Recommended Design Guideline

RECOMMENDED DESIGN GUIDELINE

1 INTRODUCTION

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 1). Applications where the fill sides are markedly different and closer to those shown in Figure 2 (sometimes referred to as side-hill fills) are excluded from this study because they are the subject of a separate study (*1*). It should be noted from Figure 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.

Both the Système International d'Unités (SI) and inch-pound (I-P) units have been used in this guideline. SI units are shown first, and I-P units are shown in parentheses within text. Numerous figures are included for use in design. Therefore, only SI units are provided in some of the figures to avoid duplication of figures. Additionally, in some cases figures have been reproduced that use either all SI or all I-P units. These figures have not been revised to show both sets of units. However, Section 7 presents factors that can be used to convert between SI and I-P units. The one exception to the dual SI and I-P unit usage involves the quantities of density and unit weight. Density is the mass per unit volume and has units of kg/m^3 (slugs/ft^3), and unit weight is the weight per unit volume and has units of kN/m^3 (lbf/ft^3). Although density is the preferred quantity in SI, unit weight is still the common quantity in geotechnical engineering practice. Therefore, the quantity of unit weight will be used herein except when referring to EPS-block geofoam. The geofoam manufacturing industry typically uses the quantity of density with the SI units of kg/m^3 but with the I-P quantity of unit weight with units of lbf/ft^3. Therefore, the same dual-unit system of density in SI and unit weight in I-P units will be used when referring to EPS-block geofoam.

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," which 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 report, published as *NCHRP Web Document 65,* for the necessary technical background. This guideline is intended to be used in conjunction with the recommended standard that follows.

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 sideslopes 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 two-lane roadway with 1.8-m (6-ft) shoulders, four-lane roadway with two 3-m (10-ft) exterior shoulders and two 1.2-m (4-ft) interior shoulders, and a six-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) and 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 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 those with noncritical conditions.

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 because of 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.

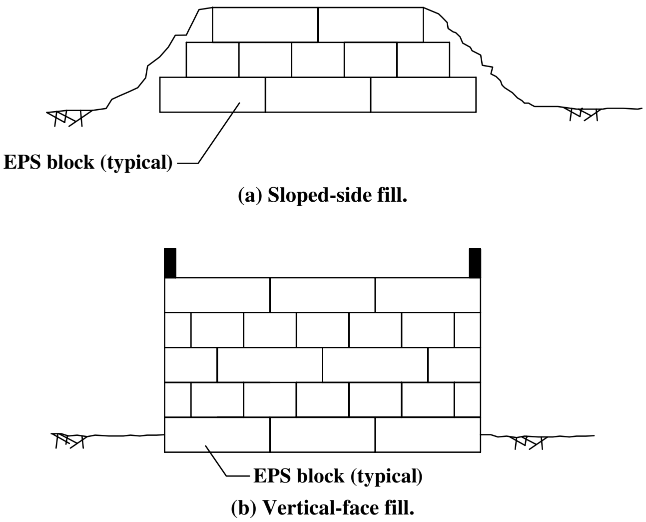


Figure 1. Typical EPS-block geofoam applications involving embankments (2).

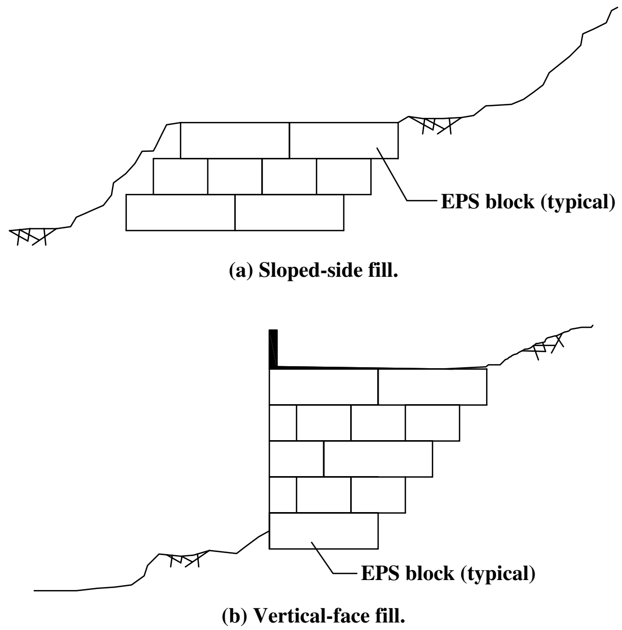


Figure 2. Typical EPS-block geofoam applications involving side-hill fills (2).

2 DESIGN GUIDELINE

2.1 Major Components of an EPS-Block Geofoam Embankment

As indicated in Figure 3, an EPS-block geofoam embankment consists of three major components:

- The existing *foundation soil,* which may or may not 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 the bottom of the EPS blocks for overall economy. In addition, depending on whether the embankment has sloped sides (trapezoidal embankment) or vertical sides (vertical embankment), 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 or portland cement concrete (PCC) to provide a smooth traveling surface for motor vehicles. Asphalt concrete appears to be the predominant road surface type because asphalt concrete pavements tend to tolerate postconstruction settlements better than PCC pavements and because asphalt concrete pavements are less expensive. However, in certain applications (e.g., vehicle escape ramps in mountainous regions and logging roads), an unbound gravel or crushed-rock surface layer may be used.

2.2 Design Phases

At the present time, 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

TABLE 1 Examples of critical and noncritical embankment design and construction conditions (*3*)

Condition	Critical	Noncritical
Stability	Large, unexpected, catastrophic movements	Slow, creep movements
	Structures involved	No structures involved
	Evidence of impending instability failure	No evidence of impending instability failure
Settlements	Large total and differential	Small total and differential
	Occur over relatively short distances	Occur over large distances
	Rapid, direction of traffic	Slow, transverse to direction of traffic
Repairs	Repair cost much greater than original construction cost	Repair cost less than original construction cost

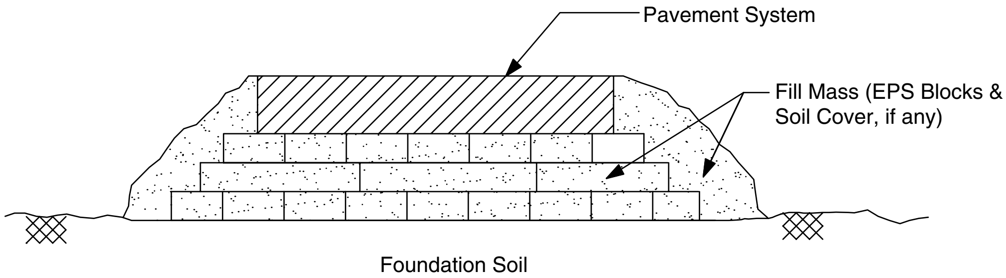


Figure 3. Major components of an EPS-block geofoam embankment.

as well as its components individually must be designed to prevent failure. As used herein, the term failure includes both of the following:

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

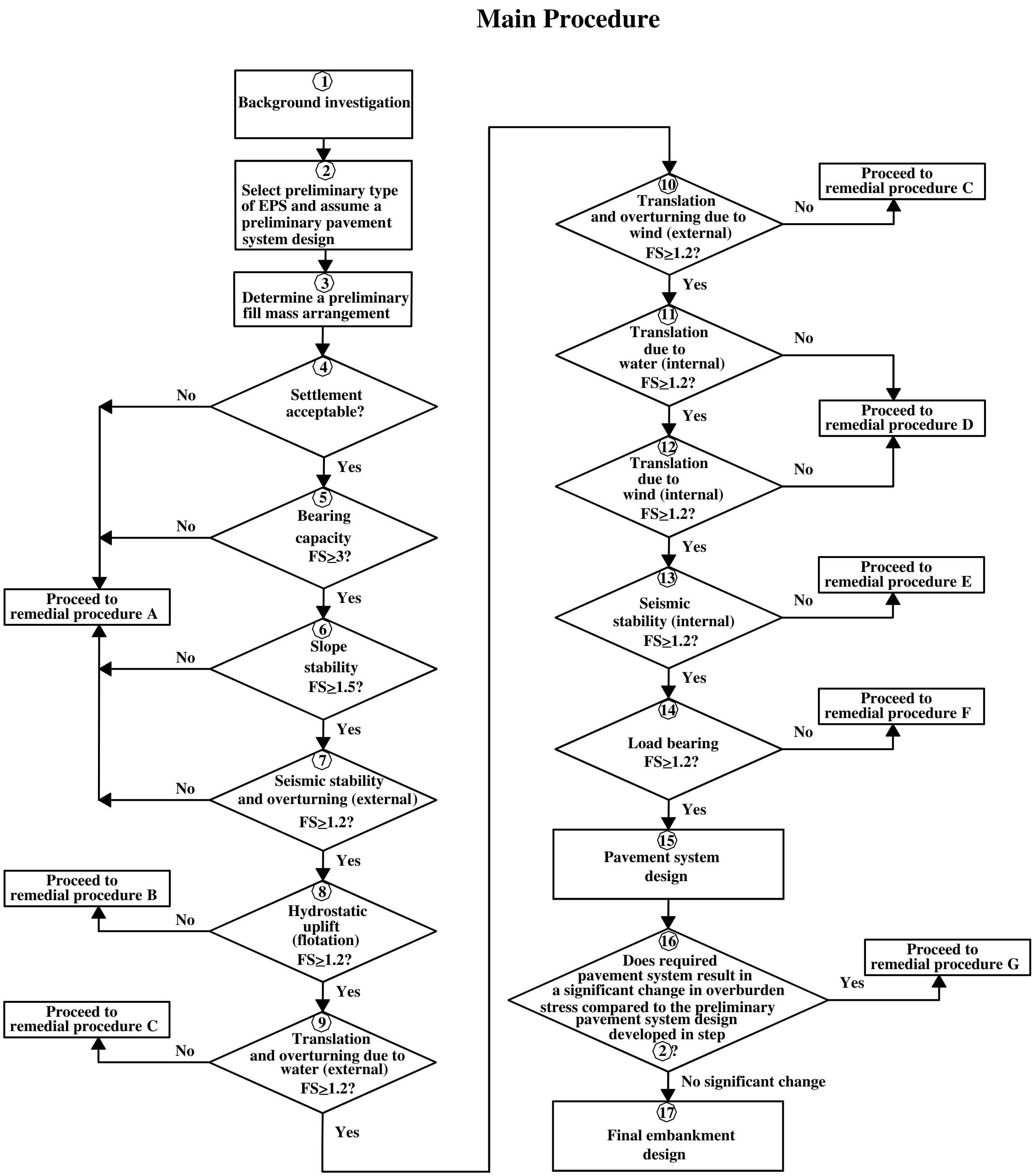
The overall design process is divided into the following three phases:

- Design for *external (global) stability* of the overall embankment, which considers how the combined fill mass and overlying pavement system interact with the existing foundation soil. External stability includes consideration of serviceability failure issues, such as global total and differential settlement, and collapse failure issues, such as bearing capacity and slope stability under various load cases (e.g., applied gravity, seismic loading, and 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 typically has a higher material cost per volume than soil, 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.
- 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 a 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 by structural anchorage, for any road hardware (e.g., guardrails, barriers, median dividers, lighting, signage, and utilities).

2.3 Design Procedure

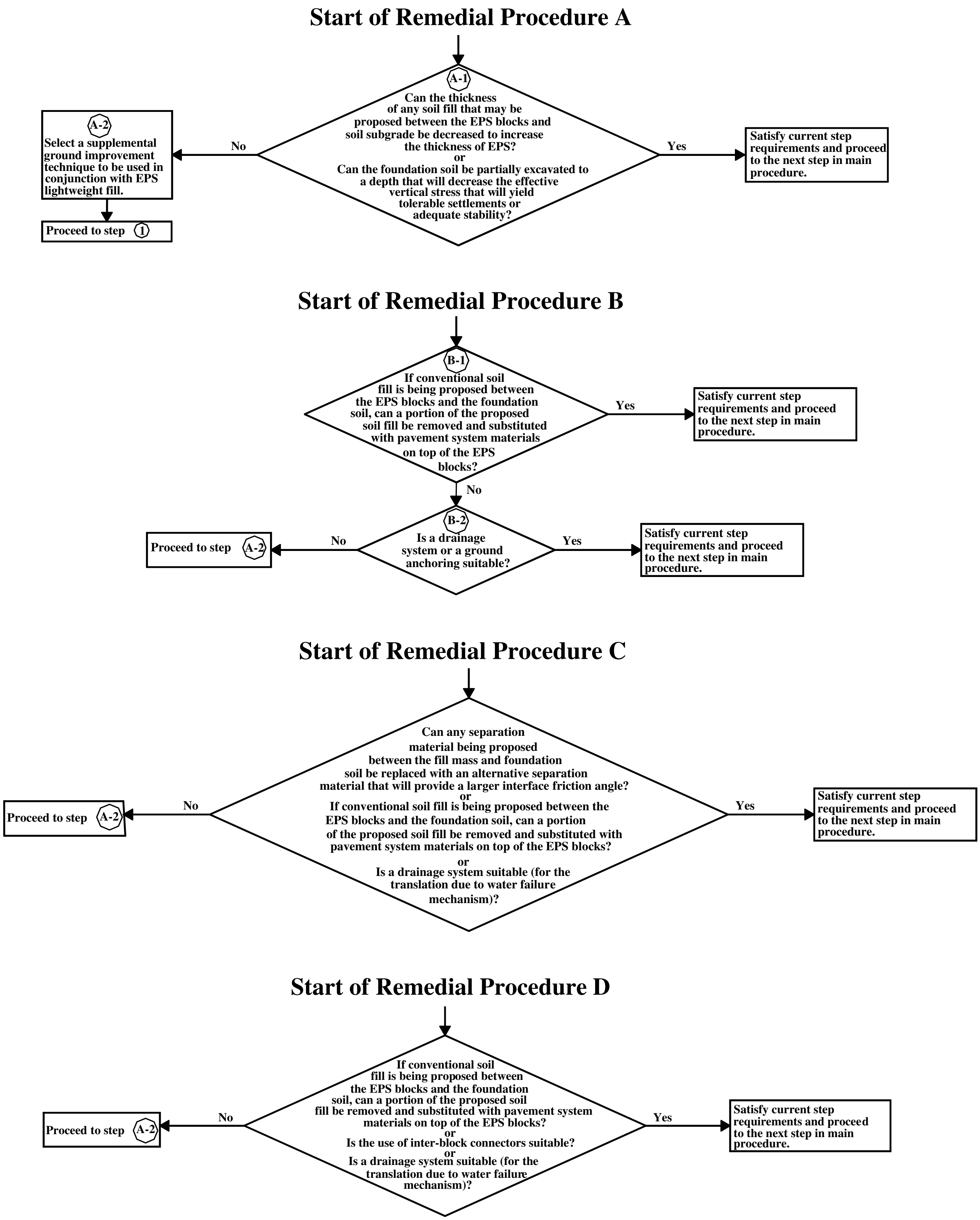
The design procedure for an EPS-block geofoam roadway embankment over soft soil considers the interaction between the three major components of the embankment: foundation soil, fill mass, and pavement system. Because of this interaction, the three-phased design procedure involves interconnected analyses among these three components. For example, some issues of pavement system design act oppositely 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 among 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 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 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.



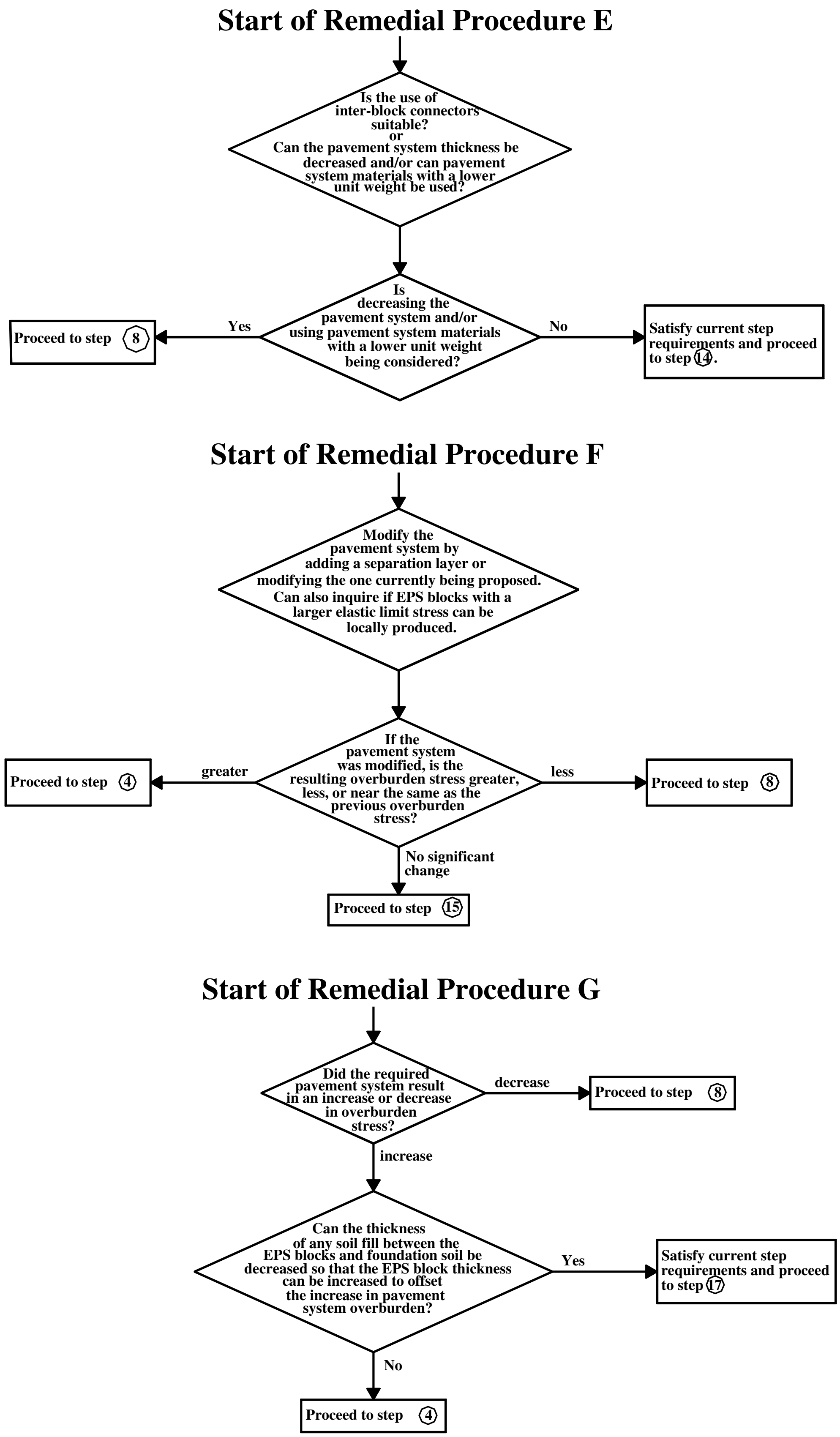
FS = Factor of Safety.

Figure 4. Flow chart of design procedure for an EPS-block geofoam roadway embankment.



Note: These remedial procedures are not applicable to overturning of a vertical embankment about the toe of the embankment at the embankment and foundation soil interface. If the factor of safety against overturning of a vertical embankment is less than 1.2, consideration can be given to adjusting the width or height of the vertical embankment.

Figure 4. (*Continued*)



Note: These remedial procedures are not applicable to overturning of a vertical embankment about the toe of the embankment at the embankment and foundation soil interface. If the factor of safety against overturning of a vertical embankment is less than 1.2, consideration can be given to adjusting the width or height of the vertical embankment.

Figure 4. (*Continued*)

2.3.1 Step 1—Background Investigation

The first step in the design procedure is background investigation, which involves obtaining the subsurface information at the project site, estimating the loads that the embankment system will be subjected to, and determining the geometrical parameters of the embankment. Background investigation is discussed in detail in Chapter 3 and was not the focus of this research so this information is not included in the guideline. The guideline focuses on the steps for designing the EPS portion of the embankment.

2.3.2 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 4 is based on obtaining a pavement system that transmits the least amount of vertical stress to 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 that 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). Chapter 4 presents the methodology for selecting a preliminary pavement system.

2.3.3 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 has, 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 number of EPS blocks necessary to 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.

2.3.4 Steps 4–10—External (Global) Stability

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 4–10 in the flow chart in Figure 4. The tolerable criteria for each operation are also shown in Figure 4. The design methodology and tolerable criteria for external (global) stability are described in more detail in Section 4.

2.3.5 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 11–14 in the flow chart in Figure 4 with the accompanying tolerable criteria. More detail of the internal stability evaluation and tolerable criteria is presented in Section 5.

2.3.6 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. Pavement system design is described in Section 3.

2.3.7 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 4. If the applied vertical stress from 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 4. If the applied vertical stress from the final pavement system is in agreement with that from the preliminary pavement system, the resulting embankment design can be used for construction purposes.

3 PAVEMENT SYSTEM DESIGN PROCEDURE

3.1 Introduction

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 a 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 2.

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 following sections present the design catalogs for flexible and rigid pavement systems. The American Association of State Highway and Transportation Officials (AASHTO) 1993 design procedure (*4*) was used to develop the flexible and rigid pavement design catalogs.

3.2 Flexible Pavement System Design Catalog

The design catalog for a flexible pavement system, shown in Table 3, is based on the following assumptions (*4*):

1. All designs are based on the structural requirement for one performance period, regardless of the time interval. The performance period is defined as the period of time for which an initial (or rehabilitated) structure will last before reaching its terminal serviceability (*4*).
2. The range of traffic levels for the performance period is limited to between 50,000 and 1 million 80-kN (18-kip) equivalent single-axle load (ESAL) applications. An ESAL is the summation of equivalent 80-kN (18-kip) single-axle loads and is used to convert mixed traffic to design traffic for the performance period (*4*).
3. The designs are based on a 50- or 75-percent level of reliability, which AASHTO considers acceptable for low-volume road design.
4. The designs are based on the resilient modulus values indicated in Table 2 for the three typical grades of EPS: EPS50, EPS70, and EPS100.
5. The designs are based on an initial serviceability index of 4.2 and a terminal serviceability index of 2. The average initial serviceability at the American Association of State Highway Officials (AASHO) road test was 4.2 for flexible pavements. AASHTO recommends a terminal serviceability index of 2 for highways with less traffic than major highways.
6. The designs are based on a standard deviation of 0.49 to account for variability associated with material properties, traffic, and performance. AASHTO recommends a value of 0.49 for the case where the variance of projected future traffic is not considered.
7. The designs do not consider the effects of drainage levels on predicted pavement performance.

Table 3 is similar in format to the design catalogs provided in the 1993 *AASHTO Guide for Design of Pavement Structures* (*4*). Although the design catalog in Table 3 is for low-volume roads, the use of EPS-block geofoam is not limited to low-volume roads; EPS-block geofoam has been used for high-volume traffic roads such as Interstate highways.

Once a design structural number (SN) is determined, appropriate flexible pavement layer thicknesses can be identified that will yield the required load-carrying capacity indicated by

TABLE 2 Equivalent soil subgrade values of EPS-block geofoam for pavement design

Proposed AASHTO Material Designation	Design Values of Engineering Parameters			
	Minimum Allowable Full-Block Density, kg/m^3(lbf/ft^3)	California Bearing Ratio, CBR (%)	Initial Tangent Young's Modulus, E_{ti}, MPa(lbs/in^2)	Resilient Modulus, M_R, MPa(lbs/in^2)
EPS50	20 (1.25)	2	5 (725)	5 (725)
EPS70	24 (1.5)	3	7 (1015)	7 (1015)
EPS100	32 (2.0)	4	10 (1450)	10 (1450)

Note: The use of EPS40 directly beneath paved areas is not recommended and thus does not appear in this table because of the potential for settlement problems. The minimum allowable block density is based on density obtained on either a block as a whole unit or an actual full-sized block. The proposed AASHTO material type designation system is based on the minimum elastic limit stress of the block as a whole in kilopascals (see Table 8).

