground during NCHRP Project 24-11.

Task 9. Develop Routine Geofoam Design Aids.

Objective: Establish recommended EPS-block geofoam properties that can be utilized in typical commercial slope stability analysis software programs and develop

design examples to facilitate the routine design of embankment projects with geofoam in slope stability applications.

Approach: This task consists of the last step, Step 10, of the design development procedure outlined in Task 2. It will consist of the implementation of Task 8 into a form that is easy and friendly for the practicing engineer to use.

Task 9 will focus on establishing recommended EPS-block geofoam properties that can be utilized in typical commercial slope stability analysis software programs to ensure strain compatibility with the adjacent and underlying soil. The use of commercial software is useful for slopes with complex geometries, numerous soil layers, and various external loading conditions.

Sensitivity analyses will be performed to evaluate the sensitivity of the recommended design procedure to various input design parameters. It is anticipated that sensitivity analyses can be performed for the case of circular slip surfaces within simple homogeneous slopes such as the charts presented in the landslide manual (Transportation Research Board, 1996) and in various slope stability textbooks (Abramson, et al., 2002, Duncan and Wright, 2005). It may also be possible to perform sensitivity analyses for non-circular slip surfaces for a range of engineering properties and geometries.

The results of the sensitivity analyses will be depicted in charts and figures, which can be used by designers to evaluate the sensitivity of the recommended design procedure to various input design parameters to develop an optimal lightweight fill design. Additionally, designers may also use these charts and figures to determine the feasibility of using lightweight fill as well as for design of simple slope geometries and homogeneous soil conditions.

Design examples will be developed that will illustrate the use of the recommended design procedure, i.e., design algorithm. In each example, detailed calculations will be shown with the appropriate equation and/or design chart number. Additionally, tables will be used to summarize design calculation input values and results. These tables can serve as the basis for developing computer design spreadsheets. The comprehensive design example of a Boston Central Artery/Tunnel geofoam vertical wall that is currently being developed by Parsons Brinckerhoff as part of an FHWA project may provide the basis for some of the examples to be prepared for this project.

The design algorithm, sensitivity charts and figures, and design examples will be incorporated in a design guideline that will summarize each step of the design process.

Task 10. Prepare Final Geofoam Report.

Objective: Prepare a final report that summarizes findings, draws conclusions, and documents the research products including a validated design and analysis method for projects using geofoam as a lightweight fill in new and existing slope stability applications; a recommended design guideline in AASHTO format; and a recommended material and construction standard in AASHTO format. Provide an implementation plan for state highway agencies to use in incorporating the research products into practice.

Approach: This task will contain two major components: (1) preparation of a final project report and (2) an implementation plan for state highway agencies. The final report and implementation plan will be

prepared by Dr. Arellano with input and review from Drs. Stark, Leshchinsky, and Horvath.

1. Preparation of the final project report.

The purpose of the final report is to provide those who have primary involvement with roadway embankment projects, including the following four groups: end users, manufacturers, contractors, and owners, with both state-of-art knowledge and state-of-practice design guidance for use of EPS-block geofoam in slope stability projects. The end users include engineers who perform the design, develop specifications, and perform field inspection; EPS block molders who manufacture the product; and construction contractors who install the product.

The key products of the research will be a recommended design guideline and a recommended material and construction standard. As with the NCHRP Project 24-11 reports, it is anticipated that the recommended design guideline and the standard will be published as a NCHRP Report and that the full report will be available as a NCHRP Web Document. However, as part of the final implementation plan, recommendations will be made on how to integrate the results of this project with the NCHRP Project 24-11 reports to make implementation of the research results in practice more effective.

It is anticipated that the full report will be organized similar to the NCHRP Project 24-11 report (Stark, et al., 2004) and will be divided into two parts. The first part will consist of eleven chapters. Chapter 1 will provide an overview and history of the use of EPS-block geofoam for slope stabilization projects and will include the research objective and approach used for the study. Chapter 2 will present a summary of updated engineering properties that are necessary to implement the proposed design methodology.

Chapter 3 will provide an overview of the design methodology and include the background for the "Recommended Design Guideline." Chapter 4 will present the details of the design procedure.

