NORWEGIAN UNIVERSITY OF
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Design of EPS lightweight Fill material in road construction

[A literature review]

By:- **Tebarek Ahmed Awol**

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Abstract:

The main target of this project is to make a literature review on design guidelines/experience of EPS fill light - weight embankment. Design experience from four countries: USA, Norway, France and Greece are presented.

Even if there are only quite a few documentation of failure in EPS fill embankments, having a common geotechnical consensus on parameter selection and design is necessary. Even if researches are carried out, there is still a tradition of using different EPS engineering material parameters from place to place. It has been a focus point for many researchers to document design procedures for EPS design and constructions for different uses.

USA has many years of experience in using EPS as an alternative light weight material. Because of the increase in uses of EPS in road construction, transportation research board has produced a preliminary design manual for design of such fills. Along with other case studies, their experience in the design is included as one chapter. Design charts for seismic analysis and material parameters of EPS along with controlling measures are documented.

Having a long experience and being the first user of EPS, Norway has an accomplished experience in using the material. Special construction procedures from Håndbook274, Håndbook016 and Håndbook 018 with information from case studies has been included.

From experiences gained through more than 200 projects in EPS, France has produced a document on how to utilize EPS. Design procedure mentioned in the guide produced by the French committee for road engineering, CFTR, is mentioned. Greece'sdesign experience gained from utilization of EPS on one of their major highway has been included.

Summarised EPS material parameters, performance parameters and European standard on EPS are presented in the last chapter.

NORWEGIAN UNIVERSITY OF SCIENCE AND TECHNOLOGY
DEPARTMENT OF CIVIL AND TRANSPORT ENGINEERING

TBA4510 Geotechnical Engineering Specialization Project, fall 2011

Project task description for
Tebarek Ahmed Awol

Project title:

Design of EPS lightweight fill material in road construction

Background
Expanded Polystyrene, EPS, is commenly used for insulation and frost protection in the construction sector. EPS is also increasingly used in in road engineering as a light weight fill material. In road construction EPS may be used as an alternative to heavy fills or embankments on soft ground to minimize settlements, EPS may be used to provide an arching effect over buried pipes or culverts, reduce earth pressure or prevent propagation of ground vibrations. The usage is encouraged by the fact that EPS is easy to handle and construction is rapid. The increase in use has put focus on design rules and design parameters. Norwegian and international experience, design procedures and design rules are the subject of this student project.

Task description
The task is to study existing guidelines for design of EPS fills and embankments from different countries with exerience in this field and some selected case studies. The literature review should address design principles in the calculation of settlement or deformation of the fill and stability of the embankment. Factor of safety used for design, EPS material parameters and design charts used, if any, should be documented. Based on the information gained from this literature review, further studies on numerical modelling of EPS fill design can be suggested.

In more detail the work will/may be focusing on:

- A study of existing design guidelines for settlements of EPS fill embankments
- Considerations regarding the stability of the light weight fill embankment
- Experience from case records including considerations on internal stability of the fill
- Recommendation for seismic considerations and seismic analysis
- Summary of EPS material parameters

This project work is proposed by the National Public Road Administration in Norway which will provide background material and advice through Dr. Jan Vaslestad, MSc. Murad Sani and MSc. Geir Refsdal.

Trondheim, December 2011
Supervisor at NTNU

Professor Steinar Nordal

Semester Project - Dec.2011 *Design guidelines for light – weight fill EPS embankments*

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Tebarek Ahmed Awol

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Chapter 1. Introduction

1. Background

1.1 History of EPS

As the necessities for expansion of infrastructures needed, searching for alternative materials for construction is natural. The necessity of constructing roads and railway lines are increasing from time to time. The geotechnical challenges in design and construction of such infrastructures in some cases are quite demanding because of the increase in design regulations, increase in comfort level of society and construction in heavily populated city areas. This increases the affinity for searching a new methodology and new materials for better performance.

Large settlements associated with large fills and weak subgrade soil, heavy loads on buried structures, large horizontal loads on approach embankments and other geotechnical problems have been issues. In early 1970’s, EPS (expanded polystyrene) was used as an alternative material to solve a problem. EPS has a unique quality in which its low density and light weight makes it suitable to use it as alternative material.

The first use of EPS in road is accomplished in 1972 in Norway near Oslo. The construction of the embankment was not the result of a planned search for an alternative light weight material for road embankments on soft ground, it was the offspring of a large research project with a totally different aim - how to frost protect roads and engineering structures (ref. 1).Since then, the use of EPS increases for different number of practices and number of countries benefiting from this is increasing from time to time.

1.2 Objective of this project

The main aim of this project is to get literature review by studying design guidelines of EPS fill embankments from USA, Norway, France and Greece. Respective countries design manual, different case studies and papers published on the topic are the main inputs.

This literature review will help for further studying of the topic and better understanding of engineering parameters of the EPS and the fill. Experience gained from this will help for possible further master thesis study on numerical modeling of EPS embankments.

2. Production process and material property of EPS

2.1 Production process

There are many kinds of plastics in the world, any plastics when react with the Blowing Agent will become "Foam" which generally called "Foam Plastics". We will only mention about foam that produced from Polystyrene / PS ($C8H8$) plastic so-called "Expanded Polystyrene". Expandable Polystyrene / EPS is a foam that use Pentane gas ($C5H12$) as the blowing agent.During the material production process called "Polymerization" the polystyrene resin granules impregnated with the blowing agent. (From Apiwat Hiranpradit, Asian manufacturers of EPS)

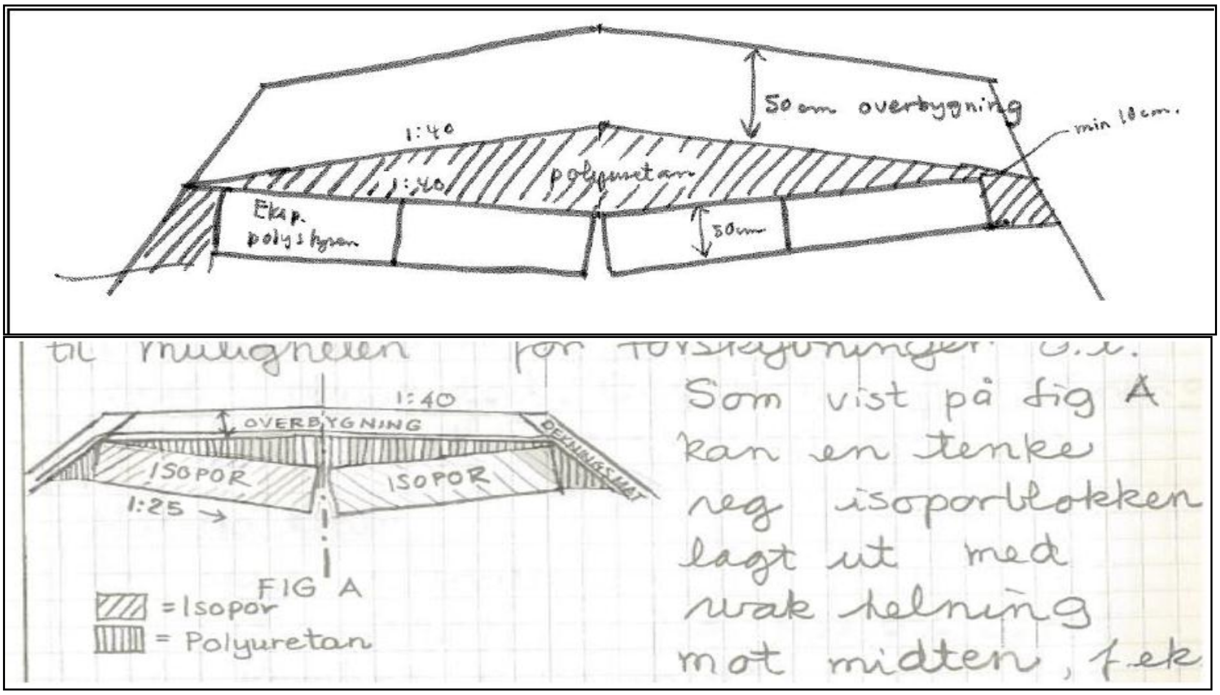


Figure 1: The first and second sketch for the EPS geoblock project at Flom Bridge (Ref 1)

1st stage - Pre-expansion

The raw material is heated in special machines called pre-expanders with steam at temperatures of between 80-100°C. The density of the material falls from some 630kg/m3 to values of between 10 and 35kg/m3.During this process of pre-expansion the raw materials compact beads turn into cellular plastic beads with small closed cells that hold air in their interior.

2nd stage - Intermediate Maturing and Stabilisation

On cooling, the recently expanded particles from a vacuum in their interior and this must be compensated for by air diffusion. This process is carried out during the material's intermediate maturing in aerated silos. The beads are dried at the same time. This is how the beads achieve greater mechanical elasticity and improve expansion capacity — very important in the following transformation stage.

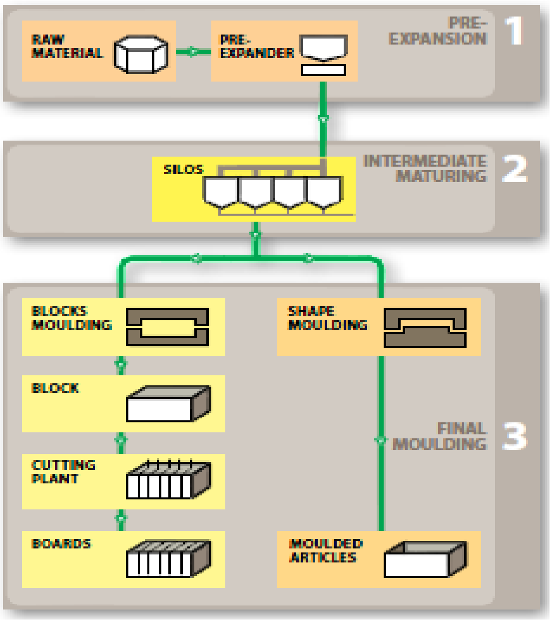


Figure 2: EPS production process (ref. 2)

3rd stage - Expansion and Final Moulding

During this stage, the stabilized pre-expanded beads are transported to moulds where they are again subjected to steam so that the beads bind together. In this way moulded shapes or large blocks are obtained (that are later sectioned to the required shape like boards, panels, cylinders etc).

2.2 Material parameters

There are different kinds of EPS based on their compressive strength at 10% deformation. Different countries have performed monitoring programme for the performance of EPS in different applications. Based on that, some empirical formulas and design limits have been proposed. Main engineering parameters of EPS are stated below but detail performance data are presented in chapter 6. The table provided below shows the summary of EPS parameter collected from literature from different lab and field tests.

Table 1: Summary parameter of EPS

Type of EPS	Compressive stress at 10 %	Unit weight(KN/m3)		Density (Kg/m3)		Water content (%)	Creep (as % of fill)	Deformation (as % of fill)
		$\gamma_{drained}$	γ_{subm}	$\rho_{drained}$	ρ_{subm}			
EPS60	60 kPa	0.5	1	50	100	5 - 10	6-7	1-3
EPS100	100 kPa							
EPS150	150 kPa							
EPS250	250 kPa							

3. Uses of EPS

Apart from the uses of EPS in food box, ice box, packaging for television, fish box, floating Krathong foam etc, it has been added as an alternative material in different aspects of construction industries. In this topic we will focus the use of EPS in road construction briefly.

3.1 As a lightweight fill in the embankment

Due to its super light weight behavior, EPS is a good candidate in large fills where the subgrade soil is believed to be very weak and sensible. For such kinds of practices we can achieve a faster construction time, less maintenance rate and better quality. The design experience of such kinds of EPS embankments construction from four different countries are presented in successive chapters.

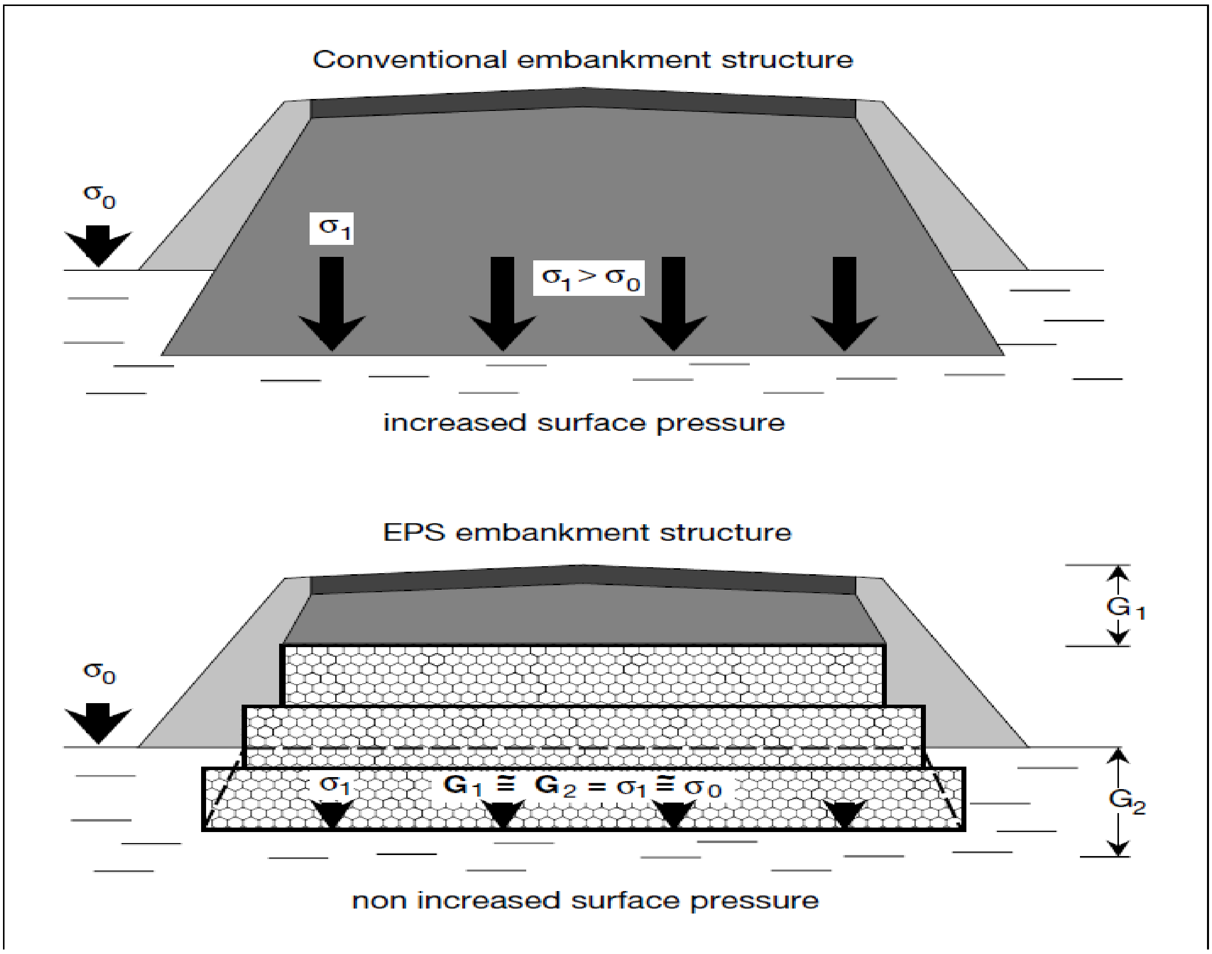


Figure 3: Comparison of conventional and EPS embankment structures (ref. 4)

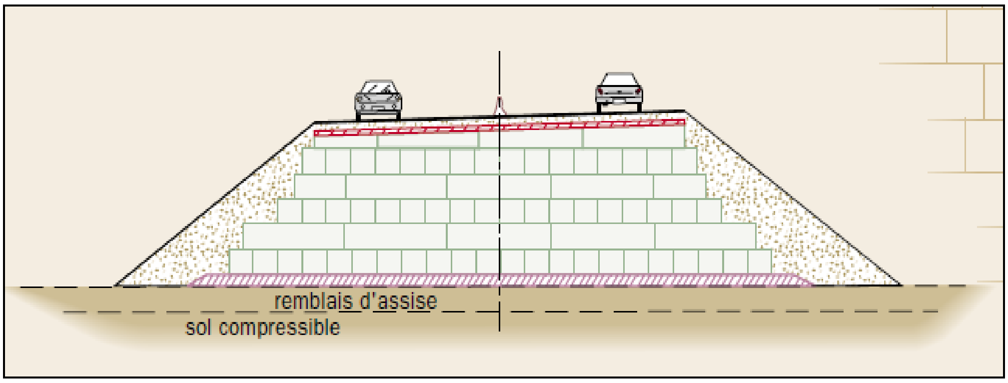


Figure 4: Fill of light weight EPS on sensible subgrade (ref. 3)

3.2 As for insulation and pavement frost damage mitigation

Experience with EPS foam boards used as frost protection for roads and railways formed the basis for the development of this construction technique. This method of construction has been applied since the middle of the 1960s, mainly in countries with severe winters (eg, alpine regions, Canada and the Scandinavian countries) where the deeply penetrating ground frosts make it necessary to provide costly frost-proof subbases for roads and railways. One can say "anti-frost construction methods" can now be ranked alongside other conventional construction techniques (ref. 4).