TABLE 3 Flexible pavement design catalog for low-volume roads

R (%)	EPS Type	Traffic Level					
		Low		Medium		High	
		50,000	300,000	400,000	600,000	700,000	1,000,000
50	EPS50	4*	5.1	5.3	5.5	5.7	5.9
	EPS70	3.5	4.6	4.7	5	5.1	5.3
	EPS100	3.1	4.1	4.2	4.5	4.6	4.8
75	EPS50	4.4	5.6	5.8	6.1	6.2	6.5
	EPS70	3.9	5	5.2	5.5	5.6	5.9
	EPS100	3.5	4.5	4.7	5	5.1	5.3

R = Reliability level.
* design structural number, SN.

the SN in accordance with the following AASHTO flexible pavement design equation:

$$SN = a_1D_1 + a_2D_2 + a_3D_3 \quad (1)$$

Where

a_1, a_2, and a_3 = layer coefficients for surface, base, and subbase course materials, respectively, and
D_1, D_2, and D_3 = thickness (in inches) of surface, base, and subbase course, respectively.

Layer coefficients can be obtained in the 1993 *AASHTO Guide for Design of Pavement Structures* (*4*) or from state department of transportation (DOT) design manuals. However, layer coefficient values for PCC slabs are not provided in the 1993 AASHTO pavement design guide (*4*). If a reinforced PCC slab is considered as a separation layer between the top of the EPS blocks and the overlying pavement system, it may be possible to incorporate the PCC slab into the AASHTO 1993 flexible pavement design procedure by determining a suitable layer coefficient to represent the PCC slab. *NCHRP Report 128* (*5*) indicates that, based on test results performed in Illinois, a PCC base with a 7-day strength of 17.2 MPa (2,500 lbs/in^2) exhibits a layer coefficient of 0.5.

It can be seen that, for a given set of layer coefficients, Equation 1 does not provide a unique solution for the thickness of the surface, base, and subbase. However, AASHTO recommends the minimum thickness values indicated in Table 4 for asphalt concrete and aggregate base to overcome placement impracticalities, ensure adequate performance, and lower costs. This recommendation provides guidance in fixing values of D_1 and D_2 so D_3 can be estimated in Equation 1. In addition, it is recommended herein that a minimum pavement system thickness of 610 mm (24 in.) be used over EPS-block geofoam to minimize the potential for differential icing and solar heating. After various layer thickness combinations have been determined and checked against construction and maintenance constraints, a cost-effective layer thickness combination is typically selected.

3.3 Rigid Pavement System Design Catalog

Design catalogs for rigid pavements developed herein and based on the AASHTO 1993 design procedure are presented in Tables 5 and 6. The rigid pavement design catalogs are similar to the rigid pavement design catalogs provided in the AASHTO 1993 procedure (*4*) except that the designs herein are based on the resilient modulus values representative of an EPS subgrade, which are shown in Table 2. Tables 5 and 6 can be used by design engineers to obtain a concrete thickness with a geofoam embankment. As with the design catalogs provided in the AASHTO 1993 procedure, Tables 5 and 6 are based on the following assumptions:

- Slab thickness design recommendations apply to all six U.S. climatic regions.
- The procedure is based on the use of dowels at transverse joints.
- The range of traffic loads for the performance period is limited to between 50,000 and 1,000,000 applications of 80-kN (18-kip) ESALs. An ESAL is the summation of equivalent 80-kN (18-kip) single-axle loads used to convert mixed traffic to design traffic for the performance period (*4*).
- The designs are based on a 50-percent or 75-percent level of reliability, which AASHTO considers acceptable for low-volume road design.
- The designs are based on a minimum thickness of high-quality material subbase equivalent to 610 mm (24 in.) less the PCC slab thickness used. This thickness minimizes the potential for differential icing and solar heating.
- The designs are based on the resilient modulus values indicated in Table 2 for EPS70 and EPS100.
- The designs are based on a mean PCC modulus of rupture (S_c') of 4.1 or 4.8 MPa (600 or 700 lbs/in^2).
- The designs are based on a mean PCC elastic modulus (E_c) of 34.5 GPa (5,000,000 lbs/in^2).
- Drainage (moisture) conditions (C_d) are fair ($C_d = 1.0$).
- The 80-kN (18-kip) ESAL traffic levels are as follows:
  - High: 700,000–1,000,000.
  - Medium: 400,000–600,000.
  - Low: 50,000–300,000.

Even though the design catalogs in Tables 5 and 6 are for low-volume roads, EPS-block geofoam can be and has been used for high-volume traffic roads, such as Interstate highways.

TABLE 4 Minimum practical thicknesses for asphalt concrete and aggregate base (*4*)

	Minimum Thickness , mm (in.)	
Traffic, ESALs	**Asphalt Concrete**	**Aggregate Base**
Less than 50,000	25 (1.0)	100 (4.0)
50,001–150,000	50 (2.0)	100 (4.0)
150,001–500,000	64 (2.5)	100 (4.0)
500,001–2,000,000	76 (3.0)	150 (6.0)
2,000,001–7,000,000	90 (3.5)	150 (6.0)
More than 7,000,000	100 (4.0)	150 (6.0)

TABLE 5 Rigid concrete thickness for low-volume roads for inherent reliability of 50 percent

INHERENT RELIABILITY %	EPS TYPE	EPS RESILIENT MODULUS MPa (lbs/in^2)	LOAD TRANSFER DEVICES	No				Yes			
			EDGE SUPPORT	No		Yes		No		Yes	
			S'c MPa (lbs/in^2)	4.1 (600)	4.8 (700)	4.1 (600)	4.8 (700)	4.1 (600)	4.8 (700)	4.1 (600)	4.8 (700)
			Traffic (ESALs)	**Rigid Concrete Thickness (in.)**							
50	EPS70	7 (1015)	50,000	5	5	5	5	5	5	5	5
	EPS100	10 (1450)		5	5	5	5	5	5	5	5
	EPS70	7 (1015)	300,000	6.5	6	6	6	6	5.5	5.5	5
	EPS100	10 (1450)		6.5	6	6	6	6	5.5	5.5	5
	EPS70	7 (1015)	400,000	7	6.5	6.5	6	6	5.5	6	5.5
	EPS100	10 (1450)		7	6.5	6.5	6	6	5.5	6	5.5
	EPS70	7 (1015)	600,000	7.5	7	7	6.5	6.5	6	6	5.5
	EPS100	10 (1450)		7.5	7	7	6.5	6.5	6	6	5.5
	EPS70	7 (1015)	700,000	7.5	7	7	6.5	6.5	6	6	6
	EPS100	10 (1450)		7.5	7	7	6.5	6.5	6	6	6
	EPS70	7 (1015)	1,000,000	8	7.5	7.5	7	7	6.5	6.5	6
	EPS100	10 (1450)		8	7.5	7.5	7	7	6.5	6.5	6

TABLE 6 Rigid concrete thickness for low-volume roads for inherent reliability of 75 percent

INHERENT RELIABILITY %	EPS TYPE	EPS RESILIENT MODULUS MPa (lbs/in^2)	LOAD TRANSFER DEVICES	No				Yes			
			EDGE SUPPORT	No		Yes		No		Yes	
			S'c MPa (lbs/in^2)	4.1 (600)	4.8 (700)	4.1 (600)	4.8 (700)	4.1 (600)	4.8 (700)	4.1 (600)	4.8 (700)
			Traffic (ESALs)	Rigid Concrete Thickness (in.)							
75	EPS70	7 (1015)	50,000	5.5	5	5	5	5	5	5	5
	EPS100	10 (1450)		5.5	5	5	5	5	5	5	5
	EPS70	7 (1015)	300,000	7	6.5	6.5	6	6.5	6	6	5.5
	EPS100	10 (1450)		7	6.5	7	6	6.5	6	6	5.5
	EPS70	7 (1015)	400,000	7.5	7	7	6.5	6.5	6	6	6
	EPS100	10 (1450)		7.5	7	7	6.5	6.5	6	6	6
	EPS70	7 (1015)	600,000	8	7.5	7.5	7	7	6.5	6.5	6
	EPS100	10 (1450)		8	7.5	7.5	7	7	6.5	6.5	6
	EPS70	7 (1015)	700,000	8	7.5	7.5	7	7	6.5	7	6
	EPS100	10 (1450)		8	7.5	7.5	7	7	6.5	7	6
	EPS70	7 (1015)	1,000,000	8.5	8	8	7.5	7.5	7	7	6.5
	EPS100	10 (1450)		8.5	8	8	7.5	7.5	7	7	6.5

3.4 Typical Dead Load Stress Range Imposed by a Pavement System

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 mm (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.

4 EXTERNAL (GLOBAL) STABILITY EVALUATION

4.1 Introduction

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.

4.2 Settlement of Embankment

4.2.1 Introduction

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 Terzaghi et al. (*6*) present inclinometer measurements that show that 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 (*6*). 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 2:

$$S_{total} = S_{if} + S_i + S_p + S_s + S_{cf} = S_p + S_{cf} \tag{2}$$

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 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). Therefore, the total settlement estimate focuses on primary and secondary consolidation of the soil foundation. Therefore, Equation 2 simplifies total settlement 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 "Settlement Analysis" (*7*).

4.2.2 Settlement Due to End-of-Primary Consolidation

The end-of-primary 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. Equation 3 can be used to estimate the end-of-primary consolidation of the soil foundation and allows for overconsolidated and normally consolidated soil deposits:

$$S_p = \frac{C_r}{1+e_o} L_o \log \frac{\sigma'_p}{\sigma'_{vo}} + \frac{C_c}{1+e_o} L_o \log \frac{\sigma'_{vf}}{\sigma'_p} \tag{3}$$

Where

S_p = settlement resulting from one-dimensional end-of-primary consolidation,
C_r = recompression index,
σ'_p = preconsolidation pressure,
σ'_{vo} = in situ effective vertical stress (i.e., effective overburden pressure),
e_o = in situ void ratio under effective overburden pressure σ'_{vo},
C_c = compression index,
L_o = preconstruction thickness of the compressible layer with void ratio e_o,
σ'_{vf} = final effective vertical stress = $\sigma'_{vo} + \Delta\sigma'_Z$, and
$\Delta\sigma'_Z$ = change in effective vertical stress.

Soils that have not been subjected to effective vertical stresses higher than the present effective overburden pressure are considered normally consolidated and have a value of σ'_p/σ'_{vo} of unity. For normally consolidated foundation soil, Equation 3 can be simplified as follows:

$$S_p = \frac{C_c}{1+e_o} L_o \log \frac{\sigma'_{vf}}{\sigma'_p} \tag{4}$$

If the estimated settlement of the proposed EPS block embankment exceeds the allowable settlement, one expedient soft ground treatment method that can be used is to partially overexcavate the existing soft foundation soil and to place EPS blocks in the overexcavation. This treatment method decreases settlement by decreasing the final effective vertical stress. Note that L_o to be used in Equation 4 is the preconstruction thickness. If an overexcavation procedure is performed, L_o will be the thickness of the soft foundation soil prior to the overexcavation procedure. If the foundation soil is overconsolidated (i.e., $\sigma'_p/\sigma'_v > 1$, where σ'_v is the existing vertical stress), but the proposed final effective vertical stress will be less than or equal to the preconsolidation pressure (i.e., $\sigma'_{vf} \le \sigma'_p$), Equation 3 can be simplified as follows:

$$S_p = \frac{C_r}{1+e_o} L_o \log \frac{\sigma'_{vf}}{\sigma'_{vo}} \tag{5}$$

4.2.3 Settlement Due to Secondary Consolidation

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. Thus, secondary consolidation occurs under the final effective vertical stress, σ'_{vf}. Equation 6 can be used to estimate the secondary consolidation of the soil foundation (*6*).

$$S_s = \frac{[C_\alpha/C_c] \times C_c}{1+e_o} L_o \log \frac{t}{t_p} \tag{6}$$

Where

S_s = settlement resulting from one-dimensional secondary compression,
C_α = secondary compression index,
t = time, and
t_p = duration of primary consolidation.

C_α is determined from the results of laboratory consolidation tests. However, for preliminary settlement analyses, empirical values of C_α/C_c, such as those provided in Table 7, can be used to estimate C_α. The validity of the C_α/C_c concept has been verified using field case histories (*8, 9*).

TABLE 7 Values of C_α/C_c for soils (*6*)

Material	C_α/C_c
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat and Muskeg	0.06 ± 0.01

Field values of t_p for layers of soil that do not contain permeable layers and peats can range from several months to many years. However, for the typical useful life of a structure, the value of t/t_p rarely exceeds 100 and is often less than 10 (*6*).

4.2.4 Allowable Settlement

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 that the settlements are uniform, occur slowly over a period of time, and do not occur next to a pile-supported structure (*10*). 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.) (*10*). 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).

4.3 External Bearing Capacity of Embankment

4.3.1 Introduction

This section presents an evaluation of external bearing capacity of an EPS-block geofoam embankment. If an external bearing capacity failure occurs, the embankment can undergo excessive vertical settlement and affect adjacent property. The general expression for the ultimate bearing capacity of soil, q_{ult}, is defined by Prandt (*11*) as follows:

$$q_{ult} = cN_c + \gamma D_f N_q + \gamma B_w N_\gamma \tag{7}$$

Where

c = Mohr-Coulomb shear strength parameter (i.e., 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.

It is anticipated that most, if not all, EPS-block geofoam embankments will be founded on soft, saturated cohesive soils because traditional fill material cannot be used in this situation without pretreatment. Narrowing the type of foundation soil to soft, saturated cohesive soils that allow c to equal the undrained strength, s_u, of the foundation soil, as well as assuming the embankment is placed on the ground surface, simplifies Equation 7 to the following:

$$q_{ult} = s_u N_c = \left(1 + 0.2\frac{B_w}{L}\right)\left(1 + 0.2\frac{D_f}{D_w}\right) = 5s_u \tag{8}$$

Where

D_W = depth from ground surface to the water table,
L = length of the embankment, and
D_f = zero because the embankment is founded on the ground surface.

For design purposes, an EPS-block geofoam embankment is assumed to be modeled as a continuous footing; thus, the length of the embankment can be assumed to be significantly larger than the width such that the term B_W/L in Equation 8 approaches zero. Upon including the B_W/L simplification in Equation 8, N_c reduces to 5. By transposing Equation 8 and using a factor of safety of 3 against external bearing capacity failure, the following expression is obtained:

$$s_u = \frac{3 * \sigma_{n@0m}}{5} = \frac{3(\sigma_{n,pavement@0m} + \sigma_{n,traffic@0m} + \sigma_{n,EPS@0m})}{5} \tag{9}$$

Where

$\sigma_{n@0m}$ = normal stress applied by the embankment at the ground surface or at a depth of 0 m, kPa
$= \sigma_{n,pavement@0m} + \sigma_{n,traffic@0m} + \sigma_{n,EPS@0m}$; (10)
$\sigma_{n,pavement@0m}$ = normal stress applied by pavement system at the ground surface, kPa;
$\sigma_{n,traffic@0m}$ = normal stress applied by traffic surcharge at the ground surface, kPa;
$\sigma_{n,EPS@0m}$ = normal stress applied by weight of EPS-block geofoam at the ground surface, kPa $= \gamma_{EPS} * T_{EPS}$ (11)
γ_{EPS} = unit weight of the EPS-block geofoam, kN/m³; and
T_{EPS} = thickness or total height of EPS-block geofoam, m.

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

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

Where

$\sigma_{n,pavement}$ = normal stress applied by pavement at top of embankment, kPa;
$\sigma_{n,traffic}$ = normal stress applied by traffic surcharge at top of embankment, kPa; and
T_W = top width of embankment, m.

Substituting the conservative design values of $\sigma_{n,pavement}$ = 21.5 kPa and $\sigma_{n,traffic}$ = 11.5 kPa and γ_{EPS} = 1 kN/m³ into Equation 12 yields the following expression for the undrained shear stress required to satisfy a factor of safety of 3 for a particular embankment height:

$$s_u = \frac{3}{5} * \left\{\left[\frac{(21.5\text{ kPa} + 11.5\text{ kPa}) * T_w}{T_W + T_{EPS}}\right] + \frac{(1\text{ kN/m}^3)T_{EPS}}{2}\right\} = \frac{99\ T_W}{5(T_W + T_{EPS})} + 0.3\ T_{EPS} \tag{13}$$

Based on Equation 13 and various values of T_{EPS}, Figure 5 presents the minimum thickness or height of geofoam required for values of foundation soil undrained shear strength. The results show that if the foundation soil exhibits a value of s_u greater than or equal to 19.9 kPa (415 lbs/ft²), external bearing capacity will not control the external stability of the EPS embankment. However, if the value of s_u is less than 19.9 kPa (415 lbs/ft²), the allowable thickness or height of the EPS-block geofoam embankment can be estimated for a particular road width from Figure 5 to prevent bearing capacity failure.

For example, the lowest value of s_u that can accommodate a six-lane embankment (road width of 34 m (112 ft)) is approximately 18.3 kPa (382 lbs/ft²) for a minimum height of EPS block equal to 12.2 m (40 ft). This means that for a six-lane embankment and an s_u value of 18.3 kPa (382 lbs/ft²), the

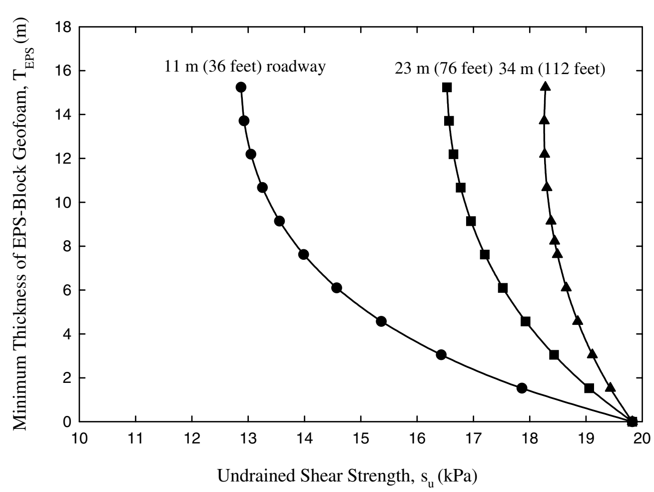


Figure 5. Design chart for obtaining the minimum thickness or height of geofoam, T_{EPS}, for a factor of safety of 3 against external bearing capacity failure of a geofoam embankment.

required T_{EPS} will be 12.2 m (40 ft). Conversely, if the height of the EPS embankment desired is 4.6 m (15 ft), an s_u of 18.9 kPa (394 lbs/ft^2) would be required.

4.4 External Slope Stability of Embankment

4.4.1 Trapezoidal Embankments

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

The soil cover is 0.46 m (1.5 ft) thick, which is typical for the sideslopes, 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 (*12*) of using 0.67 m (2 ft) of an 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).

4.4.1.2 Design Charts. The results of stability analyses using the typical cross section were used to develop the static external slope stability design charts in Figures 7 through 9 for a two-lane (road width of 11 m [36 ft]), four-lane (road width of 23 m [76 ft]), and six-lane (road width of 34 m [112 ft]) roadway embankment, respectively. Figure 7 presents the results for a two-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 effect of geofoam height, T_{EPS}, is small, whereas 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/ft^2). 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.

It can be seen from Figures 7 through 9 that the critical static factors of safety for the embankments for the two-lane, four-lane, and six-lane roadways, respectively, all exceed a value of 1.5 for values of s_u greater than or equal to 12 kPa (250 lbs/ft^2). These results indicate that external static slope stability will be satisfied (i.e., the factor of safety will be greater than 1.5) if the foundation undrained shear strength exceeds 12 kPa (250 lbs/ft^2). 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.

4.4.2 Vertical Embankments

4.4.2.1 Introduction and Typical Cross Section. This section presents an evaluation of external slope stability as a potential failure mode of EPS-block geofoam vertical embankments. The typical cross section through an EPS

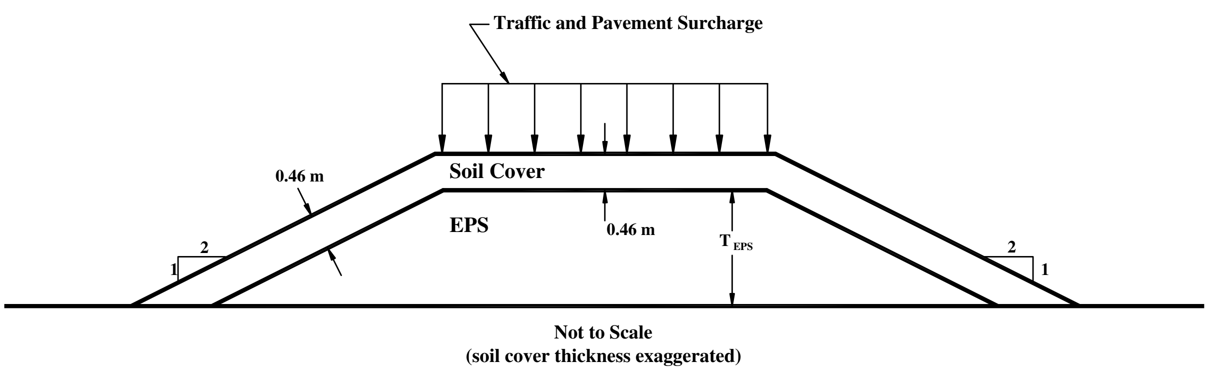


Figure 6. Typical cross section used in static external slope stability analyses of trapezoidal embankments.

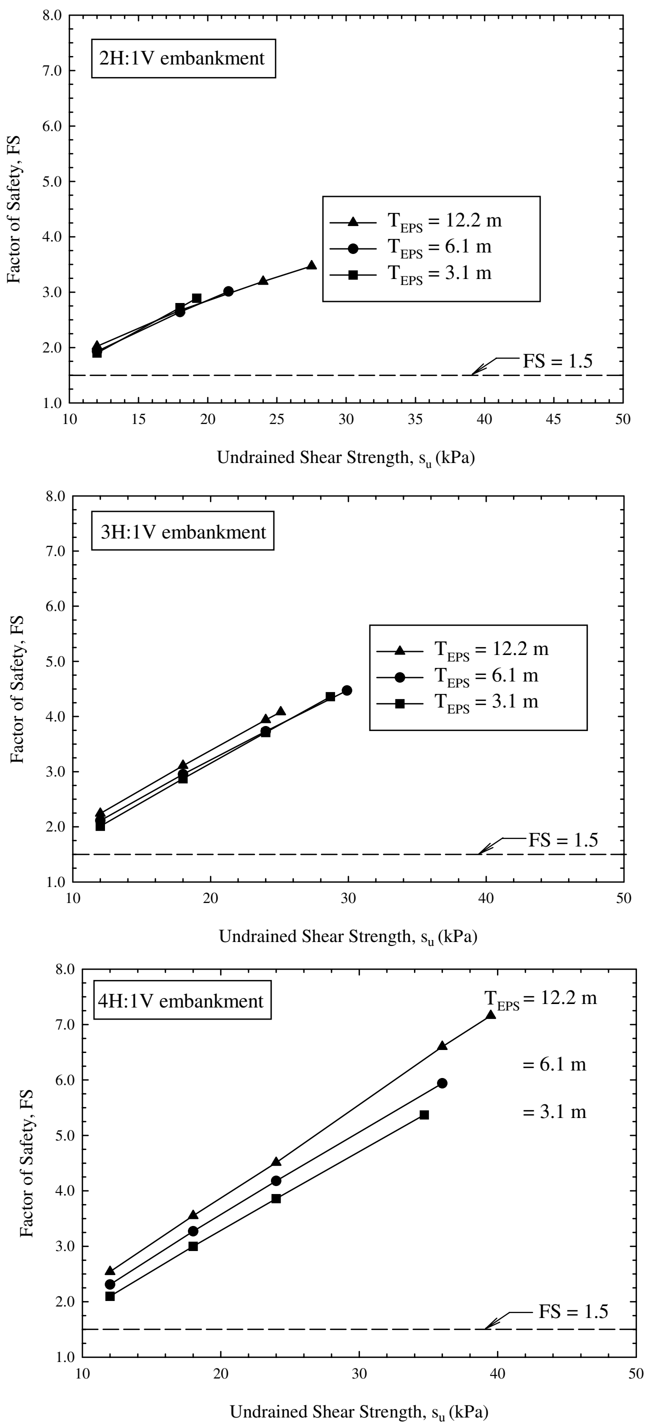


Figure 7. Static external slope stability design chart for trapezoidal embankments with a two-lane roadway with a total road width of 11 m (36 ft).

