DESIGN PROCEDURE FOR GEOFOAM APPLICATIONS IN EMBANKMENT PROJECTS

Timothy D. Stark
Professor of Civil Engineering
University of Illinois @ Urbana-Champaign
205 N. Mathews Ave.
Urbana, IL 61801
(217) 333-7394
(217) 333-9464 Fax
tstark@uiuc.edu

and

David Arellano
Graduate Research Assistant
2212 Newmark Civil Engineering Laboratory
University of Illinois @ Urbana-Champaign
205 N. Mathews Ave.
Urbana, IL 61801
(217) 333-6940
(217) 333-9464 Fax
darellan@uiuc.edu

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DESIGN PROCEDURE FOR GEOFOAM APPLICATIONS IN EMBANKMENT PROJECTS

By: Timothy D. Stark[1] and David Arellano [2]

ABSTRACT: This report is a comprehensive document that provides both state-of-the art knowledge and state-of practice design guidance for the use of geofoam in roadway embankments and bridge approaches. This document presents a design guideline or procedure as well as an appropriate material and construction standard, both in AASHTO format. It is anticipated that this document will encourage greater and more consistent use of EPS-block geofoam in roadway embankments. The ultimate benefit of this is an optimization of both the technical performance as well as cost of EPS-block geofoam embankments. It is anticipated that designers will be more willing to consider EPS-block geofoam as an alternative for construction of embankments over soft ground using the design methodology and construction standard presented herein. The research has confirmed that EPS-block geofoam can provide a safe and economical solution to problems with construction of roadway embankments on soft soils. This report is designed to produce a single source of information on the present knowledge of geofoam usage in roadway embankments.

Despite the continuing worldwide use of EPS-block geofoam, a specific design guideline or design procedure for its use as lightweight fill in roadway embankments was unavailable. Therefore, there was a need in the U.S.A. since the mid 1990s to develop a formal and detailed design document for use of EPS-block geofoam in routine practice. The purpose of this report is to fill this void with a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design guidance for engineers. This document presents a design guideline as well as an appropriate material and construction standard, both in AASHTO format. It is anticipated that this document will encourage greater and more consistent use of EPS-block geofoam in roadway embankments. The ultimate benefit of this is an optimization of both the technical performance as well as cost of EPS-block geofoam embankments. It is anticipated that designers will be more willing to consider EPS-block geofoam as an alternative for construction of embankments over soft ground using the design methodology and material and construction standard presented herein.

[1] Prof. of Civil Engrg., Univ. of Illinois, 205 N. Mathews Ave., Urbana, IL 61801.
[2] Graduate Reearch Assistant of Civil Engrg., Univ. of Illinois, 205 N. Mathews Ave., Urbana, IL 61801.

Despite the extensive and continuing worldwide use of EPS-block geofoam, including in the U.S.A., specific design guidelines for its use as lightweight fill in roadway embankments is currently unavailable. Therefore, there was a need in the U.S.A. since the mid 1990s to develop formal and detailed design documents for use of EPS-block geofoam in routine practice. These documents would include both design guidelines as well as appropriate material and construction standard, both in the American Association of State Highway and Transportation Officials (AASHTO) format. The purpose of these design documents would be to encourage wider as well as more consistent use of EPS-block geofoam in roadway embankments. The ultimate benefit of these guidelines would be an optimization of both the technical performance as well as cost of EPS-block geofoam embankments.

The purpose of this report is to provide a comprehensive document that provides both state-of-the-art knowledge and state-of-practice design guidance for engineers. It is anticipated that designers will be more willing to consider EPS-block geofoam as an alternative for construction of embankments over soft ground using the design methodology and standard presented herein.

INTRODUCTION

The use of lightweight fill materials including EPS-block geofoam for roadway embankments as an alternative to ground improvement increased during the 1990s due to four significant reasons. First, the overall time for construction is typically much shorter and less uncertain when lightweight fills are used rather than a foundation soil or ground improvement method. The shorter construction time results from the simplicity of placing the blocks and the ability to place the blocks in adverse conditions. Second, lightweight fills produce relatively small undrained (initial) and consolidation settlements whereas traditional ground improvement methodologies, such as preloading, typically produce relatively large undrained and consolidation settlements. While these settlements may not affect the final road, they can negatively affect adjacent property, roads, bridges, buildings, utilities, etc. However, it is important to note that the use of lightweight fill materials will not reduce the magnitude of secondary (creep) compression settlement that will occur without an embankment. The magnitude of secondary consolidation settlement is a function of the properties of the underlying soft foundation soil only, and is thus independent of the external stresses applied to the foundation soil. Third, lightweight fills decrease maintenance costs because of less settlement. Fourth, the durability of EPS-block geofoam has been proven by projects completed in the 1970s. In consideration of these benefits, the typically higher unit cost of lightweight fill materials (a "negative" cited in (*1*) which was prepared in the late 1980s) is usually more than offset by savings when overall project costs are considered.

Various countries have developed general design guidelines and manuals to aid in the design of an embankment on soft soil incorporating EPS-block geofoam. These countries include France (*2*), Germany (*3*,*4*), Japan (*5*), Norway (*6*-*9*), the United Kingdom (*10*), and The Netherlands (*11*). The first monograph dedicated to geofoam discusses the basic concepts for analyzing and designing EPS-block geofoam fills (*12*). An outline-type manual with a general guideline specification has even appeared in the United States (*13*). However, these design guidelines and manuals do not provide a comprehensive design procedure or a material and construction standard that makes selection of a cost-effective design practical and reliable. The general consensus that was reached at the first *International Workshop on Lightweight Geo-Materials* that was held on 26 to 27 March 2002 in Tokyo, Japan was that although new weight-reduction techniques for decreasing applied loads on soft ground or earth pressures on retaining walls using geo-materials have recently been developed by various organizations, standardization of present design and construction methods is required (*14*).

Although EPS-block geofoam for road construction is an established technology and despite the extensive and continuing worldwide use of EPS-block geofoam, it is currently underutilized in U.S. practice because a comprehensive design guideline and a material and construction standard specification for its use as lightweight fill in roadway embankments is currently unavailable. Therefore, there is a need in the U.S. to develop formal and detailed design documents for use of EPS-block geofoam in routine practice. These documents need to include design guidelines as well as an appropriate material and construction specification. The purpose of these design documents would be to encourage wider as well as more consistent use of EPS-block geofoam as an alternative for construction of roadway embankments over soft ground. The ultimate benefit of these guidelines would be an optimization of both the technical performance as well as cost of EPS-block geofoam embankments.

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 road embankments over soft ground. To accomplish this objective, AASHTO, in cooperation with the FHWA, funded a thee-year study through the National Cooperative Highway Research Program (NCHRP), which is administered by the Transportation Research Board of the National Research Council, to develop a comprehensive design procedure and a material and construction standard specification for the use of geofoam in roadway embankments. The research proposed herein is similar to the NCHRP study.

It is anticipated that the availability of a comprehensive design manual will encourage designers to consider EPS-block geofoam more in the future than they have in the past. Additionally, the proposed design manual can serve as a blueprint for development of design guidelines and material and construction standard specifications for other types of lightweight fills.

As a result, an objective of the National Cooperative Highway Research Program (NCHRP) Project No. HR 24-11 titlede "Guidelines for Geofoam Applications in Embankment Projects" is to develop design guidelines and construction specifications for the use of geofoam in roadway embankments and bridge approaches over soft ground. The design guideline is presented herein and should facilitate the use of geofoam in civil engineering projects by providing engineers with a consistent design methodology to optimize both the technical performance and cost through the development of a comprehensive design guideline including design and analysis procedures for the use of EPS-block geofoam in road embankments over soft ground. The ultimate benefit of these guidelines would be an optimization of both the technical performance as well as cost of EPS-block geofoam embankments. Because of space constraints, this paper only provides an overview of the design guideline.

Successful technology transfer and acceptance of a construction product or technique requires the availability of a comprehensive design procedure; a material, product, and construction specification; and cost data. 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. Therefore, the proposed research will consist of the following four tasks: (1) summarize relevant engineering properties, (2) develop a comprehensive design procedure, (3) update the NCHRP provisional material, product, and construction standard specification, and (4) perform an economic analysis. This paper is the first in a series of three.

BASIS OF THE DESIGN GUIDELINE

Lightweight fill embankments are typically placed over soft saturated soils that are normally or at best slightly overly consolidated. Soft soil ground conditions as defined in the provisional design guideline is a soil subgrade that is compressible and has relatively low shear strength. Design and construction of embankments over soft ground is based on avoiding failure during construction by providing adequate stability and limiting postconstruction settlement to desirable amounts (*15*). The term "failure" as used in the provisional design guideline is a *loss of function*. Failure or loss of function may occur as either a *serviceability failure* (the *serviceability limit state*, SLS) or a *collapse failure* (the *ultimate limit state*, ULS). An embankment over soft soil may experience a serviceability failure due to excessive total or differential settlement that develops over time and which produces premature failure of the pavement system. Premature failure of the pavement system may include an uneven and often cracked pavement surface that may require frequent repaving and possibly other maintenance. An embankment over soft soil may experience a collapse failure through either a rotational (slope stability), lateral spreading, or bearing capacity failure mechanism. The collapse failure may involve at least partial, if not total, collapse.

The overall design goal is to satisfy the following equations for the ULS and SLS respectively:

ULS: resistance of embankment to failure > embankment loads producing failure **(3.1)**

SLS: estimated deformation of embankment $\leq$ maximum acceptable deformation **(3.2)**

The extent that resistance exceeds loads in Equation (3.1) is interpreted as representing the "safety" incorporated into the design. Thus, the "safety" or *factor of safety* (FS) is defined as

$$FS = \frac{\sum \text{resistances of embankment to failure}}{\sum \text{embankment loads producing failure}} \qquad \textbf{(3.3)}$$

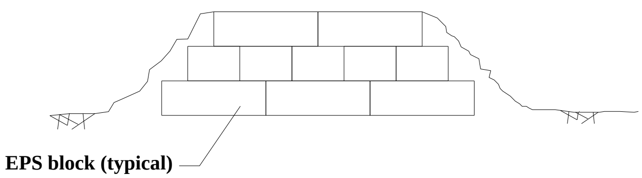
In the traditional Allowable Stress Design (ASD) method, the actual service loads are used in the design and safety is incorporated by using a single factor of safety applied to the ULS mechanism (e.g. slope stability) being investigated. The more modern Load and Resistance Factor Design (LRFD) method, safety is incorporated through the use of separate factors applied to simultaneously increase service loads and reduce material strengths or resistances. At the time this report was finalized (2002), geotechnical design practice in the U.S.A. was still in the early stages of transitioning from ASD to LRFD although road design is on the forefront of this transition with LRFD methodologies incorporated into various American Association of State Highway and Transportation Officials (AASHTO) design standards. It appears that refinements to LRFD methodologies are still required before they will be embraced for geotechnical design (*16*). At the present time, design of earthworks incorporating EPS-block geofoam are only designed deterministically using service loads and the traditional Allowable Stress Design (ASD) methodology with safety factors. Regardless of whether ASD or LRFD is used, the SLS assessment reflected in Equation (3.2) is always performed using service loads.

The common strategy for selecting a soft ground treatment alternative has been to use the traditional approach of satisfying Equations (3.1) and (3.2) by utilizing normal soil for the embankment. This means that the resistance and stiffness of the embankment and soft soil foundation must be increased artificially to be able to resist the loads with acceptable deformations. This is accomplished by employing one or more soft ground treatment techniques that collectively increase the strength and reduce the compressibility of the overall system (primarily the existing soft foundation soil but also the embankment soil itself to some degree). Chapter 1 presents an overview of soft ground treatment methods.

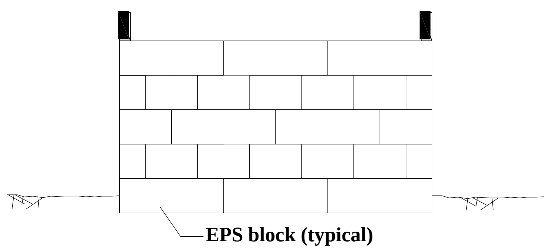
Less prevalent is the alternative approach to satisfying Equations. (3.1) and (3.2) of reducing the embankment loads. This involves accepting the natural resistance (strength and compressibility) of the existing soft foundation soil as it exists and employing strategies to sufficiently reduce the loads acting on the soft soils to achieve the goals stated in Equations (3.1) and (3.2). This is accomplished by using *lightweight fill* materials such as EPS-block geofoam. The most significant aspect of lightweight fill materials is that lightweight fill materials have densities less than that of soil and rock so that the resulting gravity or seismic forces from the fill material are less than those from normal earth materials (*17*). The use of this concept in designing embankments over soft soils may yield a more technically effective and cost efficient embankment.