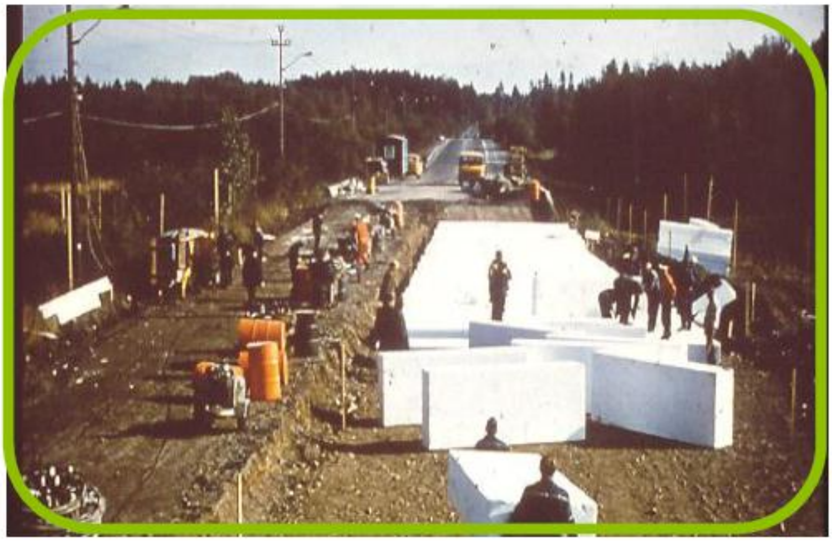


Figure 5: The first layer of EPS geofoam blocks placement for frost protection in Norway, 1972 (ref. 1)

3.3 As horizontal force reduction in approach embankments

Based on the light weightiness of the material, practical experience and the load distribution capacity of EPS, major horizontal force reduction has been found when load is applied. Such properties of the block make it a suitable construction material as a fill in approach embankments of a fill and as well as pile. But, care should be taken not too overload the EPS because o creep reasons. See figure shown below for schematic presentation of how EPS blocks can be used as a fill material in approach embankments.

3.4 To bring arching effect in deeply buried structures

The earth pressure on deeply buried culverts is significantly affected by arching. Both the magnitude and distribution of earth pressure on buried culverts are known to depend on the relative stiffness of the culvert and the soil. The so-called induced trench method (also called imperfect ditch) involves installing a compressible layer above the rigid culvert. As the embankment is constructed, the soft zone compresses more than the surrounding fill, and thus induces positive arching above the culvert (ref. 5). The problem of earth pressure on buried structures has a great practical importance in constructing embankments over pipes and culverts. It is very important to reduce the load on the structures and to bring this effect, EPS has been a successful candidate and good performance has been recorded.

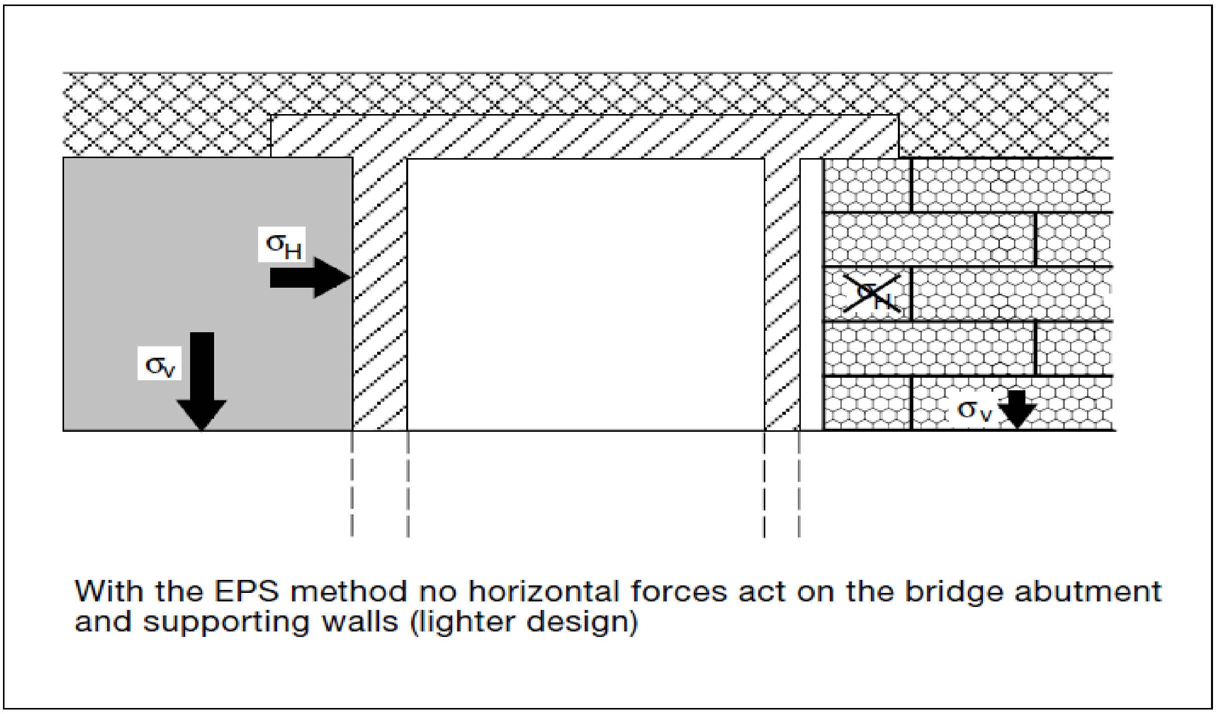


Figure 6: Schematic presentation of EPS fill in approach embankment (ref.4)

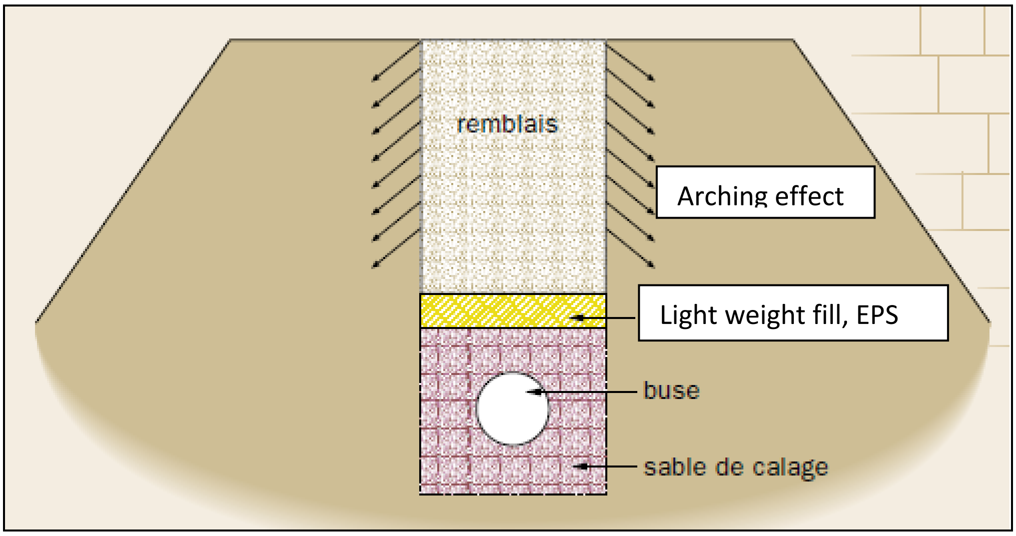


Figure 7: Arching effect in buried structures (ref. 1)

Chapter 2. USA

1. Introduction

Even though EPS geofoam was used in the United States much earlier than in most countries, subsequent progress was slow. Recently, EPS geofoam is used in a growing trend in a number of applications in the States. The largest volume of EPS geofoam in one project is about 100,000 cubic meters in Salt Lake City in the reconstruction of interstate I-15(ref.8).

Most of this report under this topic was a 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) (ref.14).

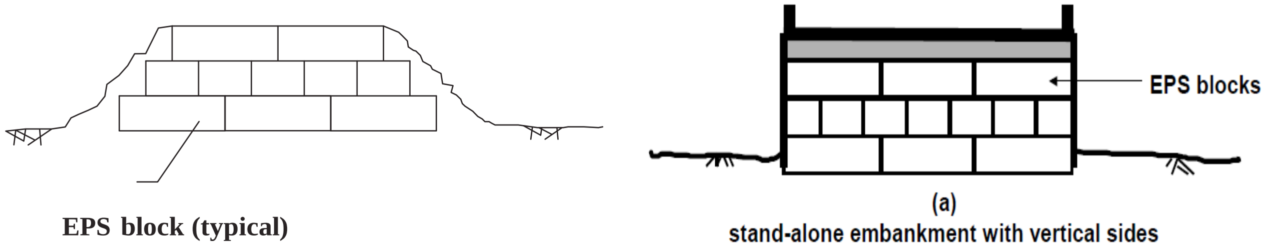
The proposed design guideline is limited to embankments that have a transverse (cross-sectional) geometry such that the two sides are more or less of equal height (*figure: 8*).

The design charts developed as part of this research and included herein as an appendices are based on embankment models with the geometric and material parameters described in this specific topic. However, most design charts are based on embankment sides lopes of 0 (horizontal, H):1 (vertical, V), 2H:1V, 3H:1V, and 4H:1V. Widths at the top of the embankment of 11 m , 23 m , and 34 m were evaluated, based on a two-lane roadway with 1.8-m shoulders, four- lane roadway with two 3-m exterior shoulders and two 1.2-m interior shoulders, and a six-lane roadway with four 3-m shoulders. Each lane was assumed to be 3.66 m wide. Embankment heights ranging between 1.5 m and 16 m were evaluated. For simplicity, the fill mass was assumed to consist entirely of EPS blocks.

Examples of critical and noncritical design conditions are provided in table 2. 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 non- critical conditions.

Table 2: Examples of critical and noncritical embankment design and construction conditions

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



(a) Sloped-side fill. (b) Vertical-face fill.

Figure 8: Typical EPS-block geofoam applications involving embankments (Ref.15).

Even if the block layout traditionally was done by the design engineer for the project since the designer knows the exact block dimensions beforehand but 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.

2. Design Guideline

2.1 Major Components of an EPS-Block Geofoam Embankment

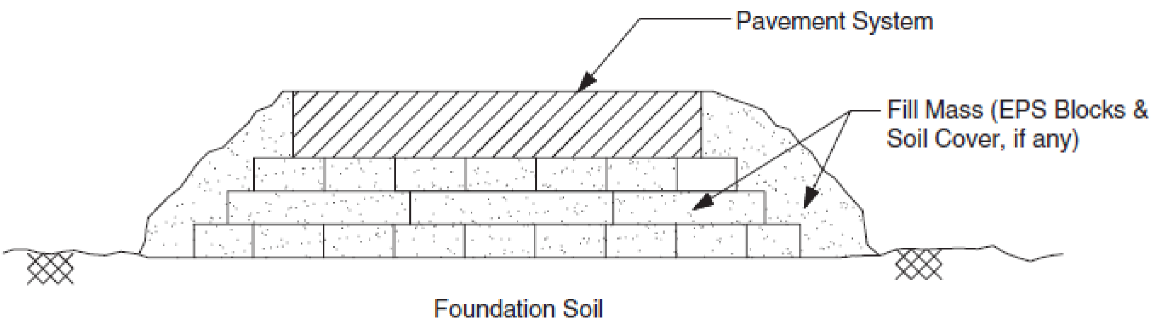


Figure 9: Major components of an EPS-block geofoam embankment.

As indicated in Figure 3 above, 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.

2.2 Design Phases

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).

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 as well as its components individually must be designed to prevent failure.

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

1. Design for external (global) stability of the overall embankment
2. Design for internal stability within the embankment mass
3. Design of an appropriate pavement system for the subgrade provided by the underlying EPS blocks.

2.3 Design Procedure

The design procedure for an EPS – block geofoam roadway embankment over soft soil considers the interaction between the three phases. The stability of the fill affects the stability of the pavement and on the other way around, the load coming from the pavement affects the stability of the fill. Therefore, in order to come up with a best solution iterative analysis is used here as a design to reach to a cost effective and stable design.

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.

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.

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 in thickness to minimize the effects of differential icing and solar heating to satisfy internal and external stability requirements. Therefore, it is recommended that the preliminary pavement system be assumed to be 610 mm thick and that the various component layers of the pavement system be assumed to have a total (moist) unit weight of 20 kN/m3. Selection of a preliminary pavement system is also provided.

2.3.3 Step 3—Select Preliminary Embankment Arrangement

To achieve the most cost effective design, use the minimum number of EPS blocks necessary to meet the external and internal stability requirements. But, we should make sure that the load transferred to the subsoil fulfills the design requirements including 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 Appendix 1.

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 Appendix 1 with the accompanying tolerable criteria.

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 Appendix 1. 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 Appendix 1. 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). This is accomplished by making the EPS fill to an equivalent soil by giving it resilient modulus or equivalent California Bearing Ratio (CBR). A summary of these design parameters is provided in Table 3.

In the USA guideline, pavement design catalogs were developed to facilitate pavement system design based on the American Association of State Highway and Transportation Officials (AASHTO) 1993 design procedure 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 4, is based on the following assumptions:

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.

2. The range of traffic levels for the performance period is limited to between 50,000 and 1 million 80-kN equivalent single-axle load (ESAL) applications.
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 3 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.

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

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

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 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. 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 a layer coefficient of 0.5 is given for the slab.

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 5 for asphalt concrete and aggregate base to overcome placement impracticalities, ensure adequate performance, and lower costs.

Table 3: 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 4: 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.