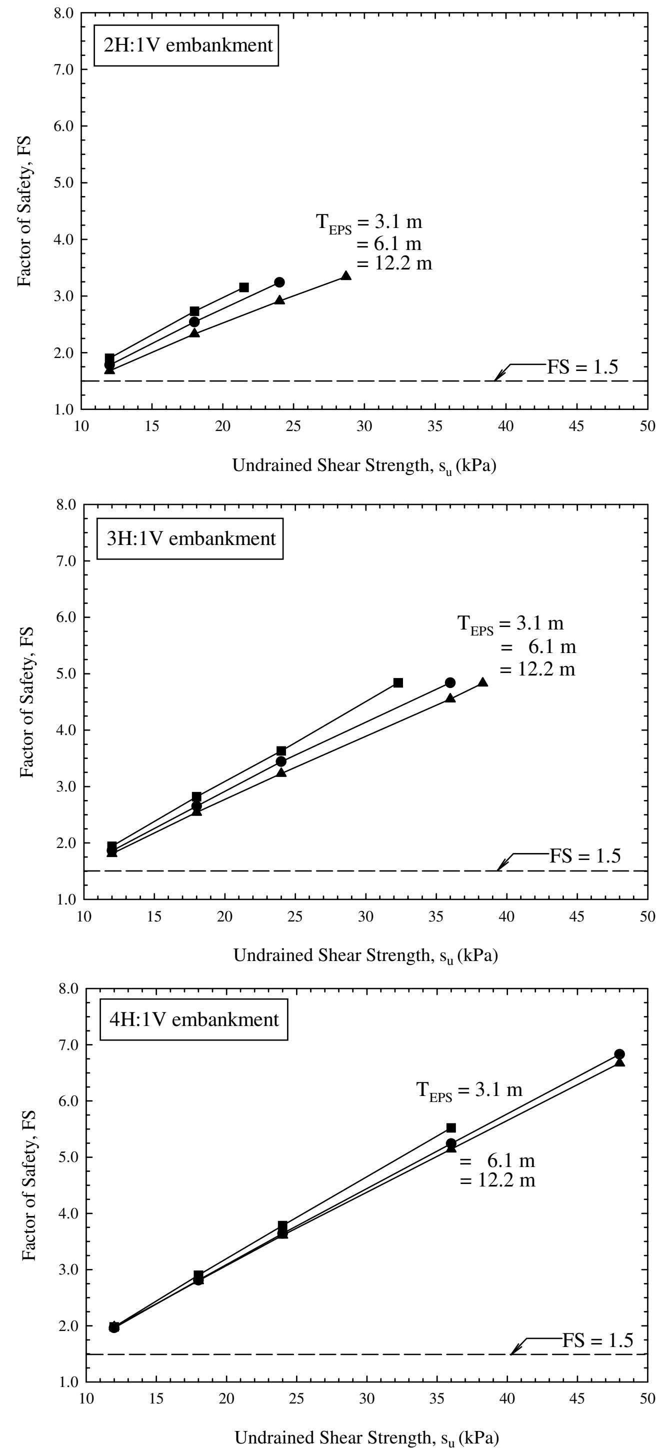


Figure 8. Static external slope stability design chart for a trapezoidal embankment with a four-lane roadway with a total road width of 23 m (76 ft).

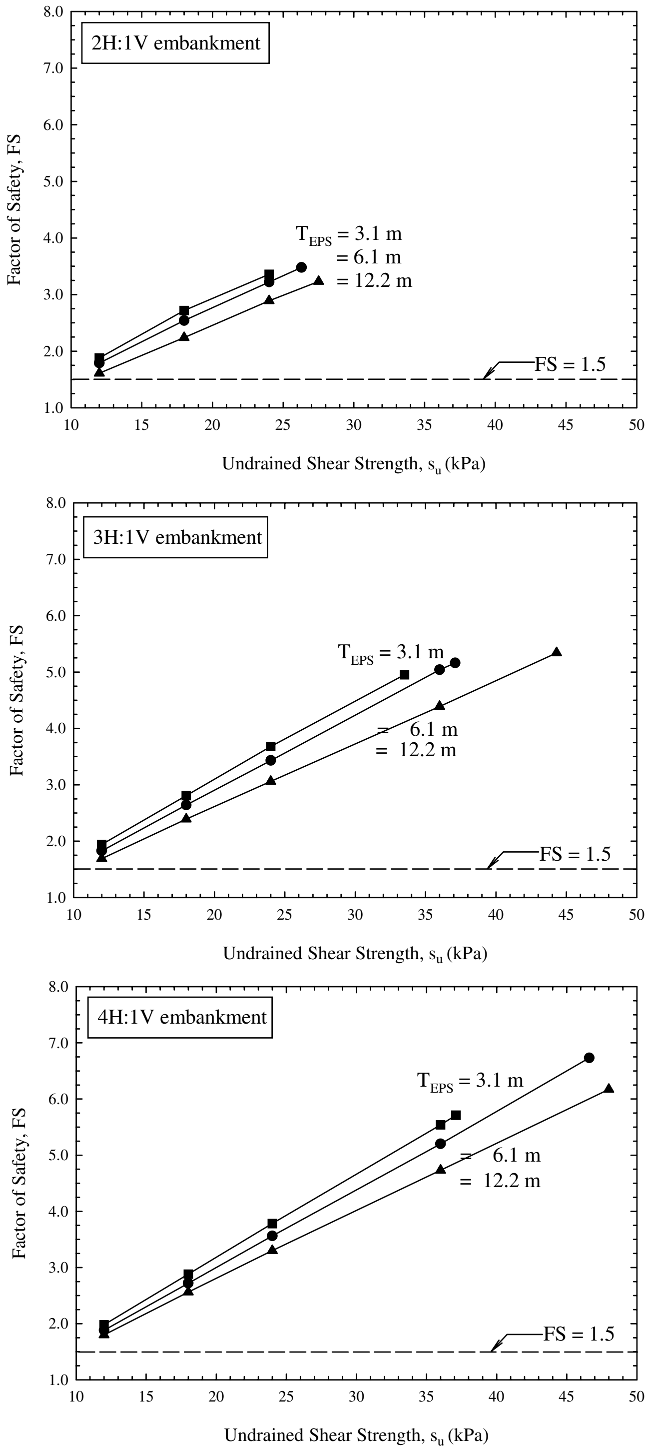


Figure 9. Static external slope stability design chart for a trapezoidal embankment with a six-lane roadway with a total road width of 34 m (112 ft).

vertical embankment used in the external static stability analyses is shown in Figure 10. This cross section differs from the cross section used for the static analyses of trapezoidal embankments in Figure 6 because 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 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 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 4.5. In a pseudo-static analysis, a seismic coefficient cannot be applied to a surcharge in limit equilibrium stability analyses, only to 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 instead of a surcharge for the static stability analyses of vertical embankments, 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 used (i.e., one embankment modeled with a surcharge and one modeled with a soil layer).

4.4.2.2 Design Charts. The results of the stability analyses were used to develop the static external slope stability design chart in Figure 11. Figure 11 presents the results for a two-lane (road width of 11 m [36 ft]), four-lane (road width of 23 m [76 ft]), and six-lane (road width of 34 m [112 ft]) roadway embankment, respectively, and the three graphs correspond to the three embankment heights considered—i.e., 3.1 m (10 ft), 6.1 m (20 ft), and 12.2 m (40 ft)—for various values of foundation soil s_u. As shown in Figure 11, 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)-tall 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; 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.

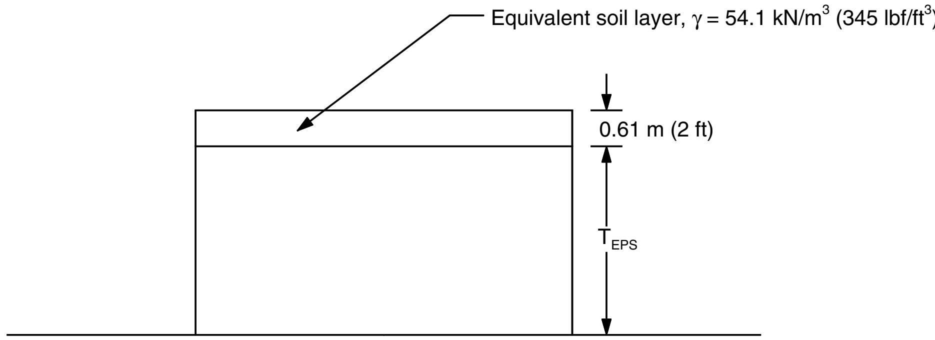


Figure 10. Typical cross section used in static and seismic external slope stability analyses of vertical embankments.

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.

4.5 External Seismic Stability of Embankment

4.5.1 Trapezoidal Embankments

4.5.1.1 Introduction and Typical Cross Section. 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 5.4. External seismic stability is evaluated using a pseudo-static slope stability analysis (*13*) involving circular failure surfaces through the foundation soil. The steps in a pseudo-static analysis are as follows:

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 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 (11 m [36 ft], 23 m [76 ft], and 34 m [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 use geofoam. The seismic analyses used 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 was being evaluated. As a result, the design charts for seismic stability terminated at the s_u value for the foundation soil that corresponded 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

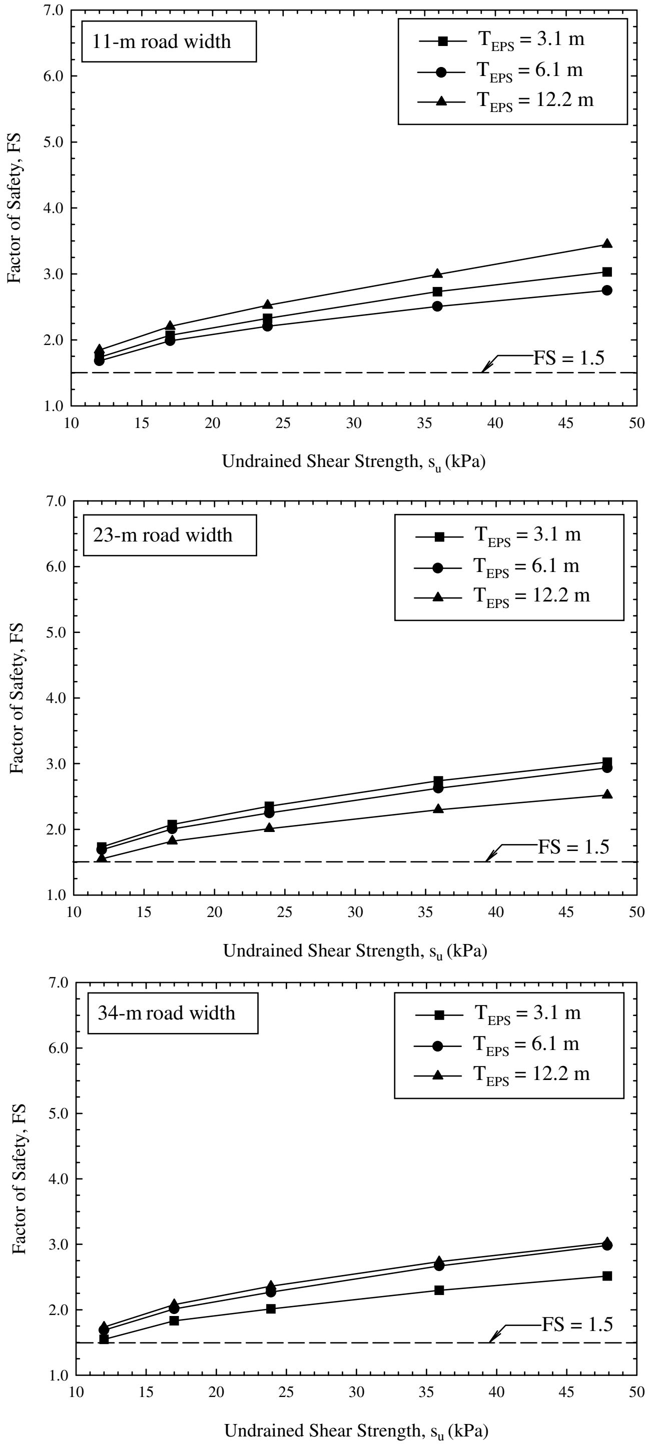


Figure 11. Effect of embankment height on static external slope stability for vertical embankments and a road width of 11 m (36 ft), 23 m (76 ft), and 34 m (112 ft).

at the same value of s_u as the static stability charts in Figures 7 through 9.

A typical cross section through an EPS embankment with sideslopes of 2H:1V used in the pseudo-static stability analyses is shown in Figure 12. This cross section differs from the cross section used for the static analyses in Figure 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/ft^2]) and traffic surcharge (11.5 kPa [240 lbs/ft^2]) used previously for external bearing capacity and slope stability. The surcharge in Figure 6 had to be replaced because a seismic coefficient is not applied to a surcharge in limit equilibrium stability analyses, only to material layers, because the horizontal force that represents the seismic loading must be applied at the center of gravity of the material layer.

4.5.1.2 Design Charts. Figures 13 through 15 present the seismic external stability design charts for a six-lane (road width of 34 m [112 ft]) geofoam roadway embankment and the three values of horizontal seismic coefficient (0.05, 0.10, and 0.20, respectively). The six-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 result leads to seismic stability concerns for the smallest horizontal seismic coefficient (see Figure 10), the shortest embankment height of 3.1 m (10 ft) (see Figure 14), and the flattest slope inclination of 4H:1V (see Figure 15). These design charts can be used to estimate the critical values of the seismic factor of safety.

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.

4.5.2 Vertical Embankments

4.5.2.1 Introduction and Typical Cross Section. 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 4.5.1.1 and circular failure surfaces through the foundation soil.

In seismic design of vertical embankments, the following two analyses should be performed: (1) pseudo-static slope stability analysis involving circular failure surfaces through the foundation soil and (2) overturning of the entire

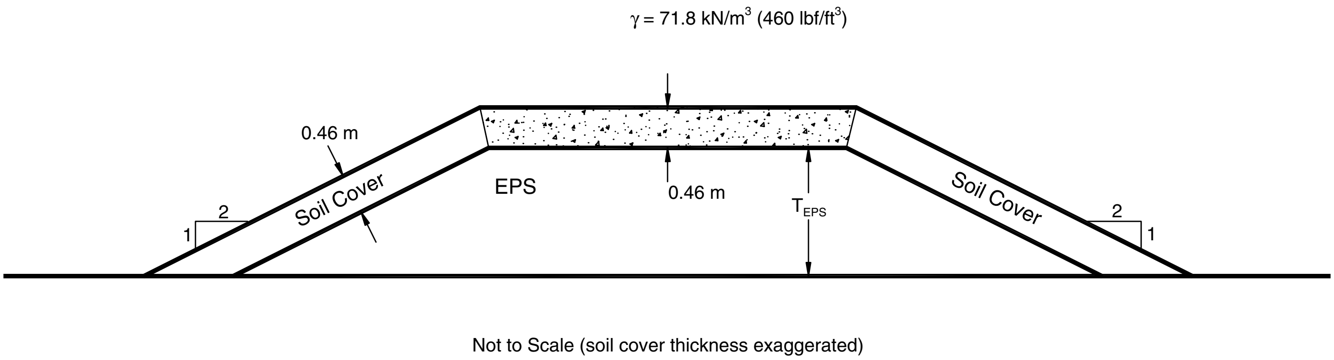


Figure 12. Typical cross section used in seismic external slope stability analyses of trapezoidal embankments.

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.

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 use geofoam. The seismic analyses use 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 10.

4.5.2.2 Design Charts. Figures 16 through 18 present the seismic external stability results for an 11-m (36-ft), 23-m (76-ft), and 34-m (112-ft) geofoam roadway embankment with vertical walls, respectively. Each figure shows the critical factor of safety versus foundation s_u for the three values of horizontal seismic coefficient—i.e., 0.05, 0.10, and 0.20. Comparison of these figures results in the following conclusions:

- 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 are greater than those obtained for the embankments with 2H:1V sideslopes. This conclusion is in agreement with the conclusion made for trapezoidal embankments that flatter embankments are more critical than 2H:1V embankments because the weight of the soil cover materials above the critical static failure surface increases as the sideslope 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; 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.
- Unlike the observations made for trapezoidal embankments, a wider roadway does not necessarily result in a decrease in seismic stability.
- 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 16 through 18. 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.

4.5.2.3 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 19.

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

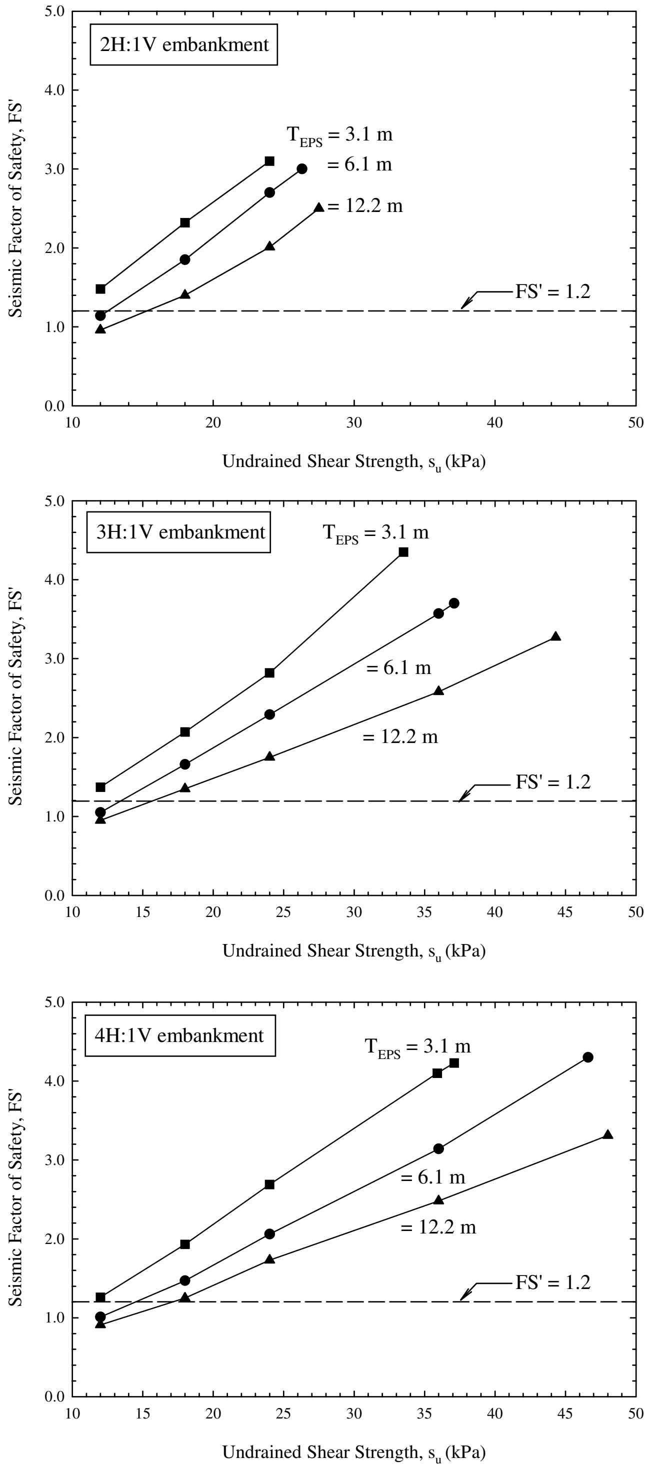


Figure 13. Seismic external slope stability design chart for trapezoidal embankments with a six-lane roadway with a total road width of 34 m (112 ft) and a k_h of 0.05.

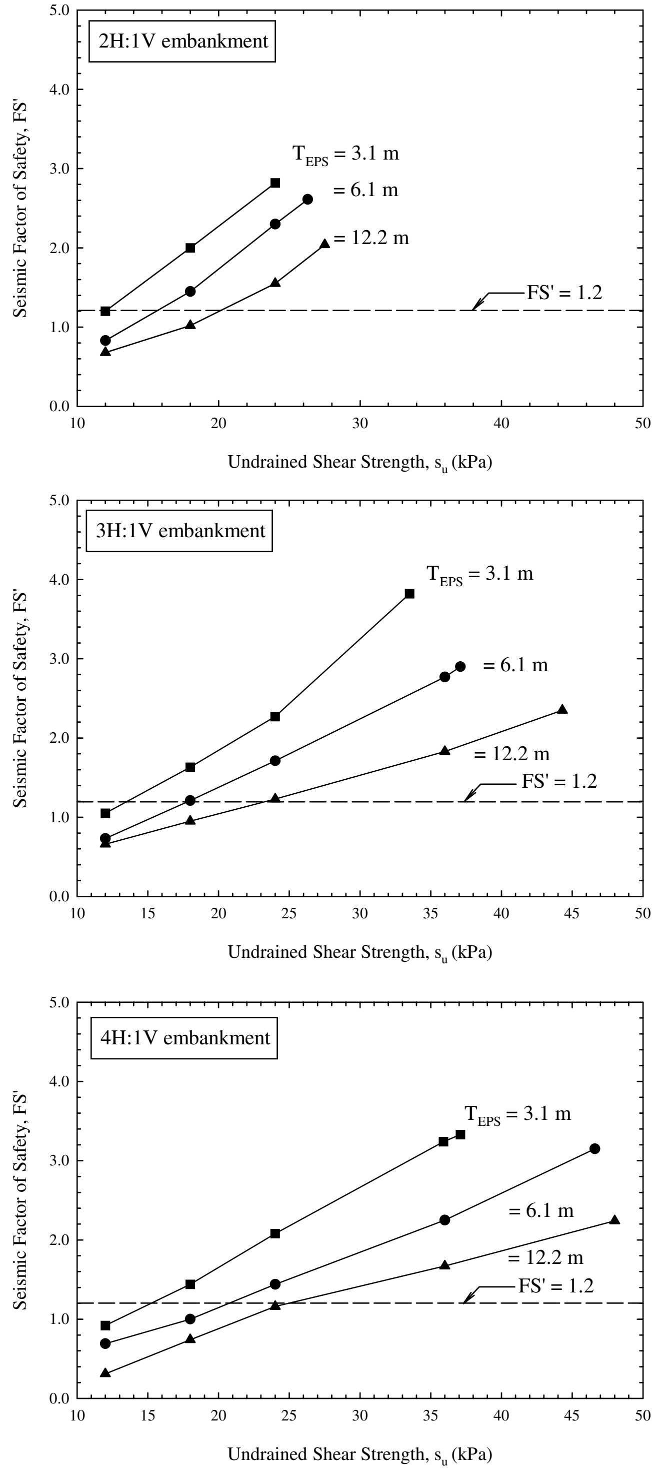


Figure 14. Seismic external slope stability design chart for trapezoidal embankments with a six-lane roadway with a total road width of 34 m (112 ft) and a k_h of 0.10.