As with any design in geotechnical engineering practice, the design procedure of embankments over soft foundations must incorporate tolerable criteria such as minimum factor of safety and maximum allowable settlement. Minimum factor of safety values typically utilized in geotechnical design are based on precedence and can be found in local codes, design manuals, and geotechnical literature. The recommended factors of safety for each failure mechanism calculation that can be considered in preliminary design is summarized in Figure 3.3. However, tolerable settlements for highway embankments are not well established in practice nor is information concerning tolerable settlements available in the geotechnical literature. Postconstruction settlements as much as 0.3 to 0.6 m (1 to 2 ft) during the economic life of a roadway are generally considered tolerable provided that the settlements are uniform, occur slowly over a period of time, and do not occur next to a pile-supported structure (*18*). If postconstruction 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 undergone 0.3 to 0.6 m (1 to 2 ft) of uniform settlement without distress or objectionable riding roughness, flexible pavements are usually selected where doubt exists about the uniformity of postconstruction settlements and some states utilize a flexible pavement when predicted settlements exceed 150 mm (6 in.) (*18*). Tolerable settlements of bridge approach embankments depend on the type of structure, location, foundation conditions, operational criteria, etc (*1*). The following references are recommended for information on abutment movements: (*19-22*). Settlement considerations are further discussed in Chapter 5.

This design guideline is intended to provide design guidance to civil engineers experienced in geotechnical engineering and pavement engineering when designing lightweight fills that incorporate expanded polystyrene (EPS)-block geofoam. The proposed design guideline is limited to embankments that have a transverse (cross-sectional) geometry such that the two sides are more or less of equal height, see Figure B.1. Applications where the fill sides are markedly different and closer to those shown in Figure B.2 (sometimes referred to as side-hill fills) are excluded from this study because they are the subject of a separate study (*23*). It should be noted from Figure B.1 (b) that unlike other types of lightweight fill embankments, a vertical embankment can be utilized with EPS-block geofoam. The use of a vertical embankment, sometimes referred to as a geofoam wall, will minimize the amount of right-of-way needed and will also minimize the impact of the embankment loads on nearby structures. The types of fills considered in this document are also limited to approaches with conventional jointed-deck bridges (including fill behind the abutments of such bridges). In both the embankment and bridge approach cases the underlying foundation soil consists of soft soil defined as relatively compressible and weak. For the purposes of this design guideline, such earthworks will be referred to simply as embankments on soft soil.



(a) Sloped-side fill.



(b) Vertical-face fill.

Figure B.1. Typical EPS-block geofoam applications involving embankments (*12*).

Figure B.2. Typical EPS-block geofoam applications involving side-hill fills (*12*).

The design charts developed as part of this research and included herein are based on embankment models with the geometric and material parameters described in the report. However, most design charts are based on embankment side slopes of 0 (horizontal, H):1 (vertical, V), 2H:1V, 3H:1V, and 4H:1V. Widths at the top of the embankment of 11 m (36 ft), 23 m (76 ft), and 34 m (112 ft) were evaluated. These widths are based on a 2-lane roadway with 1.8 m (6 ft) shoulders, 4-lane roadway with two 3 m (10 ft) exterior shoulders and two 1.2 m (4 ft) interior shoulders, and a 6-lane roadway with four 3 m (10 ft) shoulders. Each lane was assumed to be 3.66 m (12 ft) wide. Embankment heights ranging between 1.5 m (4.9 ft) to 16 m (52 ft) were evaluated. For simplicity, the fill mass was assumed to consist entirely of EPS blocks.

This design guideline is expected to be suitable for the preliminary design of most typical projects (projects with either critical or noncritical conditions) and for final design for projects with predominantly noncritical conditions. Examples of critical and noncritical design conditions are provided in Table B.1. Engineering judgment is required to determine if critical or noncritical design conditions exist for a specific project situation. More detailed design is required for embankments with critical conditions than with noncritical conditions.

Table B.1. Examples of critical and noncritical embankment design and construction **conditions (*1*).**

With regard to 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 the 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.

MAJOR COMPONENTS OF AN EPS – BLOCK GEOFOAM EMBANKMENT

As indicated in Figure 3.2, an EPS – block geofoam embankment consists of three major components:

- The existing foundation soil which may have undergone ground improvement prior to placement of the fill mass.

- The proposed fill mass, which primarily consists of EPS-block geofoam although some amount of soil fill is often used between the foundation soil and bottom of the EPS blocks for overall economy. In addition, depending on whether the embankment has sloped (trapezoidal embankment) or vertical (vertical embankment) sides, there is either soil or structural cover over the sides of the EPS blocks.

- The proposed pavement system, which is defined as including all material layers, bound and unbound, placed above the EPS blocks. The uppermost pavement layer, which serves as the finished road surface, is usually either asphaltic concrete (AC) or portland-cement concrete (PCC) to provide a smooth traveling surface for motor vehicles. AC appears to be the most predominant road surface type because AC pavements tend to tolerate postconstruction settlements better than PCC pavements as well as for economic reasons. However, in certain applications (e.g., vehicle escape ramps in mountainous regions, logging roads) an unbound gravel or crushed-rock surface layer may be utilized.

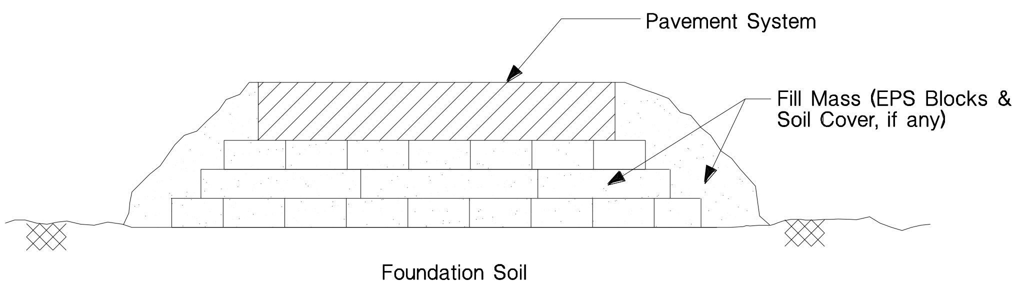


Figure 3.2. Major components of an EPS-block geofoam embankment.

DESIGN PHASES

At the present time, design of earthworks incorporating EPS-block geofoam are only designed deterministically using service loads and the traditional Allowable Stress Design (ASD) methodology with safety factors. The embankment overall as well as its components individually must be designed to prevent failure. As used herein, the term failure includes both:

- *serviceability failure* (e.g., excessive settlement of the embankment, premature failure of the pavement system). In this document, this will be referred to as the *serviceability limit state* (SLS) and

- *collapse* or *ultimate failure* (e.g., slope instability of the edges of the embankment). In this document, this will be referred to as the *ultimate limit state* (ULS).

The overall design process is divided into the following three phases:

- Design for external (global) stability of the overall embankment. This considers how the combined fill mass and overlying pavement system interact with the existing foundation soil. It includes consideration of serviceability issues (SLS), such as global total and differential settlement, and collapse failure issues (ULS), such as bearing capacity and slope stability under various load cases, e.g., applied gravity, seismic, water and wind loading. 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 design criteria concerning settlement and stability. Therefore, it is not necessary for the EPS blocks to extend the full height vertically from the top of the foundation soil to the bottom of the pavement system.
- Design for internal stability within the embankment mass. The primary consideration is the proper selection and specification of EPS properties so that the geofoam mass can support the overlying pavement system without excessive immediate and time-dependent (creep) compression that can lead to excessive settlement of the pavement surface (an SLS consideration).
- Design of an appropriate pavement system for the subgrade provided by the underlying EPS blocks. This design criterion is to prevent premature failure of the pavement system, as defined by rutting, cracking, or similar criterion, which is an SLS type of failure. Also, when designing the pavement cross-section overall consideration should 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 summary of the three-phased design procedure is shown in Table 3.1. Each of the three primary design phases shown in Table 3.1 has been divided into various failure mechanisms that need to be considered for each design phase. Each failure mechanism has also been categorized into either an ULS or SLS failure. These three design phases are conceptually similar to those used in the design process for other types of geosynthetic structures used in road construction, e.g. mechanically stabilized earth walls (MSEWs).

Table 3.1. Summary of Proposed Design Procedure for EPS-block geofoam roadway embankments.

DESIGN PROCEDURE

The design procedure for an EPS – block geofoam roadway embankment over soft soil considers the interaction between the three major components of the embankment, i.e., foundation soil, fill mass, and pavement system. Because of this interaction, the three-phased design procedure involves interconnected analyses between these 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. Additionally, the dead load imposed by the pavement system and fill mass may decrease the factor of safety of some failure mechanisms, e.g., slope stability, while increasing it in others, e.g., uplift. Because of the interaction between these components, overall design optimization of a roadway embankment incorporating EPS-block geofoam requires an iterative analysis to achieve a technically acceptable design at the lowest overall cost. In order to minimize the iterative analysis, the design procedure shown in Figure B.4 was developed to obtain an optimal design. The design procedure considers a pavement system with the minimum required thickness, a fill mass with the minimum thickness of EPS-block geofoam, and the use of an EPS block with the lowest possible density. Therefore, the design procedure will produce a cost efficient design. Figure B.4 also presents remedial measures that can be employed if one of the design criteria is not satisfied.

The design procedure is similar for both trapezoidal and vertical embankments except that overturning of the entire embankment at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation soil as a result of horizontal forces should be considered for vertical embankments as part of seismic stability (Step 7), translation due to water (Step 9), and translation due to wind (Step 10) analysis during the external stability design phase

Figure 3.3. Flow chart of design procedure for an EPS-block geofoam roadway embankment.

Step 1 – Background Investigation

The first step in the design procedure is background investigation, which involves obtaining the subsurface information at the project site, estimates of the loads that the embankment system will be subjected to, and determining the geometrical parameters of the embankment.

Step 2 – Preliminary Selection of EPS and Pavement System

The second step of the design procedure is to select a preliminary type of EPS-block geofoam and pavement system. Although the pavement system has not been designed at this point, it should be equal to or greater than 610 mm (24 in.) in thickness to minimize the effects of differential icing and solar heating. The design procedure depicted in Figure B.4 is based on obtaining a pavement system that provides the least amount of vertical stress to the of the EPS-block geofoam embankment to satisfy internal and external stability requirements. Therefore, it is recommended that the preliminary pavement system be assumed to be 610 mm (24 in.) thick and the various component layers of the pavement system be assumed to have a total (moist) unit weight of 20 kN/m^3 (130 lbf/ft^3).

A minimum of two layers of blocks should be used for lightweight fills beneath roads because a single layer of blocks can shift under traffic loads and lead to premature failure (*24*). Block thicknesses typically range between 610 mm (24 in.) to 1000 mm (39 in.). Therefore, it is recommended that a minimum of two EPS blocks with a thickness of 610 mm (24 in.) each or a total initial height of 1.2 m (4 ft) be considered for the EPS block height to determine the preliminary embankment arrangement during the design process. The preliminary height of conventional soil fill is the total embankment height required based on the background investigation less the preliminary pavement system thickness of 600 mm (24 in.) and the thickness of two EPS blocks of 1.2 m (4 ft).

The proposed EPS-block geofoam embankment design procedure requires that a preliminary pavement system design be assumed to estimate the gravity loads for use in the external and internal stability analyses prior to performing the final pavement design. It is recommended that the preliminary system be assumed to be 610 (24 in.) thick and the various component layers, i.e. asphalt-concrete, crushed stone, and sandy gravel subbase, of the pavement system be assumed to have a total (moist) unit weight of 20 kN/m^3 (130 lbf/ft^3) for initial design purposes.

Step 3 – Select Preliminary Embankment Arrangement

The third step of the design procedure is to determine a preliminary embankment arrangement. Because EPS-block geofoam typically has a higher material cost-per-volume than soil, it is desirable to optimize the volume of EPS used yet still satisfy design criteria concerning settlement and stability. Therefore, to achieve the most cost-effective design, a design goal is to use the minimum amount of EPS blocks possible that will meet the external and internal stability requirements. The design failure mechanisms that will dictate the maximum stress that can be imposed on the soft foundation soil, which dictates the minimum thickness of EPS blocks needed, include settlement, bearing capacity, slope stability, and external seismic stability.

Steps 4-10 – External (global) Stability

INTRODUCTION

After the design loads, subsurface conditions, embankment geometry, preliminary type of EPS, preliminary pavement design, and preliminary fill mass arrangement have been obtained, the design continues with external (global) stability evaluation. External (global) stability is illustrated in Steps 4 through 10 in the flow chart in Figure B.4. The tolerable criteria for each operation is also shown in Figure B.4.

Design for external (global) stability of the overall EPS-block geofoam embankment involves consideration of how the combined fill mass and overlying pavement system will interact with the foundation soil. External stability consideration in the proposed design procedure includes consideration of Serviceability Limit State (SLS) issues, such as total and differential settlement caused by the soft foundation soil and Ultimate Limit State (ULS) issues, such as bearing capacity, slope stability, seismic stability, hydrostatic uplift (flotation), translation due to water (hydrostatic sliding), and translation due to wind.