Chapter 5 will present design examples that will demonstrate the design methodology outlined in Chapter 3 and detailed in Chapter 4 that can be used by design engineers to facilitate design of their projects.

Chapters 6, 7, and 8 will discuss geofoam construction practices, MQC/MQA, and design details, respectively. These chapters will provide the background for understanding the recommended EPS-block geofoam standard. Chapter 9 will provide a summary of case histories that have successfully incorporated EPS-block geofoam into slope stability applications. Chapter 10 will provide cost information to allow a cost estimate to be prepared during the design phase so that an optimal geofoam design can be selected. The designer can then use this optimal geofoam design to perform a cost comparison with other slope stabilization techniques. Finally, Chapter 11 will present recommended areas of future research for EPS-block geofoam for roadway embankments.

The second part of the report will be composed of six appendices. Appendix A will describe the geofoam usage survey that will be developed and distributed during this study and will also present the responses to the survey. Appendix B will present the recommended design guideline for EPS-block geofoam in slope stability applications that will be outlined in Chapter 3. Appendix C will present the recommended standard for the use of EPS-block geofoam in slope stability applications, which should facilitate DOTs in specifying, and thus contracting for

the use of geofoam in slope stability projects. Appendix D will include an extensive bibliography of all references encountered during this study that relate to EPS-block geofoam. Finally, Appendix E will present a glossary of the terms used in the report and Appendix F will provide conversion factors that can be used to convert between Système International d'Unités (SI) and inch-pound (I-P) units.

It is anticipated that the design guideline and the material and construction standard will be published separately to facilitate use as was done for NCHRP Project 24-11.

2. An implementation plan for state transportation agencies.

An implementation plan will be developed for state highway agencies to use in incorporating the research products into practice.

Two key issues about the final report that will be addressed prior to publication and dissemination of the final report is the feasibility of integrating the research results for slope applications with the research results of stand-alone embankments, NCHRP Project 24-11 results. Also, the feasibility of creating a web-based link between the recommended design guideline and material and construction standard for geofoam applications in slope stability applications with the appropriate key sections of the full report will be considered.

PROJECT TIMELINE

The proposed project timeline, planned expenditures, and planned progress schedule are shown in Figures 2, 3, and 4, respectively. The project timeline and planned expenditures are in accordance with the timeline and itemized budget included in the proposal.

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Appendix K

Project Phase II Work Plan

REVISED PHASE II WORK PLAN

to the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM (NCHRP)

on Project 24-11 (02)

Guidelines for Geofoam Applications in Slope Stability Projects

Revisions based on the results of the Interim Report Panel Meeting of March 30, 2009.

LIMITED USE DOCUMENT

This Work Plan is furnished only for review by members of the NCHRP project panel and is regarded as fully privileged. Dissemination of information included herein must be approved by the NCHRP.

July 7, 2009

from

David Arellano, Ph.D., P.E.
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WORK PLAN PHASE II

PROJECT OBJECTIVE

The objective of this research is to develop a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design guidance to engineers for the use of EPS-block geofoam for the function of lightweight fill in slope stability applications. The completed research document will consist of the following five primary research products: (1) summary of relevant engineering properties, (2) a comprehensive design guideline, (3) a material and construction standard, (4) economic data, and (5) a detailed numerical example. To develop these key research products and, thereby accomplish the research objective, the research consists of two phases.

The objective of the Phase I research was to review, document, and synthesize the worldwide experience of using EPS-block geofoam as lightweight fill in new and existing slope stability applications and develop an interim recommended design guideline and material and construction standard. Phase I consisted of the following six tasks: (1) perform literature search, (2) summarize design methods, (3) summarize geofoam construction practices, (4) review and modify the NCHRP Project 24-1(01) recommended EPS-block geofoam material and construction standard, (5) perform an economic analysis of geofoam versus other lightweight fill material for slope stabilization purposes, and (6) prepare an interim report that summarizes the results of Phase I. As the phase I work progressed, it was determined that it would be better to include a preliminary design algorithm as part of the interim design guideline instead of developing the algorithm during Phase II as was initially planned. Therefore, Task 8 (develop a design algorithm) was completed during Phase I. The results of the Phase I work are included in the interim report dated January 14, 2009.