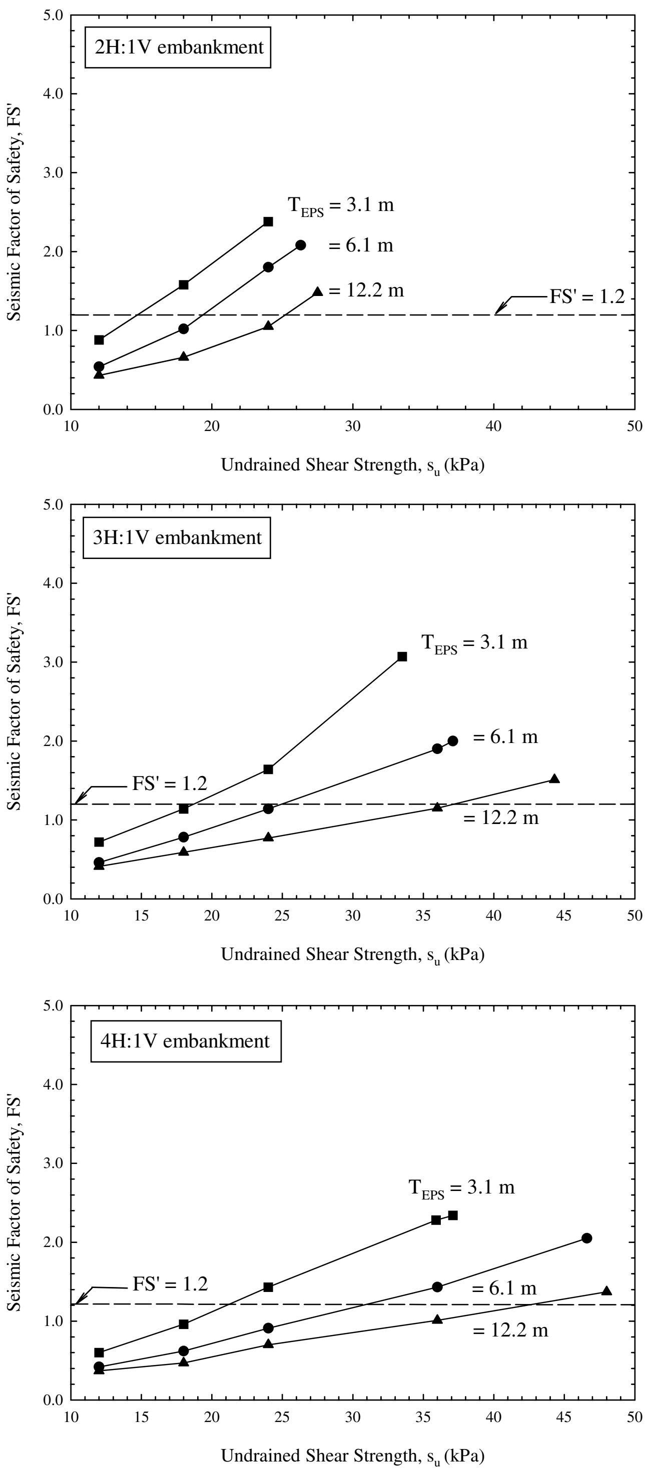


Figure 15. Seismic external slope stability design chart for trapezoidal embankments with a six-lane roadway with a total road width of 34 m (112 ft) and a k_h of 0.20.

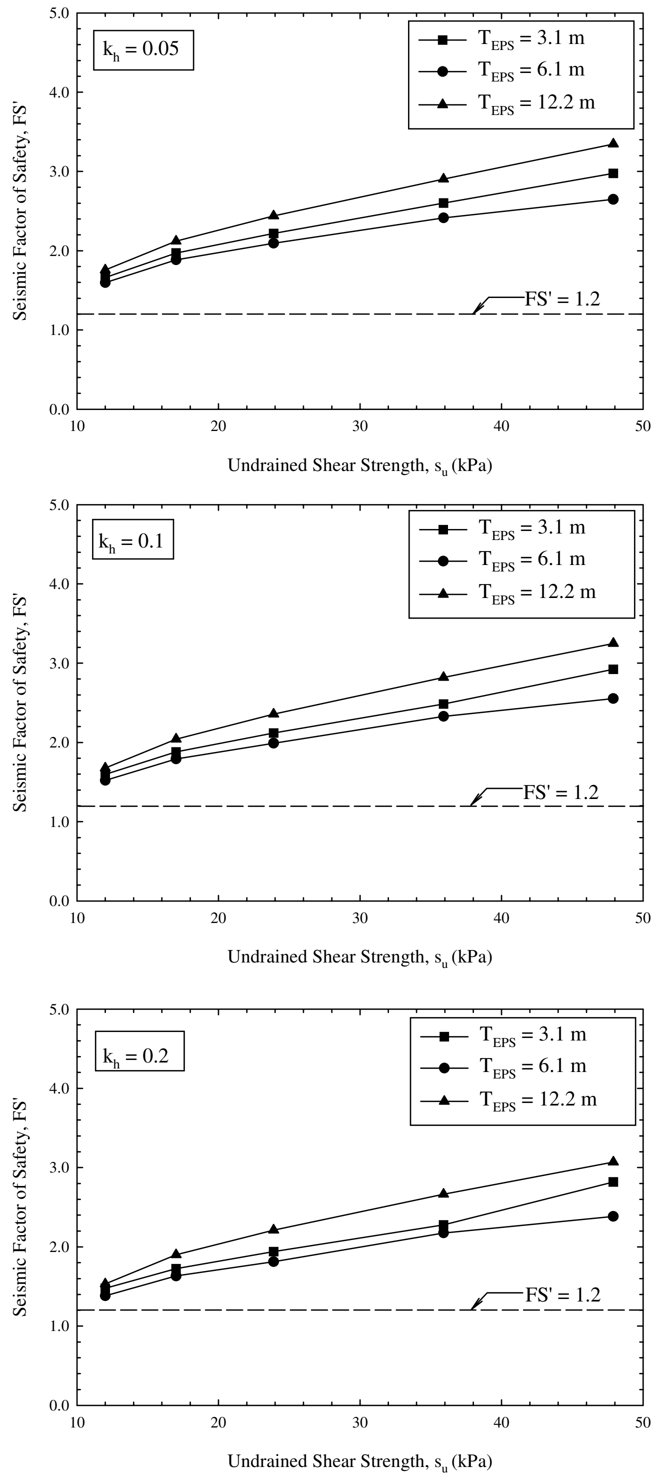


Figure 16. Seismic external stability design chart for a two-lane roadway vertical embankment and a total width of 11 m (36 ft).

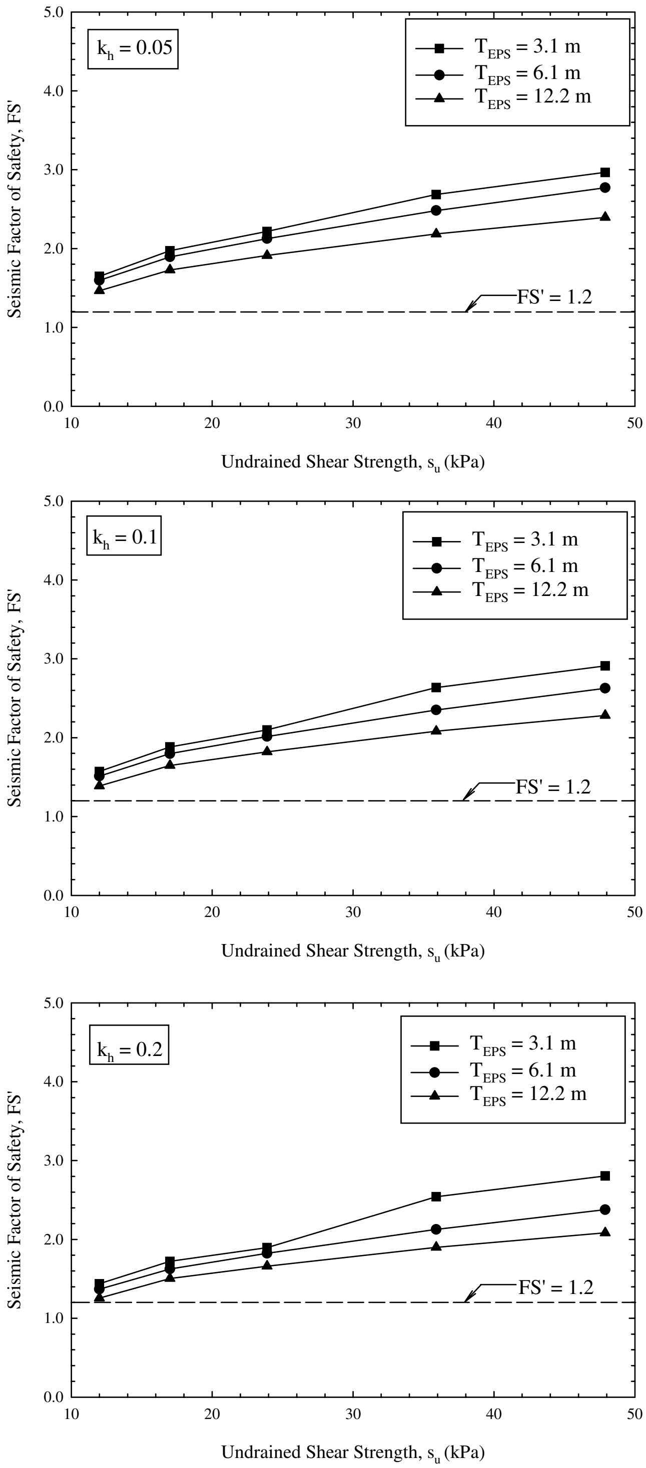


Figure 17. Seismic external stability design chart for a four-lane roadway vertical embankment and a total width of 23 m (76 ft).

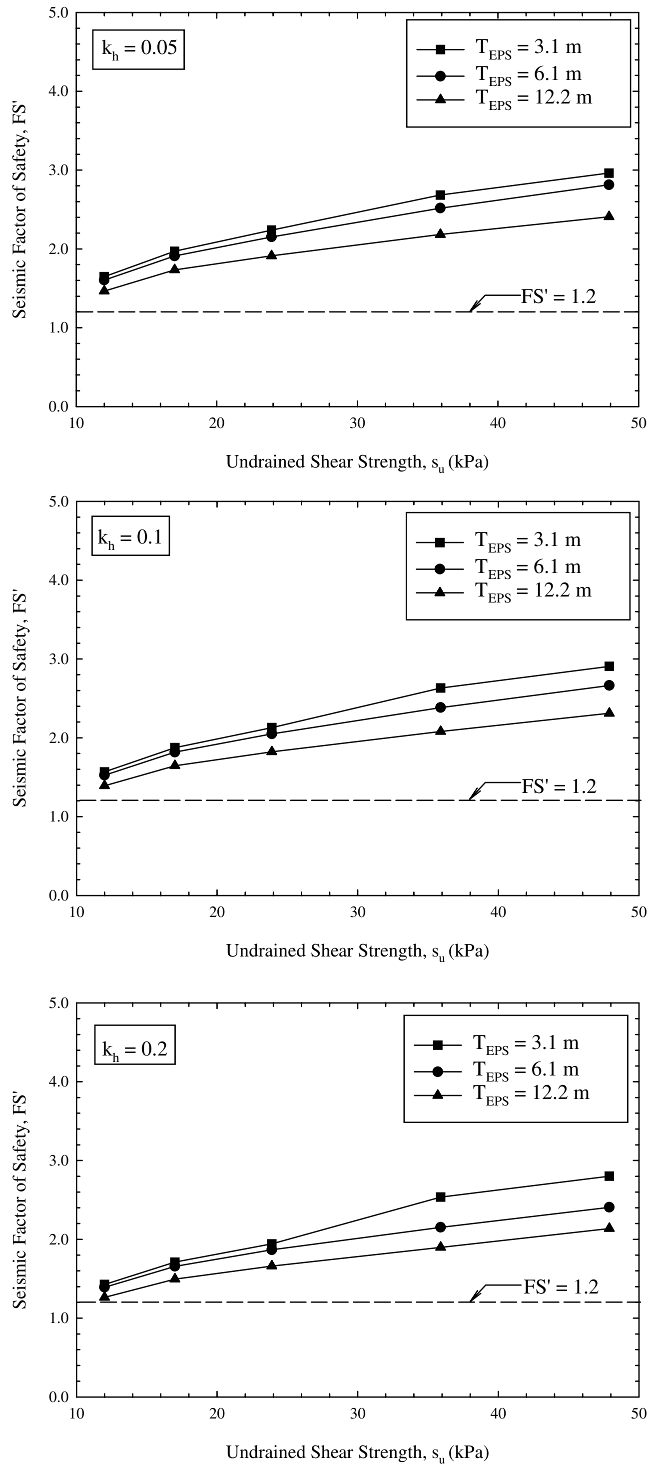


Figure 18. Seismic external stability design chart for a six-lane roadway vertical embankment and a total width of 34 m (112 ft).

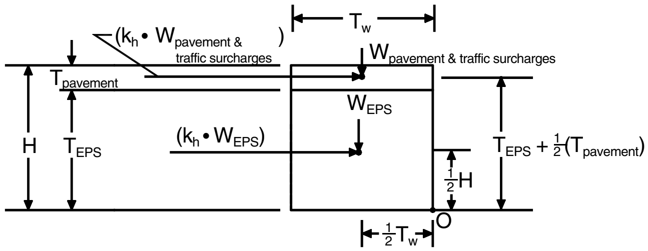


Figure 19. Variables for determining the factor of safety against overturning of a vertical embankment due to pseudo-static horizontal forces used to represent an earthquake loading.

earthquake loading is a temporary loading condition. The factor of safety against overturning is expressed as follows:

$$FS = \frac{\sum \text{stabilizing moments}}{\sum \text{overturning moments}}$$
$$= \frac{\left(\frac{1}{2} * T_w\right) * \left(W_{EPS} + W_{\text{pavement \& traffic surcharges}}\right)}{\left[\left(\frac{1}{2} * H\right) * \left(k_h * W_{EPS}\right)\right] + \left[\left(T_{EPS} + \left(\frac{1}{2} * T_{\text{pavement}}\right)\right) * \left(k_h * W_{\text{pavement \& traffic surcharges}}\right)\right]} \quad (14)$$

Where

T_W = top width,
W_{EPS} = weight of EPS-block geofoam embankment,
$W_{\text{pavement \& traffic surcharges}}$ = weight of the pavement and traffic surcharges,
T_{pavement} = pavement thickness,
k_h = horizontal seismic coefficient used in pseudo-static method,
T_{EPS} = thickness of EPS-block geofoam embankment, and
H = full height of the embankment.

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, $e \leq (T_W/6)$, 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 15 can be used to determine the location of the resultant a distance x from the toe of the embankment, and Equation 16 can be used to determine e. Equation 17 can be used to estimate the maximum and minimum pressures under the embankment.

$$x = \frac{\sum \text{stabilizing moments} - \sum \text{overturning moments}}{\sum N} \quad (15)$$

Where

x = location of the resultant of the forces from the toe of the embankment and
$\sum N$ = summation of normal stresses.

$$e = \frac{T_W}{2} - x \quad (16)$$

Where

e = eccentricity of the resultant of the forces with respect to the centerline of the embankment and
T_W = top width of the embankment.

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

Where

q = soil pressure under the embankment and
q_a = allowable soil pressure.

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

4.6 Hydrostatic Uplift (Flotation)

4.6.1 Introduction

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 20), the factor of safety against upward vertical uplift of the embankment is as follows:

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

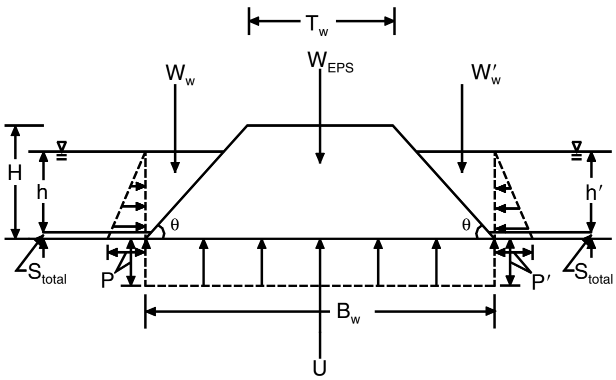


Figure 20. Variables for determining hydrostatic uplift for the case of water equal on both sides of the embankment. (P = pressure exerted on the side of the embankment and U = uplift pressure acting on the base of the embankment.)

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 2,
B_W = bottom embankment width, and
O_{REQ} = additional overburden force required above the EPS blocks to obtain the desired factor of safety.

Equation 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 because 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:

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

The value of O_{REQ} is the additional overburden force required above the EPS blocks to obtain the desired factor of safety in Equation 18 or a factor of safety of 1.2 in Equation 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 the 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, $\gamma_{pavement}$, 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 sideslopes of the embankment and can be calculated using the variables in Figure 21. Therefore, to ensure that the desired factor of safety in Equation 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:

$$O_{REQ} < \left(\gamma_{pavement} * T_{pavement} * T_W\right) + W_{cover} + W_{other} \quad (20)$$

Where

W_{cover} = weight of the cover soil and
W_{other} = other weights.

Design charts (see Figures 22 through 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, H (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 20 as shown below:

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

Where

γ_{EPS} = unit weight of the geofoam.

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. For example, Figure 22 presents the hydrostatic uplift design charts for a 4H:1V (14-degree) embankment with the tailwater level equal to the upstream water level. If the proposed geofoam embankment has a four-

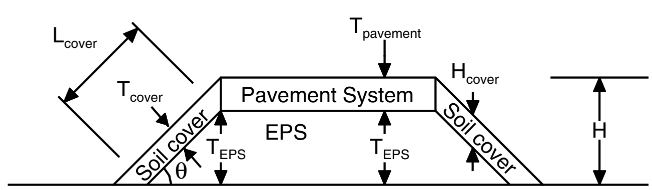


Figure 21. Variables for the weight induced by the soil cover. (L_{cover} = length of soil cover on the side of the embankment, T_{cover} = perpendicular thickness of the soil cover, and H_{cover} = vertical thickness of the soil cover.)

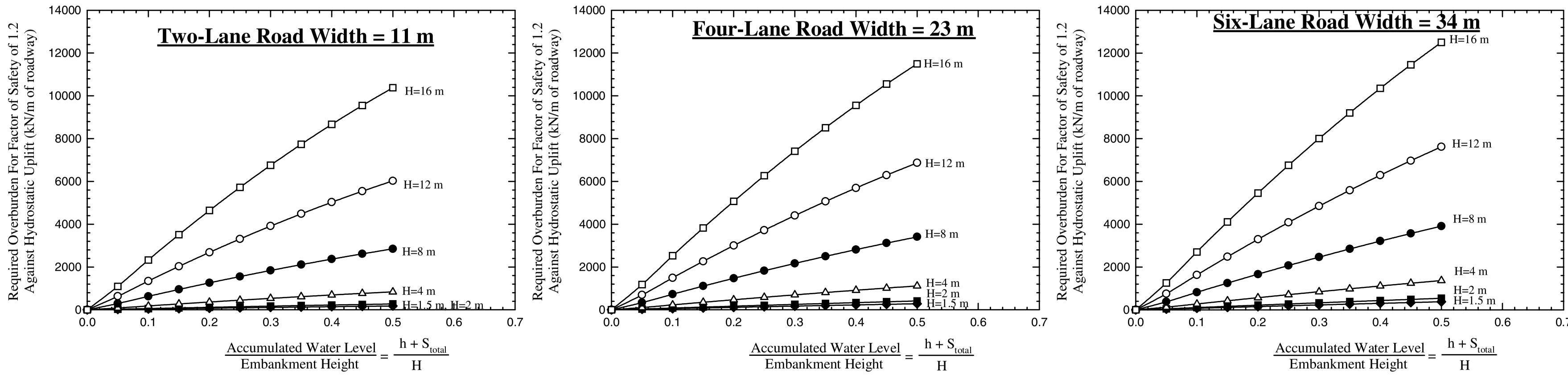


Figure 22. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with tailwater level equal to upstream water level, 4H:1V embankment slope, and three road widths.

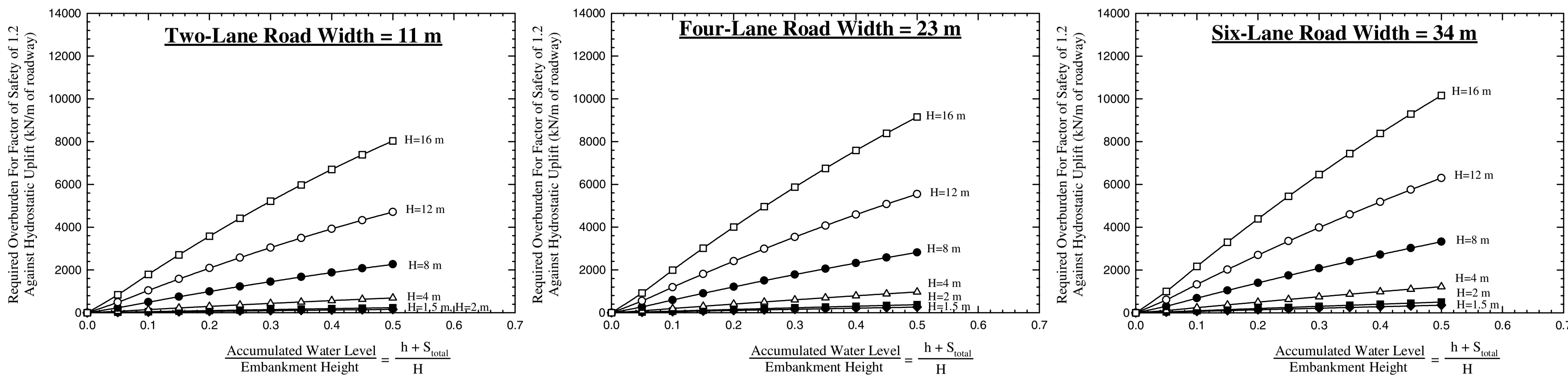


Figure 23. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with tailwater level equal to upstream water level, 3H:1V embankment slope, and three road widths.