Step 4 – Settlement of Embankment

Settlement is the amount of vertical deformation that occurs from immediate or elastic settlement of the fill mass or foundation soil, consolidation and secondary compression of the foundation soil, and long-term creep of the fill mass at the top of a highway embankment. Settlement caused by lateral deformation of the foundation soil at the edges of an embankment is not considered because (*25*) presents inclinometer measurements that show the settlements from lateral deformation are generally small compared with the five previously mentioned settlement mechanisms if the factor of safety against external instability during construction remains greater than about 1.4. If the factor of safety remains greater than 1.4, settlement caused by lateral deformation is likely to be less than 10 percent of the end-of primary settlement (*25*). The proposed design procedure recommends a factor of safety against bearing capacity failure and slope instability greater than 1.5. Therefore, settlement resulting from lateral deformations is not considered herein.

Total settlement of an EPS-block geofoam embankment considered herein, S_{total}, consists of five components as shown by Equation (B.2):

$$S_{total} = S_{if} + S_i + S_p + S_s + S_{cf} = S_p + S_c \qquad \textbf{(B.2)}$$

where 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.

Immediate or elastic settlement of both the fill mass and foundation soil occur during construction and will not impact the condition of the final pavement system. It is concluded that the value of S_{cf} is expected to be within tolerable limits (less than 1 percent over 50 years) because the design procedure limits to immediate strain in the EPS to 1 percent. Therefore, the total settlement estimate focuses on primary and secondary consolidation of the soil foundation. Therefore Equation (B.2) simplifies as shown above. However, immediate settlement of the soil foundation should be considered if the embankment will be placed over existing utilities. Immediate settlement can be estimated by elastic theory and is discussed in (*26*).

Procedures for estimating both the EOP consolidation of the soil foundation is the amount of compression that occurs during the period of time required for the excess porewater pressure to dissipate for an increase in effective stress for both overconsolidated and normally consolidated soil deposits and Secondary consolidation of the soil foundation is the amount of compression that occurs after the dissipation of the excess porewater pressure induced by an increase in effective stress occurs and thus secondary consolidation occurs under the final effective vertical stress, σ'_{vf}. can be found in most geotechnical engineering textbooks.

Postconstruction settlements of 0.3 to 0.6 m (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 (*18*). If postconstruction 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 0.6 m (1 to 2 ft) of uniform settlement, flexible pavements are usually selected where doubt exists about the uniformity of postconstruction settlements and some states utilize a flexible pavement when predicted settlements exceed 150 mm (6 in.) (*18*). The transition zone between geofoam and embankment soil should be gradual to minimize differential settlement. The calculated settlement gradient within the transition zone should not exceed 1:200 (vertical:horizontal).

Step 5 – External Bearing Capacity of Embankment

If an external bearing capacity failure occurs, the embankment can undergo excessive vertical settlement and impact adjacent property. The general expression for the ultimate bearing capacity of soil, q_{ult}, is defined by (*27*) as:

$$q_{ult} = cN_c + \gamma D_f N_q + \gamma B_w N_\gamma \qquad \textbf{(B.7)}$$

where:

c = Mohr-Coulomb shear strength parameter termed cohesion, kPa,

N_c, N_γ, N_q = Terzaghi shearing resistance bearing capacity factors,

γ = unit weight of soil, kN/m^3,

B_W= bottom width of embankment, m, and

D_f = depth of embedment, m.

Narrowing the type of foundation soil to soft, saturated cohesive soils which allows c to equal the undrained strength, s_u of the foundation soil, and assuming the embankment is placed on the ground surface. For design purposes, an EPS-block geofoam embankment is assumed to be modeled as a continuous footing; and thus, the length of the embankment can be assumed to be significantly larger than the width.

Incorporating stress distribution theory into Equation (B.9), the undrained shear strength required to satisfy a factor of safety of 3 for a particular embankment height is as follows:

$$s_u = \frac{3}{5} * \left\{ \left[\frac{(\sigma_{n,pavement} + \sigma_{n,traffic}) * T_W}{(T_W + T_{EPS})} \right] + \frac{(\gamma_{EPS} * T_{EPS})}{2} \right\} \qquad \textbf{(B.12)}$$

where:

□$_{n,\ pavement}$ = normal stress applied by pavement at top of embankment, kPa,

□$_{n,\ traffic}$ = normal stress applied by traffic surcharge at top of embankment, kPa, and

T_W = top width of embankment, m.

□$_{n@0m}$ = normal stress applied by the embankment at the ground surface or at a depth of 0 metres, kPa

□□□□□□□□□ = □$_{n,\ pavement@0m}$ + □$_{n,\ traffic@0m}$ + □$_{n,geofoam@0m}$, **(B.10)**

□$_{n,\ pavement@0m}$ = normal stress applied by pavement system at the ground surface, kPa,

□$_{n,\ traffic@0m}$ = normal stress applied by traffic surcharge at the ground surface, kPa,

□$_{n,EPS@0m}$ = normal stress applied by weight of EPS-block geofoam at the ground surface, kPa

$= \gamma_{EPS} * T_{EPS}$, **(B.11)**

γ_{EPS} = unit weight of the EPS-block geofoam, kN/m³, and

T_{EPS} = thickness or total height of EPS-block geofoam, m.

Step 6 – Slope Stability

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This section presents an evaluation of external slope stability as a potential failure mode of EPS-block geofoam embankments. If a slope stability failure occurs, the embankment can undergo substantial vertical settlement and impact adjacent property. A typical cross-section through a trapezoidal EPS embankment with side slopes of 2H:1V is shown in Figure B.6 and was used to develop the external slope stability design charts for trapezoidal embankments.

A similar cross-section was used to evaluate the external slope stability of vertical embankments except that no soil cover is used on the sides of vertical embankments and the the surcharge used to represent the pavement and traffic surcharges is replaced by placing a 0.61 m (2 ft) soil layer on top of the embankment with a unit weight of 54.1 kN/m^3 (345 lbf/ft^3). The soil layer is 0.61 m (2 ft) thick to represent the minimum recommended pavement section thickness discussed in Section B.3. Therefore, the vertical stress applied by this soil layer equals 0.61 m (2 ft) times the increased unit weight of 54.1 kN/m^3 (345 lbf/ft^3) or 33.0 kPa (690 lbs/ft^2). A vertical stress of 33.0 kPa (690 lbs/ft^2) corresponds to the sum of the design values of pavement surcharge (21.5 kPa (450 lbs/ft^2)) and traffic surcharge (11.5 kPa (240 lbs/ft^2)) used previously for external bearing capacity and slope stability of trapezoidal embankments. The pavement and traffic surcharge in Figure B.6 was replaced by an equivalent soil layer because a seismic slope stability analysis can only be performed with material layers and not surcharge loads as discussed in section B.4.5. In a pseudo-static analysis a seismic coefficient cannot be applied to a surcharge in limit equilibrium stability analyses only material layers because the horizontal force that represents the seismic loading must be applied at the center of gravity of the material layer. The equivalent soil layer, which is equivalent to the pavement and traffic surcharge, was used for the static stability analyses of vertical embankments instead of a surcharge, as was done for static stability analyses of trapezoidal embankments, to minimize the number of stability analyses that would be required if two models were utilized, i.e., an embankment modeled with a surcharge and one modeled with a soil layer.

The soil cover is 0.46 m (1.5 ft) thick, which is typical for the side slopes, and is assigned a moist unit weight of 18.9 kN/m^3 (120 lbf/ft^3). The pavement system is modeled using a surcharge of 21.5 kPa (450 lbs/ft^2). The traffic surcharge is 11.5 kPa (240 lbs/ft^2) based on the AASHTO recommendation (*28*) of using 0.67 m (2 ft) of a 18.9 kN/m^3 (120 lbf/ft^3) soil to represent the traffic surcharge at the top of the embankment. Therefore, the total surcharge used to represent the pavement and traffic surcharges is 21.5 kPa (450 lbs/ft^2) plus 11.5 kPa (240 lbs/ft^2) or 33.0 kPa (690 lbs/ft^2).

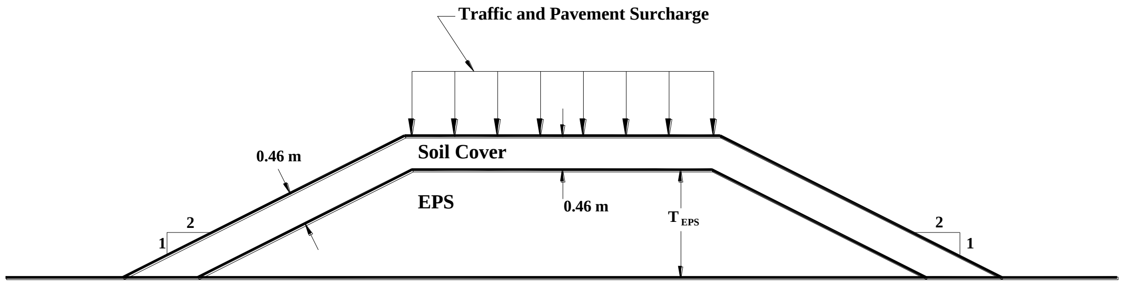

Figure B.6. Typical cross-section used in static external slope stability analyses of trapezoidal embankments.

The results of stability analyses using the typical cross-section were used to develop the static external slope stability design charts in Figures B.7 through B.9 for a 2-lane (road width of 11 m (36 ft)), 4-lane (road width of 23 m (76 ft)), and 6-lane (road width of 34 m (112 ft)) roadway embankment, respectively. Figure B.7 presents the results for a 2-lane geofoam embankment and the three graphs correspond to the three slope inclinations considered, i.e., 2H:1V, 3H:1V, and 4H:1V, for various values of s_u for the foundation soil. It can be seen that for a 2H:1V embankment the affect of geofoam height, T_{EPS}, is small where as geofoam height is an important variable for a 4H:1V embankment. The geofoam height corresponds to only the thickness or height of the geofoam, T_{EPS}, and thus the total height of the embankment is T_{EPS} plus the thickness of the pavement system. In the graph for the 4H:1V embankment, it can be seen that each relationship terminates at a different s_u value for the foundation soil. The value of s_u at which each relationship terminates signifies the transition from external slope stability being critical to internal stability being critical. For example, for a geofoam height of 12.2 m (40 ft), external slope stability controls for s_u values less than approximately 40 kPa (825 lbs/ft²). Therefore, a design engineer can enter this figure with an average value of s_u for the foundation soil and determine whether external or internal stability controls the design. If internal stability controls, a static internal slope stability analysis does not have to be performed because the factor of safety against internal slope stability failure is expected to exceed 1.5. If external stability controls, the designer can use this design chart to estimate the critical static factor of safety for the embankment, which must exceed a value of 1.5.

The analyses performed to develop the design charts revealed that the critical static factor of safety for the embankment for the 2-lane, 4-lane, and 6-lane roadway embankment, respectively, all exceed a value of 1.5 for values of s_u greater than or equal to 12 kPa (250 lbs/ft²). These results indicate that external static slope stability will be satisfied, i.e., factor of safety greater than 1.5, if the foundation undrained shear strength exceeds 12 kPa (250 lbs/ft²). Thus, external slope stability does not appear to be the controlling external failure mechanism, instead it appears that settlement will be the controlling external failure mechanism.

The analyses performed for vertical embankments revealed that as the foundation s_u increases, the overall embankment slope stability factor of safety increases. It can be seen that for a 23 m (76 ft) and 34 m (112 ft) wide embankment, as the geofoam thickness or height, T_{EPS}, increases for a given foundation s_u , the critical factor of safety decreases. The geofoam height corresponds to only the thickness or height of the geofoam and thus the total height of the embankment is T_{EPS} plus the thickness of the pavement system. However, for the narrower embankment of 11 m (36 ft), the geofoam height of 12.2 m (40 ft) yielded a larger factor of safety than the shorter embankments of 3.1 m (10 ft) and 6.1 m (20 ft). Narrow and tall embankments yield larger factors of safety because the failure surface will extend further out from the toe of the embankment and, consequently, the heavier foundation soil below the toe of the embankment provides more resisting force to the failure surface. The failure surface extends further out because if the failure is assumed to be circular, the failure surface must extend further out for narrow and tall embankments to accommodate the circular failure surface.

It can be seen that roadway width has little influence on the critical factor of safety for short embankments, e.g., at a height of 3.1 m (10 ft), but the influence of embankment width increases with increasing embankment height. This conclusion is supported by the observation made previously on the behavior of the critical static failure surface that narrow and tall embankments with vertical walls will yield larger factors of safety because the failure surface will extend further out from the toe of the embankment and, consequently, the heavier foundation soil below the toe of the embankment provides more of the resisting load to the failure surface.

Figure B.7. Static external slope stability design chart for trapezoidal embankments with a 2-lane roadway with a total road width of 11 m (36 ft).