The work plan included herein includes the recommended tasks for the Phase II research. This revised work plan includes the revisions that were discussed during the March 30, 2009 interim report panel meeting. The key changes made to the work plan included in the draft interim report dated January 14, 2009 includes the removal of the task to develop a method that considers the strength of the EPS blocks for use with limit equilibrium methods of slope stability analysis (previous Task 7) and the task involving clarification and improvement of the moment reduction method (previous Task 9) and the addition of the task to further evaluate pavement design considerations (current Task 7) and the task to provide updated information that focuses on performance related issues that can contribute to the improvement of the recommended standard (current Task 9).

OVERVIEW OF PHASE II RESEARCH APPROACH

Although an interim design guideline, which is included in Appendix B of the interim report, was developed during Phase I, it is based on an assessment of existing technology and literature that involved primarily stand-alone embankments over soft ground. The Phase I research revealed important analysis and design differences between the use of EPS-block geofoam for the lightweight fill function in slope applications versus stand-alone applications over soft ground. The primary differences between slope applications versus stand-alone embankments over soft ground are summarized below:

- Site characterization is usually much more complex and difficult because it typically involves explorations made on an existing slope and concomitant access difficulties. The slope cross-section often consists of multiple soil and rock layers that vary in geometry both parallel and perpendicular to the road alignment and piezometric conditions may be very complex and even seasonal in variation.
- The governing design issue is usually based on an ultimate limit state (ULS) failure involving the analysis of shear surfaces using material strength and limit-equilibrium techniques.

Serviceability limit state (SLS) considerations involving material compressibility and global settlement of the fill are rarely a concern.

- There is always an unbalanced earth load, often relatively significant in magnitude, acting on the EPS mass that must be addressed as part of the design process.
- Piezometric conditions are often a significant factor to be addressed in design. In fact, if the use of EPS geofoam is being considered to reconstruct a failed or failing area, piezometric issues typically contribute to the cause of the failure in the first place.
- The volume of EPS placed within the overall slope cross-section may be relatively limited. Furthermore, the optimal location of the EPS mass within the overall slope cross-section is not intuitively obvious.
- The road pavement may not overlie the portion of the slope where the EPS is placed. Therefore load conditions on the EPS blocks may be such that blocks of relatively low density can be used which can achieve cost savings in the overall design.

Therefore, since a majority of analysis and design methods available in the literature focused on stand-alone embankments over soft ground, further study is required to address various uncertainties in the current state-of-practice of analyzing various failure mechanisms included in the interim design procedure. The Phase II study is required to address these uncertainties, refine the interim design procedure, and complete the comprehensive design guideline for the use of EPS-block geofoam in slope stabilization and repair applications. The details of the proposed Phase II research are described subsequently.

OBJECTIVE OF PHASE II

The objective of Phase II is to develop a comprehensive design methodology that optimizes both technical performance and cost for geofoam as lightweight fill in new and existing slope stability applications for highway projects. The Phase II work will consist of the below eight tasks. As previously noted, Task 8 (develop a design algorithm) was completed during Phase I and is not included as part of the total eight task count.

TASKS

Task 7. Pavement design considerations.

Objective: To recommend a method to design flexible and rigid pavement systems overlying EPS blocks based on the current AASHTO Mechanistic-Empirical Design Guide (MEPDG) and to summarize constructability issues related to the pavement system.

Approach: The need to revisit the design of pavement systems overlying EPS blocks is demonstrated by the fact that four of the fifteen responses to Question D3 of the geofoam usage questionnaire that was developed and distributed during Phase I involved pavement design and construction. Question D3 and a summary of the pavement design and construction related responses are provided below.

Question D3
Overall, what one item would you like us to consider or include in the NCHRP 24-11(02) project documents that would be of greatest use to you in designing, supplying or installing EPS-block geofoam for road construction?

Pavement design and construction related responses

- Revisit wheel loads and stresses recommended in 24-11(1). They seem to be overly conservative.
- More information on preparation of base material and top surface treatment (concrete slab, liner, etc.).
- Alternatives to use of a concrete capping slab over the geofoam.
- More details on protection of EPS from hydrocarbons and design procedure for load distribution slab.