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 and they can be used by design engineer 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 ESALs. An ESAL is the summation of equivalent 80-kN single-axle loads used to convert mixed traffic to design traffic for the performance period .

- 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 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 = 4.1)
- The designs are based on a mean PCC elastic modulus (E_c) of 34.5 GPa.
- Drainage (moisture) conditions (C_d) are fair (C_d = 1.0).
- The 80-kN 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 5: Minimum practical thicknesses for asphalt concrete and aggregate base (4)

Traffic, ESALs	Minimum Thickness , mm (in.)	
	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)

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 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/m3 for initial design purposes.

4. External (global) stability analysis

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

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 they 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. 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, Stotal, consists of five components, as shown by Equation 2:

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

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 Scf 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 .

4.2.1 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:

$$S_p = \frac{C_r}{1+e_o} L_o \log \frac{\sigma'_p}{\sigma'_{vo}} + \frac{C_c}{1+e_o} L_o \log \frac{\sigma'_{vf}}{\sigma'_p} \quad \text{...(3)}$$

Where

Sp= Settlemnt 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.

For normally *consolidated foundation soil*, Equation 3 can be simplified as follows:

$$S_p = \frac{C_c}{1+e_o} L_o \log \frac{\sigma'_{vf}}{\sigma'_p} \quad \text{...(4)}$$

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 over excavate the existing soft foundation soil and to place EPS blocks in the over excavation. 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, Lo will be the thickness of the soft foundation soil prior to the over excavation procedure. If the foundation soil is over consolidated (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., $\sigma'_{vf} \leq \sigma'_p$), Equation 3 can be simplified as follows:

$$S_p = \frac{C_r}{1+e_o} L_o \log \frac{\sigma'_{vf}}{\sigma'_{vo}} \qquad \text{..(5)}$$

4.2.2 Settlement Due to Secondary Consolidation

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 .

$$S_s = \frac{[C_\alpha / C_c] \times C_c}{1+e_o} L_o \log \frac{t}{t_p} \qquad \text{.....................................(6)}$$

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 6, can be used to estimate C_α.

Table 6: Values of C□ /Cc for soils

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 tp 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 /tp rarely exceeds 100 and is often less than 10.

4.2.3 Allowable Settlement

Post construction settlements of 0.3 to 0.6 m 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 . If post construction settlement occurs over a long period of time, any pavement distress caused by settlement can be repaired when the pavement is resurfaced. Although rigid pavements have performed well after 0.3 to 0.6 m of uniform settlement, flexible pavements are usually selected where doubt exists about the uniformity of post construction settlements, and some states utilize a flexible pavement when predicted settlements exceed 150 mm .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, qult, is defined by Prandt as follows:

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

Where

c = Mohr-Coulomb shear strength parameter (i.e., cohesion), kPa;
Nc, Nγ, Nq = Terzaghi shearing resistance bearing capacity factors;
γ = unit weight of soil, kN/m3;
B_W = bottom width of embankment, m; and
Df = 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, su, of the foundation soil, as well as assuming the embankment is placed on the ground surface, simplifies Equation 7 to the following:

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

Where

D_W = depth from ground surface to the water table,
L = length of the embankment, and
Df = 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 BW/L in Equation 8 approaches zero. Upon including the BW/L simplification in Equation 8, Nc 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:

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

Where

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

= $\sigma_{n,pavement@0m} + \sigma_{n,traffic@0m} + \sigma_{n,EPS@0m}$;

$\sigma_{n,pavement@0m}$ = 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 EPSblock 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:

$$s_u = \frac{3}{5} * \left\{\left[\frac{\left(\sigma_{n,pavement} + \sigma_{n,traffic}\right) * T_w}{(T_W + T_{EPS})}\right] + \frac{(\gamma_{EPS} * T_{EPS})}{2}\right\}$$ ....(10)

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 σn,pavement = 21.5 kPa and σn,traffic = 11.5 kPa and γEPS = 1 kN/m3 into Equation 10 yields the following expression for the undrained shear stress required to satisfy a factor of safety of 3 for a particular embankment height:

$$s_u = \frac{3}{5} * \left\{\left[\frac{(21.5\ \text{kPa} + 11.5\ \text{kPa}) * T_w}{T_W + T_{EPS}}\right] + \frac{(1\ \text{kN/m}^3)T_{EPS}}{2}\right\} = \frac{99\ T_W}{5(T_W + T_{EPS})} + 0.3\ T_{EPS}$$ ....(11)

Based on Equation 11 and various values of TEPS, Figure 10 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 su greater than or equal to 19.9 kPa , external bearing capacity will not control the external stability of the EPS embankment. However, if the value of su is less than 19.9 kPa, the allowable thickness or height of the EPS-block geofoam embankment can be estimated for a particular road width from Figure 10 to prevent bearing capacity failure.

For example, the lowest value of su that can accommodate a six-lane embankment (road width of 34 m is approximately 18.3 kPa for a minimum height of EPS block equal to 12.2 m . This means that for a six-lane embankment and an su value of 18.3 kPa ,the required T_{EPS} will be 12.2 m . Conversely, if the height of the EPS embankment desired is 4.6 m an su of 18.9 kPa 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. A typical cross section through a trapezoidal EPS embankment with side slopes of 2H:1V is shown in Figure11 and was used to develop the external slope stability design charts for trapezoidal embankments. The soil cover is 0.46 m thick, which is typical for the side slopes, and is assigned a moist unit weight of 18.9 kN/m3.

The pavement system is modeled using a surcharge of 21.5 kPa .The traffic surcharge is 11.5 kPa based on the AASHTO recommendation of using 0.67m of an18.9-kN/m3 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 plus 11.5 kPa or 33.0 kPa.

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 Appendix 2 for a two-lane (road width of 11 m), four-lane (road width of 23 m), and six-lane (road width of 34 m) roadway embankment, respectively. The first graph 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 su 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 TEPS 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 su value for the foundation soil. The value of su at which each relationship terminates signifies the transition from external slope stability being critical to internal stability being critical.

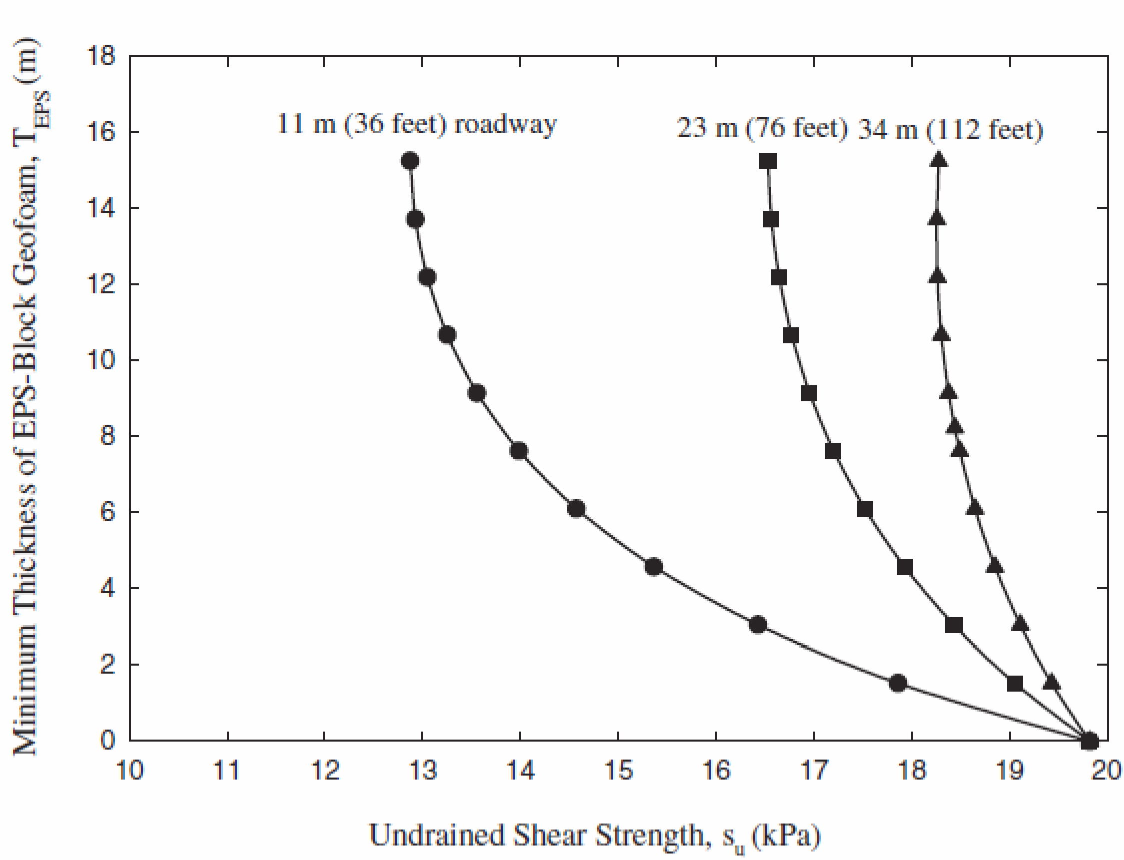


Figure 10: Design chart for obtaining the minimum thickness or height of geofoam, TEPS, for a factor of safety of 3 against external bearing capacity failure of a geofoam embankment.

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 vertical embankment used in the external static stability analyses is shown in Figure 12.

This cross section differs from the cross section used for the static analyses of trapezoidal embankments in Figure 11 because the surcharge used to represent the pavement and traffic surcharges is replaced by placing a 0.61-m soil layer on top of the embankment with a unit weight of 54.1kN/m3. The soil layer is 0.61 m thick to represent the minimum recommended pavement section thickness. Therefore, the vertical stress applied by this soil layer equals 0.61 m times the increased unit weight of 54.1 kN/m3, or 33.0 kPa. A vertical stress of 33.0 kPa corresponds to the sum of the design values of pavement surcharge (21.5 kPa) and traffic surcharge (11.5 kPa) used previously for external bearing capacity and slope stability of trapezoidal embankments.

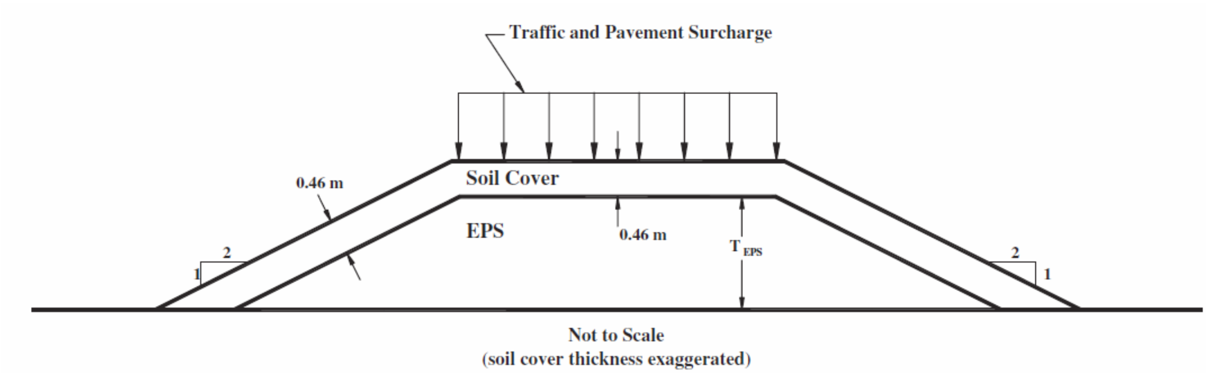


Figure 11: Typical cross section used in static external slope stability analyses of trapezoidal embankments.

The pavement and traffic surcharge in Figure 11 was replaced by an equivalent soil layer because a seismic slope stability analysis can only be performed with material layers and not surcharge loads.

4.4.2.2 Design Charts.

The results of the stability analyses were used to develop the static external slope stability design chart in Appendix 3. Appendix 3 presents the results for a two-lane (road width of 11 m), four-lane (road width of 23 m), and six-lane (road width of 34 m) roadway embankment, respectively, and the three graphs correspond to the three embankment heights considered—i.e., 3.1 m, 6.1 m, and 12.2 m—for various values of foundation soil su. As shown in Appendix 3, as the foundation su increases, the overall embankment slope stability factor of safety increases. 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.

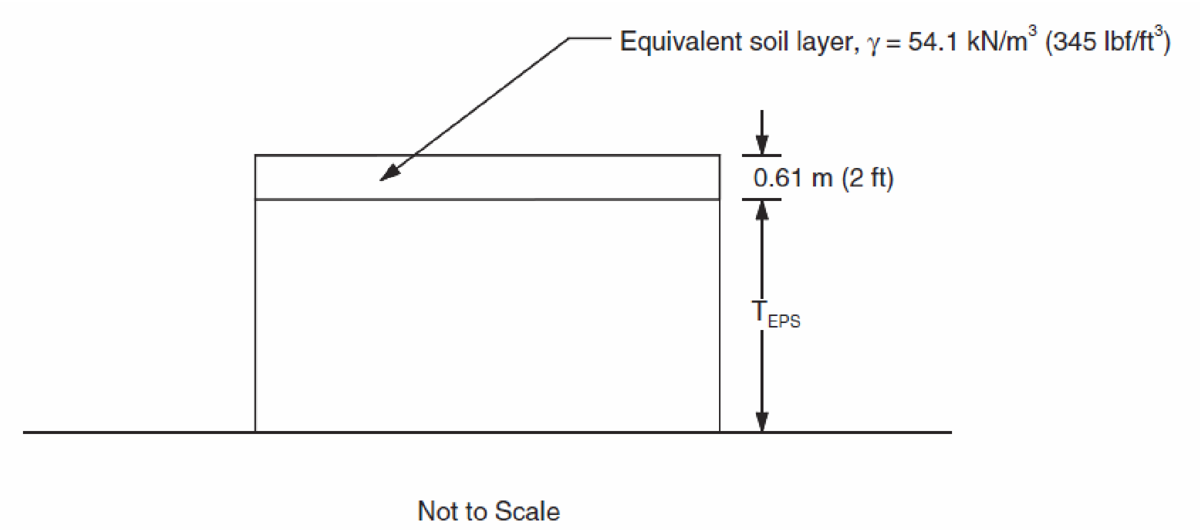


Figure 12: Typical cross section used in static and seismic external slope stability analyses of vertical embankments.

4.5 Hydrostatic Uplift (Flotation)

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

$$FS = \frac{W_{EPS} + W_W + W'_W + O_{REQ}}{\gamma_W * (h + S_{total}) * B_W} \quad \text{...(12)}$$

Equation 12 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, series of calculation to determine the value of O_{REQ} performed for various cross sections.