Step 6 – Seismic Stability and Overturning (external)

Seismic loading is a short-term event that must be considered in geotechnical problems including road embankments. Seismic loading can affect both external and internal stability of an embankment containing EPS-block geofoam. This section considers external seismic slope stability of EPS-block geofoam trapezoidal embankments while internal seismic stability is addressed in Section B.5.4. External seismic stability is evaluated using a pseudo-static slope-stability analysis (*29*) involving circular failure surfaces through the foundation soil. The steps in a pseudo-static analysis are:

1. Locate the critical static failure surface, i.e., the static failure surface with the lowest factor of safety, that passes through the foundation soil using a slope stability method that satisfies all conditions of equilibrium. This value of factor of safety should satisfy the required value of static factor of safety of 1.5 before initiating the pseudo-static analysis.
2. Reduce the static shear strength values for cohesive (20 percent) or liquefiable (80-90 percent) soils situated along the critical static failure surface to reflect a strength loss due to earthquake shaking.
3. Determine the appropriate value of horizontal seismic coefficient, k_h, that will be applied to the center of gravity of the critical static failure surface. A search for a new critical failure surface should not be conducted with a seismic force applied because the search may and usually does not converge.
4. Calculate the pseudo-static factor of safety, FS', for the critical static failure surface and ensure it meets the required value of 1.2.

Pseudo-static slope stability analyses were conducted on the range of embankment geometries used in the external static stability analyses to investigate the effect of various embankment heights (3.1 m (10 ft) to 12.2 m (40 ft)), slope inclinations (2H:1V, 3H:1V, and 4H:1V), and road widths of 11, 23, and 34 m (36, 76, and 112 ft) on external seismic slope stability. Three seismic coefficients, low (0.05), medium (0.10), and high (0.20), were used for each roadway embankment. The results of these analyses were used to develop design charts to facilitate seismic design of roadway embankments that utilize geofoam. The seismic analyses utilized the critical static failure surfaces identified for each geometry in the external static stability analyses. A pseudo-static analysis was conducted on only the critical failure surfaces that passed through the foundation soil because external stability is being evaluated. As a result, the design charts for seismic stability terminate at the s_u value for the foundation soil that corresponds to the transition from a critical failure surface in the foundation soil to the geofoam embankment determined during external static stability analysis. This resulted in the seismic stability design charts terminating at the same value of s_u as the static stability charts in Figures B.7 through B.9.

A typical cross-section through an EPS embankment with side-slopes of 2H:1V used in the pseudo-static stability analyses is shown in Figure B.12. This cross-section differs from the cross-section used for the static analyses in Figure B.6 because the surcharge used to represent the pavement and traffic surcharges is replaced by assigning the soil cover layer on top of the embankment a unit weight of 71.8 kN/m^3 (460 lbf/ft^3). The soil cover is 0.46 m (1.5 ft) thick so the stress applied by this soil cover equals 0.46 m times the increased unit weight or 33.0 kPa (690 lbs/ft^2). A stress of 33.0 kPa (690 lbs/ft^2) corresponds to the sum of the design values of pavement surcharge (21.5 kPa (450 lbs/ft²)) and traffic surcharge (11.5 kPa (240 lbs/ft²)) used previously for external bearing capacity and slope stability. The surcharge in Figure B.6 had to be replaced because a seismic coefficient is not applied to a surcharge in limit equilibrium stability analyses only material layers because the horizontal force that represents the seismic loading must be applied at the center of gravity of the material layer.

Figure B.12. Typical cross-section used in seismic external slope stability analyses of trapezoidal embankments.

Figures B.13 through B.15 present the seismic external stability design charts for a 6-lane (road width of 34 m (112 ft)) geofoam roadway embankment and the three values of horizontal seismic coefficient, i.e., 0.05, 0.10, and 0.20, respectively. The 6-lane roadway results in the most critical seismic stability condition because the widest roadway results in the largest critical slide mass and thus the largest horizontal seismic force. This results in seismic stability concerns for the smallest horizontal seismic coefficient (see Figure B.10), the shortest embankment height of 3.1 m (10 feet) (see Figure B.14), and the flattest slope inclination of 4H:1V (see Figure B.15). These design charts can be used to estimate the critical values of the seismic factor of safety.

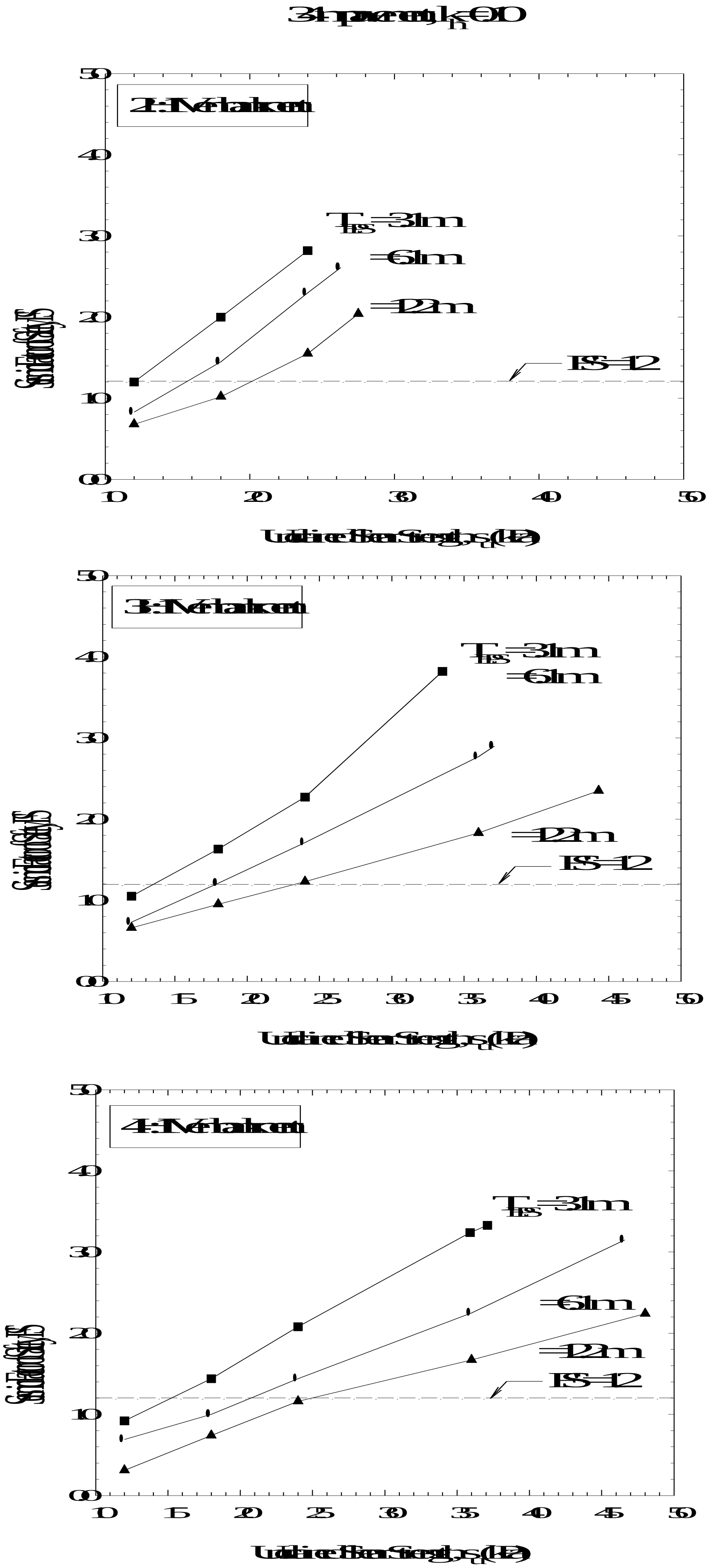

Figure B.14. Seismic external slope stability design chart for trapezoidal embankments with a 6-lane roadway with a total road width of 34 m (112 ft) and a k_h of 0.10.

In summary, seismic external slope stability can control the design of a trapezoidal geofoam roadway embankment depending on the width, or number of roadway lanes, on the embankment and the magnitude of the horizontal seismic coefficient. Most of the geometries considered herein are safe for a horizontal seismic coefficient of less than or equal to 0.10. If the particular embankment is expected to experience a horizontal seismic coefficient greater than or equal to 0.20, seismic external slope stability could control the design of the embankment.

In seismic design of vertical embankments the following two analyses should be performed: 1) psuedo-static slope-stability analysis involving circular failure surfaces through the foundation soil, and 2) overturning of the entire embankment about one of the bottom corners of the embankment at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation soil due to pseudo-static horizontal forces acting on the embankment especially for tall and narrow vertical embankments.

This section focuses on the effect of seismic forces on the external slope stability of vertical EPS-block geofoam embankments. This analysis uses the same pseudo-static slope stability analysis used for external seismic stability of trapezoidal embankments presented in Section B.4.5.1.1 and circular failure surfaces through the foundation soil.

Pseudo-static slope stability analyses were conducted to investigate the effect of various embankment heights (3.1 m (10 ft) to 12.2 m (40 ft)) and road widths of 11, 23, and 34 m (36, 76, and 112 ft) on external seismic slope stability. The results of these analyses were used to develop design charts to facilitate seismic design of vertical roadway embankments that utilize geofoam. The seismic analyses utilize the critical static failure surfaces identified for each geometry in the external static stability analyses. A pseudo-static analysis was conducted on only the critical failure surfaces that passed through the foundation soil because external stability is being evaluated.

The same typical cross-section through an EPS embankment used in the static slope stability analysis of embankments with vertical walls was also used for the pseudo-static stability analyses and is shown in Figure B.10.

The pseudo-static external slope stability analyses results revealed the following conclusions:

(1) Seismic stability is not a concern for vertical embankments with the geometries considered and horizontal seismic coefficients of 0.05, 0.10, and 0.20 because all of the computed values of factor of safety exceed the required value of 1.2. The factor of safety values obtained for embankments with vertical walls is greater than the embankment with 2H:1V side slopes. This conclusion is in agreement with the conclusion made for trapezoidal embankments that flatter embankments are more critical than the 2H:1V embankment because the weight of the soil cover materials above the critical static failure surface increases as the side slope becomes flatter which results in a greater seismic force being applied in the 3H:1V and 4H:1V embankments versus the 2H:1V embankment. The flatter embankments are more critical and thus a higher foundation undrained shear strength will be required to satisfy a factor of safety of 1.2 especially for the 4H:1V embankment.

(2) Unlike the observations made for trapezoidal embankments, a wider roadway does not necessarily result in a decrease in seismic stability.

(3) The narrower embankment width of 11 m (36 ft) produces a higher factor of safety because the heavier foundation soil below the toe of the embankment provides more resisting force to the failure surface than the wider embankments for a given height. The failure surface extends further out because if the shape of the failure surface is assumed to be circular, the failure surface must extend further out for narrow and tall embankments to accommodate the circular failure surface. Additionally, a narrower embankment yields a smaller length of the failure surface that is subjected to the pavement and traffic driving stresses. This same behavior is exhibited in the external seismic stability analysis shown in Figures B.16 through B.19. At embankment widths of 23 m (76 ft) and 34 m (112 ft), the seismic factors of safety are similar. However, the narrower embankment with a width of 11 m (36 ft) yields a higher factor of safety.

Overturning

For tall and narrow vertical embankments the overturning of the entire embankment at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation soil as a result of pseudo-static horizontal forces should be considered. These horizontal forces create an overturning moment about the toe at point O as shown in Figure B.19.

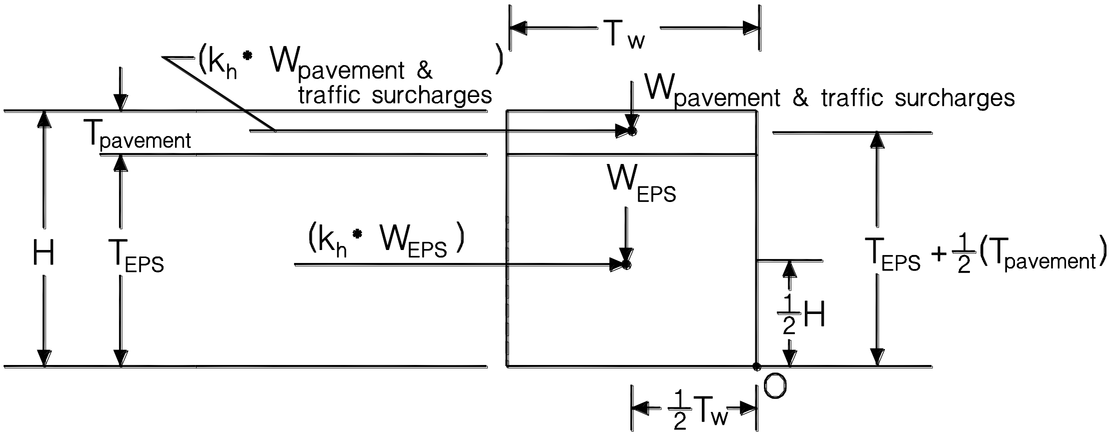

Figure B.19 Variables for determining the factor of safety against overturning of a vertical embankment due to pseudo-static horizontal forces used to represent an earthquake loading.