These replies to Question D3 of the geofoam usage questionnaire in conjunction with the fact that the cost of the pavement system may be a large proportion of the overall cost of a stabilized slope that includes a paved roadway, especially if a Portland cement concrete load distribution slab is part of the pavement system, substantiate the need to further study the pavement design and construction aspect of EPS-block geofoam technology.

This task will consist of four subtasks. First, the summary of pavement systems that have been incorporated over EPS-block geofoam that was included in the 24-11(01) report will be updated. As part of this subtask we will solicit feedback on the condition of the existing pavement system.

Second, a summary of pavement structural analysis methods that may be applicable to pavement systems overlying EPS blocks will be prepared and a method to design flexible and rigid pavement systems overlying EPS blocks will be recommended. Emphasis during this task will be to update the pavement design recommendations included in the 24-11(01) report so that the updated recommended pavement design procedures are in alignment with the current AASHTO Mechanistic-Empirical Design Guide (MEPDG) (2008). The wheel loads and stresses recommended in the 24-11(1) report will be revisited to address the concern expressed in the questionnaire response that the wheel loads and stresses may be overly conservative. Consideration will also be given to design of pavement systems with as well as without a concrete load distribution slab. Additionally, the results of a differential icing application study (Arellano, 2007) will be summarized and included in the pavement design procedure.

Emphasis will be placed on the unique aspect of pavement design over EPS-block geofoam that the final pavement system design must also consider the impact of the pavement system on external and internal stability of the slope. Additionally, the final pavement cross-section must also provide sufficient support, either by direct embedment or structural anchorage, for any proposed utilities and road hardware.

The third subtask will consist of evaluating and summarizing potential alternatives to a Portland cement concrete load distribution slab. The fourth subtask will consist of summarizing constructability issues related to the pavement system. This summary will include constructability issues related to the limitations of achieving a specified level of compaction immediately above the EPS blocks. As part of this subtask, a procedure for design of a concrete load distribution slab will be investigated and summarized. General guidance with regards to estimating the minimum thickness of material required over the EPS blocks to minimize damage to the blocks during construction will be provided. Current design procedures of unpaved roads to include construction haul roads may provide guidance on stresses imposed by typical construction equipment through granular base material. Additionally, the orientation of the final layer of the upper most blocks will be revisited.

Task 8. Develop a design algorithm.

Objective: Develop a design algorithm that optimizes both technical performance and cost.

Approach: As the phase I work progressed, it was determined that it would be better to develop a preliminary design algorithm during Phase I and include it as part of the interim design guideline instead of developing the algorithm during Phase II as was initially planned. Therefore, this task has been completed.

Task 9. Performance based issues related to the standard.

Objective: Provide updated information that focuses on performance related issues that can contribute to the improvement of the recommended standard.

Approach: The following performance related issues will be investigated further to determine if current knowledge of these performance issues can contribute to the improvement of the recommended standard.

- Dimensional stability and seasoning time requirements.
- The current requirements for short-term UV protection and the related impact on EPS block properties.
- The impact of regrind on the integrity of the block as a whole and the impact on the overall density/unit weight of the block.
- Potential improvements to current testing protocols to make the overall QC/QA process more manageable. Investigate current available data on specimen size affects. More importantly, investigate current knowledge regarding the relationship between specimen properties and the property of a block as a whole. Can other tests such as flexural tests provide a better indication of the properties of a block as a whole than the current tests included in the recommended standard?

Task 10. Evaluate the applicability of the simplified seismic response methodology for stand-alone embankments to slope applications.

Objective: Perform detailed analyses to evaluate the influence of adjacent slope material on seismic stability and the applicability of the simplified seismic response methodology for stand-alone embankments to slope applications.

Approach: The seismic design procedure included in the interim recommended design guideline is based on the work performed predominantly on stand-alone embankments over soft soils and does not consider the influence of the adjacent slope material. Therefore, further study is required to evaluate the impact of seismic inertial forces from the adjacent slope material on the recommended seismic design procedure incorporated in the interim design guideline.