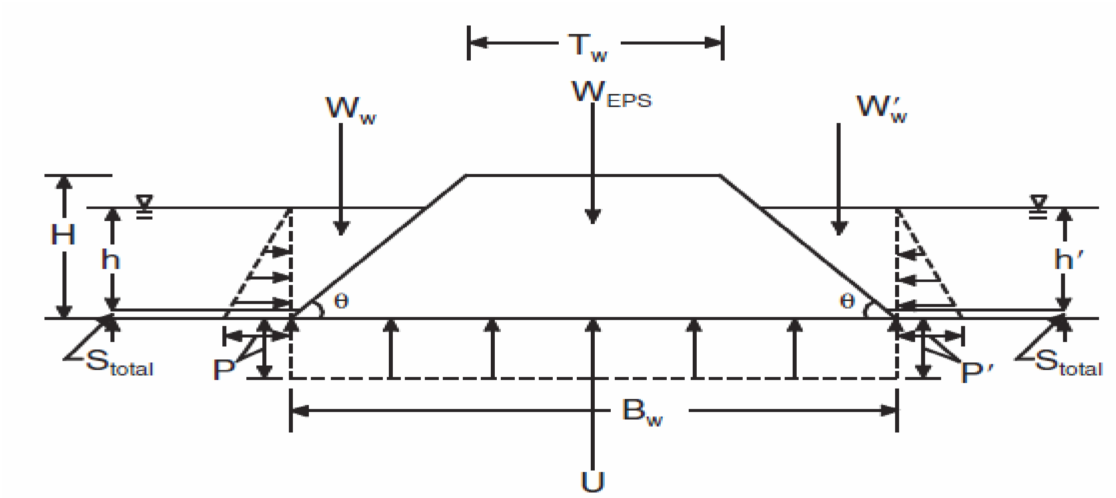


Figure 13: Variables for determining hydrostatic uplift for the case of water equal on both sides of the embankment. (P = pressure exerted on the side of the embankment and U = uplift pressure acting on the base of the embankment.)

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.

The components usually contributing to O_{REQ} are the weight of the pavement system and the cover soil on the embankment side slopes.

$$O_{REQ} < (\gamma_{pavement} * T_{pavement} * T_W) - (\gamma_{EPS} * T_{pavement} * T_W) + W_{cover} + W_{other} \quad \text{..........................(14)}$$

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 + Stotal. The design engineer then compares this value of O_{REQ} with the weight of the pavement system and cover soil.

$$W_{cover} = 2 * \left(\gamma_{cover} * \frac{T_{EPS}}{\sin\theta} * \frac{T_{cover}}{\cos\theta} \right) \qquad \text{.......(15)}$$

Where

$\gamma cover$ = 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.

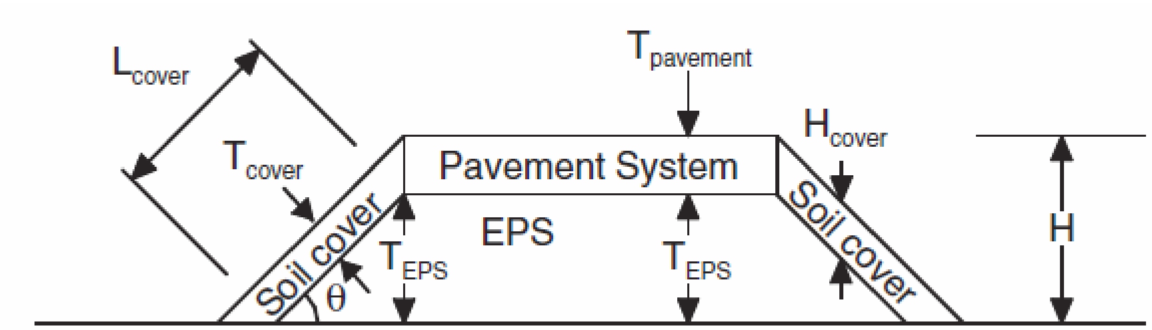


Figure 14: Variables for the weight induced by the soil cover. (Lcover =length of soil cover on the side of the embankment, Tcover = perpendicular thickness of the soil cover, and Hcover = vertical thickness of the soil cover.)

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.

Appendix 4 present the design charts for all of the embankment geometries mentioned earlier for equal upstream and tailwater levels and uplift at the EPS block/foundation soil interface. The values of OREQ shown in Appendix 4 are the required weight of material over the EPS blocks in kilo Newtons per linear meter of embankment length.

Figure15 shows the variable for determining hydrostatic uplift analysis for the case of water on one side of the embankment only. Equation 16 can be used to obtain the factor of safety against hydrostatic uplift.

$$FS = \frac{W_{EPS} + W_W + O_{REQ}}{\frac{1}{2} * \gamma_W * (h + S_{total}) * B_W} \qquad \text{.......(16)}$$

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 16 can be rearranged and used to obtain the value of OREQ required to obtain the desired factor of safety of 1.2 against hydrostatic uplift.

$$O_{REQ} = \left[1.2 * \left(\frac{1}{2} * \gamma_W * (h + S_{total}) * B_W\right)\right] - [(W_{EPS} + W_W)] \quad \text{...(17)}$$

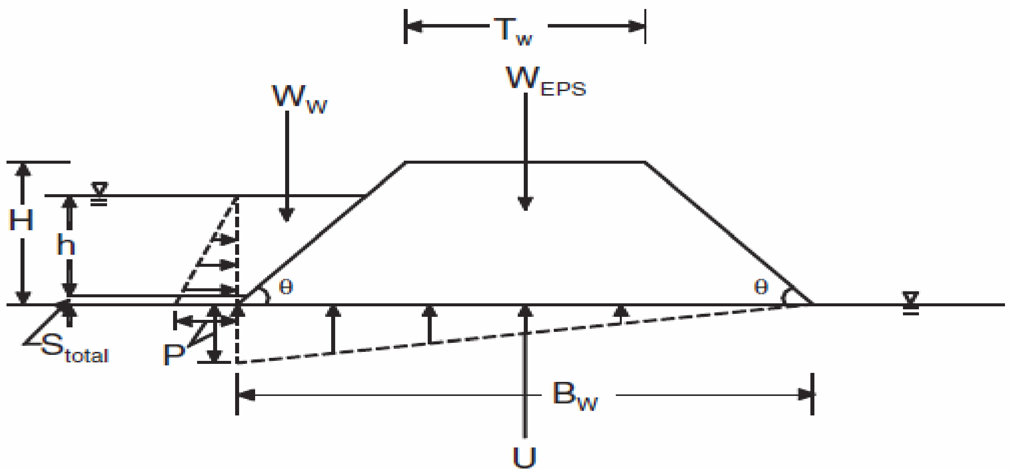


Figure 15: Variable for determining hydrostatic uplift analysis for the case of water on one side of the embankment only. (P=pressure exerted on the side of the embankment and U =uplift pressure acting on the base of the embankment.)

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

4.6.1 Introduction

Because of the extraordinarily low density of EPS-block geofoam, the potential for translation for entire embankment is 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 corner

4.6.2 Translation

The tendency of the entire embankment to slide under an unbalanced water pressure is resisted primarily by EPS/foundation soil interface friction. Evenif, friction angle,δ, for this interface is relatively high, the resisting force will be small because the dead weight of the overall embankment is small. The factor of safety for translation (horizontal sliding) of the entire embankment in a direction perpendicular to the proposed road alignment should be :

$$FS = \frac{\left[(W_{EPS} + W_W + O_{REQ}) - \left(\frac{1}{2}(h + S_{total}) * \gamma_W * B_W\right)\right] * \tan\delta}{\frac{1}{2}\left(\gamma_W * (h + S_{total})^2\right)} \quad \text{..................(18)}$$

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,

Stotal = total settlement as defined by Equation 2, and

BW = bottom of embankment width.

And for O_{REQ}, Equation 18 becomes:

$$O_{REQ} = \frac{1.2\left(\frac{1}{2}\right)\left(\gamma_W * (h + S_{total})^2\right)}{\tan\delta} + \left(\frac{1}{2}((h + S_{total}) * \gamma_W) * B_W\right) - W_{EPS} - W_W \quad \text{...(19)}$$

Equation 19 can be used to obtain the required value of O_{REQ} for a factor of safety of 1.2 against hydrostatic sliding.

4.6.3 Overturning

Overturning may be critical for tall and narrow vertical embankments. These horizontal forces create an overturning moment about the toe at point O. The worst-case scenario is water accumulating on only one side of the embankment, as shown in Figure 15.

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

$$FS = \frac{\sum \text{stabilizing moments}}{\sum \text{overturning moments}} = \frac{\left(\frac{1}{2} * T_W\right) * (W_{EPS} + O_{REQ})}{\frac{1}{3}(h + S_{total}) * R_p} \quad \text{…………………...………………………(20)}$$

Where:

Rp = 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 and Equation 20 becomes:

$$O_{REQ} = \frac{1.2 * \left(\frac{1}{3}\right) * (h + S_{total}) * (R_p)}{\left(\frac{1}{2} * T_W\right)} - W_{EPS} \quad \text{...(21)}$$

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 $\leq$(Bw/6), to minimize the potential for the wall to overturn. Additionally, the maximum and minimum soil pressures under the embankment should not exceed the allowable soil pressure, qa.

4.7 Translation and Overturning Due to Wind

Too conservative result from present wind analysis and lack of documented failure, analysis for wind is not included in the US design guidelines.

5. Internal stability evaluation

5.1 Introduction

Design for internal stability of an EPS-block geofoam embankment includes consideration of SLS issues and ULS issues.

5.2 Translation Due to Water (Hydrostatic Sliding)

The main task here is that to verify whether enough friction exists between blocks. Equation 19 can be used to determine the required overburden force, O_{REQ}, to achieve a factor of safety of 1.2 against horizontal sliding.

$$O_{REQ} < \left(\gamma_{pavement} * T_{pavement} * T_W\right) + W_{cover} + W_{other} \quad \text{..(22)}$$

Appendix 5 can be used to determine the required overburden force, OREQ, to achieve a factor of safety of 1.2 against horizontal sliding.

5.3 Translation Due to Wind

Too conservative result from present wind analysis and lack of documented failure, analysis for wind is not included in the US design guidelines.

5.4 Load Bearing

5.4.1 Introduction

The primary internal stability issue for EPS-block geofoam embankments is the load bearing of the EPS-geofoam mass. It is important in the selection of EPS type in each layer based on the stress coming from the pavement. In US the design approach used for load bearing is based on deformation-based design methodology. It is based on the elastic limit stress, σe, to evaluate the load bearing of EPS. This method might seem conservative since the elastic limit stress of the whole block might be greater than the minimums.

Table 7: Minimum allowable values of elastic limit stress and initial tangent Young's modulus for the proposed AASHTO EPS material designation

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)

5.4.2 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.

6. Seismic Analysis

6.1 External Seismic Stability of Embankment

6.1.1 Trapezoidal Embankments

6.1.1.1 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. External seismic stability is evaluated using a pseudo-static slope stability analysis 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 to 12.2 m), slope inclinations (2H:1V, 3H:1V, and 4H:1V), and road widths (11 m , 23 m, and 34 m) 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.

A typical cross section through an EPS embankment with side slopes of 2H:1V used in the pseudo-static stability analyses is shown in Figure 16.

4.5.1.2 Design Charts.

Refer to appendix 6 for design charts of trapezoidal embankments.

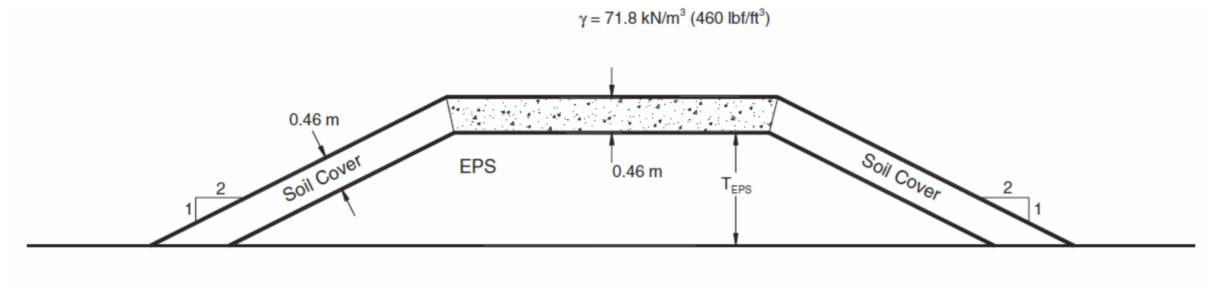


Figure 16: Typical cross section used in seismic external slope stability analyses of trapezoidal embankments.

6.1.2 Vertical Embankments

6.1.2.1 Introduction and Typical Cross Section.

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 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 to 12.2 m) and road widths of 11, 23, and 34 m (36, 76, and 112 ft) on external seismic slope stability. The same typical cross section as Figure 12 is used

6.1.2.2 Design Charts.

Refer to appendix 6 for design charts of vertical embankments.

6.1.2.3 Overturning.

For tall and narrow vertical embankments, the overturning of the entire embankment at Point O, as shown in Figure 17 is needed. A factor of safety against overturning of 1.2 is recommended for design purposes because overturning due to earthquake loading is a temporary loading condition.

For this cases the eccentricity of the loads should be considered since the play role in determining the direction of the soil pressure. Therefore, as e increases, the potential for overturning of the embankment increases. 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.

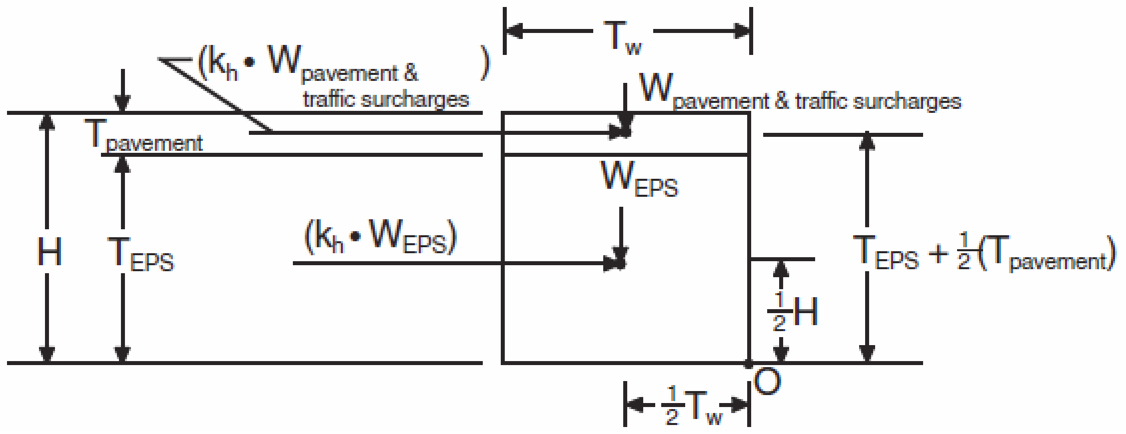


Figure 17: Variables for determining the factor of safety against overturning of a vertical embankment due to pseudo-static horizontal forces used to represent an earthquake loading.

$$FS = \frac{\sum \text{stabilizing moments}}{\sum \text{overturning moments}} = \frac{\left(\frac{1}{2} * T_w\right) * \left(W_{EPS} + W_{pavement\ \&\ traffic\ surcharges}\right)}{\left[\left(\frac{1}{2} * H\right) * \left(k_h * W_{EPS}\right)\right] + \left[\left(T_{EPS} + \left(\frac{1}{2} * T_{pavement}\right)\right) * \left(k_h * W_{pavement\ \&\ traffic\ surcharges}\right)\right]} \quad (23)$$

Where

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

$$X = \frac{\sum \text{stabilizing moments} - \sum \text{Overturning Moments}}{\sum N} \quad \text{...(24)}$$

Where

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

e = TW/2 - x ……………………………………………………………………….(25)

Where

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

TW = top width of the embankment.

$$q = \frac{\sum N}{T_w}\left(1 \pm \frac{6e}{T_w}\right) \leq q_a \qquad \text{.....(26)}$$

Where

q = soil pressure under the embankment and

qa = allowable soil pressure.