Vertical loads such as the weight of the EPS blocks and the pavement system and traffic surcharges will provide a stabilizing moment. A factor of safety against overturning of 1.2 is recommended for design purposes because overturning due to earthquake loading is a temporary loading condition. The factor of safety against overturning is expressed as follows:

$$FS = \frac{\sum \text{stabilizing moments}}{\sum \text{overturning moments}}$$

$$= \frac{\left(\frac{1}{2} * T_W\right) * \left(W_{EPS} + W_{pavement\ \&\ traffic\ surcharges}\right)}{\left[\left(\frac{1}{2} * H\right) * \left(k_h * W_{EPS}\right)\right] + \left[\left(T_{EPS} + \left(\frac{1}{2} * T_{pavement}\right)\right) * \left(k_h * W_{pavement\ \&\ traffic\ surcharges}\right)\right]} \qquad \textbf{(B.14)}$$

The soil pressure under a vertical embankment is a function of the location of the vertical and horizontal forces. It is generally desirable that the resultant of the vertical and horizontal forces be located within the middle third of the base of the embankment, i.e., eccentricity, e ≤ (T_W/6), to minimize the potential for overturning. If e = 0, the pressure distribution is rectangular. If e < (T_W/6), the pressure distribution is trapezoidal, and if e = (T_W/6), the pressure distribution is triangular. Therefore, as e increases, the potential for overturning of the embankment increases. Note that if e > (T_W/6), the minimum soil pressure will be negative, i.e., the foundation soil will be in tension. Therefore, separation between the vertical embankment and foundation soil may occur, which may result in overturning of the embankment, because soil cannot resist tension. This is the primary reason for ensuring that $e \leq (T_W/6)$. Equation (B.15) can be used to determine the location of the resultant a distance x from the toe of the embankment and Equation (B.16) can be used to determine e. Equation (B.17) can be used to estimate the maximum and minimum pressures under the embankment.

$$x = \frac{\sum \text{stabilizing moments} - \sum \text{overturning moments}}{\sum N} \qquad \textbf{(B.15)}$$

where x = location of the resultant of the forces from the toe of the embankment

ΣN = summation of normal stresses

$$e = \frac{T_W}{2} - x \qquad \textbf{(B.16)}$$

where e = eccentricity of the resultant of the forces with respect to the centerline of the embankment

T_W = top width of the embankment

$$q = \frac{\sum N}{T_w}\left(1 \pm \frac{6e}{T_w}\right) \leq q_a \qquad \textbf{(B.17)}$$

where q = soil pressure under the embankment

q_a = allowable soil pressure

The soil pressures should not exceed the allowable soil pressure, q_a.

Step 8 - HYDROSTATIC UPLIFT (FLOTATION)

EPS-block geofoam used as lightweight fill usually has a density that is approximately 1 percent of the density of earth materials. Because of this extraordinarily low density, the potential for hydrostatic uplift (flotation) of the entire embankment at the interface between the bottom of the assemblage of EPS blocks and the foundation soil must be considered in external stability evaluations.

For the case of the vertical height of accumulated water to the bottom of the embankment at the start of construction, h, equal to the vertical height of tailwater to bottom of the embankment at the start of construction, h', (see Figure B.20), the factor of safety against upward vertical uplift of the embankment is:

$$FS = \frac{W_{EPS} + W_W + W'_W + O_{REQ}}{\gamma_W * (h + S_{total}) * B_W} \quad \textbf{(B.18)}$$

where W_{EPS} = weight of EPS-block geofoam embankment,

W_w = vertical component of weight of water on the embankment face above the base of the embankment on the accumulated water side,

W'_w = vertical component of weight of water on the face of the embankment on the tailwater side,

γ_W = unit weight of water,

S_{total}=total settlement as defined by Equation (B.2),

B_W = bottom embankment width, and

O_{REQ} = additional overburden force required above the EPS blocks to obtain the desired factor of safety.

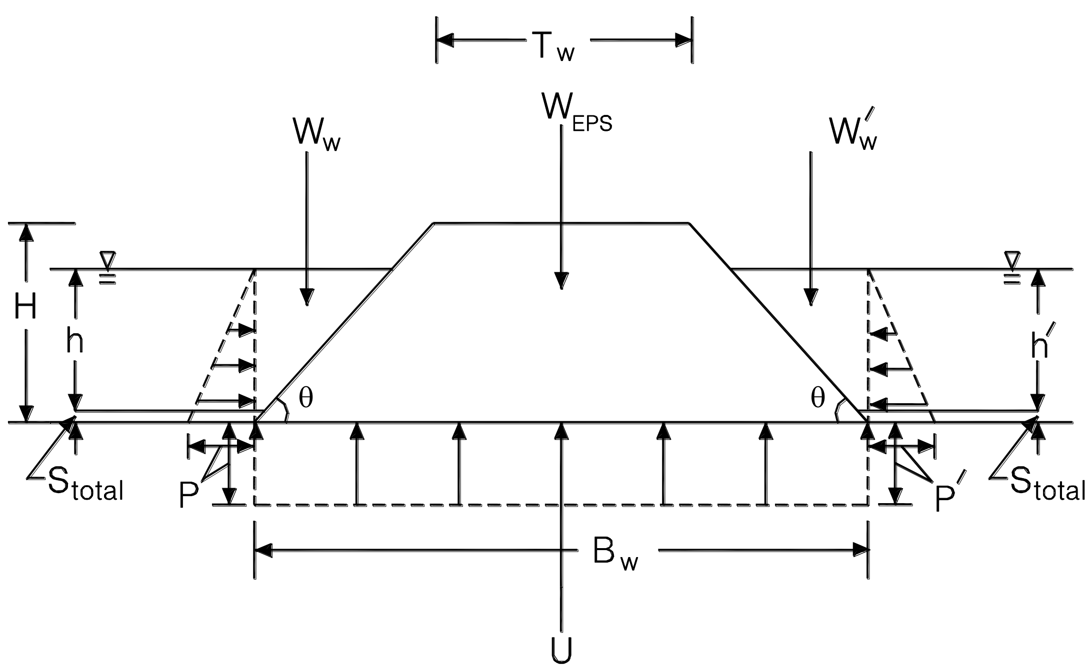

Figure B.20. Variables for determining hydrostatic uplift for the case of water equal on both sides of the embankment.

Equation (B.18) can be used to obtain the value of O_{REQ} required to obtain any desired factor of safety. A factor of safety against hydrostatic uplift of 1.2 is recommended for design purposes because hydrostatic uplift is a temporary loading condition and a factor of safety of 1.2 is being used for other temporary loading conditions in the design procedure, such as seismic loading. Therefore, the value of O_{REQ} corresponding to a factor of safety of 1.2 and the various embankment geometries considered during this study was calculated to develop design charts for hydrostatic uplift. This rearrangement results in the following expression:

$$O_{REQ} = \left[1.2 * (\gamma_W * (h + S_{total}) * B_W)\right] - \left[(W_{EPS} + W_W + W'_W)\right] \qquad \textbf{(B.19)}$$

The value of O_{REQ} is the additional overburden force required above the EPS blocks to obtain the desired factor of safety in Equation (B.18) or a factor of safety of 1.2 in Equation (B.19). The components usually contributing to O_{REQ} are the weight of the pavement system and the cover soil on the embankment sideslopes. The weight of pavement system can be taken to be equal to the pavement surcharge of 21.5 kPa (450 lbs/ft²) used previously for external bearing capacity and slope stability multiplied by the width, T_w, or it can be calculated by multiplying the unit weight of the pavement system, $\square_{pavement}$, by the pavement thickness, $T_{pavement}$, and width, T_W. The traffic surcharge of 11.5 kPa (240 lbs/ft²) used previously is not included in O_{REQ} because it is a live or transient load and may not be present at the time of the design hydrostatic uplift condition. The weight of the cover soil imposes overburden weight on the EPS blocks on both side slopes of the embankment and can be calculated using the variables in Figure B.21. Therefore, to ensure the desired factor of safety in Equation (B.19) is satisfied for hydrostatic uplift, the calculated value of O_{REQ} should be less than the sum of the pavement, cover soil, and other weights applied to the embankment as shown below:

$$O_{REQ} < (\square_{pavement} * T_{pavement} * T_W) + W_{cover} + W_{other} \qquad \textbf{(B.20)}$$

Design charts, see Figures B.22 through B.25 were prepared for each embankment geometry because calculation of W_{EPS}, W_W, and W'_W is cumbersome. The design charts simplify the process because a design engineer can enter a design chart and obtain the value of O_{REQ} corresponding to a factor of safety of 1.2. The values of O_{REQ} provided by the design charts are based on the assumption that the EPS blocks extend for the full height of the embankment, i.e., $H = T_{EPS}$. Therefore, the weight of the EPS equivalent to the height of the pavement system times the unit weight of the EPS must be subtracted in the result of O_{REQ} in Equation (B.20) as shown below:

$$O_{REQ} < \left(\gamma_{pavement} * T_{pavement} * T_W\right) - \left(\gamma_{EPS} * T_{pavement} * T_W\right) + W_{cover} + W_{other} \qquad \text{(B.21)}$$

The accumulated water level indicated in the design charts is the sum of the vertical accumulated water level to the bottom of the embankment at the start of construction and the estimated total settlement, $h+S_{total}$. The design engineer then compares this value of O_{REQ} with the weight of the pavement system and cover soil.

Equal water level on both sides of the embankment is the worst-case scenario and construction measures should be taken to try avoid the situation of equal water level being created on both sides of the embankment. Figures B.22 through B.25 present the design charts for all of the embankment geometries considered during this study for equal upstream and tailwater levels and uplift at the EPS block/foundation soil interface. The values of O_{REQ} shown in Figures B.22 through B.25 is the required weight of material over the EPS blocks in kN per linear meter of embankment length. The accumulated water level is the total water depth to include the estimated total settlement, i.e., $h+S_{total}$. The design charts only extend to a maximum ratio of accumulated water level to embankment height of 0.5, which means the total water depths to include the estimated total settlement is limited to 50 percent of the embankment height, because an embankment with a high accumulated water level is essentially a dam structure that may require unreasonable overburden forces on top of the EPS blocks to obtain the desired factor of safety.

Figure B.22 Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with tailwater level equal to upstream water level, 4H:1V embankment slope, and three road widths.

For the case of the vertical height of tailwater to bottom of the embankment, $h'+S_{total}$, equals zero (see Figure B.26), and Equation (B.25) can be used to obtain the factor of safety against hydrostatic uplift.

$$FS = \frac{W_{EPS} + W_W + O_{REQ}}{\frac{1}{2} * \gamma_W * (h+S_{total}) * B_W} \qquad \textbf{(B.25)}$$

Figure B.26. Variable for determining hydrostatic uplift analysis for the case of water on one side of the embankment only.

Equation (B.25) can be rearranged and used to obtain the value of O_{REQ} required to obtain the desired factor of safety of 1.2 against hydrostatic uplift. Therefore, the value of O_{REQ} corresponding to a factor of safety of 1.2 and the various embankment geometries considered during this study was calculated to develop design charts for hydrostatic uplift with zero tailwater as shown below:

$$O_{REQ} = \left[1.2 * \left(\frac{1}{2} * \gamma_W * (h + S_{total}) * B_W \right) \right] - \left[(W_{EPS} + W_W) \right] \quad \textbf{(B.26)}$$

Figures B.27 through B.30 present the design charts for all of the embankment geometries considered during this study for a total tailwater depth of zero. These charts can be used to estimate the value of O_{REQ} required to obtain the desired factor of safety of 1.2 against hydrostatic uplift at the EPS block/foundation soil interface. The same conditions used to generate the design charts for the equal upstream and tailwater levels were used to develop the design charts for zero tailwater.

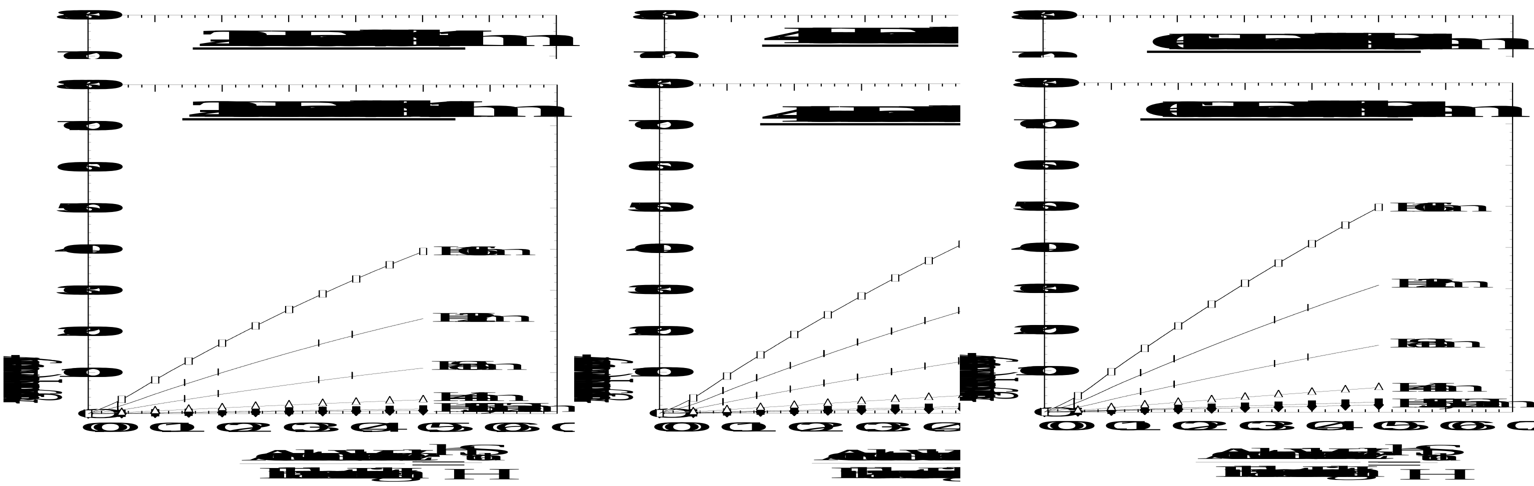

Figure B.27 Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with no tailwater, 4H:1V embankment slope, and three road widths.