The current state-of-practice of seismic analysis is to decouple the determination of the overall seismic response acceleration of the EPS-block geofoam embankment into the determination of the seismic response of the natural slope material followed by the seismic response of the EPS-block fill mass. Additionally, it is current state-of-practice to evaluate each potential seismic failure mechanism separately. The failure mechanisms that are considered for external seismic stability analysis include slope instability, horizontal sliding of the entire EPS-block geofoam fill mass, overturning of a vertical-sided embankment, bearing capacity failure of the existing foundation earth material, and settlement of the existing foundation material. The failure mechanisms that are considered for internal seismic stability analysis include horizontal sliding between layers of blocks and/or between the pavement system and the upper layer of blocks and load bearing failure of the EPS blocks.

Therefore, seismic analysis and design of EPS-block geofoam embankments can be separated into the following three primary steps: (1) estimating the seismic-response acceleration at the existing ground surface or base (subgrade level) of the EPS fill mass by performing a site-specific assessment, (2) estimating the seismic-response acceleration at the top of the EPS fill mass, (3) performing pseudo-static limit equilibrium stability analyses of the various failure mechanisms. The current recommended procedure for performing Step 2 does not consider the seismic interaction between the adjacent slope material and the EPS-block geofoam fill mass for estimating the seismic-response acceleration at the top of the EPS fill mass. Therefore, the focus of this task is to develop a recommended procedure that

considers the influence of the adjacent slope material on the seismic-response acceleration of the EPS fill mass and/or on seismic stability.

It is anticipated that numerical analyses using FLAC or FLAC/Slope will be required to study the interaction between the EPS-block fill mass and the adjacent earth slope. Dr. Stark's extensive experience in analyzing slopes under seismic loads and using FLAC will be valuable for this analysis. Dr. Leshchinsky's extensive experience in analyzing reinforced-earth slopes and walls will also be valuable for this analysis.

The case history involving a numerical analysis of an EPS-block geofoam stand-alone, vertical-sided embankment constructed as part of the I-15 Reconstruction Project in Salt Lake City, Utah that was presented by Bartlett and Lawton (2008) will be initially used to develop the slope model. Bartlett and Lawton analyzed the behavior of the embankment under seismic loads generated by a nearby M7.0 earthquake using the finite difference program FLAC. An overview of the numerical study was included in Chapter 3 of the interim report.

Several unique aspects of the numerical analysis included (1) allowance of two degrees of freedom (horizontal and vertical) movement, (2) use of nonlinear stress-strain relations for all materials, except for the lumped mass model used to represent the pavement system, which was treated as an elastic material, (3) allowance of horizontal sliding between the geofoam layers by including interfaces nodes, and (4) input of both the horizontal and vertical components of the strong motion records into the model to explore their combined effects on the dynamic response and potential sliding (Bartlett and Lawton, 2008).

Task 11. Determine the impact of typical centrifugal loads on an EPS-block geofoam fill mass.

Objective: Perform detailed analyses to determine the impact of typical centrifugal loads on an EPS-block geofoam fill mass.

Approach: Although loads due to centrifugal forces are not typically considered in the design of earth structures, they may prove significant in the design of slopes incorporating EPS-block geofoam. The AASHTO centrifugal load (CF) category is intended to account for the reaction forces exerted on a curved highway bridge as vehicles go around the curve. The vehicle's tires exert a force on the roadway to overcome the vehicle's inertia and accelerate it around the curve. This in turn produces a reaction force on the roadway surface which is eventually transmitted to the subgrade. The force exerted by the vehicle tires that overcomes the vehicle's inertia is technically a centripetal or "center-seeking" force; that is, it is oriented in such a way as to push the vehicle toward the center of the curve. The force of interest for the design of EPS-block slopes is the reaction force of this centripetal force. The reaction force is equal in magnitude and opposite in direction to the centripetal force, pushing the roadway away from the center of the curve, hence the term centrifugal or "center-fleeing."