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

6.2 Internal Seismic Stability

6.2.1 Trapezoidal Embankments

6.2.1.1 Introduction and Typical Cross Section.

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 used in USA guideline is presented 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 pseudostatic 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 high EPS trapezoidal embankment with side slopes of 2H:1V that was used in the pseudo-static internal stability analyses is shown in Figure 18.

Weight on top of embankment = 71.8 kN/m3
Soil cover = 0.46 m (0.46*71.8 kN/m3= 33.0 kPa).
Pavement and traffic load = 33.0

Figure 19 also presents the three failure surfaces or modes considered in the internal seismic stability analyses.

Mode I = translational sliding at the pavement system/EPS interface at the top of the EPS blocks.

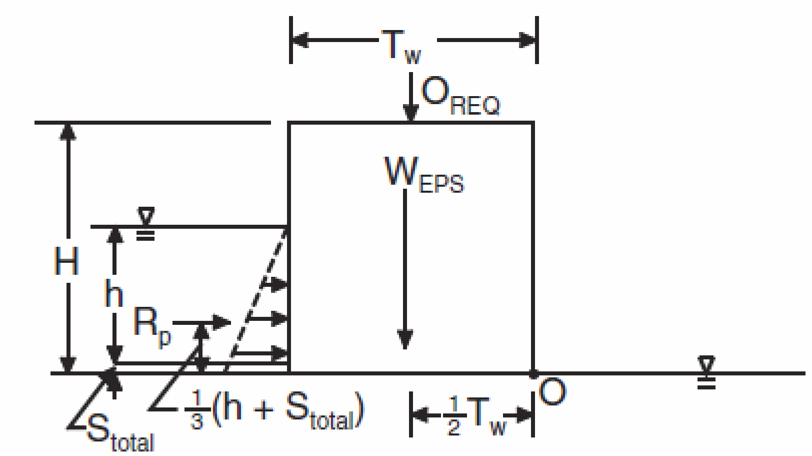


Figure 18: Variables for determining the factor of safety against overturning due to hydrostatic horizontal forces for the case of water on one side of the embankment.

Mode II = 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.

Mode III = translational sliding at the EPS/ foundation soil interface at the base of the EPS blocks.

6.2.1.2 Design Chart.

Geometry used:- embankment heights of 3.1 m to 12.2 m ; slope inclinations of 2H:1V, 3H:1V, and 4H:1V; and roadway widths of 11 m, 23 m, and 34 m .

6.2.2 Vertical Embankments

6.2.2.1 Introduction and Typical Cross Section.

The main thing here is that sliding is assumed to occur only within the geofoam embankment or along an EPS interface.

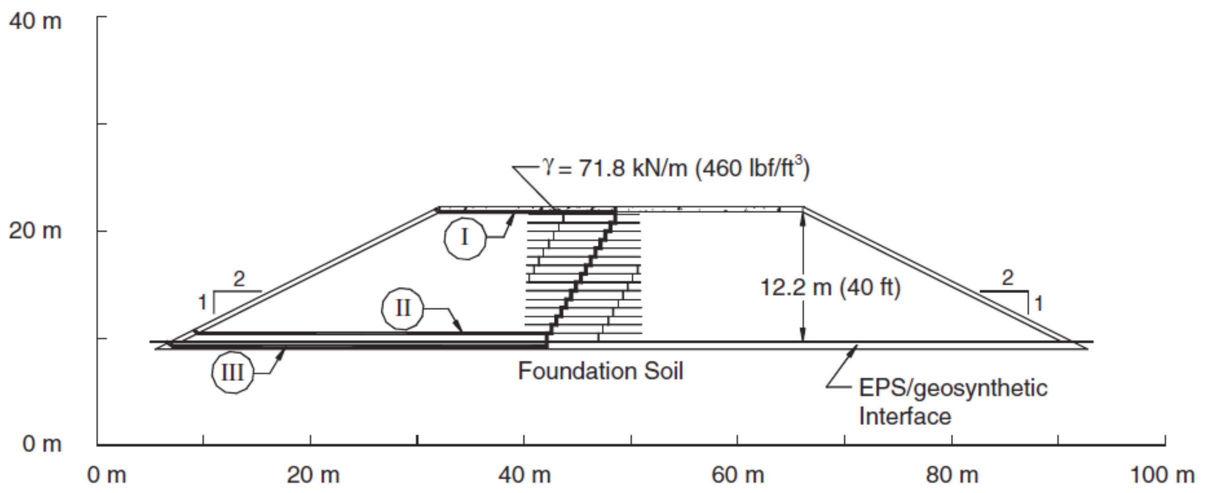


Figure 19: Typical trapezoidal cross section used in seismic internal slope stability analyses with the three applicable failure modes.

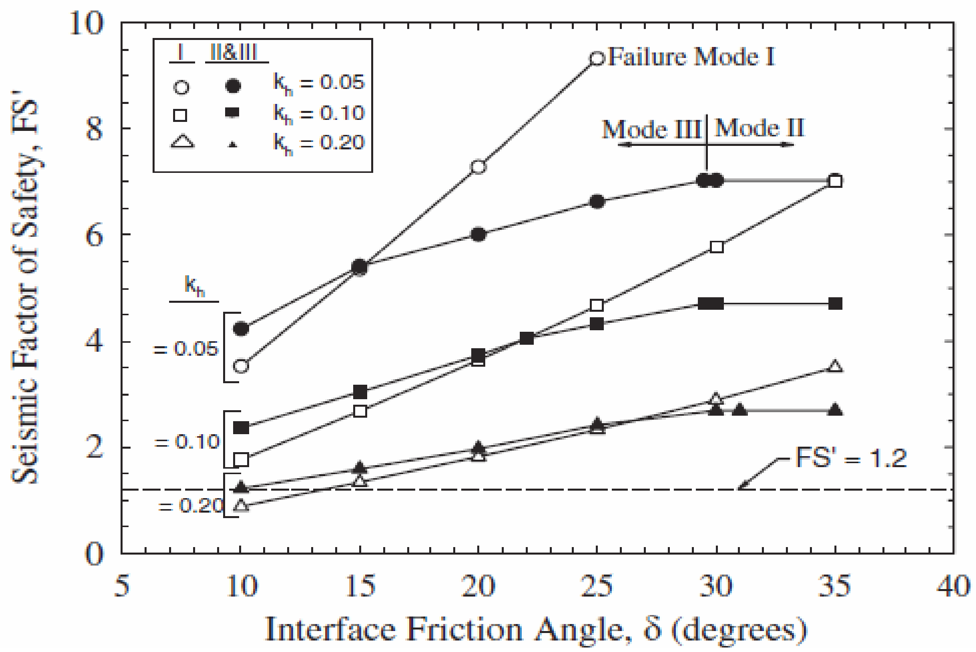


Figure 20: Design chart for internal seismic stability of EPS trapezoidal embankments.

This analysis uses the same pseudo-static slope stability analysis used for internal seismic stability of trapezoidal embankments in and uses the same cross section similar to the cross section used for static analyses of vertical embankments.

Weight on top of embankment = 71.8 kN/m3

Soil cover = 0.61m including the equivalent traffic load

6.2.2.2 Design Chart.

See figure 21 and 22.

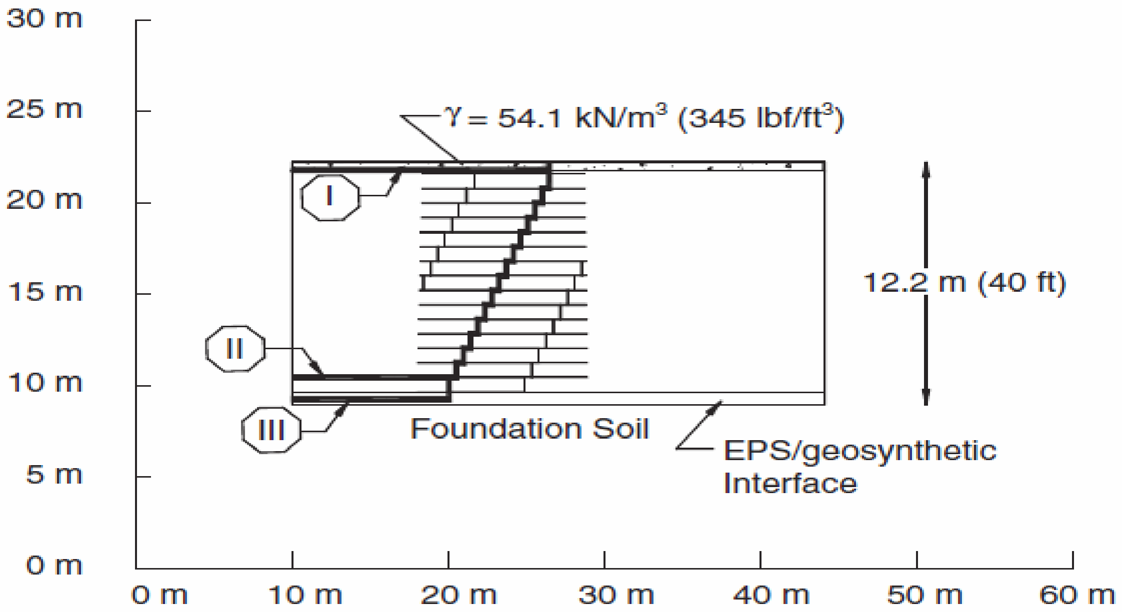


Figure 21: Typical cross section used in seismic internal slope stability analyses for vertical embankments with the three applicable failure modes.

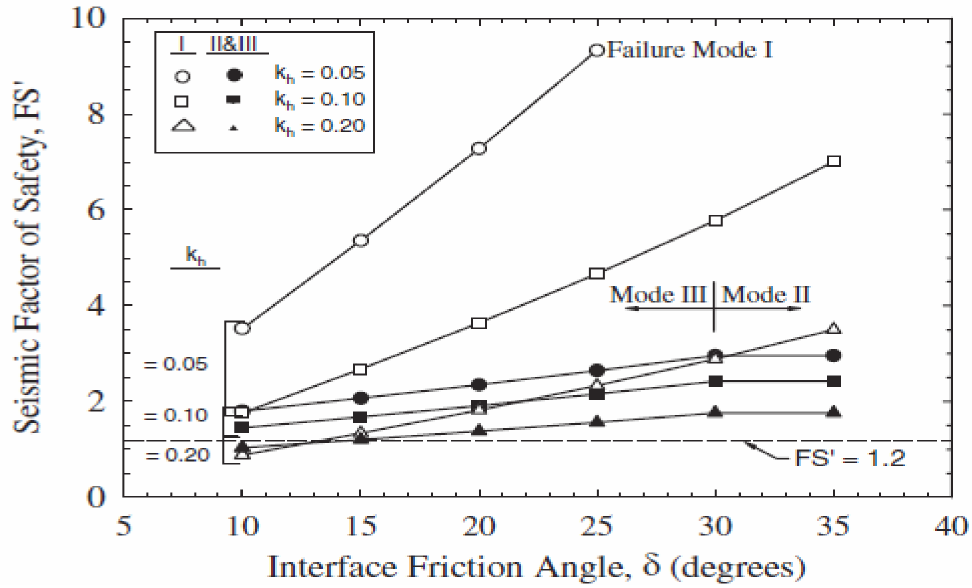


Figure 22: Design chart for internal seismic stability of EPS vertical embankments.

Chapter 3. NORWAY

1. Introduction

In Norway, there is an experience of using light weight materials in road construction. The Norwegian public road authorities applied sawdust, lightweight aggregate and cellular concrete blocks as an alternative light weight materials. The usage of EPS as light weight fill material started here in Norway when it was used for roads as frost protecting layer in 1972. Since then it has been used in more than 500 projects in Norway alone in different areas for different purposes.

EPS light weight embankments are proven to be a good solution to large fill areas in soft grounds. The ultimate benefit of EPS lies in the fact that it brings good strength and stability by imposing light weight to the soft subgrade underneath it. In the next topics, we will refer to some design criteria's stated in Hándbook and other literatures for how to design an EPS fill embankment in Norway.

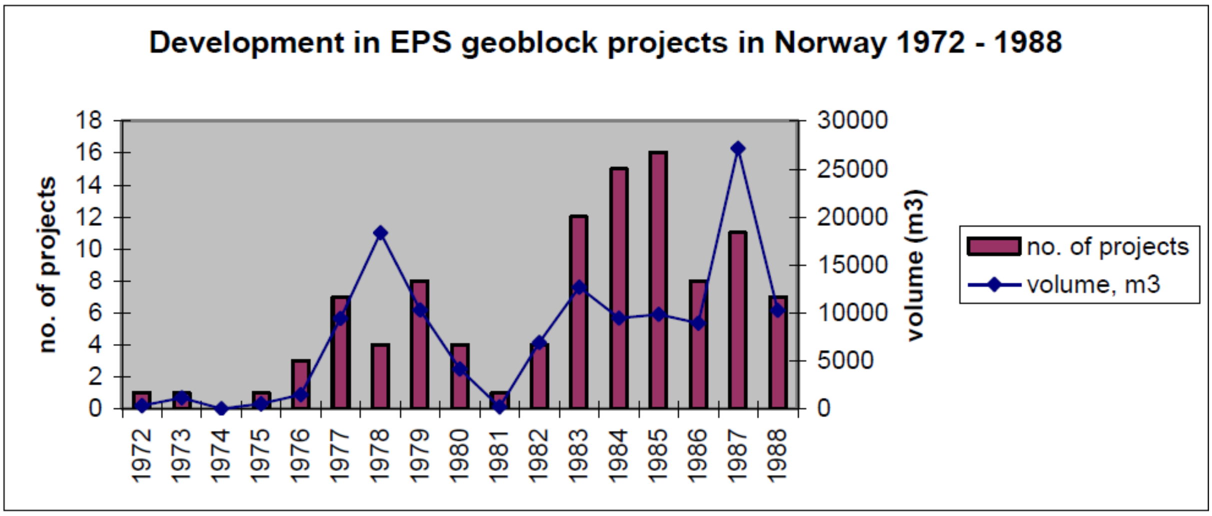


Figure 23: Development of EPS usage in Norway(ref.1)

2. Design guidelines

There is no as such a document providing procedures for how to design EPS fill embankments. But there are provisions stated in NPRA handbook for construction details as well as general regulations for EPS constructions to be safe. Lab tests and field tests on EPS fills has been carried out by the Norwegian road research laboratory including monitoring activities. Those reports will be used as a reference for this literature reviews.

Road embankments in general have three major components: the pavement structure, the fill (EPS) and the subgrade. For stability of such a fill, the stability of each component should be acquired. As adesign guideline, general outline of Norwegian pavement design, geotechnical stability requirements of the subgrade and as well as the EPS is included.

3. Pavement Design

3.1 Introduction

The purpose of pavement design is to find a structure that could withstand the climatic and traffic loads in a technical and economical satisfactory way in a reasonable lifetime. A long historical development in both road building and pavement design could be followed. Practical judgment has always played an important role. Trial and error have led to new knowledge and understanding. In recent years material testing have proven a useful tool for better understanding of the structures, and theoretical analyses of stress/strains and detoriation have become useful (ref.16)

In the Norwegian design system, the main emphasis has been on the bearing capacity. The main aim is that the structure should spread the load from the traffic to protect the ground from excessive stress and avoid damages from deformations. To fulfill this requirement, it is necessary that the materials used are stable, not moisture susceptible and with satisfactory bearing capacity.