Step 9 - TRANSLATION AND OVERTURNING DUE TO WATER (HYDROSTATIC SLIDING AND OVERTURNING)

Because of the extraordinarily low density of EPS-block geofoam, the potential for translation (horizontal sliding) of the entire embankment at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation soil due to an unbalanced water pressure must be considered. This scenario is similar to the hydrostatic uplift case with zero tailwater but the failure mode is sliding and not uplift. Additionally, for vertical geofoam embankments, the potential for overturning of the entire embankment about one of the bottom corners of the embankment at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation soil due to an unbalanced water pressure must be considered.

The tendency of the entire embankment to slide under an unbalanced water pressure is resisted primarily by EPS/foundation soil interface friction. Although the friction angle, δ, for this interface is relatively high (it approaches the Mohr-Coulomb angle of internal friction, ϕ, of the foundation soil), the resisting force (which equals the dead weight times the tangent of δ) is small because the dead weight of the overall embankment is small. Consequently, the potential for the entire embankment to slide under an unbalanced water pressure loading is a possible failure mechanism and the potential for translation (horizontal sliding) of the entire embankment in a direction perpendicular to the proposed road alignment should be considered.

For the case of no interface cohesion along the sliding surface, which is typical for geosynthetic interfaces, the expression for factor of safety against hydrostatic sliding simplifies to the following:

$$FS=\frac{\left[\left(W_{EPS}+W_W+O_{REQ}\right)-\left(\frac{1}{2}\left((h+S_{total})*\gamma_W\right)*\left(B_W\right)\right)\right]*\tan\delta}{\frac{1}{2}\left(\gamma_W*(h+S_{total})^2\right)} \quad \textbf{(B.27)}$$

where δ = interface friction angle along the sliding surface,

γ_W = unit weight of water,

h = vertical height of accumulated water to bottom of embankment,

S_{total}=total settlement as defined by Equation (B.2), and

B_W = bottom of embankment width, and

the other variables were previously defined in Section B.4.6.1.

For a factor of safety of 1.2 and solving for O_{REQ}, Equation (B.27) becomes:

$$O_{REQ}=\frac{1.2(\frac{1}{2})\left(\gamma_W*(h+S_{total})^2\right)}{\tan\delta}+\left(\frac{1}{2}\left((h+S_{total})*\gamma_W\right)*\left(B_W\right)\right)-W_{EPS}-W_W \quad \textbf{(B.28)}$$

Equation (B.28) can be used to obtain the required value of O_{REQ} for a factor of safety of 1.2 against hydrostatic sliding. To ensure the desired factor of safety in Equation (B.28) is satisfied for hydrostatic sliding, the calculated value of O_{REQ} should be less than the sum of the pavement, cover soil, and other weights applied to the embankment as shown by Equation (B.20). Figures B.31 through B.34 present the design charts for all of the embankment geometries considered during this study for horizontal sliding caused by accumulation of water on one-side of the embankment. These charts can be used to estimate the value of O_{REQ} per linear meter of embankment length required to obtain the desired factor of safety of 1.2 against hydrostatic sliding at the EPS block/foundation soil interface as was demonstrated for the hydrostatic uplift design charts.

The design charts are based on the assumption that the EPS blocks extend the full height of the embankment, i.e., $H = T_{EPS}$. Therefore, the weight of the EPS equivalent to the height of the pavement system times the unit weight of the EPS must be subtracted in the result of O_{REQ} as shown by Equation (B.21). The accumulated water level used in the design charts is the sum of the vertical accumulated water level to the bottom of the embankment at the start of construction and the estimated total settlement, i.e., $h+S_{total}$.

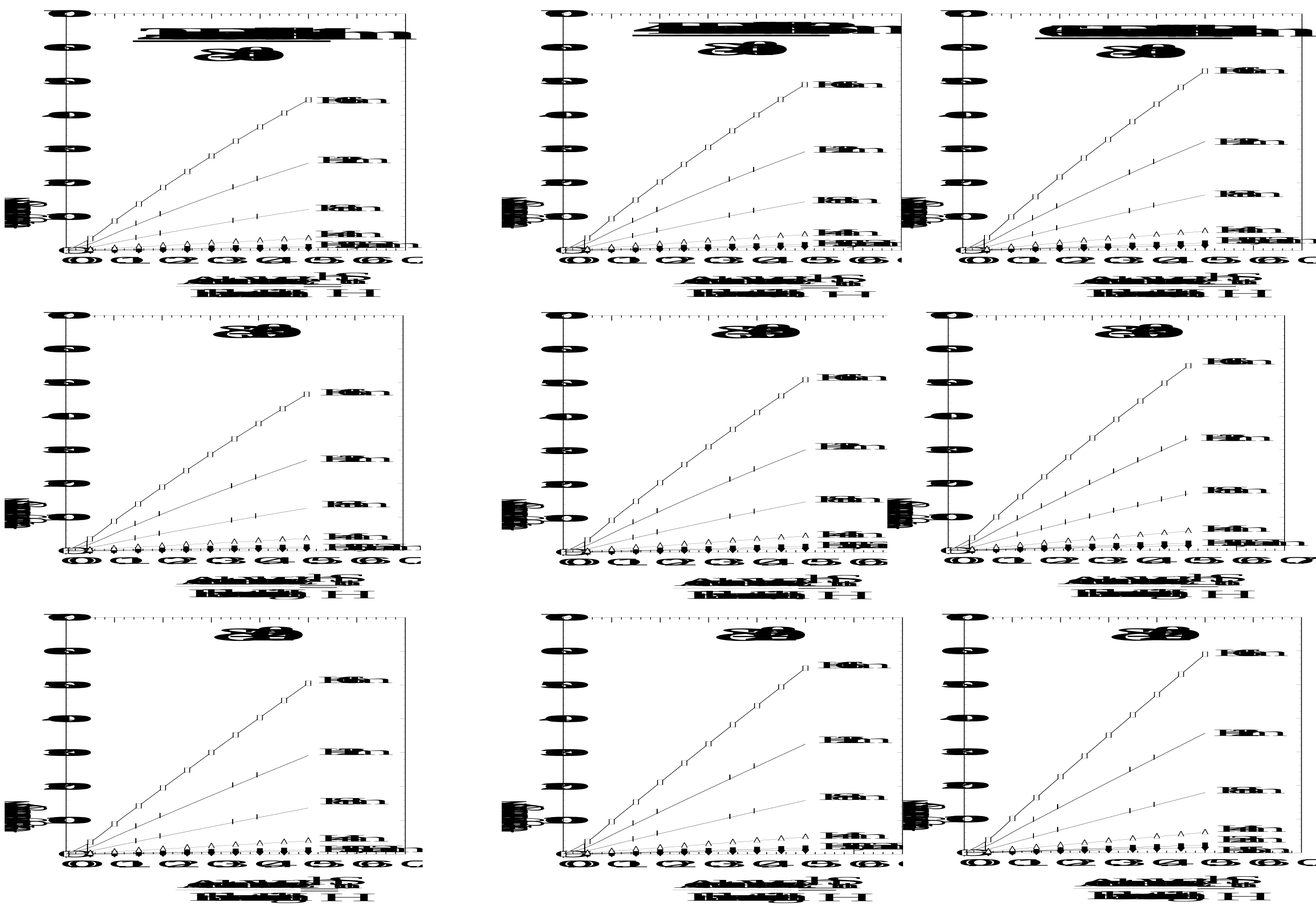

Figure B.31. Hydrostatic sliding (translation due to water) design for a factor of safety of 1.2 with no tailwater, 4H:1V embankment slope, and three road widths for various interface friction angles.

Overturning

For vertical embankments, the tendency of the entire embankment to overturn at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation soil is a result of an unbalanced water pressure acting on the embankment. Overturning may be critical for tall and narrow vertical embankments. These horizontal forces create an overturning moment about the toe at point O as shown in Figure B.35. The worst case scenario is for water accumulating on only one side of the embankment as shown in Figure 5.52. Vertical loads, such as the weight of the EPS blocks, the pavement system, and traffic surcharges, will provide a stabilizing moment. As described for the analysis of hydrostatic uplift, O_{REQ} is the additional overburden force required above the EPS blocks to obtain the desired factor of safety.

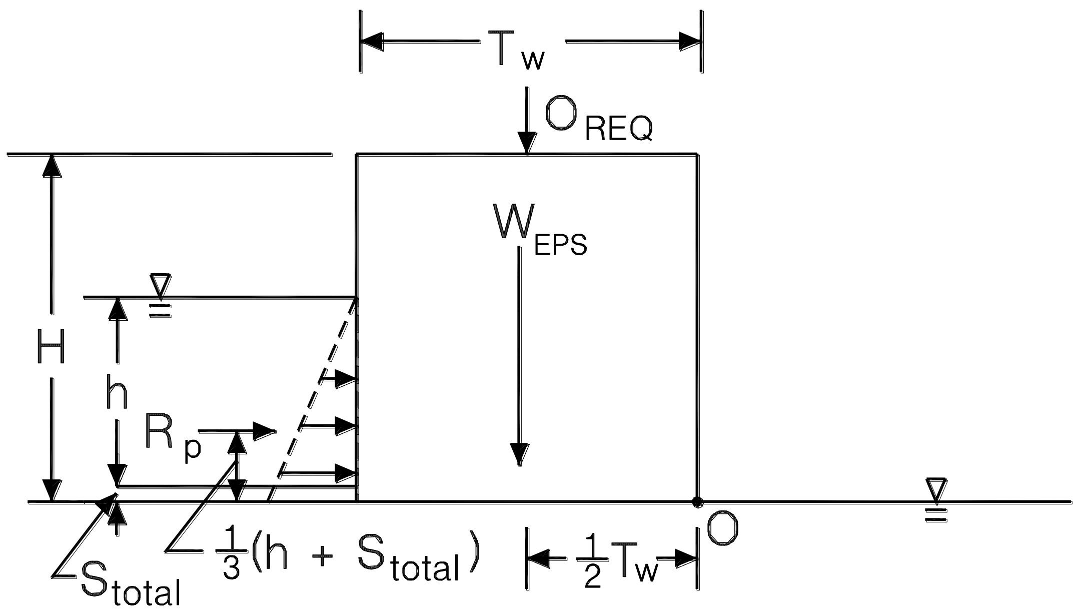

Figure B.35 Variables for determining the factor of safety against overturning due to hydrostatic horizontal forces for the case of water on one side of the embankment.

The factor of safety against overturning due to horizontal hydrostatic forces is expressed as

$$FS = \frac{\sum \text{stabilizing moments}}{\sum \text{overturning moments}} = \frac{(\frac{1}{2} * T_W) * (W_{EPS} + O_{REQ})}{\frac{1}{3}\left(h + S_{total}\right) * R_p} \qquad \textbf{(B.29)}$$

A factor of safety against hydrostatic overturning of 1.2 is recommended for design purposes because hydrostatic overturning is a temporary loading condition and a factor of safety of 1.2 is being used for other temporary loading conditions, such as hydrostatic uplift and sliding and seismic loading. For a factor of safety of 1.2 and solving for O_{REQ}, Equation (B.29) becomes

$$O_{REQ} = \frac{1.2 * \left(\frac{1}{3}\right) * \left(h + S_{total}\right) * \left(R_p\right)}{\left(\frac{1}{2} * T_W\right)} - W_{EPS} \qquad \textbf{(B.30)}$$

Equation (B.30) can be used to obtain the required value of O_{REQ} for a factor of safety of 1.2 to resist hydrostatic overturning.

The resultant of the vertical and horizontal forces should be checked to verify that the resultant is located within the middle third of the base, i.e., eccentricity, $e \leq (B_w/6)$, to minimize the potential for the wall to overturn. Equations (B.15) and (B.16) can be used to determine e. Additionally, the maximum and minimum soil pressures under the embankment should not exceed the allowable soil pressure, q_a. Equation (5.17) can be used to determine the maximum and minimum pressures under the embankment.