These centrifugal loads are dependent on the volume of traffic that the roadway is designed to carry, as well as the design speed of the roadway. An interstate highway designed to carry high traffic loads at high design speeds will obviously exert much greater centrifugal loads on its underlying subgrade than a low traffic rural road. For most earth structures, the sum of the reaction forces developed at the roadway is so small compared to the mass of the underlying subgrade that centrifugal loading can be safely ignored; however, because EPS-block geofoam has such an extremely low density, the inertia of the fill mass may not be large enough to justify neglecting the centrifugal forces developed at a curved roadway surface. A study on the impact of typical centrifugal loads on an EPS-block geofoam fill mass is required. As with seismic loading, any lateral loads applied to the EPS-block geofoam fill must be given special consideration to prevent shifting and shearing at the interfaces between layers of blocks.

In addition to evaluating the possibility of external slope instability, the nature of EPS-block geofoam construction is such that centrifugal loads may also tend to contribute to other failure mechanisms besides simply external slope instability, such as sliding between block layers or internal instability. Centrifugal forces were included in the design of EPS-block geofoam stand-alone

embankments utilized as part of the Boston Central Artery/Tunnel (BCA/T) project (Parsons Brinckerhoff). However, further analyses are required to determine the importance of these centrifugal loads in designing an EPS-block geofoam slope system. If it becomes apparent that centrifugal forces play a significant role, methods for incorporating this load group into the design procedure will be developed.

Task 12. Obtain higher density block test data.

Objective: Obtain test data for higher density block for inclusion in the recommended interim material and construction standard developed during Phase I.

Approach: The most recent version of ASTM D 6817 includes material properties for higher density EPS-blocks, ASTM designation EPS46. This higher density block has a density of 45.7 kg/m^3 and an elastic limit stress of 128 kPa. The highest density currently included in the recommended standard is 32 kg/m^3. Therefore, an effort will be made to obtain test data for higher density blocks to include in the recommended standard.

Task 13. Update the design algorithms developed in Task 8.

Objective: Update the design algorithms developed in Task 8 with the results of Tasks 7 through 12.

Approach: The proposed two potential design procedures shown in Figures 3.17 and 3.18 of the interim report developed in Task 8 during Phase I will be re-evaluated to determine if any changes will be required based on the research results of Tasks 7 through 12.

Task 14. Develop routine design aids.

Objective: Establish recommended EPS-block geofoam properties that can be utilized in typical commercial slope stability analysis software programs and develop design examples to facilitate the routine design of EPS-block geofoam in slope stabilization and repair.

Approach: This task will consist of the implementation of Task 13 into a form that is easy and friendly for the practicing engineer to use. Based on the results of the project questionnaire that is included in Appendix A of the interim report, the use of limit equilibrium slope stability software is the predominant method that DOTs utilize to analyze the stability of slopes. Therefore, Task 14 will focus on establishing recommended EPS-block geofoam properties that can be utilized in typical commercial slope stability analysis software programs.

Sensitivity analyses will be performed to evaluate the sensitivity of the recommended design procedure to various input design parameters. It is anticipated that sensitivity analyses can be performed for the case of circular slip surfaces within simple homogeneous slopes such as the charts presented in the landslide manual (Transportation Research Board, 1996) and in various slope stability textbooks (Abramson, et al., 2002, Duncan and Wright, 2005). It may also be possible to perform sensitivity analyses for non-circular slip surfaces for a range of engineering properties and geometries.

The results of the sensitivity analyses will be depicted in charts and figures, which can be used by designers to evaluate the sensitivity of the recommended design procedure to various input design parameters and to develop an optimal lightweight fill design. Additionally, designers may also use these charts and figures to determine the feasibility of using lightweight fill as well as for design of simple slope geometries and homogeneous soil conditions.

Design examples will be developed that will illustrate the use of the recommended design procedure. In each example, detailed calculations will be shown with the appropriate equation and/or design chart number. Additionally, tables will be used to summarize design calculation input values and results. These tables can serve as the basis for developing computer design spreadsheets.

The design algorithm, sensitivity charts and figures, and design examples will be incorporated in the final recommended design guideline that will summarize each step of the design process.

Task 15. Prepare final report.

Objective: Prepare a final report that summarizes findings, draws conclusions, and documents the research products including a validated design and analysis method for projects using geofoam as a lightweight fill in slope stability applications; a recommended design guideline in AASHTO format; and a recommended material and construction standard in AASHTO format. Provide an implementation plan for state highway agencies to use in incorporating the research products into practice.