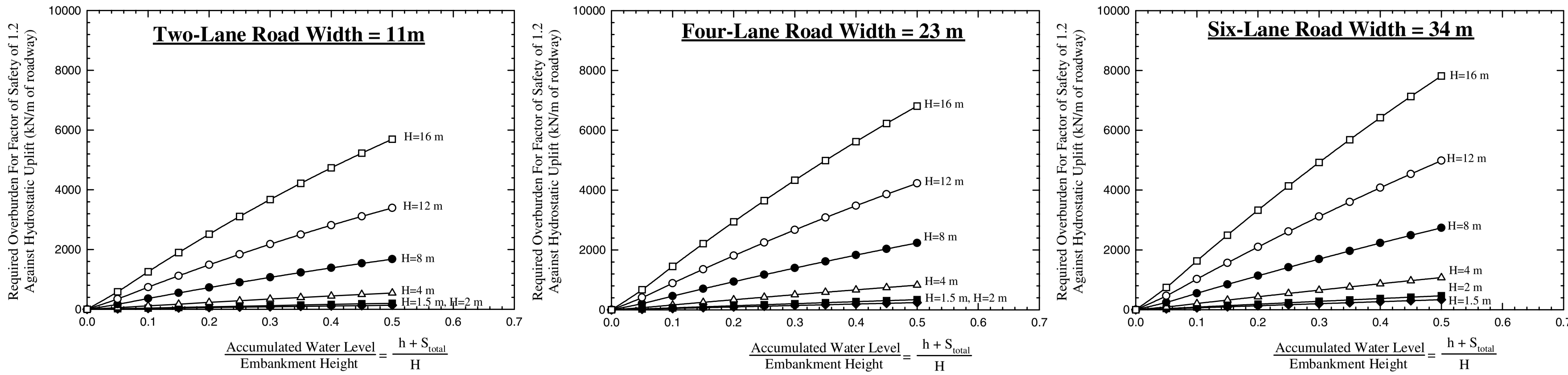


Figure 24. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with tailwater level equal to upstream water level, 2H:1V embankment slope, and three road widths.

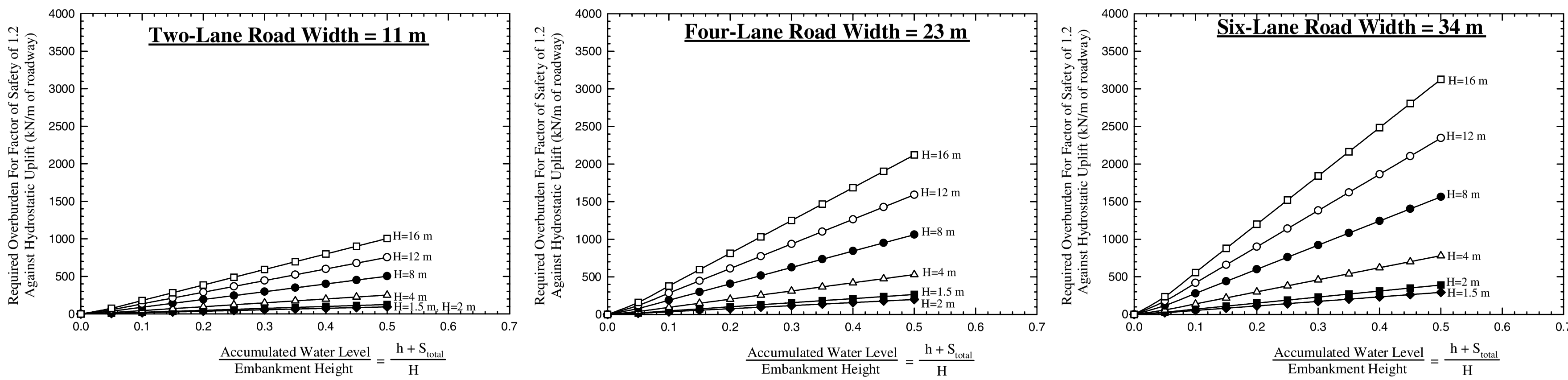


Figure 25. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with tailwater level equal to upstream water level, vertical embankment (0H:1V), and three road widths.

lane roadway (middle chart), a height of 12 m (40 ft), and a ratio of accumulated water level to embankment height of 0.2, which means the total water depth to include the estimated total settlement is 20 percent of the embankment height, the required value of O_{REQ} is approximately 936 kN/m (62 kip/ft) per length of embankment. If the typical pavement system with a $T_{pavement}$ of 1,000 mm (39 in.) used in previous external stability calculations is used, the pavement weight, $W_{pavement}$, equals the surcharge times the pavement width:

$$W_{pavement} = 21.5\,kN/m^2 * 23.2\ m$$
$$= 498.8\,kN/m\ \text{of roadway} \tag{22}$$

If the typical cover soil thickness of 0.46 m (1.5 ft) and moist unit weight of 18.9 kN/m^3 (120 lbf/ft^3) used in previous external stability calculations is used, the cover soil weight equals:

$$W_{cover} = 2 * \left(\gamma_{cover} * \frac{T_{EPS}}{\sin\theta} * \frac{T_{cover}}{\cos\theta}\right)$$
$$= 2 * \left(18.9\,kN/m^3 * \frac{12\ m}{\sin 14^\circ} * \frac{0.46\ m}{\cos 14^\circ}\right)$$
$$= 889\,kN/m \tag{23}$$

Where

γ_{cover} = unit weight of the cover,
T_{EPS} = thickness of EPS-block geofoam embankment, and
T_{cover} = thickness of the cover soil over the EPS-geofoam embankment.

From Equation 17 and assuming an EPS40,

$$936\ kN/m = O_{REQ} < 498.8\,kN/m$$
$$-\left(0.16\,kN/m^3 * 1\ m * 23\ m\right)$$
$$+889\,kN/m \tag{24}$$
$$936\ kN/m = O_{REQ} < 1{,}384.1\,kN/m\ \text{of roadway}$$

Thus, the pavement and cover soil will provide sufficient overburden for a factor of safety of 1.2.

Equal water level on both sides of the embankment is the worst-case scenario, and construction measures should be taken to try to avoid the situation of equal water level being created on both sides of the embankment. Figures 22 through 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 22 through 25 are the required weight of material over the EPS blocks in kilonewtons per linear meter of embankment length. Embankment top widths of 11m (36 ft), 23 m (76 ft), and 34 m (112 ft); sideslope inclinations of 0H:1V, 2H:1V, 3H:1V, and 4H:1V; and six heights between 1.5 m (4.92 ft) and 16 m (52.49 ft) were used in developing the charts. 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 are 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 26 shows the variable for determining hydrostatic uplift analysis for the case of water on one side of the embankment only. Equation 25 can be used to obtain the factor of safety against hydrostatic uplift.

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

Where

W_{EPS} = weight of EPS-block geofoam embankment,
W_W = vertical component of weight of water on the geofoam embankment fact above the base of the embankment on the accumulated water side,
γ_W = unit weight of water, and
B_W = bottom embankment width.

Equation 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:

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

Figures 27 through 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

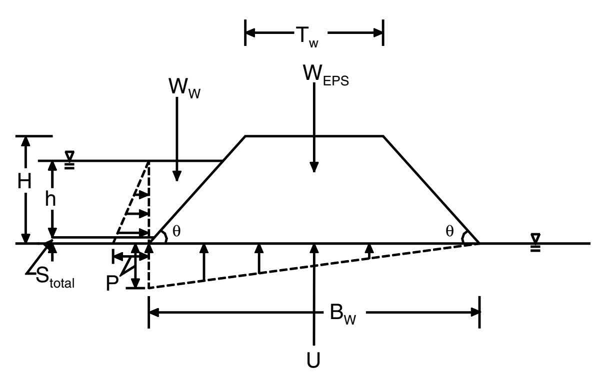


Figure 26. Variable for determining hydrostatic uplift analysis for the case of water on one side of the embankment only. (P = pressure exerted on the side of the embankment and U = uplift pressure acting on the base of the embankment.)

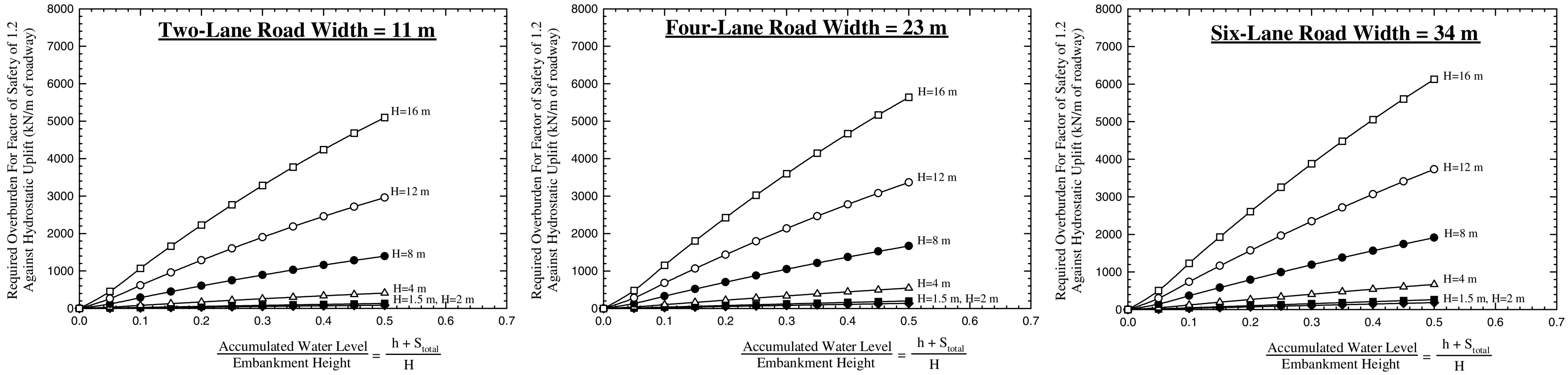


Figure 27. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with no tailwater, 4H:1V embankment slope, and three road widths.

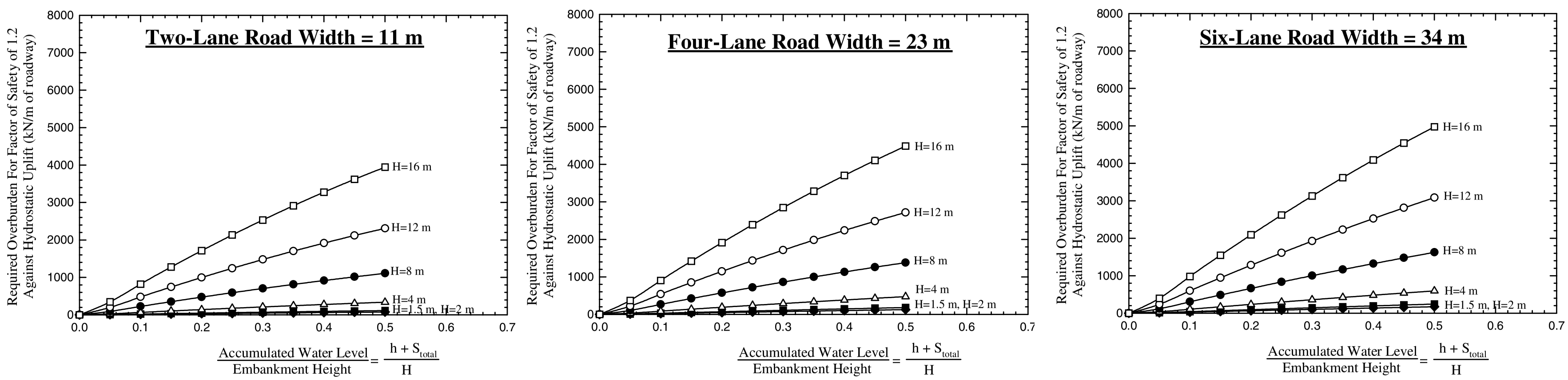


Figure 28. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with no tailwater, 3H:1V embankment slope, and three road widths.

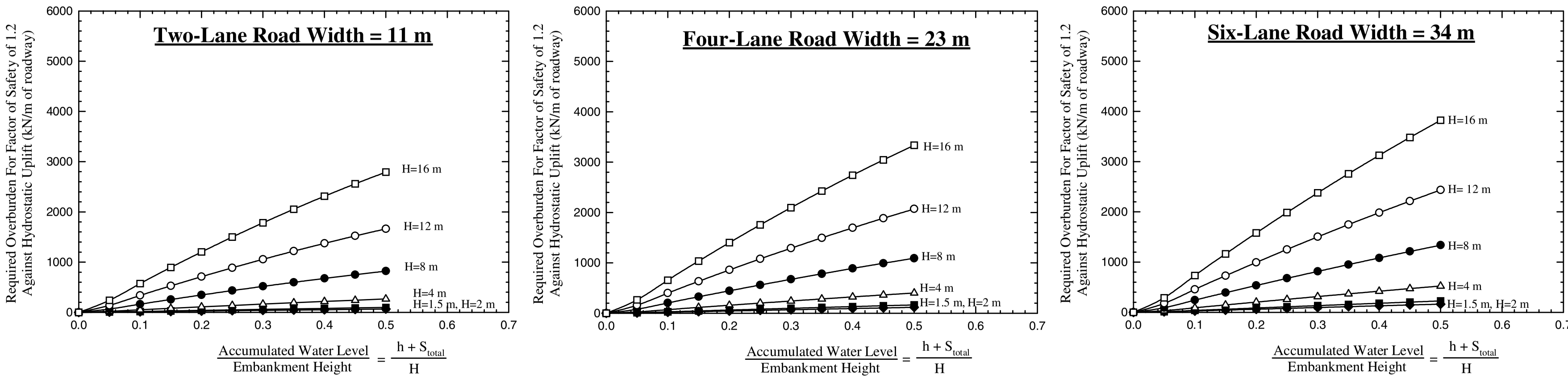


Figure 29. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with no tailwater, 2H:1V embankment slope, and three road widths.

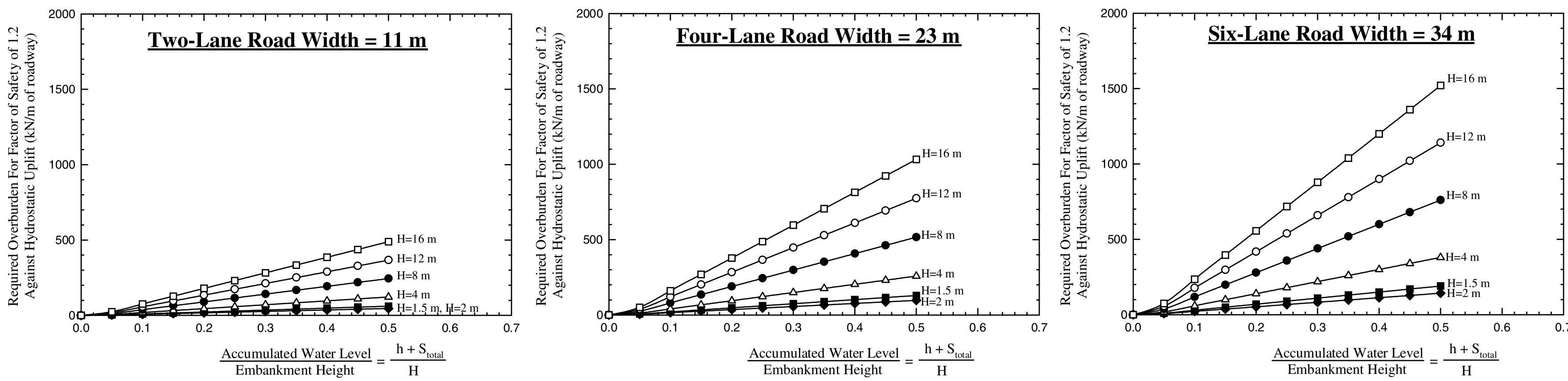


Figure 30. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with no tailwater, vertical embankment (0H:1V), and three road widths.

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.

4.7 Translation and Overturning Due to Water (Hydrostatic Sliding and Overturning)

4.7.1 Introduction

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, one must consider 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.

4.7.2 Translation

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, a possible failure mechanism is for the entire embankment to slide under an unbalanced water pressure loading. 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 is simplified to the following:

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

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 2, and
B_W = bottom of embankment width.

The other variables were previously defined in Section 4.6.1. For a factor of safety of 1.2 and solving for O_{REQ}, Equation 27 becomes:

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

Equation 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 that the desired factor of safety in Equation 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 20. Figures 31 through 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. Embankment top widths of 11m (36 ft), 23 m (76 ft), and 34 m (112 ft); sideslope inclinations of 0H:1V, 2H:1V, 3H:1V, and 4H:1V; and six heights between 1.5 m (4.9 ft) and 16 m (52.5 ft) were used in developing the charts. For example, the design charts for Figures 31 through 34 correspond to slope inclinations of 4H:1V, 3H:1V, 2H:1V, and 0H:1V, respectively. 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 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., $h + S_{total}$.

4.7.3 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

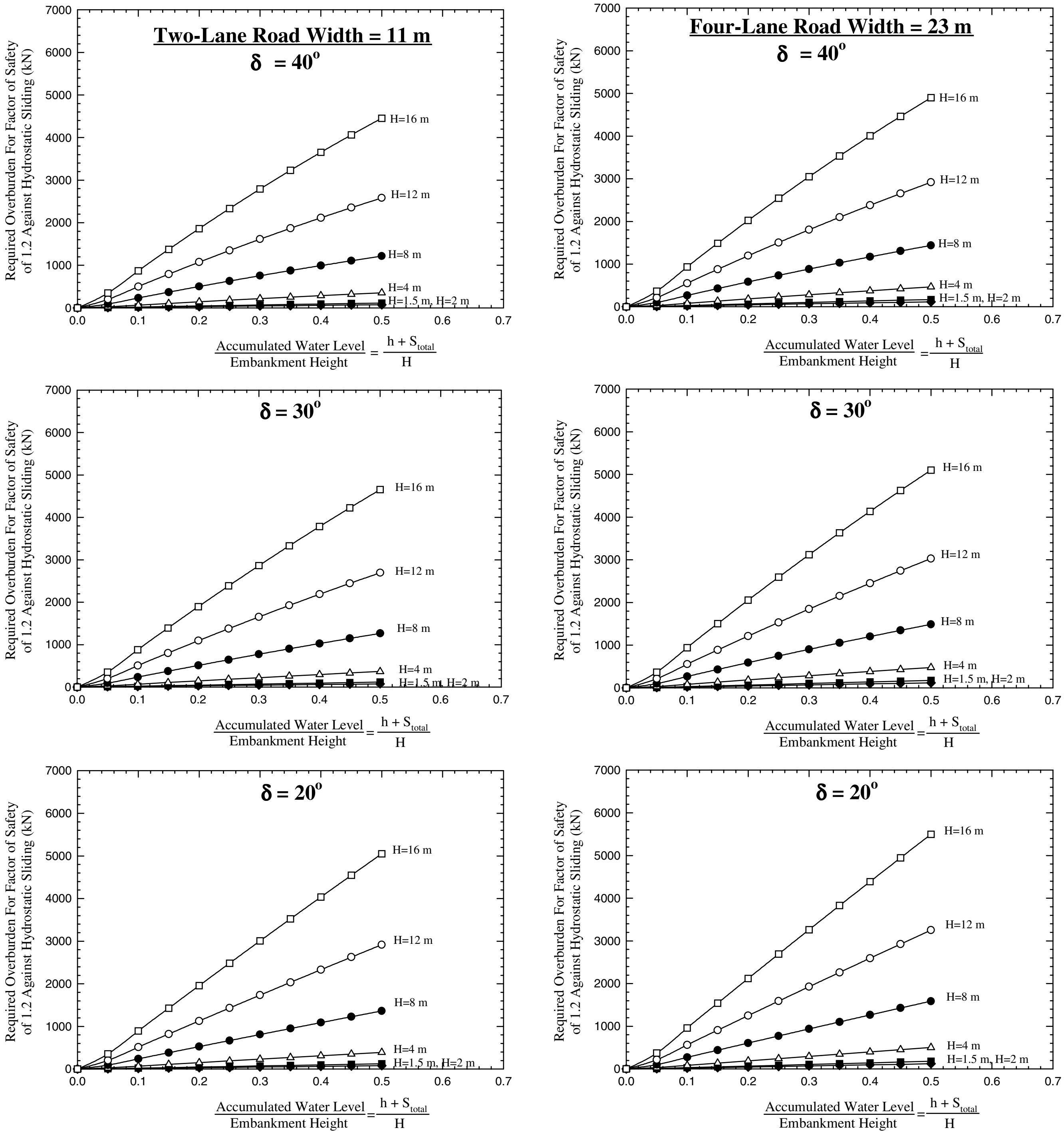


Figure 31. Hydrostatic sliding (translation due to water) design for a factor of safety of 1.2 with no tailwater, 4H:1V embankment slope, and three road widths for various interface friction angles.

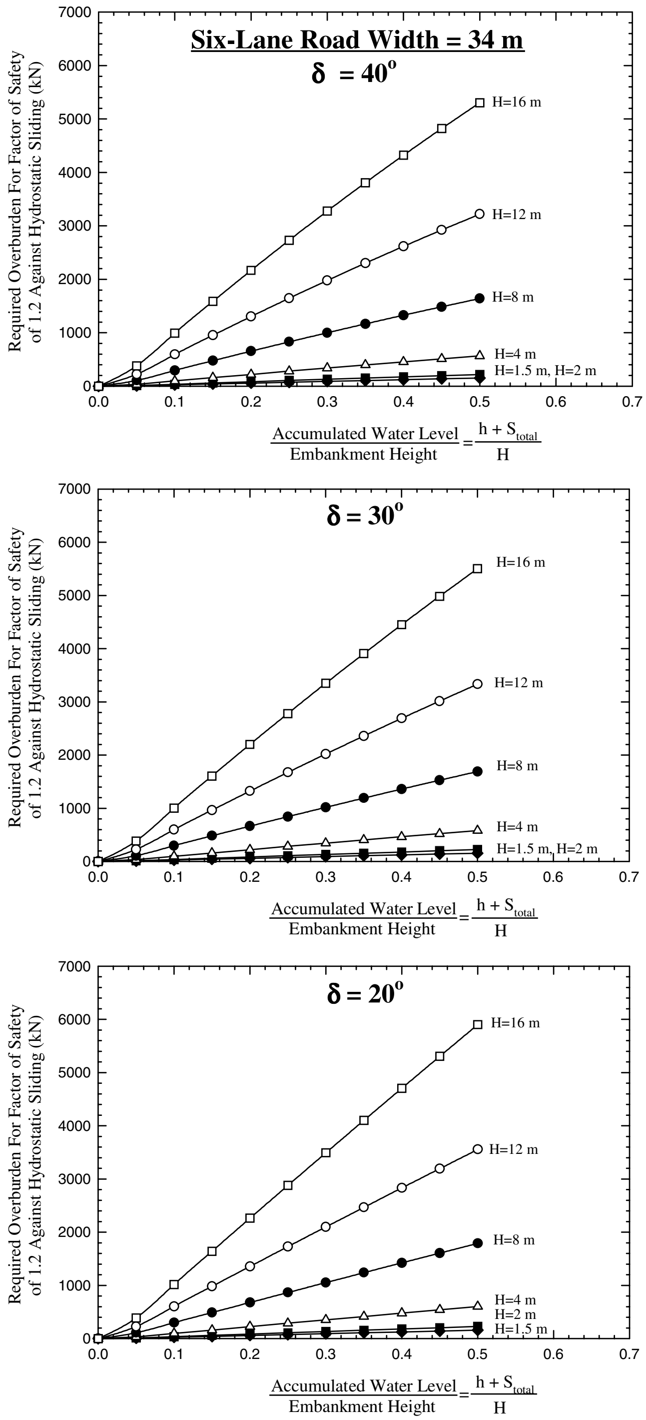


Figure 31. (*Continued*)

Figure 35. The worst-case scenario is water accumulating on only one side of the embankment, as shown in Figure 26. 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.

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

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

$$= \frac{\left(\frac{1}{2} * T_W\right) * \left(W_{EPS} + O_{REQ}\right)}{\frac{1}{3}\left(h + S_{total}\right) * R_p} \tag{29}$$

Where

R_p = resultant force acting on the side of the embankment.

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 29 becomes:

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

Equation 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 15 and 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 17 can be used to determine the maximum and minimum pressures under the embankment.