The Norwegian design system could be used at three different levels:

- Level 1 (empirical design)
- Level 2 (semi-empirical system)
- Level 3 (theoretical system)

This sub topic discusses design according to level 1, also called design from tables or design with fixed load distributing coefficients. It is mainly based on chapter 51 in the Norwegian Public Roads Administration PDG (Handbook 018).

Design level 1 is a typical empirical design system, meaning that the method is based on generalized experience and analytical results. The subgrade materials are classified in different classes according to grain size, grading, frost susceptibility, drainage conditions, etc. The thicknesses of the different layers are found in tables dependent on subgrade class and traffic loading. The tables have been specified based on index values and fixed load distributing coefficients. Input values for use of the tables are:

1. Ground condition (Subgrade class)
2. Road construction materials (type and quality)
3. Traffic load
4. Standardized road structures
5. Climatic conditions (insulation against frost heave on frost susceptible soil)

By using the principle of the index method and the load distributing coefficients, it is possible to transfer the layered pavement structure into a homogeneous structure with equivalent bearing capacity. The index value for a layer is its thickness (cm) multiplied with its load distributing coefficient. The total equivalent thickness for the whole pavement structure could be found by:

he = h1*a1 + h2*a2 + …. + hn*an ……………..…………………………………………(27)

This is commonly referred to as the Strength index of the pavement structure. The index method is illustrated on figure 24 below.

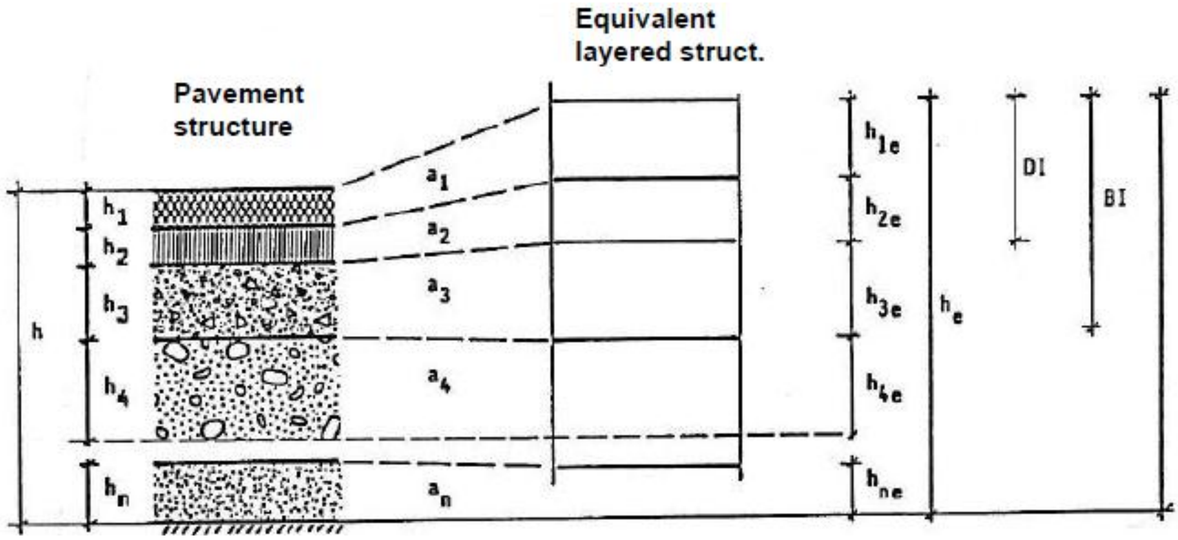


Figure 24: Principles and symbols used in the index method

It is important to notice that the load distribution coefficients **only** express how well the materials distribute load. Other important material properties like stability, wearing resistance, drainage capacity, and surface structure should also be considered when selecting material.. Based on results from field and laboratory testing, standardized values for load distributing coefficients have been found for all common road construction materials. These values are shown in Norwegian design manual Hb 018, figure 512.1 on page 214.

The requirement to equivalent thickness for the pavement structure is dependent on the traffic load (see table 9: traffic group classification) and the ground conditions or frost susceptibility (design manual Hb 018, figure 510.1 on page 211). Based on experience and theoretical analyses, this is implemented into the design table that gives the thickness for each layer. In the Norwegian manuals Hb018, figure 512.7, page 220, table is put to select SI value for different traffic groups of main roads and gives a suggested thickness for different layers. Due to stress concentration in the top layers, specific materials are specified for the base course and wearing course layers. Thickness requirements are implemented as index requirements, and there are defined requirements for a Base layer index (BIk) and a Strength index (SIk).

Current wearing course index, WI = $\sum a_i h_i$ is the sum of the index values for all the layers from the top including all subsequent layers with load distribution coefficient > 2.5. For AADT < 3000, asphalt layers with load distribution coefficient < 2.5 could also be accepted.
Current base course index, BI = DI +$\sum a_i h_i$ is the sum of the index values for all the layers from the top including all subsequent layers with load distribution coefficient > 1.25. For AADT > 1500, the top base course layer should be constructed with materials with load distribution coefficient > 1.25.

Current strength index, SI = BI + $\sum a_i h_i$ is the sum of index values for all layers in the structure.

When we use EPS as a light weight fill material , it is considered as bearing capacity group 6 and over that a concrete plate with thickness of 10cm with load distribution coefficient,a, of 3 should be applied before asphalt. The strength or quality of the plate should be B35 MF45 made of a reinforced mesh with a rod diameter 5 mm and 15 x 15 cm or 3 mm with 10 x 10 cm squares.

Calculation should be made for SI, BI and WI and check with the recommended numbers presented in the manuals.

Table 8: Traffic group classification

Trafikkgruppe	Ekv. 10 t aksler mill.
A	< 0,5
B	0,5 - 1,0
C	1,0 - 2,0
D	2,0 - 3,5
E	3,5 - 10
F	> 10

3.2 EPS material control

When laying out EPS, they should be evenly distributed. The number of EPS blocks that should be used is shown in table B and sampling should be performed as shown in figure A. Dimensions and eveness are checked once for every 25 EPS blocks and heights for screeds used are checked for every 10m along the profile (ref. 17).

Dimensions and evenness of expanded polystyrene checked for each block. The blocks should be square and have flat surfaces. Maximum tolerance for the dimension (height, width, length), is ± 1%. The maximum allowable deviation of uniformity is 5 mm as measured by 3 m straightedge. Thickness Difference between neighboring blocks (blocks in the same team) will be no more than 5 mm.

Table 9: Frequency of control for compressive strength

Filling volume	Total number of blocks
< 500 m3	Min. 3 blocks
500 – 1000 m3	Min. 5 blocks
>1000 m3	Min. 5 blocks per 1000m3

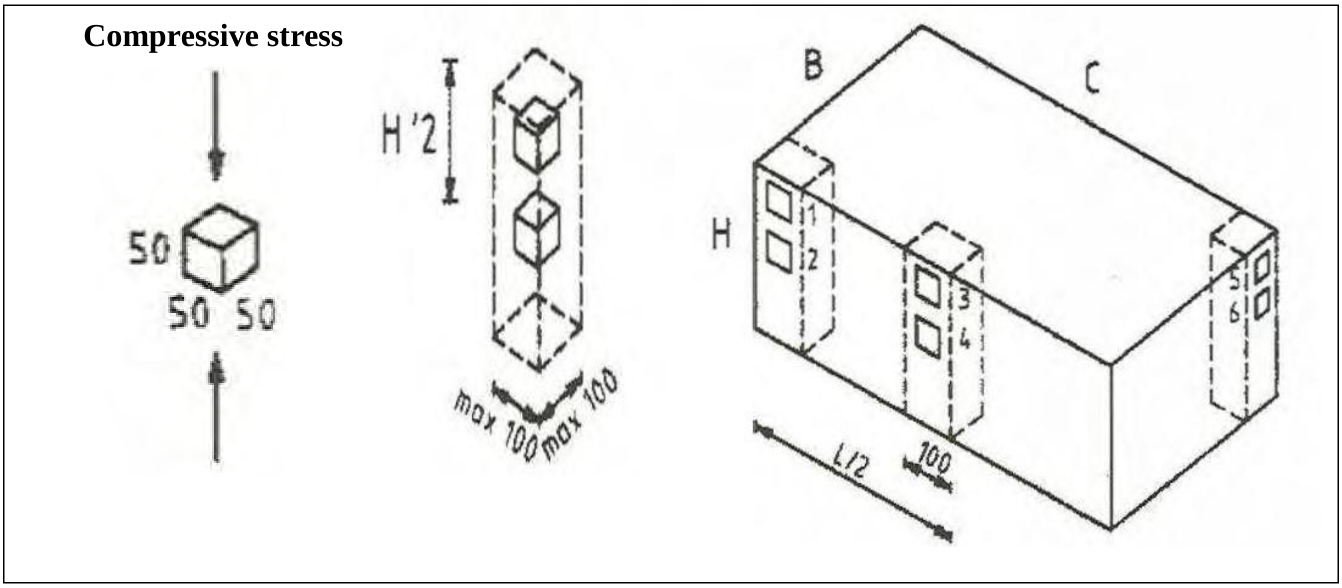


Figure 25: Removing samples for compressive strength control

Table 10: Recommended light weight fill material

Material	EPS block
Material quality	min. 100 KN/m2 compressive strength at 5% deformation

4. External (global) stability evaluation

4.1 Introduction

For the composite structure to be externally stable, the cumulative safety is needed. Settlements of the embankment, bearing capacity evaluation, hydraulic uplift and slope stability will be discussed under this topic (ref 18).

4.2 Settlement of embankment

Settlement is a time dependent process caused by a rearrangement of grain particles due to the load imposed or by process during time. Three types of settlements are dealt here: immediate settlement, primary consolidation and secondary consolidation.

$$\delta_{tot} = \delta_i + \delta_p + \delta_s \qquad (28)$$

Immediate settlement is an elastic and/ plastic deformation that takes place immediately due to the burden applied on the material. Most of this settlement comes when construction is carried out. Both EPS and base soil should be accounted for this.

Primary consolidation is a time dependence settlement as a result of extrusion or withdrawal of pore water from the void spaces.

Secondary consolidation is due to plastic creep of the material and may continue for a long time after the primary consolidation is completed.

Settlements may also occur because of additional stresses in the earth from adjacent structural elements as well as from groundwater lowering.

Settlements can be determined by field and laboratory experiments or estimated from previous experience on basis of current soils.

The following information is needed to carry out the settlement calculations:

- Existing effective stresses in the ground (P_o') calculated from the weight density (density) and pore pressure. Since there is often a reason to assume that pore pressure distribution is not hydrostatic, it is necessary to make pore pressure measurements in different depths.
- Oedometer modulus (M), modulus number (m), preconsolidation stress (P_c’) and consolidation coefficient (Cv) from Oedometer test (mostly) or any other relevant test methods. Typical values for each parameter can be found in Chapter 2 of Hb 018.

Acceptance Criteria for permissible settlement, differential settlement and rate of settlement must be determined in each case based on requirements of factor of safety and functionality for the construction.

Load distribution to be used in this case dependent on whether elastic (Janbu/Jaky) or plastic method such as Boussinesq is used and the calculated results will be different in both cases. The designer must choose the one which contributes the most in the settlement. Soil is rarely flexible. The closest to an elastic situation may be initial settlement, settlement from primary consolidation and the primary settlement of sand and gravel. Otherwise, it is closer to plastic situation.

Normally, settlement calculation for road embankments (our case) Janbu's distribution curves are used and the method is based on plasticity.

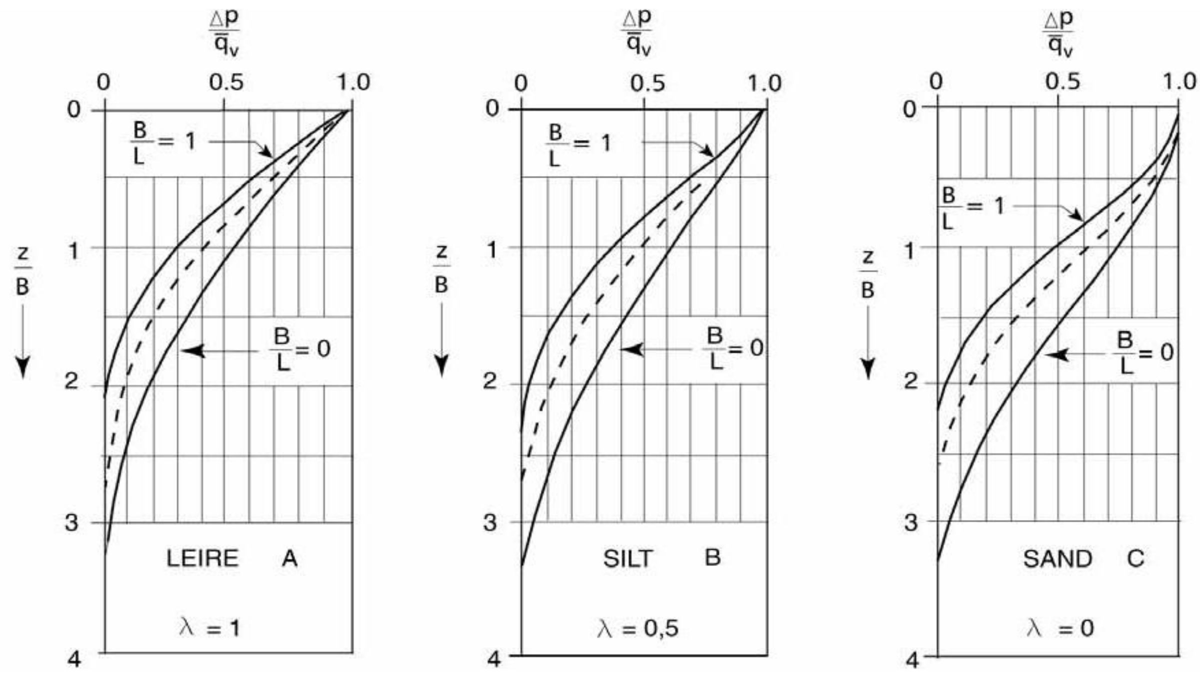


Figure 26: Load distribution for surface stress - plastic equilibrium theory (by: Janbu 1973; Basis of Soil, Ref. 8)

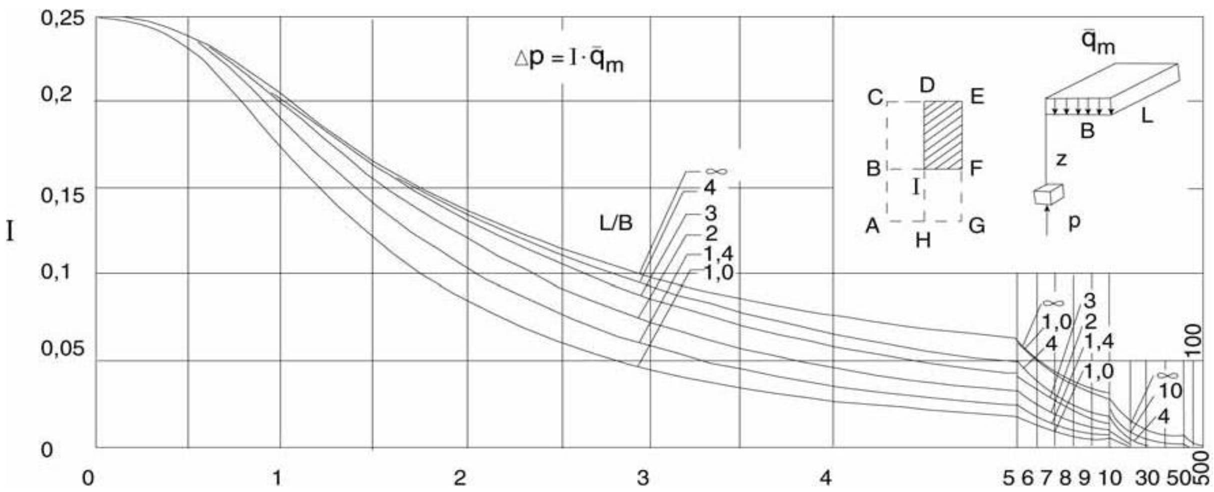


Figure 27: Load Distribution of surface stress- elastic theory. (Boussinesq)

4.2.1 Immediate settlement

Immediate settlement is not a major concern since most of it comes during construction but for computational design it is necessary to quantify the value.