Approach: This task will contain two major components: (1) preparation of a final project report and (2) an implementation plan for state highway agencies. The final report and implementation plan will be prepared by Dr. Arellano with input and review from Drs. Stark, Leshchinsky, and Horvath.

1. Preparation of the final project report.

The primary purpose of the final report will be to provide those who have primary involvement with roadway embankment projects, i.e., end users, manufacturers, contractors, and owners, with both state-of-art knowledge and state-of-practice design guidance for use of EPS-block geofoam in slope stability projects. The end users include engineers who perform the design, develop specifications, and perform field inspection; EPS block molders who manufacture the product; and construction contractors who install the product.

The key products of the research will be a recommended design guideline and a recommended material and construction standard. As with the NCHRP Project 24-11(01) reports, it is anticipated that the recommended design guideline and the standard will be published as a NCHRP report and that the full report will be available as a NCHRP web document. However, as part of the final implementation plan, recommendations will be made on how to integrate the results of this project with the NCHRP Project 24-11(01) reports to make implementation of the research results in practice more effective.

It is anticipated that the full Project 24-11(02) report will be organized similar to the Project 24-11(01) report (Stark, et al., 2004) and will be divided into two parts. The first part will consist of ten chapters. Chapter 1 will provide an overview of the use of EPS-block geofoam for slope stabilization projects and will include the research objective and approach used for the study. Chapter 2 will present a summary of updated engineering properties that are necessary to implement the proposed design methodology.

Chapter 3 will provide an overview of the design methodology and include the background for the "Recommended Design Guideline." Chapter 4 will present the details of the design procedure. Chapter 5 will present design examples that will demonstrate the design methodology outlined in Chapter 3 and detailed in Chapter 4 that can be used by design engineers to facilitate design of their projects.

Chapter 6 will provide an overview of the recommended material and construction standard and Chapter 7 will discuss geofoam construction practices. Chapter 8 will provide a summary of case histories that have successfully incorporated EPS-block geofoam into slope stability applications. Chapter 9 will provide cost information to allow a cost estimate to be prepared during the design phase so that an optimal geofoam design can be selected. The designer can then use this optimal geofoam design to perform a cost comparison with other slope stabilization techniques. Finally, Chapter 10 will present recommended areas of future research for EPS-block geofoam for slope stabilization applications.

The second part of the report will be composed of six appendices. Appendix A will describe the geofoam usage survey that was developed and distributed during Phase I of this study and will also present the responses to the survey. Appendix B will present the recommended design guideline for EPS-block geofoam in slope stability applications that will be outlined in Chapter 3. Appendix C will present the recommended standard for the use of EPS-block geofoam in slope stability applications, which should facilitate DOTs in specifying, and thus contracting for the use of geofoam in slope stability projects.

Appendix D will include an extensive bibliography of all references encountered during this study that relate to EPS-block geofoam. Finally, Appendix E will present a glossary of the terms used in the report and Appendix F will provide conversion factors that can be used to convert between Système International d'Unités (SI) and inch-pound (I-P) units.

2. An implementation plan for state transportation agencies.

An implementation plan will be developed for state highway agencies to consider for incorporating the research products into practice. Two key issues about the final report that will be addressed is the feasibility of integrating the research results for slope applications with the NCHRP Project 24-11(01) research results for stand-alone embankments. Also, the feasibility of creating a web-based link between the recommended design guideline and the recommended material and construction standard with the appropriate related sections of the full report will be considered.

PROJECT TIMELINE

The proposed Phase II project timeline, planned expenditures, and planned progress schedule are shown in Figures 6.1, 6.2, and 6.3, respectively. As indicated during the interim report panel meeting, it is anticipated that Phase II can be accomplished by June 2010. The time expended during Phase I was 35 percent, which is 5 percent over the initially planned estimate of 30 percent because Task 8 (develop a design algorithm) was completed during Phase I instead of during Phase II as was initially planned. Therefore, Phase II is 65 percent of the overall project duration.

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Appendix L

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