Step 10 - TRANSLATION AND OVERTURNING DUE TO WIND

Translation due to wind is an external failure mechanism that is unique to embankments containing EPS-block geofoam because of the extremely low density of EPS blocks compared to other types of lightweight fill. Additionally, for vertical geofoam embankments, the potential for overturning of the entire embankment about one of the bottom corners of the embankment at the interface between the bottom of the assemblage of EPS blocks and the underlying foundation soil due to horizontal wind forces must be considered.

However, the findings of the NCHRP Project HR 24-11 revealed that the wind pressures obtained from the current wind analysis equations may be too conservative because there is no documented sliding failure of an embankment containing EPS-block geofoam due to wind loading. Therefore, based on the results of the NCHRP Project HR 24-11 and the absence of documented sliding failure due to wind loading, it is recommended that the translation due to wind failure mechanism not be considered until further research is performed on the applicability of the existing design equations to EPS-block geofoam embankments. However, wind loading should be considered if the embankment will be subjected to hurricane force winds.

A wind analysis procedure is presented in the NCHRP report for completeness and because future research may develop a more realistic design procedure for evaluating the potential for basal translation (sliding) due to wind loading especially under Atlantic hurricane conditions. Development of new design procedure is a topic for future research.

Steps 11-14 – Internal Stability

After external stability, internal stability, e.g., translation due to water and wind, seismic stability, and load bearing, of the embankment is evaluated. This evaluation is illustrated in Steps 11 through 14 in the flow chart in Figure B.4 with the accompanying tolerable criteria. Design for internal stability of an EPS-block geofoam embankment includes consideration of SLS issues such as the proper selection and specification of EPS properties so the geofoam mass can provide adequate load bearing capacity to the overlying pavement system without excessive settlement and ULS issues such as translation due to water (hydrostatic sliding), translation due to wind, and seismic stability.

Step 11 - TRANSLATION DUE TO WATER (HYDROSTATIC SLIDING)

Internal stability for translation due to water consists of verifying that adequate shear resistance is available between EPS block layers and between the pavement system and the EPS blocks to withstand the forces of an unbalanced water head. Equation (B.28) can be used to determine the required overburden force, O_{REQ}, to achieve a factor of safety of 1.2 against horizontal sliding. The components usually contributing to O_{REQ} are the weight of the pavement system and the cover soil on the embankment sideslopes. Therefore, to ensure the desired factor of safety, the calculated value of O_{REQ} should be less than the sum of the pavement and cover soil weights as shown in Equation (B.20). Figures B.31 through B.34 can be used to determine the required overburden force, O_{REQ}, to achieve a factor of safety of 1.2 against horizontal sliding. The accumulated water level used in the design charts is the sum of the height from the top of the accumulated water level to the interface that will be analyzed and the estimated total settlement, i.e., $h+S_{total}$. Figures B.30 through B.31 are based on the assumption that the EPS blocks extend the full height of the embankment, i.e., $H = T_{EPS}$. Therefore, the weight of the EPS equivalent to the height of the pavement system times the unit weight of the EPS must be subtracted in the result of O_{REQ} as shown by Equation (B.21).

The thickness of EPS blocks typically range between 610 mm (24 in.) to 1,000 mm (39 in.). Therefore, if the water level to be analyzed is less than about 610 mm (24 in.), an internal stability analysis for hydrostatic sliding is not required.

Step 12 - TRANSLATION DUE TO WIND

Internal stability for translation due to wind consists of verifying that adequate shear resistance is available between EPS block layers and between the pavement system and EPS blocks to withstand the design wind forces. However, the findings of the NCHRP Project HR 24-11 revealed that the wind pressures obtained from the current wind analysis equations may be too conservative because there is no documented sliding failure of an embankment containing EPS-block geofoam due to wind loading. Therefore, based on the results of the NCHRP Project HR 24-11 and the absence of documented sliding failure due to wind loading, it is recommended that the translation due to wind failure mechanism not be considered until further research is performed on the applicability of the existing design equations to EPS-block geofoam embankments. However, wind loading should be considered if the embankment will be subjected to hurricane force winds.

A wind analysis procedure is presented in the NCHRP report for completeness and because future research may develop a more realistic design procedure for evaluating the potential for basal translation (sliding) due to wind loading especially under Atlantic hurricane conditions. Development of new design procedure is a topic for future research.

Step 13 - INTERNAL SEISMIC STABILITY

This section focuses on the effect of seismic forces on the internal stability of EPS-block geofoam trapezoidal embankments. The main difference in this analysis and the external seismic stability analysis is that sliding is assumed to occur only within the geofoam embankment or along an EPS interface. This analysis uses a pseudo-static slope stability analysis and non-circular failure surfaces through the EPS or the EPS interface at the top or bottom of the embankment. The steps in an internal pseudo-static stability analysis are:

1. Identify the potential critical static failure surfaces, i.e., the static failure surface with the lowest factor of safety, that passes through the EPS embankment or an EPS interface at the top or bottom of the EPS. This is accomplished by measuring the interface strength between EPS

blocks and the interfaces at the top and bottom of the EPS blocks and determining which of the interfaces yield the lowest factor of safety. In the analyses presented subsequently, it was found that the critical interface varies as the interface friction angle varies. Therefore, the factor of safety for all three interfaces should be calculated unless one of the interfaces exhibits a significantly lower interface friction angle than the other two interfaces and can be assumed to control the internal stability.

2. Determine the appropriate value of horizontal seismic coefficient to be applied at the center of gravity of the slide mass delineated by the critical static failure surface. Estimation of the horizontal seismic coefficient can utilize empirical site response relationships and the horizontal acceleration within the embankment can be assumed to vary linearly between the base and crest values.

3. Calculate the internal seismic factor of safety, FS', for the critical internal static failure surface and ensure that it meets the required value of 1.2. A minimum factor of safety of 1.2 is recommended for internal seismic stability of EPS geofoam embankments because earthquake shaking is a temporary loading. The seismic factor of safety for the EPS/pavement system interface is calculated using a sliding block analysis and a pseudo-static stability analysis is used for the EPS/EPS and EPS/foundation soil interfaces. The pseudo-static factor of safety should be calculated using a slope stability method that satisfies all conditions of equilibrium.

A typical cross-section through a 12.2 m (40 ft) high EPS trapezoidal embankment with side-slopes of 2H:1V that was used in the pseudo-static internal stability analyses is shown in Figure B.36. The material layer at the top of the embankment is used to model the pavement and traffic surcharges and has a unit weight of 71.8 kN/m^3 (460 lbf/ft^3). The soil cover is 0.46 m (1.5 ft) thick so the stress applied by this soil cover equals 0.46 m times the unit weight of 71.8 kN/m^3 (460 lbf/ft^3) or 33.0 kPa (690 lbs/ft^2). A stress of 33.0 kPa (690 lbs/ft^2) corresponds to the sum of the design values of pavement surcharge (21.5 kPa (450 lbs/ft^2)) and traffic surcharge (11.5 kPa (240 lbs/ft^2)) used previously in the external seismic slope stability analyses of trapezoidal embankments. The pavement and traffic surcharges had to be modeled with a high unit weight material layer instead of a surcharge. A surcharge could not be used because a seismic coefficient cannot be applied to a surcharge in limit equilibrium stability analyses. The soil cover on the side slopes of the embankment is also 0.46 m (1.5 ft) thick, which is typical for the side slopes, and is assigned a typical moist unit weight of 18.9 kN/m^3 (120 lbf/ft^3).

Figure B.36 also presents the three failure surfaces or modes considered in the internal seismic stability analyses. It can be seen that the first failure mode, i.e., Mode I, corresponds to translational sliding at the pavement system/EPS interface at the top of the EPS blocks. This interface could involve a separation material such as a geomembrane placed over the EPS to protect the EPS against hydrocarbon spills or a geotextile to provide separation between the pavement system and the EPS. If a geosynthetic is not used on the top of the EPS blocks, the interface would consist of a pavement system material overlying the EPS blocks or a separation layer material that is not a geosynthetic placed between the pavement system and EPS blocks. The second failure mode, i.e., Mode II, corresponds to translational sliding between adjacent layers of EPS blocks, e.g., at the top of the last layer of EPS blocks, and thus consists of sliding along an EPS/EPS interface. The third failure mode, i.e., Mode III, corresponds to translational sliding at the EPS/foundation soil interface at the base of the EPS blocks. If a geosynthetic is not used at the base of the EPS blocks, the interface would consist of EPS overlying either a leveling soil or the in situ foundation soil. All three of these failure modes were assumed to initiate at or near the embankment centerline because it is anticipated that a pavement joint or median will exist near the embankment centerline in the field and provide a discontinuity that allows part of the embankment to displace. In addition, the embankment is symmetric.

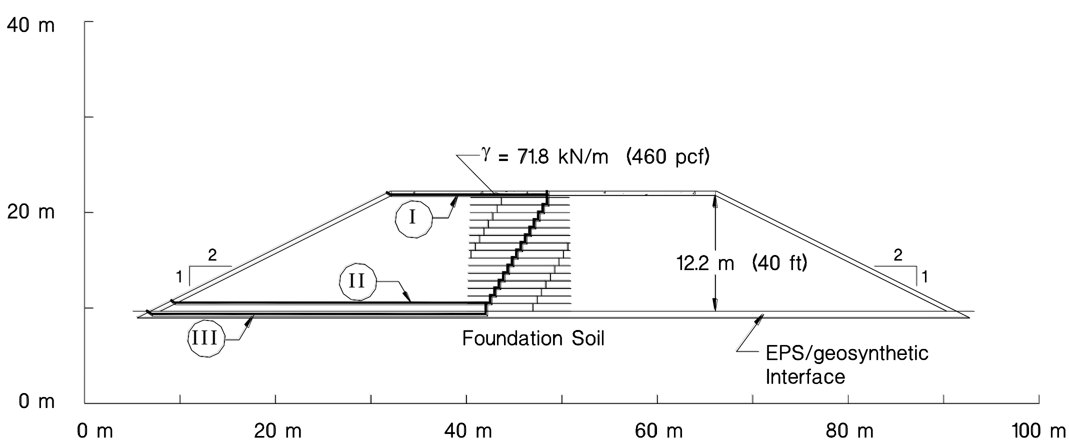

Figure B.36. Typical trapezoidal cross-section used in seismic internal slope stability analyses with the three applicable failure modes.

Slope stability analyses were conducted on a range of trapezoidal embankment geometries to investigate the effect of embankment height (3.1 m (10 ft) to 12.2 m (40 ft)), slope inclination (2H:1V, 3H:1V, and 4H:1V), and roadway width (11 m (36 ft), 23 m (76 ft), and 34 m (112 ft)) on internal seismic slope stability and to develop a design chart to facilitate internal design of trapezoidal roadway embankments that utilize geofoam. Three seismic coefficients, low (0.05), medium (0.10), and high (0.20), were used for each roadway embankment.

The internal seismic stability design chart in Figure B.37 presents the seismic factor of safety for each seismic coefficient as a function of interface friction angle. This chart can be used for any of the geometries considered during this study, i.e., embankment heights of 3.1 m (10 ft) to 12.2 m (40 ft), slope inclinations of 2H:1V, 3H:1V, and 4H:1V, and roadway widths of 11 m, 23 m, and 34 m (36, 76, and 112 feet), even though it is based on a side-slope inclination of 2H:1V, a 2-lane roadway, and an embankment height from 3.1 m (10 ft) to 12.2 m (40 ft) because this represents the worst-case scenario and is not sensitive to the range of geometries considered. It can be seen that an EPS embankment will exhibit a suitable seismic factor of safety if the minimum interface friction angle exceeds approximately 15 degrees. However, an important aspect of Figure B.37 is to develop the most cost-effective internal stability design by selecting the lowest interface friction angle for each interface that results in a seismic factor of safety of greater than 1.2. For example, a lightweight geotextile can be selected for the EPS/foundation interface because the interface only needs to exhibit a friction angle greater than 10 degrees. More importantly, the EPS/EPS interface within the EPS also only needs to exhibit a friction angle greater than 10 degrees, which suggests that mechanical connectors are not required between EPS blocks for internal seismic stability because the interface friction angle for an EPS/EPS interface is approximately 30 degrees. In summary, it appears that internal seismic stability will be controlled by the shear resistance of the pavement system/EPS interface.

Figure B.37. Design chart for internal seismic stability of EPS trapezoidal embankments.

A typical cross-section through a vertical EPS embankment used in the internal static stability analyses is shown in Figure B.38. This cross-section is similar to the cross-section used for static analyses of vertical embankments in Figure B.10 but differs from the cross-section used for the static analyses of trapezoidal embankments in Figure B.6 because the surcharge used to represent the pavement and traffic surcharges is replaced by placing a 0.61 m (2 ft) thick soil layer on top of the embankment with a unit weight of 54.1 kN/m^3 (345 lbs/ft^3). The soil layer is 0.61 m (2 ft) thick to represent the minimum recommended pavement section thickness discussed in Section B.3. Therefore, the vertical stress applied by this soil layer equals 0.61 m (2 ft) times the increased unit weight or 33.0 kN/m^2 (690 lbs/ft^2). A vertical stress of 33.0 kN/m^2 (690 lbs/ft^2) corresponds to the sum of the design values of pavement surcharge (21.5 kN/m^2 (450 lbs/ft^2)) and traffic surcharge (11.5 kN/m^2 (240 lbs/ft^2)) used previously for external bearing capacity and static slope stability of trapezoidal embankments. The surcharge in Figure B.6 had to be replaced by an equivalent soil layer for the seismic slope stability analysis because a seismic coefficient cannot be applied to a surcharge in limit equilibrium stability analyses.