Steps:

a) Additional stresses (ΔP) are calculated at the point of interest using the load distribution methods

b) Deformation moduli (Mi) for the different soil layers are determined

c) Vertical strain is calculated as $\varepsilon i = \Delta P/Mi$

d) Immediate settlement (δi) as $\delta i = \int \varepsilon_i dH$

4.2.2 Primary consolidation settlement

Steps for calculation of the primary settlement:

a) Calculate the vertical effective stress (P_o') to the point needed.

b) Determine the additional distributed external load (ΔP).

c) Determine the pre-consolidation stress (Pc') from oedometer test.

d) Find the oedometer modulus (M or M_{oed}) and modulus number (m) of the materials and calculate the strain (ε_p) at each depth.

Using Janbu's method of settlement calculation:

$M = m.Pa.[P/Pa]^{1-n}$ …………………………………………………………………….(29)

Where

Pa = reference pressure = 100KN/m^2

n = 1 for for rock, OC-clay, firm moraine
= 0,5 for sandy soils
= 0 for NC-clay and clay silts

1) The primary vertical strain for clay and clay silt are calculated using the following formulas:

ε_{oc} = (Pc' – Po')/ Mc for (n=1)

$\varepsilon_{NC} =$ 1/m . ln(Po' + ΔP + Pr')/(Pc' + Pr) for (n=0)

$\varepsilon_{NC} = \varepsilon_{OC} + \varepsilon_{NC}$…………………….....…………………………………………….(30)

Where

Pr is as defined on figure 28 below and sign will be changed if the sloped line crosses the horizontal axis after the origin.

ΔP =P'-P_o'

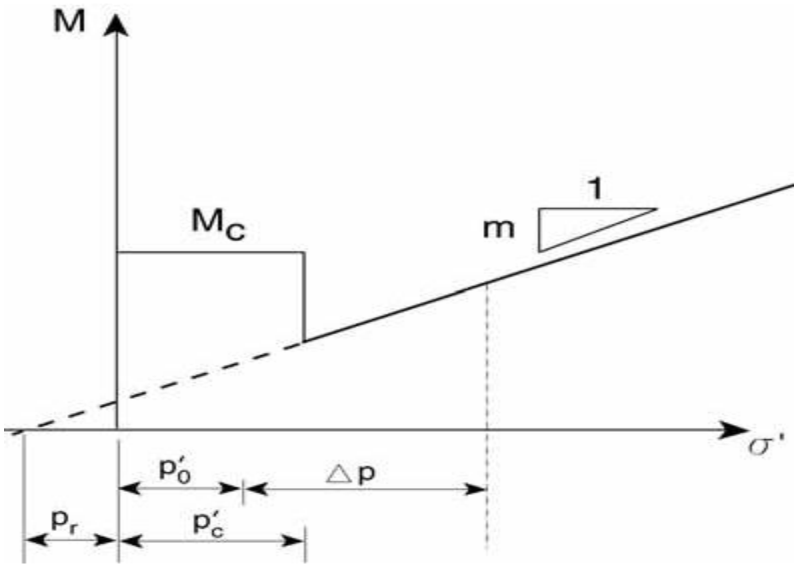


Figure 28: M-σ curve for silt and clay silt

If Po' + ΔP ≤ Pc' then ε_{OC}= ΔP/Mc and ε_{NC}=0.

For a clay that is loaded beyond Pc'

If $Po' + \Delta P \geq Pc'$ then $\varepsilon_{NC} = 1/m \,.\, \ln[(Po' + \Delta P + Pr')/(Pc' + Pr)]$

2) The primary vertical strain in coarse silt, sand and gravel can be calculated using the following formulas:

$M = m.\sqrt{P'.Pa}$

$\varepsilon_p = 2/m \,.\, \sqrt{[(Po' + \Delta P)/Pr]} - \sqrt{(P_o'/Pa)}$

e) Finally the settlement can be computed as:

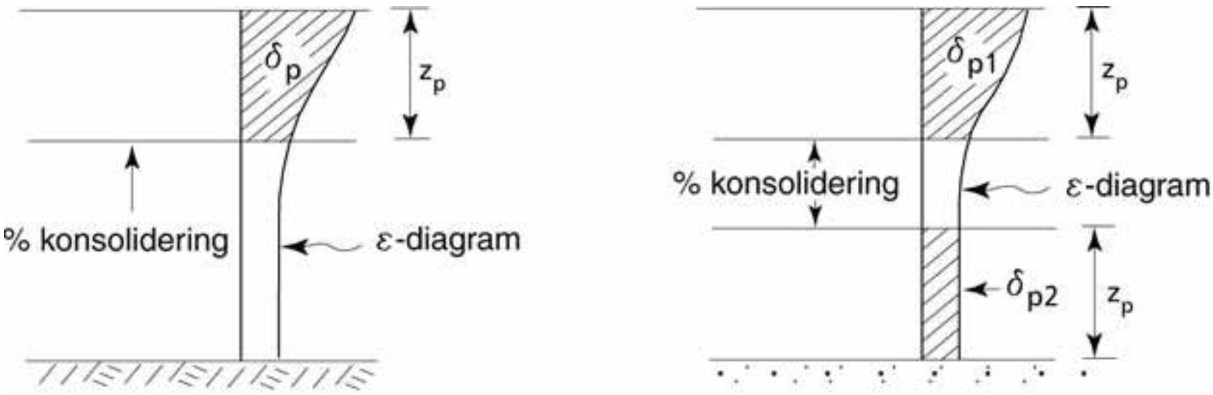
$\delta p = \int \varepsilon_p dH$

f) Coefficient of consolidation

$t_p = H^2/ Cv$

$Z_p = \sqrt{Cv.\, t_p}$

$Cv = \sum (\delta_{pn}\, C_{vn})/ \sum \delta_{pn}$ (weighted average from each layer)



a) One sided drainage b) two sided drainage

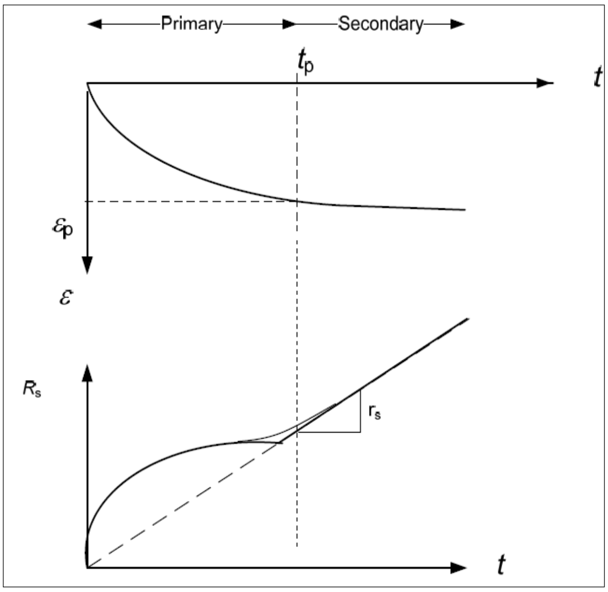
Figure29: Model for estimation of the time course of the primary consolidation

If coarse silt, sand and gravel contain >2% humus and if they are poorly compacted or vibrations altered them, the above formulas are not used.

4.2.3 Secondary consolidation settlement

Procedure:

a) t_p is calculated as before b) settlement calculation



$dt/d\varepsilon_s = R = r(t-t_r)$

$\varepsilon_s = 1/r_s \ \ln[(t-t_r)/(t_p-t_r)]$

$\delta s = \int \varepsilon_s dH$

Figure 30: Determination of time resistance number R and resistance number rs.

4.2.4 Deformation and creep in EPS

From tests undergone in Norwegian road research laboratory (NRRL) and field test measurements at Løkkeberg, the deformation in the EPS can be in the range of 1%of the thickness of EPS fill while an average creep of 6 -7% can be assumed or one can calculate creep by using the general power rule formula or Findley's equation (ref.7).

Total settlement of EPS fill can be the total settlement of the subgrade soil added to the deformation and long term creep of the EPS fill.

4.2.5 Allowable settlement

Settlement differences along and across the road should not exceed requirements given below in the design period. Settlement requirements here in Norway are mainly adapted from the Swedish-guidelines.

Higher standards may set out from the aesthetic or operational technical reasons (storm water system, etc.).

4.2.5.1 Allowable Settlement along the road

Allowable settlement difference along the road length, L, is:

$$\Delta_S = \Delta_{tot} - \Delta_R \quad \text{.....(31)}$$

where:

L is the distance in the longitudinal direction of the settlement difference, measured in meters

R is the vertical radius, expressed in meters

The values of Δtot and Δ_R from figure 31 below is in accordance with the speed limit presented in Hb 017 of street and road design manual.

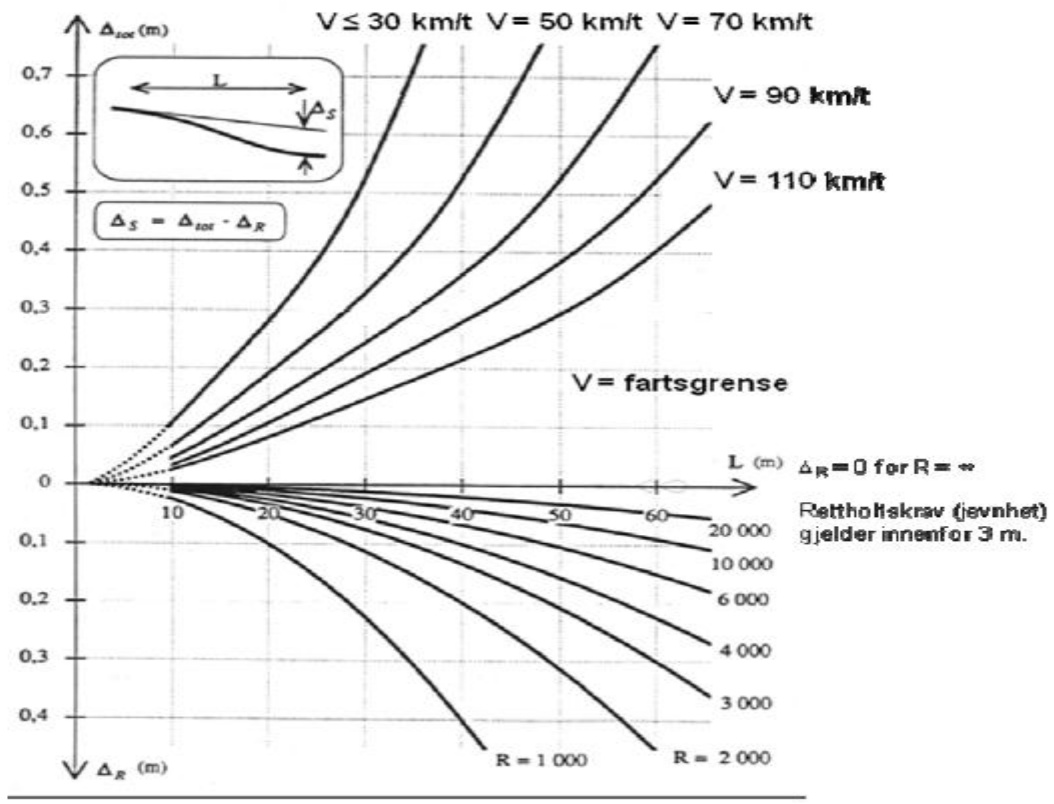


Figure 31: Largest allowable settlement difference Δs for stretch length L.

4.2.5.2 Allowable Settlement across the road

The maximum allowable cross-sectional deviation of the road should be 1% and embankments connected to a bridge, the deviation should be 0 and increases linearly to 1% within a transitional distance of:

· 30 m at the speed limit from 50 to 70 km / h
· 50 m at the speed limit from 90 to 110 km / h

4.3 Bearing capacity Evaluation

On Hb 016 chapter 06, there are two methods of calculating bearing capacity: effective and total stress analysis. With the same principle, bearing capacity of embankment can be dealt in the same way.

4.3.1 Effective stress analysis

First, formulas for flat terrain is provided and by adding a correction factors further, the formula is upgraded to include for slopped terrain and rock fill on the side slope. Since the B/L nearly 0 we can use curve reading for that.

Steps to be followed:

1) Effective parameters (tanφ & a) is determined from standard lab tests as described in chapter 02 of Hb 016.

2) Factor of safety $\gamma_M \geq 1{,}4$ can be used if the quality of the data is good. Factor of safety considerations are presented on chapter 0 of Hb 016 which are similar to Eurocode 7: geotechnical standard.

$$\tan \varphi_d = \tan\varphi / \gamma_M \quad \ldots\ldots(32)$$

3) External loads, i.e vertical load Fv and F_h horizontal load and moment M are determined. It included static drained loading (weight of construction) and short-term undrained aditional loads (traffic, etc.). Partial safety factors for dead load and variable loads are determined According to EN 1997-1:2004 + NA: 2008 (see Chapter 0 of Hb016 and Hb 185;)

$$F_v = F_{vu} + E_{vd}$$

$$F_h = F_{hu} + F_{hd}$$

$$M = M_u + M_d \quad \ldots\ldots(33)$$

u – Undrained loading

d – Drained loading

4) Calculate the effective width B_0: $B_0 = B - 2\,|F/V|$ ……….……………………..(34)

5) Assuming the load is distributed uniformly throughout the effective base:

$$q_v = (F_{vu} + E_{vd}) / B_0 \quad \ldots\ldots(35)$$

6) Horizontal shear is expected to develop on the base soil and its coefficient is calculated as:

$$r_b = \frac{\bar{\tau}_h}{\left(\bar{q}_v + a - \Delta\bar{u}_b\right)\cdot \tan\phi_d} = \frac{F_h / B_0}{\left(\bar{q}_v + a - B_q \cdot \bar{q}_{vu}\right)\cdot \tan\phi_d} \quad \ldots\ldots(36)$$

Where:

A = attraction of the foundation soil

$q_{vu} = q_v - q_{vd}$ = undrained portion of the vertical pressure

$\Delta U_b = Bq^* \, q_{vu}$ = undrained pore pressure

B_q = pore pressure response factor

D = pore pressure parameter

7) Determination of N_q, N_γ and N_u

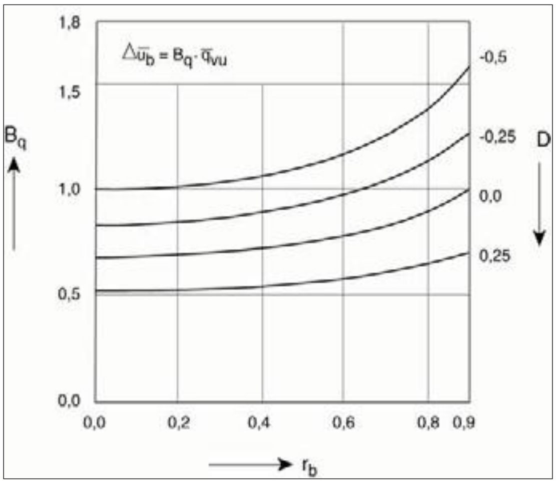


Figure 32: porepressure response factor determination

8) Mean vertical bearing capacity:

For flat terrain

$$\bar{\sigma}_v = N_q \cdot (p' + a) + \tfrac{1}{2} \cdot N_\gamma \cdot \gamma'_{under} \cdot B_0 - N_u \cdot \Delta\bar{u}_b - a \qquad (37)$$

For slopped terrain

$$\bar{\sigma}_v = f_{sq} \cdot \left(N_q \cdot p' + \tfrac{1}{2} \cdot N_\gamma \cdot \gamma'_{under} \cdot B_0\right) + \left(N_q \cdot f_{sa} - 1\right) \cdot a - N_u \cdot \Delta\bar{u}_b \qquad (38)$$

For stone filling

$$\bar{\sigma}_v' = f_{ss} \cdot \left(N_q \cdot (p' + a) + \tfrac{1}{2} \cdot N_\gamma \cdot \gamma' \cdot B_0\right) - a \qquad (39)$$

Where:

$P' = \gamma'_{under} * z$

γ'_{over} = effective unit weight for the material above the foundation

γ'_{under} = effective unit weight for foundation material

ΔU_b = undrained pore water pressure

$f_{sq} = (1 - 0.55\tan\beta)^5$ and $\tan\beta < 0.95\tan \varphi_d$ (reduction factor for sloped terrain)

$f_{sa} = e^{-2\beta \tan \varphi d}$ (reduction factor for sloped terrain)

fss = reduction factor for stone filling (refer Hb 016, figure 6.12)

9) Finally, compare whether σv≥qv, if not bearing capacity requirement is not met.