4.8 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 with other types of lightweight fill. Additionally, for vertical geofoam embankments, one must consider 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.

However, the findings of NCHRP Project 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 NCHRP Project 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

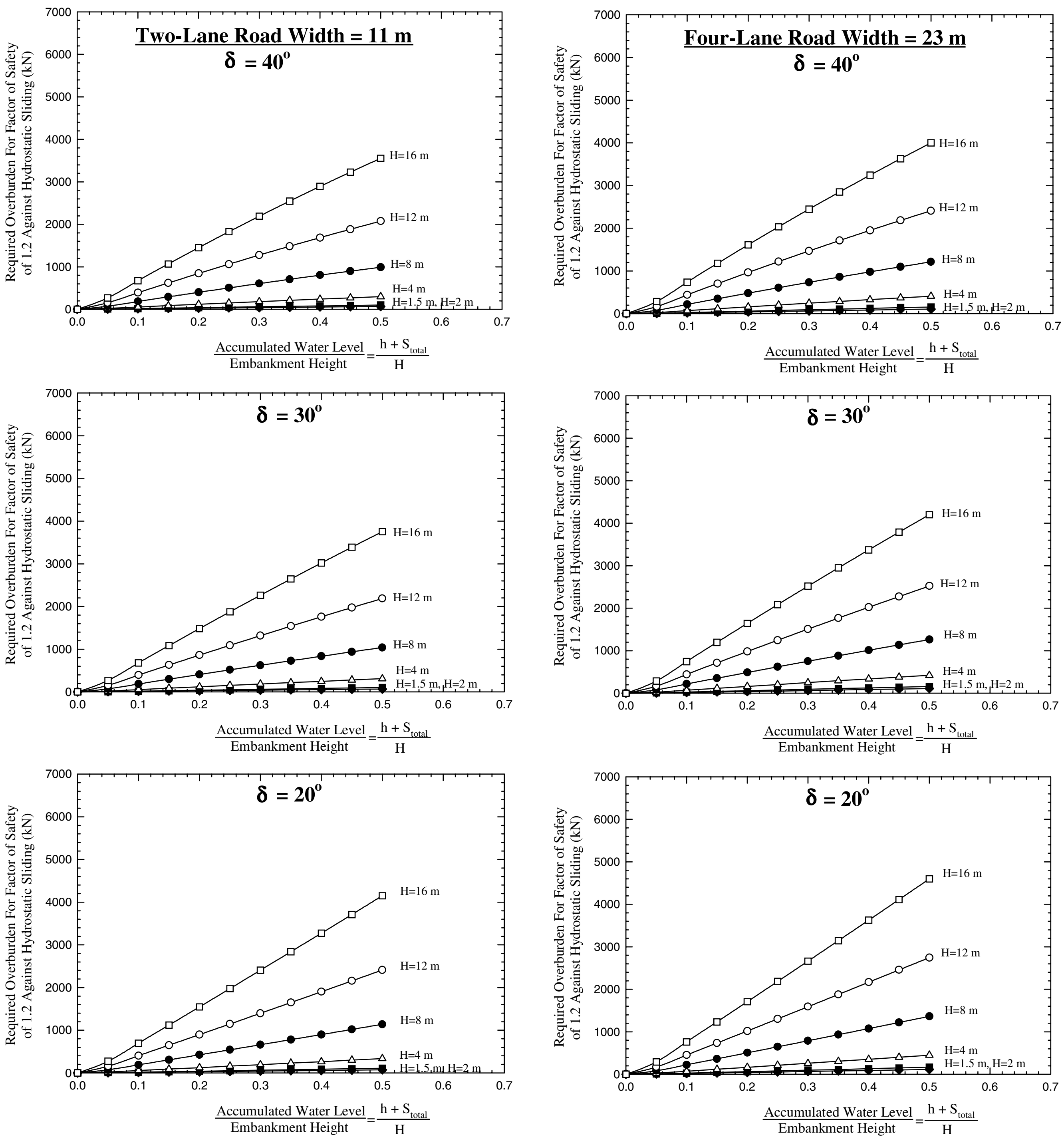


Figure 32. Hydrostatic sliding (translation due to water) design for a factor of safety of 1.2 with no tailwater, 3H:1V embankment slope, and three road widths for various interface friction angles.

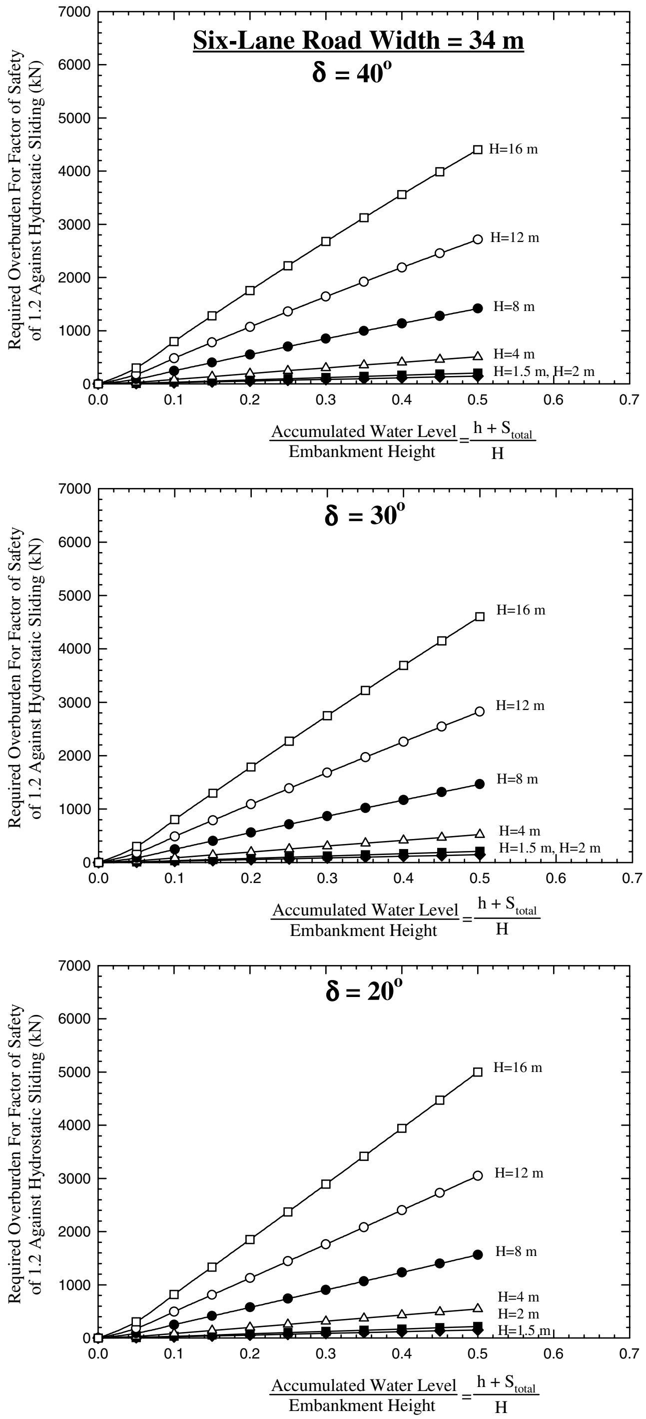


Figure 32. (*Continued*)

considered if the embankment will be subjected to hurricane force winds.

A wind analysis procedure is presented in the NCHRP Project 24-11 final report (available online as *NCHRP Web Document 65*) 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 a new design procedure is a topic for future research.

5 INTERNAL STABILITY EVALUATION

5.1 Introduction

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 that 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).

5.2 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 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 20. Figures 31 through 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 30 through 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 21.

The thickness of EPS blocks typically ranges from 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.

5.3 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 NCHRP Project 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 NCHRP Project 24-11 and the absence of documented sliding failure due to wind loading, it is recommended that the

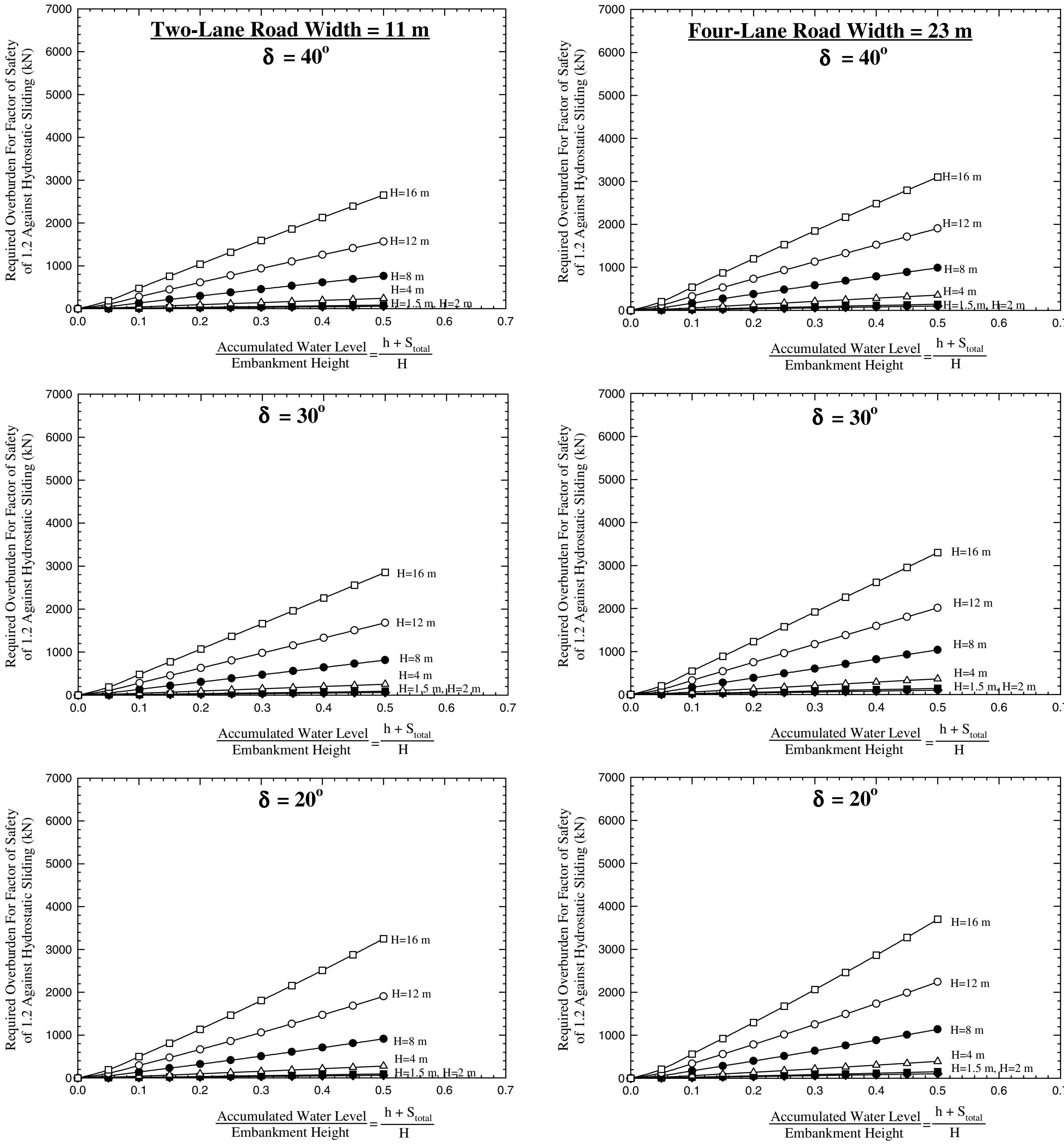


Figure 33. Hydrostatic sliding (translation due to water) design for a factor of safety of 1.2 with no tailwater, 2H:1V embankment slope, and three road widths for various interface friction angles.

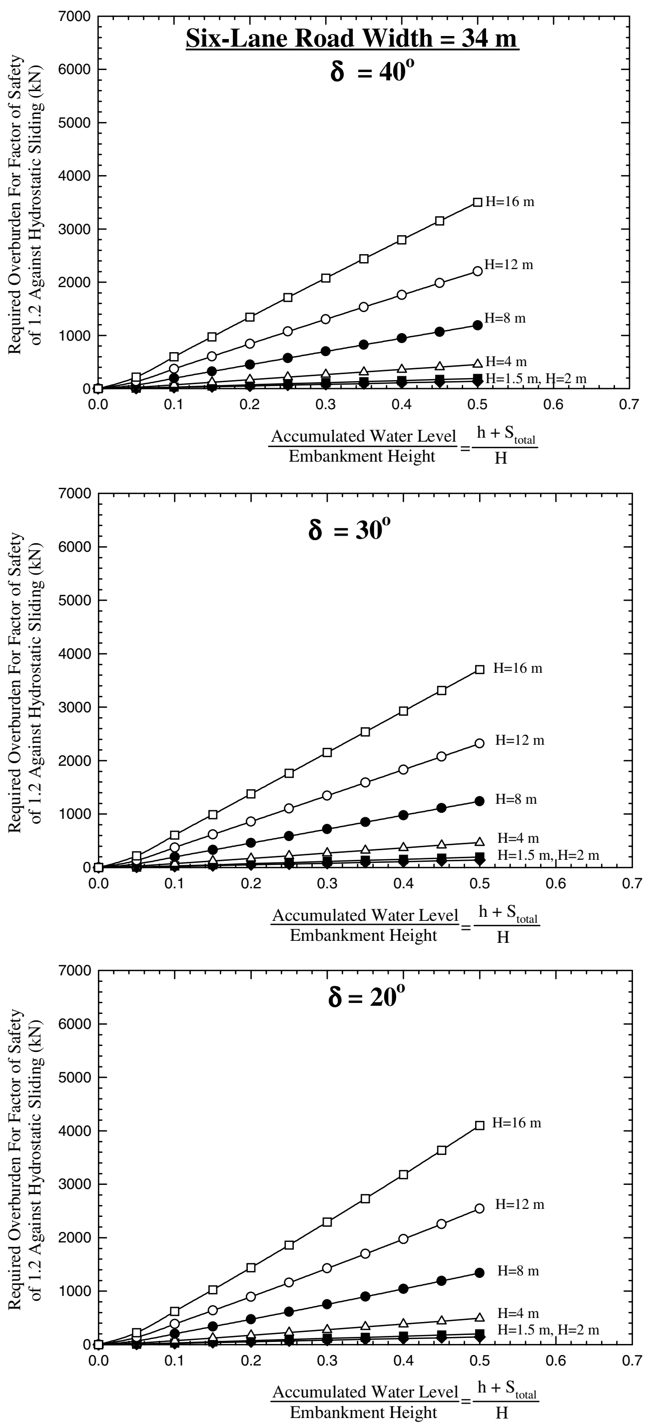


Figure 33. (*Continued*)

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 *NCHRP Web Document 65* 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 a new design procedure is a topic for future research.

5.4 Internal Seismic Stability

5.4.1 Trapezoidal Embankments

5.4.1.1 Introduction and Typical Cross Section. This section focuses on the effect of seismic forces on the internal stability of EPS-block geofoam trapezoidal embankments. The main difference between 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 noncircular 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 as follows:

1. Identify the potential critical static failure surfaces (i.e., the static failure surface with the lowest factor of safety) that pass through the EPS embankment or an EPS interface at the top or bottom of the EPS. This step 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 the 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 use 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 sideslopes of 2H:1V that was used in the pseudo-static internal stability analyses is shown in

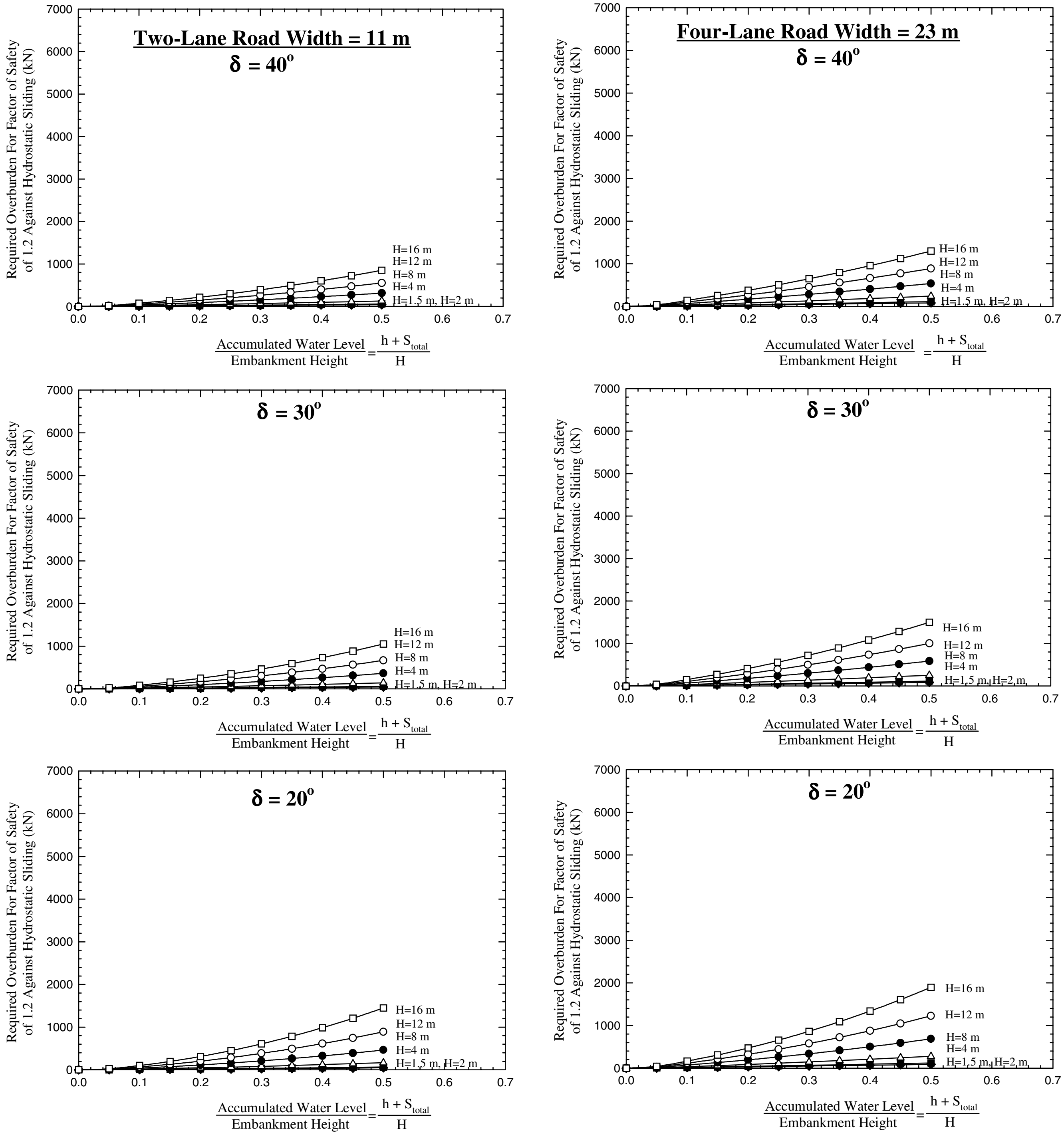


Figure 34. Hydrostatic sliding (translation due to water) design for a factor of safety of 1.2 with no tailwater, vertical embankment (0H:1V), and three road widths for various interface friction angles.

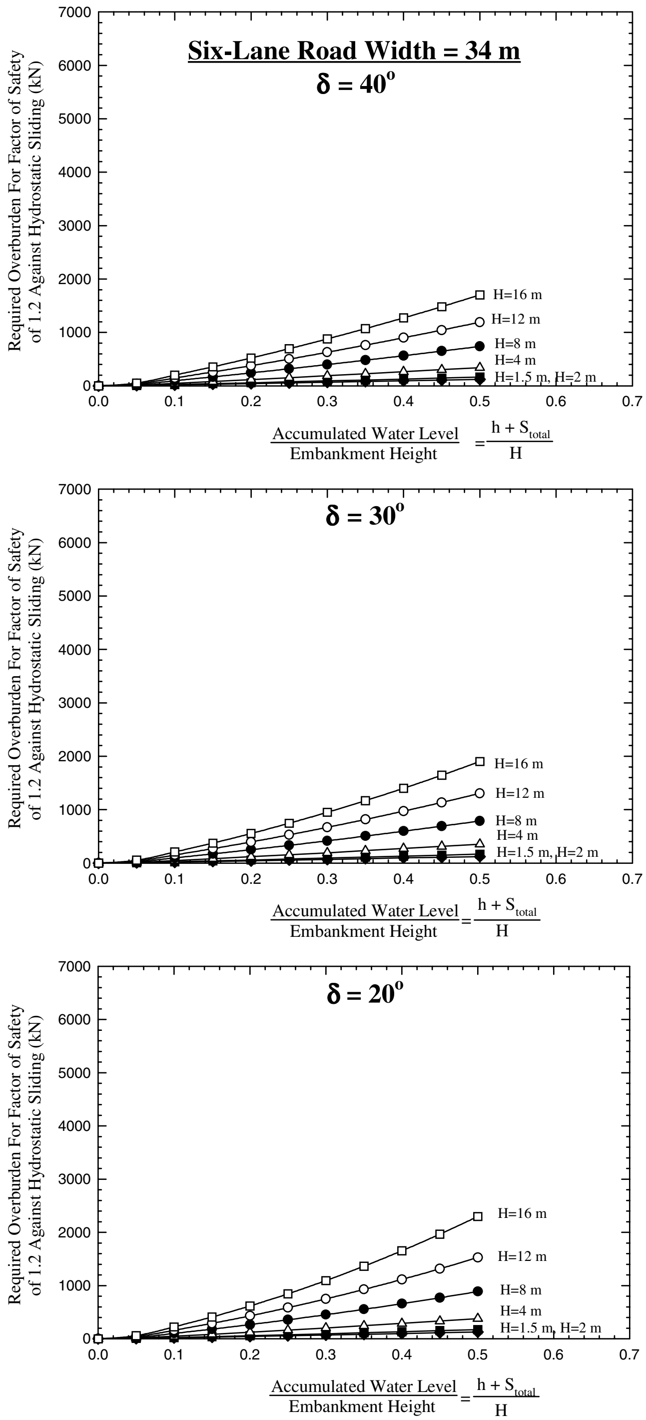


Figure 34. (*Continued*)

Figure 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^2), 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 sideslopes of the embankment is also 0.46 m (1.5 ft) thick, which is typical for the sideslopes, and is assigned a typical moist unit weight of 18.9 kN/m^3 (120 lbf/ft^3).

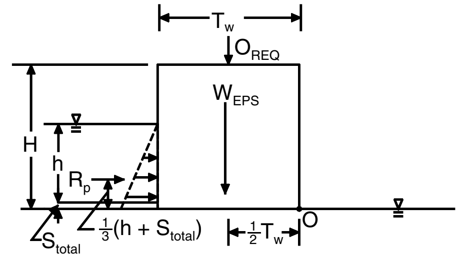


Figure 35. Variables for determining the factor of safety against overturning due to hydrostatic horizontal forces for the case of water on one side of the embankment.

Figure 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 can 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 will 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 will 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.

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 use geofoam. Three seismic coefficients—low (0.05), medium (0.10), and high (0.20)—were used for each roadway embankment.