Figure B.38. Typical cross-section used in seismic internal slope stability analyses for vertical embankments with the three applicable failure modes.

Figure 6.7 also presents the three failure modes considered in the internal seismic stability analyses for vertical geofoam embankments. These failure modes are similar to the three failure modes analyzed in seismic internal slope stability analysis of trapezoidal embankments and a description of each is included in Section B.5.4.1.1.

Slope stability analyses were conducted on a range of vertical embankment geometries to investigate the effect of embankment height (3.1 m (10 ft) to 12.2 m (40 ft)) and roadway width (11 m (36 ft), 23 m (76 ft), and 34 m (112 ft)) on internal seismic slope stability. The results of these analyses were used to develop design charts to facilitate internal design of roadway embankments with vertical walls that utilize geofoam. Three seismic coefficients, low (0.01), medium (0.10), and high (0.20), were used for each roadway embankment.

The internal seismic stability design chart for vertical embankments in Figure B.39 presents the seismic factor of safety for each seismic coefficient as a function of interface friction angle. This chart provides estimates of seismic internal factor of safety for vertical embankments with any of the geometries considered during this study, i.e., embankment heights of 3.1 m (10 ft) to 12.2 m (40 ft) and roadway widths of 11 m, 23 m, and 34 m (36, 76, and 112 feet), even though it is based on a roadway width of 11 m (36 ft) and an embankment height from 3.1 m (10 ft).

It can be seen that an EPS embankment will exhibit a suitable seismic factor of safety if the minimum interface friction angle exceeds approximately 15 degrees, which is similar for trapezoidal embankments (see Figure B.7). However, an important aspect of Figure B.39 is that it can be used to develop the most cost-effective internal stability design by selecting the lowest interface friction angle for each interface that results in a seismic factor of safety of greater than 1.2. For example, a lightweight geotextile can be selected for the EPS/foundation interface because the interface only needs to exhibit a friction angle greater than 15 degrees. More importantly, the EPS/EPS interface within the EPS also only needs to exhibit a friction angle greater than 15 degrees, which suggests that mechanical connectors are not required between EPS blocks for internal seismic stability because the interface friction angle for an EPS/EPS interface is approximately 30 degrees. In summary, as with trapezoidal embankments, it appears that internal seismic stability will be controlled by the shear resistance of the pavement system/EPS interface.

Figure B.39. Design chart for internal seismic stability of EPS vertical embankments.

Step 14 - LOAD BEARING

The primary internal stability issue for EPS-block geofoam embankments is the load bearing of the EPS geofoam mass. A load bearing capacity analysis consists of selecting an EPS type with adequate properties to support the overlying pavement system and traffic loads without excessive EPS compression that could lead to excessive settlement of the pavement surface. The design approach utilized herein is an explicit deformation-based design methodology. It is based on the elastic limit stress, σ_e, to evaluate the load bearing of EPS.

Table B.8 provides the minimum recommended values of elastic limit stress for various EPS densities. The use of the elastic limit stress values indicated in Table B.8 is slightly conservative because the elastic limit stress of the block as a whole is somewhat greater than these minimums, but this conservatism is not unreasonable and would ensure that no part of a block (where the density might be somewhat lower than the overall average) would become overstressed.

Table B.8. Minimum allowable values of elastic limit stress and initial tangent Young's Modulus for the proposed AASHTO EPS material designations.

The procedure for evaluating the 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*, or *EPS100*.
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 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 that exhibits an elastic limit stress that is greater than the calculated or required elastic limit stress at the depth being considered. The load bearing design procedure can be divided into two parts. Part 1 consists of Steps 1 through 8 and focuses on the determination of the 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 (see steps above). Part 2 consists of Steps 9 through 13 and focuses on the determination of the traffic and gravity load stresses applied at various depths within the EPS blocks and selection of the appropriate EPS for use at these various depths within the embankment. Each of the design steps are subsequently described.

Select an EPS type from Table B.8 that exhibits an elastic limit stress greater than or equal to the required σ_e determined in Step 12. *EPS40* is not recommended for directly beneath the pavement system (see Step 7) but can be used at depths below 610 mm (24 in) in the embankment if the required elastic limit stress is less than 40 kPa (5.8 lbs/in^2). However, for constructability reasons, it is recommended that no more than two different EPS block types be used

Step 15 – Pavement System Design

Step 15 involves designing the pavement system and verifying that the EPS type selected in Step 14 directly below the pavement system will provide adequate support for the pavement system. However, pavement design aspects must also be considered in Step 2 to obtain a preliminary pavement system design.

The objective of pavement system design is to select the most economical arrangement and thickness of pavement materials that will be founded on EPS blocks. The design criterion is to prevent premature failure of the pavement system, as defined by rutting, cracking, or similar criterion.

Traditional pavement design procedures may be used by considering the EPS to be an equivalent soil subgrade. The resilient modulus or equivalent California Bearing Ratio (CBR) value of the EPS can be used in the design procedure. A summary of these design parameters is provided in Table B.2.

Table B.2. Equivalent soil subgrade values of EPS-block geofoam for pavement design.

As part of the research reported herein, pavement design catalogs were developed to facilitate pavement system design. A design catalog is a means for designers to obtain expedient pavement layer thicknesses that can be used to design the pavement system. The AASHTO 1993 design procedure (*30*) was used to develop the flexible and rigid pavement design catalogs.

Step 16 – Comparison of Applied Vertical Stress

Step 16 involves verifying that the vertical stress applied by the preliminary pavement system (Step 2) and the final pavement system (Step 15) are in agreement. If the vertical stress of the final pavement system is greater than the vertical stress imposed by the preliminary pavement, the design procedure may have to be repeated at Step 4 with the higher vertical stress as shown by Remedial Procedure G of Figure B.4. If the vertical stress applied by the final pavement system is less than the applied vertical stress from the preliminary pavement system, the design procedure will have to be repeated at Step 8 as shown by Remedial Procedure G of Figure B.4 in the flow chart. If the vertical stress applied by the final pavement system is in agreement with the preliminary pavement system, the resulting embankment design can be used for construction purposes.

SUMMARY

Design charts were developed as part of this research to aid in obtaining a technically optimal design. Therefore, these design aids can be used with the proposed design algorithm to assist in developing a cost efficient design. As indicated previously, the design algorithm will assist the designer in developing a cost efficient design.

This guideline was prepared as part of the National Cooperative Highway Research Program (NCHRP) Project HR 24-11 titled "Guidelines for Geofoam Applications in Embankment Projects" that was administered by the Transportation Research Board (TRB). The report provides the commentary accompanying this guideline and the design charts and equations used in the guideline. It is suggested that users of this guideline review the accompanying report for the necessary technical background. This guideline is intended to be used in conjunction with the provisional material and construction standard that is presented in Appendix C of the NCHRP report.

Acknowledgments

References

1. Holtz, R. D., "Treatment of Problem Foundations for Highway Embankments." *NCHRP Synthesis 147,* Transportation Research Board, Washington, D.C. (1989) 72 pp.
2. "Utilisation de Polystyrene Expanse en Remblai Routier; Guide Technique." Laboratoire Central Ponts et Chaussées/SETRA, France (1990) 18 pp.
3. "Merkblatt für die Verwendung von EPS-Hartschaumstoffen beim Bau von Straßendämmen." Forschungsgesellschaft für Straßen- und Verkehrswesen, Arbeitsgruppe Erd- und Grundbau, Köln, Deutschland (1995) 27 pp.
4. "Code of Practice; Using Expanded Polystyrene for the Construction of Road Embankments." BASF AG, Ludwigshafen, Germany (1995) 14 pp.
5. "Design and Construction Manual for Lightweight Fill with EPS." The Public Works Research Institute of Ministry of Construction and Construction Project Consultants, Inc., Japan (1992) Ch. 3 and 5.
6. "Guidelines on the Use of Plastic Foam in Road Embankment." Public Roads Administration, Road Research Laboratory, Oslo, Norway (1980) 2 pp.
7. "Expanded Polystyrene Used in Road Embankments - Design, Construction and Quality Assurance." *Form 482E,* Public Roads Administration, Road Research Laboratory, Oslo, Norway (1992) 4 pp.
8. "Material Requirements for Expanded Polystyrene Used in Road Embankments." *Form 483E,* Public Roads Administration, Road Research Laboratory, Oslo, Norway (1992) 2 pp.
9. "Quality Control of Expanded Polystyrene Used in Road Embankments." *Form 484E,* Public Roads Administration, Road Research Laboratory, Oslo, Norway (1992) 4 pp.
10. Sanders, R. L., and Seedhouse, R. L., "The Use of Polystyrene for Embankment Construction." *Contractor Report 356,* Transport Research Laboratory, Crowthorne, Berkshire, U.K. (1994) 55 pp.
11. Duskov, M., and Houben, L. J. M., "Design Guidelines for Light-weight Pavement Structures with EPS Geofoam." CROW, Ede, The Netherlands (2000) 85 pp.
12. Horvath, J. S., *Geofoam Geosynthetic*, Horvath Engineering, P.C., Scarsdale, NY (1995) 229 pp.
13. Elias, V., Welsh, J., Warren, J., and Lukas, R., "Ground Improvement Technical Summaries." *FHWA-SA-98-086,* Vol. 2, 2 Vols, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C. (1999).
14. "A Report on the *International Workshop on Lightweight Geo-Materials*." *IGS News,* Vol. 18, No. 2 (July) p. 9.
15. Hunt, R. E., *Geotechnical Engineering Analysis and Evaluation*, McGraw-Hill, Inc., New York (1986) 729 pp.
16. Goble, G., "Geotechnical Related Development and Implementation of Load and Resistance Factor (LRFD) Methods, *NCHRP Synthesis of Highway Practice 276*." Transportation Research Board, Washington, D.C. (1999) 69 pp.
17. Horvath, J. S., "Geofoam and Geocomb: Lessons from the Second Millennium A.D. as Insight for the Future." *Research Report No. CE/GE-99-2*, Manhattan College, Bronx, NY (1999) 24 pp.
18. "Treatment of Soft Foundations for Highway Embankments." *NCHRP Synthesis 29,* Transportation Research Board, Washington, D.C. (1975) 25 pp.
19. "Bridge Approach Design and Construction Practices." *NCHRP Synthesis of Highway Practice 2,* Transportation Research Board, Washington, D.C. (1969) 30 pp.
20. Wahls, H. E., "Shallow Foundations for Highway Structures." *NCHRP Synthesis of Highway Practice 107,* Transportation Research Board, Washington, D.C. (1983) 38 pp.
21. Moulton, L. K., GangaRao, H. V. S., and Halvorsen, G. T., "Tolerable Movement Criteria for Highway Bridges." *FHWA/RS-85/107,* Federal Highway Administration, Washington D.C. (1985).
22. Moulton, L. K., "Tolerable Movement Criteria for Highway Bridges." *FHWA-TS-85-228,* Federal Highway Administration, Washington, D.C. (1986) 93 pp.

23. “Slope Stabilization with Geofoam (in prep.).” Syracuse University and The Society of Plastics Industries.
24. Horvath, J. S., “Lessons Learned from Failures Involving Geofoam in Roads and Embankments.” *Research Report No. CE/GE-99-1*, Manhattan College, Bronx, NY (1999) 18 pp.
25. Terzaghi, K., Peck, R. B., and Mesri, G., *Soil Mechanics in Engineering Practice*, 3rd, John Wiley & Sons, Inc., New York (1996).
26. “Settlement Analysis.” *Technical Engineering and Design Guides as Adapted From the U.S. Army Corps of Engineers, No. 9*, ASCE, New York (1994) 144 pp.
27. Prandtl, L., “Über die Eindringungsfestigkeit (Härte) plastischer Bauttoffe und die Festigkeit von Schneiden (On the penetrating strengths (hardness) of plastic construction materials and the strength of cutting edges).” *Zeit. angew. Math. Mech.*, Vol. 1, No. 1 (1921) pp. 15-20.
28. American Association of State Highway and Transportation Officials, *Standard Specifications for Highway Bridges*, 16th, American Association of State Highway and Transportation Officials, Washington, D.C. (1996).
29. Kavazanjian, E., Jr., Matasovic, N., Hadj-Hamou, T., and Sabatini, P. J., “Geotechnical Engineering Circular No. 3; Design Guidance: Geotechnical Earthquake Engineering for Highways; Volume I - Design Principles.” *FHWA-SA-97-076*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C. (1997) 186 pp.
30. American Association of State Highway and Transportation Officials, *AASHTO Guide for Design of Pavement Structures, 1993*, American Association of State Highway and Transportation Officials, Washington, D.C. (1993).