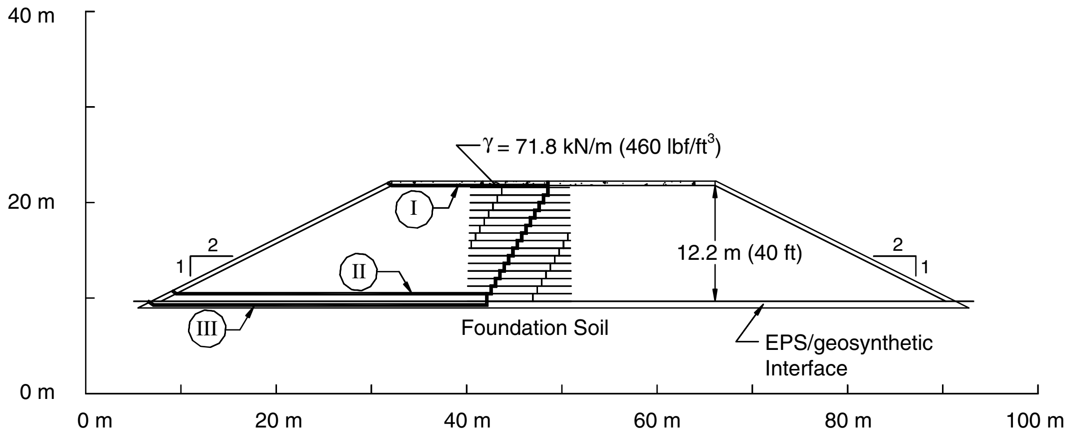


Figure 36. Typical trapezoidal cross section used in seismic internal slope stability analyses with the three applicable failure modes.

5.4.1.2 Design Chart. The internal seismic stability design chart in Figure 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 ft)—even though the chart is based on a sideslope inclination of 2H:1V, a two-lane roadway, and an embankment height from 3.1 m (10 ft) to 12.2 m (40 ft). The chart represents the worst-case scenario and is not sensitive to the range of geometries considered. 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 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 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.

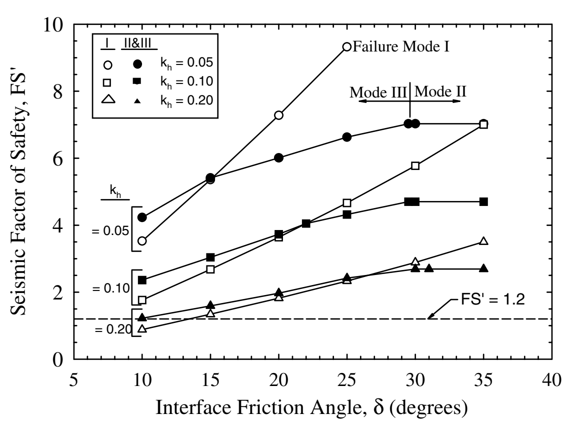


Figure 37. Design chart for internal seismic stability of EPS trapezoidal embankments.

5.4.2 Vertical Embankments

5.4.2.1 Introduction and Typical Cross Section. This section focuses on the effect of seismic forces on the internal stability of EPS-block geofoam embankments with vertical walls. The main difference between this analysis and the analysis for external seismic stability of embankments with vertical walls in Section 4.5.2 is that sliding is assumed to occur only within the geofoam embankment or along an EPS interface. This analysis uses the same pseudo-static slope stability analysis used for internal seismic stability of trapezoidal embankments in Section 5.4.1 and noncircular failure surfaces through the EPS or the EPS interface at the top or bottom of the embankment.

A typical cross section through a vertical EPS embankment used in the internal static stability analyses is shown in Figure 38. This cross section is similar to the cross section used for static analyses of vertical embankments in Figure 10, but differs from the cross section used for the static analyses of trapezoidal embankments in Figure 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/m³ (345 lbf/ft³). The soil layer is 0.61 m (2 ft) thick to represent the minimum recommended pavement section thickness discussed in Section 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/m² (690 lbs/ft²). A vertical stress of 33.0 kN/m²

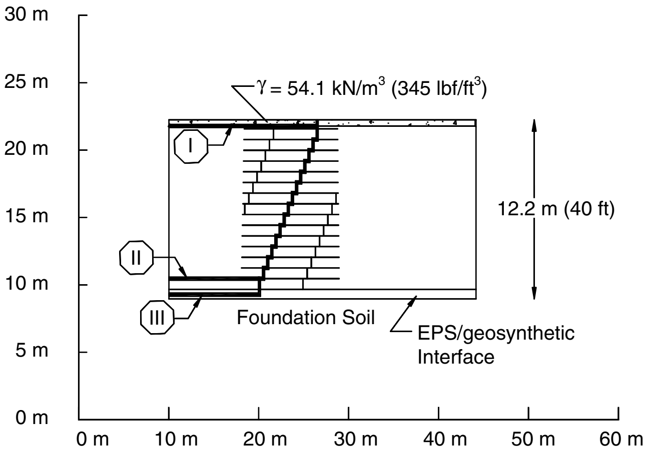


Figure 38. Typical cross section used in seismic internal slope stability analyses for vertical embankments with the three applicable failure modes.

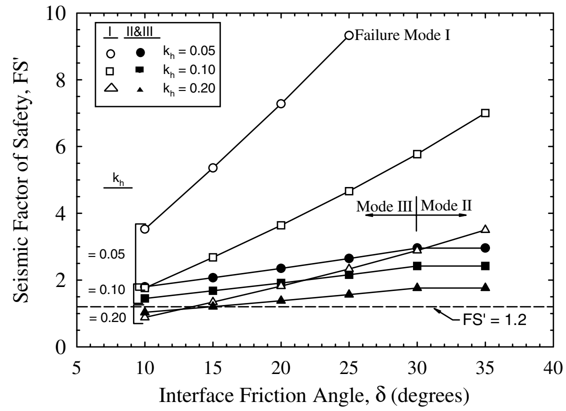


Figure 39. Design chart for internal seismic stability of EPS vertical embankments.

(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 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 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 the seismic internal slope stability analysis of trapezoidal embankments, and a description of each is included in Section 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 use geofoam. Three seismic coefficients—low (0.01), medium (0.10), and high (0.20)—were used for each roadway embankment.

5.4.2.2 Design Chart. The internal seismic stability design chart for vertical embankments in Figure 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 factors 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 ft)—even though the chart 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 7). However, an important aspect of Figure 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.

5.5 Load Bearing

5.5.1 Introduction

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 used 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 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 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 will ensure that no part of a block (where the density might be somewhat lower than the overall average) becomes overstressed.

TABLE 8 Minimum allowable values of elastic limit stress and initial tangent Young's modulus for the proposed AASHTO EPS material designations

Material Designation	Dry Density of Each Block as a Whole, kg/m^3 (lbf/ft^3)	Dry Density of a Test Specimen, kg/m^3 (lbf/ft^3)	Elastic Limit Stress, kPa (lbs/in^2)	Initial Tangent Young's Modulus, MPa (lbs/in^2)
EPS40	16 (1.0)	15 (0.90)	40 (5.8)	4 (580)
EPS50	20 (1.25)	18 (1.15)	50 (7.2)	5 (725)
EPS70	24 (1.5)	22 (1.35)	70 (10.1)	7 (1015)
EPS100	32 (2.0)	29 (1.80)	100 (14.5)	10 (1450)

5.5.2 Design Procedure

The procedure for evaluating the load-bearing capacity of EPS as part of internal stability is outlined in the following thirteen steps:

1. Estimate the traffic loads.
2. Add impact allowance to the traffic loads.
3. Estimate traffic stresses at the top of EPS blocks.
4. Estimate gravity stresses at the top of EPS blocks.
5. Calculate total stresses at the top of EPS blocks.
6. Determine the minimum required elastic limit stress for EPS under the pavement system.
7. Select the appropriate EPS block to satisfy the required EPS elastic limit stress for underneath the pavement system, e.g., EPS50, EPS70, or EPS100.
8. Select the preliminary pavement system type and determine whether 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 the minimum required elastic limit stress at various depths.
13. Select the 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.

5.5.2.1 Step 1: Estimate the Traffic Loads. Figure 40 shows the wheel configuration of a typical semitrailer truck with a tandem axle with dual tires at the rear. Trucks with a tridem axle, each spaced at 122 to 137 cm (48 to 54 in.) apart, and dual tires also exist. The largest live or traffic load expected on the roadway above the embankment should be used for design. The magnitude and vehicle tire configuration that will provide the largest live load is typically not known during the preliminary design phase. Therefore, the AASHTO standard classes of highway loading (*12*) can be used for preliminary load-bearing analyses.

5.5.2.2 Step 2: Add Impact Allowance to the Traffic Loads. Allowance for impact forces from dynamic, vibratory, and impact effects of traffic is generally only considered where these forces act across the width of the embankment or adjacent to a bridge abutment. An impact coefficient of 0.3 is recommended for design of EPS-block geofoam (*15*). Equation 31 can be used to include the impact allowance to the live

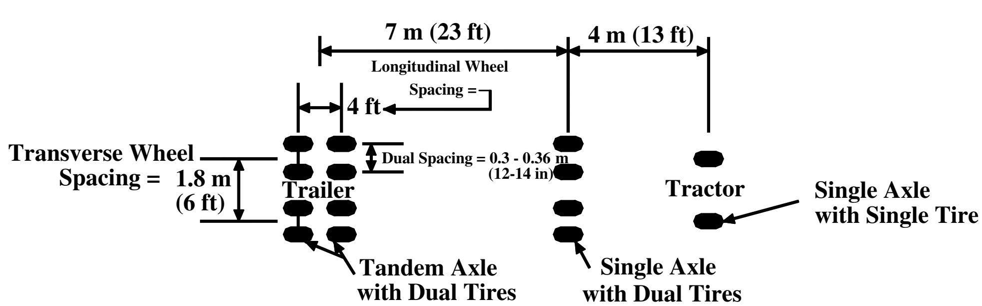


Figure 40. Wheel configuration of a typical semitrailer truck (14).

loads estimated in Step 1 if impact loading is deemed necessary for design:

$$Q = LL * (1 + I) = LL * 1.3 \tag{31}$$

Where

Q = traffic load with an allowance for impact,
LL = live load for traffic from AASHTO standard classes of highway loading (*12*) obtained in Step 1, and
I = impact coefficient = 0.3.

5.5.2.3 Step 3: Estimate Traffic Stresses at the Top of EPS Blocks. The objective of this step is to estimate the dissipation of vertical stress through the pavement system so that an estimate of the traffic stresses at the top of the EPS blocks can be obtained. The vertical stress at the top of the EPS is used to evaluate the load-bearing capacity of the blocks directly under the pavement system. Various pavement systems, with and without a separation layer between the pavement system and the EPS blocks, should be evaluated to determine which alternative is the most cost-effective.

The contact pressure applied by a tire to the top of the pavement is typically assumed to be equal to the tire pressure (*14*), and the tire and pavement surface interface is assumed to be free of shear stress. A tire pressure of 689 kPa (100 lbs/in^2) appears to be representative and is recommended for preliminary design purposes. This tire pressure is near the high end of typical tire pressures, but is used for analysis purposes by transportation software such as ILLI-PAVE (*16*).

The contact pressure is converted to a traffic load by multiplying by the contact area of the tire. For the case of a single axle with a single tire, the contact area is given by Equation 32, and the radius of the contact area is given by Equation 33:

$$A_C = \frac{Q_t}{q} \tag{32}$$

$$r = \left(\frac{A_C}{\pi}\right)^{\frac{1}{2}} \tag{33}$$

Where

A_C = contact area of one tire,
Q_t = live load on one tire,
q = contact pressure = tire pressure, and
r = radius of contact area.

For the case of a single axle with dual tires, the contact area can be estimated by converting the set of duals into a singular circular area by assuming that the circle has an area equal to the contact area of the duals, as indicated by Equation 34. The radius of contact is given by Equation 35. Equation 34 yields a conservative value, i.e., smaller area, for the contact area because the area between the duals is not included.

$$A_{CD} = \frac{Q_D}{q} \tag{34}$$

$$r = \left(\frac{A_{CD}}{\pi}\right)^{\frac{1}{2}} \tag{35}$$

Where

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

The recommended procedure for estimating the stress at the top of the EPS is Burmister's elastic layered solution (*17*). Burmister's elastic layered solution is based on a uniform pressure applied to the surface over a circular area on top of an elastic half-space mass. The primary advantage of Burmister's theory is that it considers the influence of layers with different elastic properties within the system being considered. Design charts are presented in Figures 41 through 43 that alleviate the use of computer software to use Burmister's elastic layered solutions. The procedure used to obtain the results shown in these figures is presented in Chapter 6.

Figure 41 presents values of vertical stress on top of the EPS blocks due to a single or dual wheel load, σ_{LL}, on an asphalt concrete pavement system. The asphalt concrete pavement system represented by Figure 41 consists of the indicated asphalt thickness with a corresponding crushed stone base thickness equal to 610 mm (24 in.) less the thickness of the asphalt. Figure 42 presents values of σ_{LL} on a portland cement concrete (PCC) pavement system. The PCC pavement system represented in Figure 42 consists of the indicated PCC thickness with a corresponding crushed stone base thickness equal to 610 mm (24 in.) less the thickness of the PCC. Figure 43 presents values of σ_{LL} on a composite pavement system defined here as an asphalt concrete pavement system with a PCC slab separation layer placed between the asphalt concrete pavement system and the EPS-block geofoam. The composite pavement system represented in Figure 43 consists of the indicated asphalt thickness plus a 102-mm (4-in.) concrete separation layer with a corresponding crushed stone base thickness equal to 610 mm (24 in.) less the thickness of the asphalt concrete and the separation slab. In all three figures, the minimum recommended pavement system thickness of 610 mm (24 in.) to minimize the potential for differential icing and solar heating was used. Both a single tire and a set of dual tires were modeled as a single contact area. Therefore, both a single tire and a set of dual tires can be represented by the total load of a single tire or of the dual tires and a contact area.

In summary, the vertical stress charts in Figures 41 through 43 can be used to estimate the applied vertical stress on top of the EPS due to a tire load, σ_{LL}; on top of an asphalt concrete, PCC system; and on top of a composite pavement system, respectively. For example, the vertical stress applied to the top of the EPS blocks under a 178-mm (7-in.)-thick asphalt pavement with a total wheel load of 100 kN (225 kips) is approximately 55 kPa (8 lbs/in^2) (see Figure 41). This value of 55 kPa (8 lbs/in^2) is then used in the load-bearing analysis described subsequently.

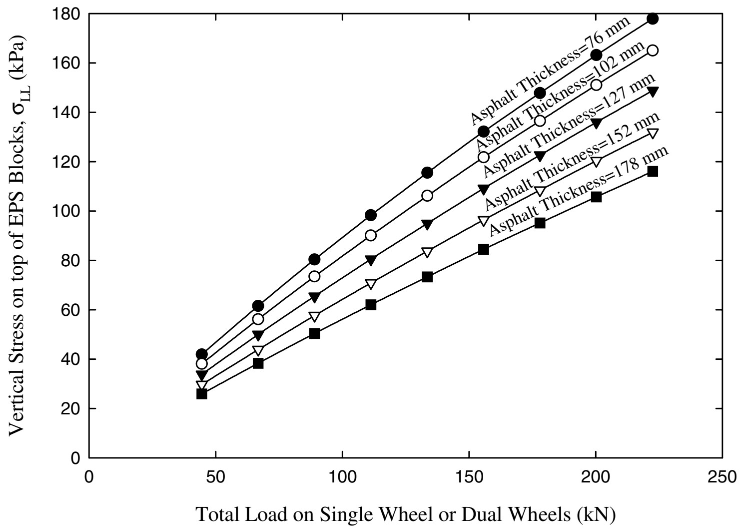


Figure 41. Vertical stress on top of the EPS blocks, σ_{LL}, due to traffic loads on top of a 610-mm (24-in.) asphalt concrete pavement system.

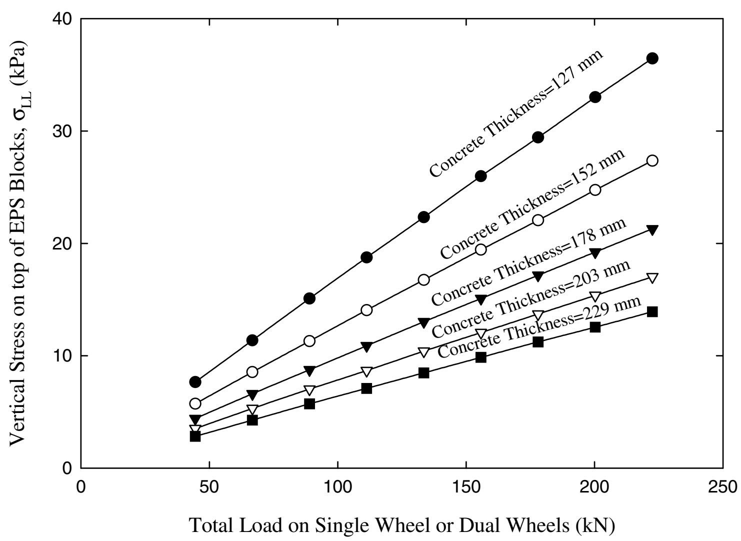


Figure 42. Vertical stress on top of the EPS blocks, σ_{LL}, due to traffic loads on top of a 610-mm (24-in.) portland cement concrete pavement system.

5.5.2.4 Step 4: Estimate Gravity Stresses at the Top of EPS Blocks. Stresses resulting from the gravity load of the pavement system and any road hardware placed on top of the roadway must be added to the traffic stresses obtained in Step 3 to conduct a load-bearing analysis of the EPS. The gravity stress from the weight of the pavement system is as follows:

$$\sigma_{DL} = T_{pavement} * \gamma_{pavement} \tag{36}$$

Where

σ_{DL} = gravity stress due to dead loads,
$T_{pavement}$ = pavement system thickness, and
$\gamma_{pavement}$ = average unit weight of the pavement system.

The various components of the pavement system can be assumed to have an average unit weight of 20 kN/m^3 (130 lbf/ft^3). Because the traffic stresses in Figures 41 through 43 are based on a pavement system with a total thickness of 610 mm (24 in.), a value of $T_{pavement}$ equal to 610 mm (24 in.) should be used to estimate σ_{DL} to ensure consistency.

5.5.2.5 Step 5: Calculate Total Stresses at the Top of EPS Blocks. The total vertical stress at the top of EPS blocks underlying the pavement system from traffic and gravity loads, σ_{total}, is as follows:

$$\sigma_{total} = \sigma_{LL} + \sigma_{DL} \tag{37}$$

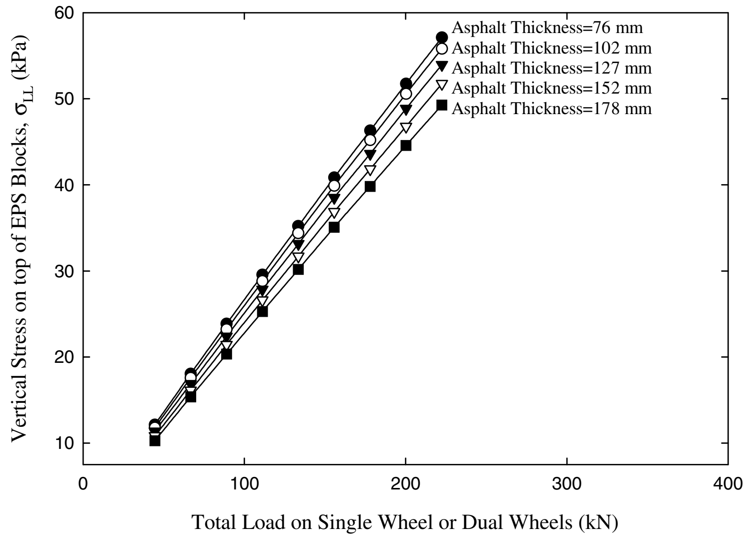


Figure 43. Vertical stress on top of the EPS blocks, σ_{LL}, due to traffic loads on top of a 610-mm (24-in.) asphalt concrete pavement system with a 102-mm (4-in.) concrete separation layer between the pavement system and EPS blocks.

Where

σ_{LL} = vertical stress applied to the top of EPS-block geofoam due to traffic loads under a particular pavement system.

5.5.2.6 Step 6: Determine the Minimum Required Elastic Limit Stress for EPS Under the Pavement System. The minimum required elastic limit stress of the EPS block under the pavement system can be calculated by multiplying the total vertical stress from Step 5 by a factor of safety, as shown in Equation 38:

$$\sigma_e \geq \sigma_{total} * FS \tag{38}$$

Where

σ_e = minimum elastic limit stress of EPS and
FS = factor of safety = 1.2.

The main component of σ_{total} is the traffic stress and not the gravity stress from the pavement. Because traffic is a main component of σ_{total} and traffic is a transient load like wind loading, a factor of safety of 1.2 is recommended for the load-bearing analysis. This is the same value of factor of safety recommended for other transient or temporary loadings, such as wind, hydrostatic uplift, sliding, and seismic loading used for external stability analyses.

5.5.2.7 Step 7: Select the Appropriate EPS Block to Satisfy the Required EPS Elastic Limit Stress for Underneath the Pavement System, e.g., EPS50, EPS70, or EPS100. Select an EPS type from Table 8 that exhibits an elastic limit stress greater than or equal to the required σ_e determined in Step 6. The EPS designation system in Table 8 defines the minimum elastic limit stress of the block as a whole in kilopascals. For example, EPS50 will have a minimum elastic limit stress of 50 kPa (7.2 lbs/in^2). The EPS selected will be the EPS block type that will be used directly beneath the pavement system for a minimum depth of 610 mm (24 in.) in the EPS fill. This minimum depth is recommended because it is typically the critical depth assumed in pavement design for selection of an average resilient modulus for design of the pavement system (*14*). Thus, the 610-mm (24-in.) depth is only an analysis depth and is not based on the thickness of the EPS blocks. Of course, if the proposed block thickness is greater than 610 mm (24 in.), the block selected in this step will conservatively extend below the 610-mm (24-in.) zone. The use of EPS40 is not recommended directly beneath paved areas because an elastic limit stress of 40 kPa (5.8 lbs/in^2) has resulted in pavement settlement problems.

5.5.2.8 Step 8: Select the Preliminary Pavement System Type and Determine Whether a Separation Layer Is Required. A cost analysis should be performed in Step 8 to preliminarily select the optimal pavement system that will be used over the type of EPS blocks determined in Step 7. The cost analysis can focus on one or all three of the pavement systems evaluated in Step 3—i.e., asphalt concrete, PCC, and a composite pavement system. The EPS selected for a depth of 610 m (24 in.) below the pavement system is a function of the pavement system selected because the vertical stress induced at the top of the EPS varies with the pavement system, as shown in Figures 41 through 43. Therefore, several cost scenarios can be analyzed—e.g., a PCC versus asphalt concrete pavement system and the accompanying EPS for each pavement system—to determine the optimal combination of pavement system and EPS. The cost analysis will also determine whether a concrete separation layer between the pavement and EPS is cost-effective by performing a cost analysis on the composite system. The resulting pavement system will be used in Steps 9 through 13.

If a concrete separation slab will be used, the thickness of the concrete slab can be estimated by assuming that the slab is a granular material and will dissipate the traffic stresses to a desirable level at 1(horizontal): 1(vertical) stress distribution. Concrete can then be substituted for granular material using the 1 concrete to 3 gravel ratio previously discussed in Step 3 to estimate the required thickness of granular material. For example, a 102-mm (4-in.)-thick concrete separation layer can be used to replace 306 mm (12 in.) of granular material. Therefore, a 927-mm (36.5-in.)-thick asphalt concrete pavement system that consists of 127 mm (5 in.) of asphalt and 800 mm (31.5 in.) of crushed stone base will be 927 mm (36.5 in.) thick less 306 mm (12 in.) of crushed stone base, which is replaced by a 102-mm (4-in.)-thick concrete separation layer. However, it is recommended that a minimum pavement system thickness of 610 mm (24 in.) be used to minimize the potential for differential icing and solar heating.

5.5.2.9 Step 9: Estimate Traffic Stresses at Various Depths Within the EPS Blocks. This step estimates the dissipation of the traffic-induced stresses through the EPS blocks within the embankment. Using the pavement system and separation layer, if included, from Step 8, the vertical stress from the traffic loads at depths greater than 610 mm (24 in.) in the EPS is calculated. The vertical stress is usually calculated at every 1 m (3.3 ft) of depth below a depth of 610 mm (24 in.). Block thickness is typically not used as a reference depth because the block thickness that will be used on a given project will typically not be known during the design stage of the project. The first depth at which the vertical stress will be estimated is 610 mm (24 in.) because in Step 7 the EPS selected to support the pavement system will extend to a depth of 610 mm (24 in.). The traffic vertical stresses should also be determined at any depth within the EPS blocks where the theoretical 1(horizontal): 2(vertical) stress zone overlaps, as will be shown subsequently (see Figure 44). These vertical stress estimates will be used in Step 12 to determine if the EPS type selected in Step 8 for directly beneath the pavement system is adequate for a depth of more than 610 mm (24 in.) into the EPS and to determine if an EPS block with a lower elastic limit stress, i.e., lower density and lower cost, can be used at a greater depth.

Based on an analysis performed during this study and the results of a full-scale model creep test that was performed at the Norwegian Road Research Laboratory (*18, 19*), a 1(horizontal): 2(vertical) distribution of vertical stresses through EPS blocks, as shown in Figure 44, should be used to estimate the applied vertical stress at various depths in the geofoam.

In order to use the 1(horizontal): 2(vertical) stress distribution method to calculate the vertical stresses applied through the depth of the EPS block using handheld calculations, it is easiest to assume a rectangular loaded area at the top of the EPS and to assume that the total applied load at the surface of the EPS is distributed over an area of the same shape as the loaded area on top of the EPS, but with dimensions that increase by an amount equal to 1(horizontal): 2(vertical) (see Figure 44). Therefore, the live load vertical stress, σ_{LL}, obtained from Figures 41 through 43 should be converted from the assumed circular area to a rectangular area. The Portland Cement Association 1984 method, as described by Huang (*14*), can be used to convert the circular loaded area to an equivalent rectangular loaded area, as shown in Figure 45. The rectangular area shown is equivalent to a circular contact area that corresponds to a single axle with a single tire, A_C, or a single axle with dual tires, A_{CD}. The values of A_C and A_{CD} can be obtained from Equations 32 and 34, respectively, using the following procedure:

1. Estimate σ_{LL} from Figure 41, 42, or 43 depending on the pavement system being considered.
2. Use σ_{LL} in Equation 32 or 34 as the contact pressure, q, and the recommended traffic loads from Step 1 to estimate the live load on one tire in Equation 32 or 34 for a single axle with a single tire or a single axle with dual tires, respectively, to calculate A_C or A_{CD}.
3. In Figure 45, use the values of A_C or A_{CD} to calculate the value of L′, the length of a rectangle used to represent a circular contact area for a tire A_C if the axle has one tire and A_{CD} if the axle has two tires. Equate A_C or A_{CD} to $0.5227L'^2$ and solve for L′. After solving for L′, the dimensions of the rectangular loaded area in Figure 45, i.e., 0.8712L′ and 0.6L′, can be calculated.

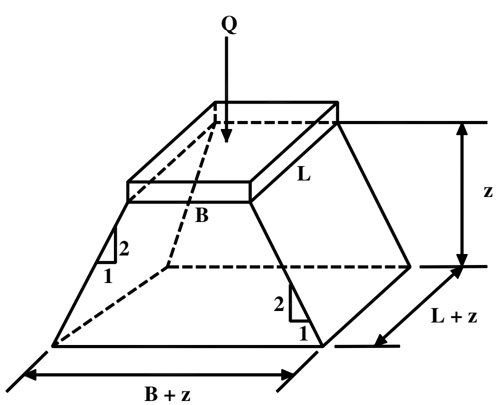


Figure 44. Approximate stress distribution by the 1(horizontal): 2(vertical) method.

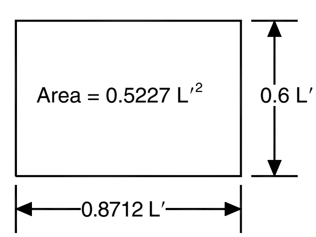


Figure 45. Method for converting a circular contact area into an equivalent rectangular contact area (14).

As shown in Figure 44, at a depth z below the EPS, the total load Q applied at the surface of the EPS is assumed to be uniformly distributed over an area (B + z) by (L + z). The increase in vertical pressure, σ_Z, at depth z due to an applied live load such as traffic is given by Equation 39. Figure 46 demonstrates the use of the 1(horizontal): 2(vertical) method to estimate overlapping stresses from closely spaced loaded areas, such as from adjacent sets of single or dual tires.

$$\sigma_{Z,LL} = \frac{Q}{(B+z)(L+z)} \tag{39}$$

Where

$\sigma_{Z,LL}$ = increase in vertical stress at depth z caused by traffic loading,
Q = applied traffic load,
B = width of the loaded area,
z = depth, and
L = length of the loaded area.

To use the 1(horizontal): 2(vertical) stress distribution method to calculate the vertical stresses through the depth of the EPS block, the assumed circular loaded area below a tire used to determine σ_{LL} in Figures 41, 42, or 43 should be converted to an equivalent rectangular area, as discussed previously. Alternatively, Equation 39 can be modified to determine $\sigma_{Z,LL}$ directly from the σ_{LL}, which is determined from Figures 41, 42, or 43, as shown below:

$$Q = \sigma_{LL} * A_{rect} \tag{40}$$

Where

A_{rect} = area of equivalent rectangle to represent a circular contact area for a tire.

σ_{LL} is obtained from Figures 41, 42, or 43.

From Figure 45,

$$L' = \left(\frac{A_{rect}}{0.5227}\right)^{\frac{1}{2}} \tag{41}$$

$$B = 0.6 * L' \tag{42}$$

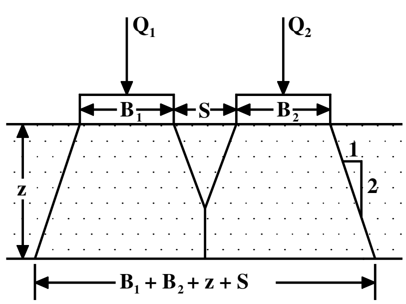


Figure 46. Approximate stress distribution of closely spaced loaded areas by the 1(horizontal): 2(vertical) method.

$$L = 0.8712 * L' \tag{43}$$

Substituting Equations 40 through 43 into 39,

$$\sigma_{Z,LL} = \frac{\sigma_{LL} * A_{rect}}{(0.6L' + z)(0.8712L' + z)} \tag{44}$$

Where A_{rect} is either A_C or A_{CD}, as determined from Equations 32 or 34, respectively.

5.5.2.10 Step 10: Estimate Gravity Stresses at Various Depths Within the EPS Blocks. Stresses resulting from the gravity load of the pavement system, any road hardware placed on top of the roadway, and the EPS blocks must be added to the traffic stresses to evaluate the load-bearing capacity of the EPS within the embankment. Equations 45, 46, and 47 can be used to determine the increase in vertical stress caused by the gravity load of the pavement system:

$$\Delta\sigma_{Z,DL} = \frac{q_t}{\pi}(\alpha + \sin\alpha) \text{ where } \alpha \text{ is in radians} \tag{45}$$

$$\alpha = 2 * \arctan\left(\frac{b}{z}\right) \text{where } \alpha \text{ is calculated in radians} \tag{46}$$

Where

b = one-half the width of the roadway.

$$q_t = q_{pavement} = \gamma_{pavement} * T_{pavement} \tag{47}$$

Where

$q_{pavement}$ = vertical stress applied by the pavement system, kN/m^3;
$\Delta\sigma_{Z,DL}$ = increase in vertical stress at depth z due to pavement system dead load, m;
$\gamma_{pavement}$ = unit weight of the pavement system, kN/m^3; and
$T_{pavement}$ = thickness of the pavement system, m.

Figure 47 shows the geometry and variables for determining the increase in vertical stress with depth.

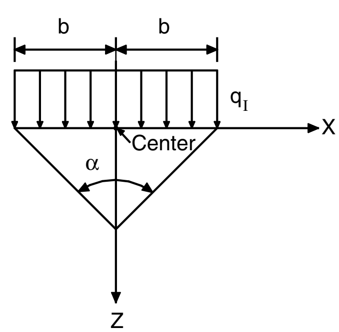


Figure 47. Geometry and variables for determining the increase in vertical stress with depth. (q_I = stress applied to the surface.)

The total gravity stress from the pavement system and the EPS blocks is as follows:

$$\sigma_{Z,DL} = (\Delta\sigma_{Z,DL}) + (z * \gamma_{EPS}) \tag{48}$$

Where

$\sigma_{Z,DL}$ = total vertical stress at depth z due to dead loads, kN/m^2;
z = depth from the top of the EPS, m; and
γ_{EPS} = unit weight of the EPS blocks, kN/m^3.

As discussed in Step 5, the various components of the pavement system can be assumed to have an average unit weight of 20 kN/m^3 (130 lbf/ft^3). Because the traffic stresses in Figures 41 through 43 are based on a pavement system with a total thickness of 610 mm (24 in.), a value of $T_{pavement}$ equal to 610 mm (24 in.) should be used to estimate q_t to ensure consistency. It is recommended that the unit weight of the EPS be assumed to be 1,000 N/m^3 (6.37 lbf/ft^3) to conservatively allow for long-term water absorption in the calculation of $\sigma_{Z,DL}$.

5.5.2.11 Step 11: Calculate Total Stresses at Various Depths Within the EPS Blocks. The total vertical stress induced by traffic and gravity loads at a particular depth within the EPS, σ_{total}, is as follows:

$$\sigma_{total} = \sigma_{Z,LL} + \sigma_{Z,DL} \tag{49}$$

5.5.2.12 Step 12: Determine the Minimum Required Elastic Limit Stress at Various Depths. Determine the minimum required elastic limit stress, σ_e, of the EPS block at each depth that is being considered using the same equation from Step 6:

$$\sigma_e \geq \sigma_{total} * 1.2 \tag{50}$$

5.5.2.13 Step 13: Select the Appropriate EPS Block to Satisfy the Required EPS Elastic Limit Stress at Various Depths in the Embankment. Select an EPS type from Table 8 that exhibits an elastic limit stress greater than or equal to the required elastic limit stress 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.

6 ABUTMENT DESIGN

6.1 Introduction

In applications where the EPS-block geofoam is used as part of a bridge approach, the EPS blocks should be continued up to the drainage layer that is placed along the back of the abutment. A geosynthetic sheet drain, not natural aggregate, should be used for this drainage layer to minimize the vertical and lateral earth pressure on the subgrade and abutment, respectively, as well as to facilitate construction. The design requirements for abutments as well as design examples can be found in *NCHRP Report 343* (*20*). The procedure for designing retaining walls and abutments consists of the following steps (*20*):

1. Select preliminary proportions of the wall.
2. Determine loads and earth pressures.
3. Calculate the magnitude of reaction force on the base.
4. Check stability and safety criteria:
   (a) location of normal component of reactions,
   (b) adequacy of bearing pressure, and
   (c) safety against sliding.
5. Revise proportions of wall, and repeat Steps 2 through 4 until stability criteria are satisfied; then check the following:
   (a) settlement within tolerable limits and
   (b) safety against deep-seated foundation failure.
6. If proportions become unreasonable, consider a foundation supported on driven piles or drilled shafts.
7. Compare economics of completed design with other wall systems.

For a bridge approach consisting of EPS-block geofoam backfill, earth pressures, which are required for Step 2 of the abutment design process, generated by the following two sources should be considered:

- The gravity load of the pavement system, W_P, and of EPS blocks, W_{EPS} (usually small), pressing directly on the back of the abutment (see Figure 48) and

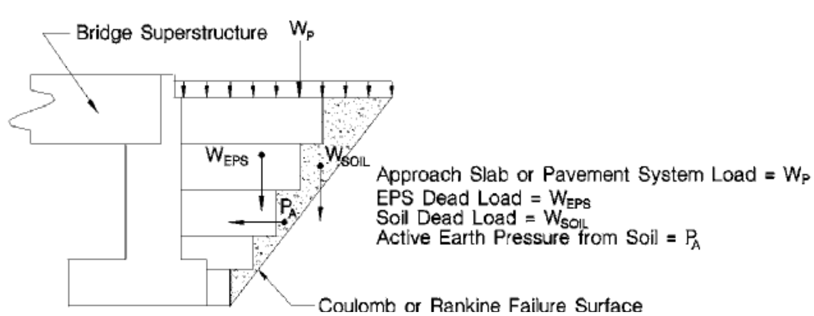


Figure 48. Loads on an EPS-block geofoam bridge approach system.

- The active earth pressure from the soil behind the geofoam fill (see Figure 48) that can be transferred through the geofoam fill to the back of the abutment.

The magnitude of these loads varies depending on whether gravity and/or seismic loading is evaluated. The procedure for estimating the gravity and seismic loads is discussed in the following sections.

6.2 Gravity Loads

The assumed components of the gravity loads acting on a vertical wall or abutment are as follows (see Figure 49):

- The uniform horizontal pressure acting over the entire depth of the geofoam caused by the vertical stress applied by the pavement system to the top of the EPS, which can be estimated from Figures 41 through 43;
- The horizontal pressure generated by the vertical stress imposed by the pavement system, which can be assumed to be equal to $\frac{1}{10}$ times the vertical stress; and
- The lateral earth pressure, P_A, generated by the soil behind the EPS/soil interface, which is conservatively assumed to be transmitted without dissipation through the geofoam to the back of the abutment. The active earth pressure acting along this interface is calculated using a coefficient of active earth pressure, K_A, because it is assumed that enough lateral deformation will occur to mobilize an active earth pressure condition in the soil behind the geofoam. The active earth pressure coefficient can be determined from Equation 51, which is based on Coulomb's classical earth-pressure theory (*21*).

The horizontal stress from the EPS blocks is neglected because it is negligible.

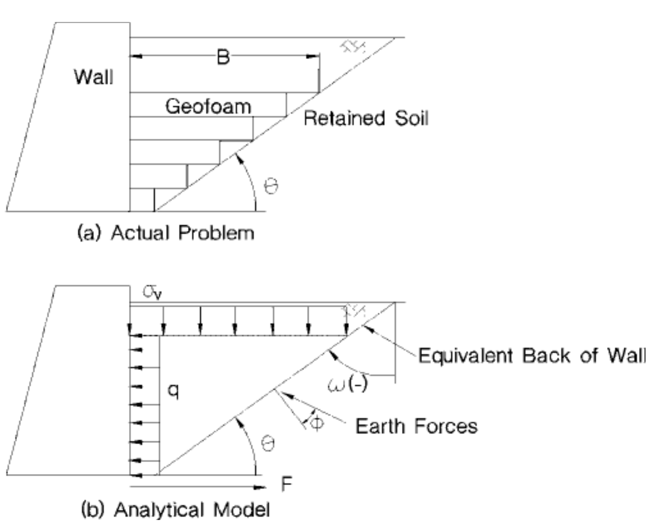


Figure 49. Gravity load components on a vertical wall. (F = resisting force against sliding.)

$$K_A = \left[\frac{\sin(\theta - \varphi)\left(\dfrac{1}{\sin\theta}\right)}{\sqrt{\sin(\theta + \delta)} + \sqrt{\dfrac{\sin(\varphi + \delta)\sin\varphi}{\sin\theta}}} \right]^2 \tag{51}$$

Where δ is the friction angle of the EPS/soil interface, which is analogous to the soil/wall interface in typical retaining wall design. The value of δ can be assumed to be equal to the friction angle of the soil, ϕ. Equation 51 is applicable only to horizontally level backfills. The active earth pressure force, P_A, is expressed in Equation 52:

$$P_A = \frac{1}{2}\gamma_{soil} H^2 K_A \tag{52}$$

Where

γ_{soil} = unit weight of the soil backfill and
H = height of the soil backfill behind the vertical wall.

6.3 Seismic Loads

The following seismic loads acting on the back of an abutment must be added to the gravity loads in Figure 50 to safely design a bridge abutment in a seismic area:

- Inertia forces from seismic excitation of the pavement system and the EPS blocks (usually negligible). These inertial forces should be reduced by the horizontal sliding resistance, $\tan\phi$, developed along the pavement system/EPS interface.
- The seismic component of the active earth pressure generated by the soil behind the EPS/soil interface, which can be calculated using the solution presented by Mononobe-Okabe (*13*).

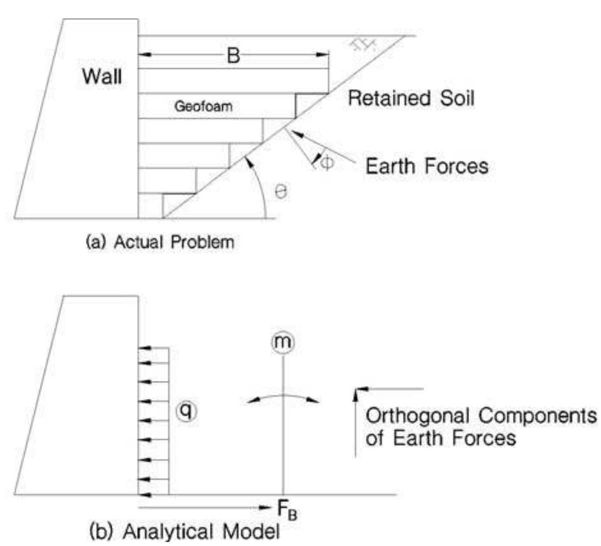


Figure 50. Seismic load components on a wall. (F_B = resisting force against sliding and m = seismic moment.)

7 CONVERSION FACTORS

7.1 Introduction

Both the Système International d'Unités (SI) and inch-pound (I-P) units have been used in this report. SI units are shown first, and I-P units are shown in parentheses within text. Numerous figures are included for use in design. Therefore, only SI units are provided in some of the figures to avoid duplication of figures. Additionally, in some cases figures have been reproduced that use either all SI or all I-P units. These figures have not been revised to show both sets of units. The one exception to the dual SI and I-P unit usage involves the quantities of density and unit weight. Density is the mass per unit volume and has units of kg/m^3 (slugs/ft^3), and unit weight is the weight per unit volume and has units of kN/m^3 (lbf/ft^3). Although density is the preferred quantity in SI, unit weight is still the common quantity in geotechnical engineering practice. Therefore, the quantity of unit weight will be used herein except when referring to EPS-block geofoam. The geofoam manufacturing industry typically uses the quantity of density with the SI units of kg/m^3, but the quantity of unit weight with the I-P units of lbf/ft^3. Therefore, the same dual-unit system of density in SI units and unit weight in I-P units will be used when referring to EPS-block geofoam.

7.2 Conversion Factors from Inch-Pound Units (I-P Units) to the Système International d'Unités (SI Units)

Length:	1 ft	=	0.3048 m
	1 ft	=	30.48 cm
	1 ft	=	304.8 mm
	1 in.	=	0.0254 m
	1 in.	=	2.54 cm
	1 in.	=	25.4 mm
	1 yd	=	0.9144 m
	1 yd	=	91.44 cm
	1 yd	=	914.4 mm
	1 mi	=	1.61 km
Area:	1 ft^2	=	929.03×10^{-4} m^2
	1 ft^2	=	929.03 cm^2
	1 ft^2	=	929.03×10^{2} mm^2
	1 in^2	=	6.452×10^{-4} m^2
	1 in^2	=	6.452 cm^2
	1 in^2	=	645.16 mm^2
	1 yd^2	=	836.1×10^{-3} m^2
	1 yd^2	=	8361 cm^2
	1 yd^2	=	8.361×10^{5} mm^2
Volume:	1 ft^3	=	28.317×10^{-3} m^3
	1 ft^3	=	28.317 cm^3
	1 in^3	=	16.387×10^{-6} m^3
	1 in^3	=	16.387 cm^3
	1 yd^3	=	0.7646 m^3
	1 yd^3	=	7.646×10^{5} cm^3
Force:	1 lb	=	4.448 N
	1 lb	=	4.448×10^{-3} kN
	1 lb	=	0.4536 kgf
	1 kip	=	4.448 kN
	1 U.S. ton	=	8.896 kN
	1 lb	=	0.4536×10^{-3} metric ton
	1 lb/ft	=	14.593 N/m
Stress, Pressure, Modulus of Elasticity:	1 lb/ft^2	=	47.88 Pa
	1 lb/ft^2	=	0.04788 kPa
	1 U.S. ton/ft^2	=	95.76 kPa
	1 kip/ft^2	=	47.88 kPa
	1 lb/in^2	=	6.895 kPa
Density:	1 slug/ft^3	=	16.018 kg/m^3
Unit Weight:	1 lbf/ft^3	=	0.1572 kN/m^3
	1 lbf/in^3	=	271.43 kN/m^3
Moment:	1 lb-ft	=	1.3558 N · m
	1 lb-in.	=	0.11298 N · m
Temperature:	1°F	=	use 5/9 (°F – 32) to obtain °C
General Note:	1 mil	=	10^{-3} in.

7.3 Conversion Factors from the Système International d'Unités (SI Units) to Inch-Pound Units (I-P Units)

Length:	1 m	=	3.281 ft
	1 cm	=	3.281×10^{-2} ft
	1 mm	=	3.281×10^{-3} ft
	1 m	=	39.37 in.
	1 cm	=	0.3937 in.
	1 mm	=	0.03937 in.
	1 m	=	1.094 yd
	1 cm	=	0.01094 yd
	1 mm	=	1.094×10^{-3} yd
	1 km	=	0.621 mi
Area:	1 m^2	=	10.764 ft^2
	1 cm^2	=	10.764×10^{-4} ft^2
	1 mm^2	=	10.764×10^{-6} ft^2
	1 m^2	=	1550 in^2
	1 cm^2	=	0.155 in^2
	1 mm^2	=	0.155×10^{-2} in^2
	1 m^2	=	1.196 yd^2
	1 cm^2	=	1.196×10^{-4} yd^2
	1 mm^2	=	1.196×10^{-6} yd^2
Volume:	1 m^3	=	35.32 ft^3
	1 cm^3	=	35.32×10^{-4} ft^3
	1 m^3	=	61,023.4 in^3
	1 cm^3	=	0.061023 in^3
	1 m^3	=	1.308 yd^3
	1 cm^3	=	1.308×10^{-6} yd^3
Force:	1 N	=	0.2248 lb
	1 kN	=	224.8 lb
	1 kgf	=	2.2046 lb
	1 kN	=	0.2248 kip
	1 kN	=	0.1124 U.S. ton
	1 metric ton	=	2204.6 lb
	1 N/m	=	0.0685 lb/ft
Stress, Pressure, Modulus of Elasticity:			
	1 Pa	=	20.885×10^{-3} lb/ft^2
	1 kPa	=	20.885 lb/ft^2
	1 kPa	=	0.01044 U.S. ton/ft^2
	1 kPa	=	20.885×10^{-3} kip/ft^2
	1 kPa	=	0.145 lb/in^2
Density:	1 kg/m^3	=	0.0624 slugs/ft^3
Unit Weight:	1 kN/m^3	=	6.361 lbf/ft^3
	1 kN/m^3	=	0.003682 lbf/in^3