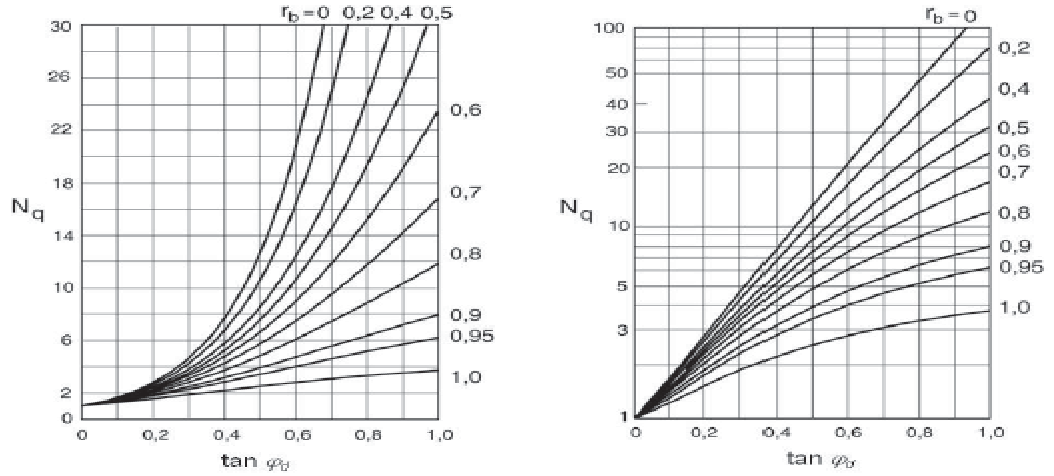


Figure 33: Bearing capacity factor Nq in linear and log scale (

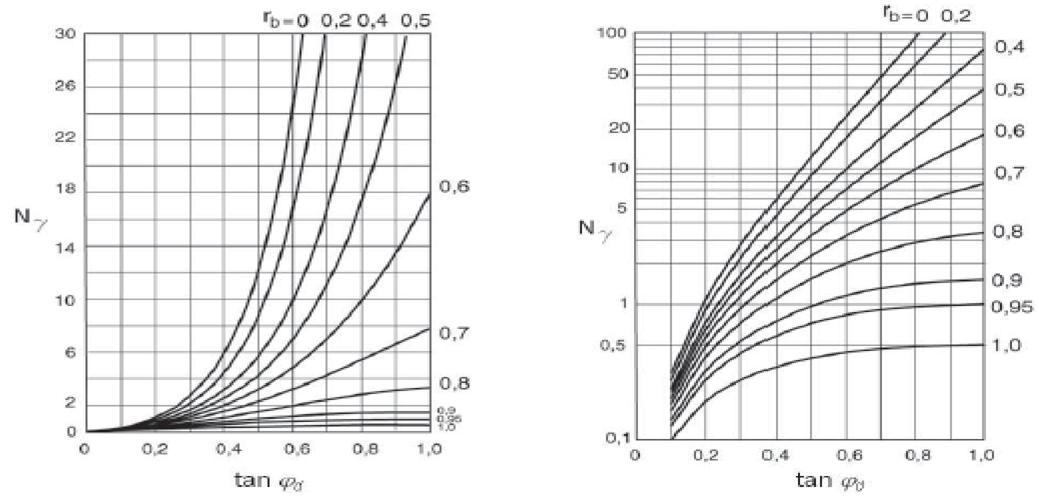


Figure 34: Bearing capacity factor Nγ in linear and log scale (janbu,

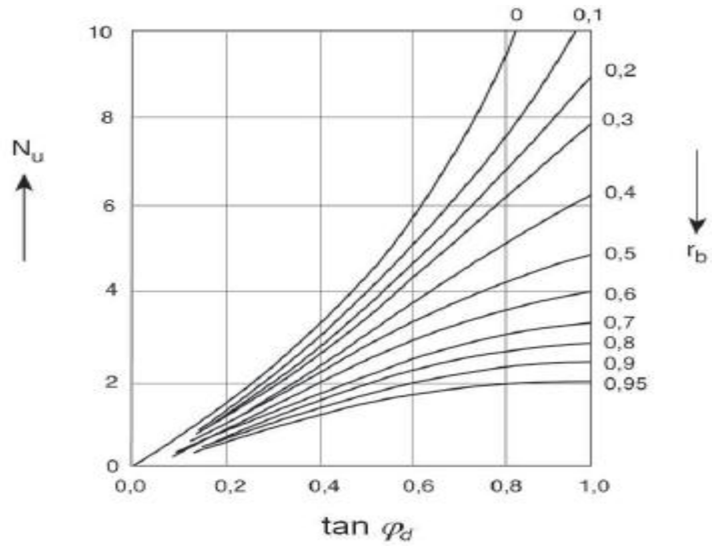


Figure 35: Bearing capacity factor Nu (Nordal, 1989)

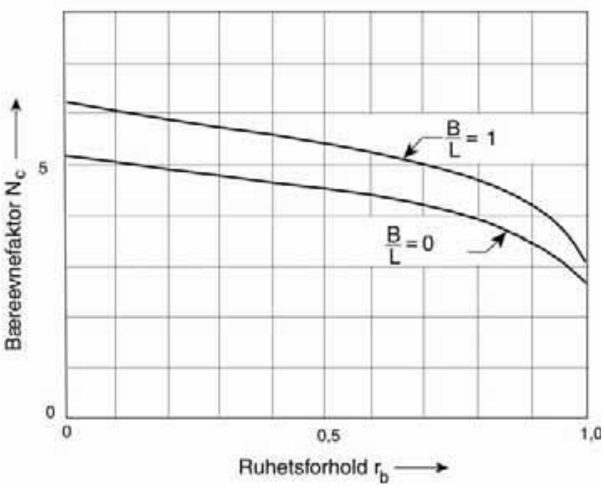
4.3.2 Total stress analysis

Recommended steps to be followed:

1) Determine Cu for the foundation soil.
2) $\tau_d = Cu/\gamma_M$; $\gamma_M = 1.4$ (suggestion on Hb 016, chapter 0.)
3) Determination of external loads, Fv and Fh, with their partial safety factor.
4) Determine the effective width B_0: $B_0 = B - 2\,|F/V|$
5) The mean vertical pressure on the ground surface: $q_v = F_v/B_0$
6) Mobilization coefficient, $r_b = F_h/(Bo * \tau_d)$
7) Bearing capacity coefficient, Nc

a) For flat terrain

b) For slopped terrain (B/L=0 & 0 ≥Fv/τ_d)



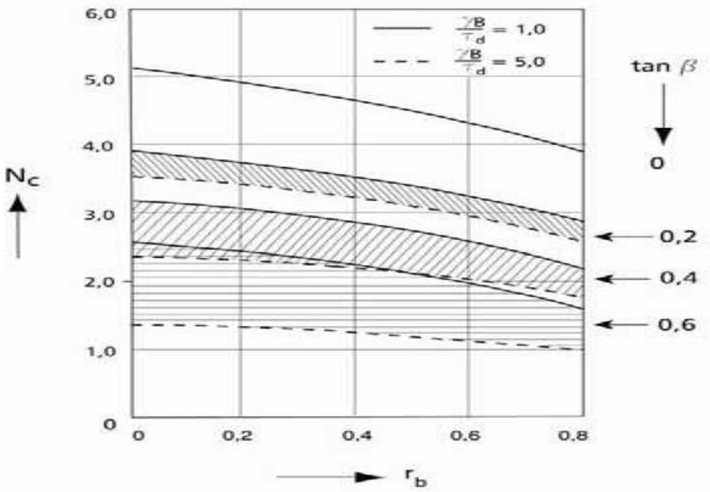


Figure 36: Bearing capacity factor Nc (Janbu, 1976)

Figure 37: Bearing capacity factor, slopped terrain (does not work if Fv/τd ≥ 1.5)

8) Mean vertical bearing capacity:

$$\sigma_v = Nc * \tau_d + P_v \quad \ldots\ldots(40)$$

9) Compare σ_v with q_o (if $\sigma_v \geq q_o$, OK!)

4.3.3 EPS bearing capacity

As explained in chapter 6, loads from traffic (both dynamic and static) can be distributed to each EPS block. EPS distribute load within a 2:1 distribution range according to performance monitoring at the Løkkeberg bridge. By bringing down the load to each level, one can check for bearing capacity by comparing it to the compression strength of the block.(Material parameters of EPS from different tests are explained in chapter 6).

4,4 External Slope Stability of Embankment

4.4.1 Introduction

According to NPRA (Norwegian public road administration) qualifies road projects with clay underneath with respect to stability as serious or very serious class and with the overall class as geotechnical category 3(ref.18).

4.4.2 Assumptions and limitations of stability calculations for classical limit equilibrium methods

Shear strength of materials are fully utilized is not reality. When a landfill with high shear strength rests on soft ground, it can take a failure in the ground before the shear strength of the fill is utilized.

Table 11: Definition of consequence class

Consequense class	Description	Example of construction
CC3	Greater consistency in terms of loss of human life, or very large economic, social or environmental consequences	Terraces, public buildings where consequences of failure are high (eg. a concert hall)
CC2	Medium-high impact in terms of loss of human life, significant economic, social or environmental consequences	Housing and office buildings, public buildings where the consequences of failure are significant (eg. An office)
CC1	Small consequence in terms of loss of human life, and little or immaterial economic, social or environmental consequences	Agricultural buildings where people usually do not present (eg. warehouses), greenhouse

- Since materials such as clay, plastic clay, sand, etc. have very different deformation characteristics, the assumption of fully developed shear along the cutting surface may be wrong.

- We expect a safety against failure which is equal in all materials cutting the failure surface.

We expect generally two-dimensional, since we assume that the cutting surface has infinite extent across the plane rather the real 3D situation infact gives a different

- Result.
- The assumption that all materials mobilize to the same level.

4.4.3 Overview of methods of stability calculation

Table 12 shows the types of stability calculation that is most used in Norway.
Table 12: Type of stability analysis methods

Calculation method	Stress	Type of analysis	Parameter	From which test:
Cu- method	Total stress	Undrained	Cu	Uniaxial, cone,CPT
ADP - method	Total stress	Undrained	S_A, S_D, S_P	Triaxial:active and passive, Direct shear test
aDφ - method	Effective stress	Undrained	a, D,φ	Triaxial
Aφ - method	Effective stress	Undrained/drained	a, φ	Triaxial,CPT

4.4.4 Design methods

The safety situation of the EPS fill embankment varies within the design life. After the filling is over and opened for traffic, pore pressure will developed within the soil mass and increase the shear strength until the excess pore water dissipates. For these situations Hb016 recommends to use Su analysis. Effective stress analysis can also be used as an alternative if proper pore pressure measurements are taken. As consolidation takes place, effective stress analysis (a-φ) can be used.

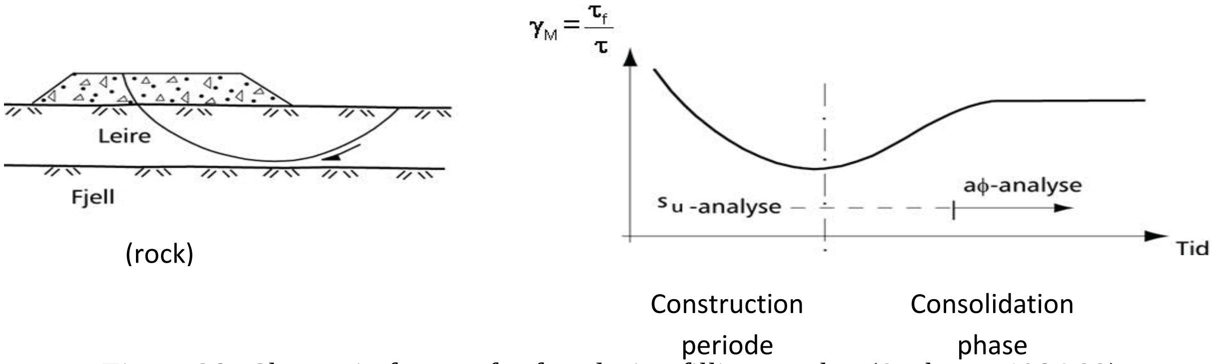


Figure 38: Change in factor of safety during filling on clay (Aarhaug, 1984,22)

Most of the different design methods rarely give the same slip surface or factor of safety, even if they met equilibrium equations.

A) Cu – method

Undrained shear strength calculated from data obtained by performing CPTU-test, uniaxial compression test, cone test and vane shear test will be used.

$\gamma_M = Cu/\tau$…………………………(41)

Where:

Cu - undrained shear strength

Cu = (au +Po') $\tan\theta_u$………….(42)

τ = shear from the embankment

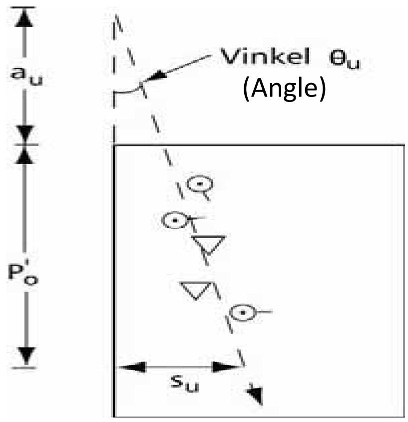


Figure 39: Cu versus Po' pot

Recommended values are provided for Cu/Po' in table 13below from Hb 016, ch04.

Table 13: Recommended values of Cu/Po' up to depth 20m (Berre,1983)

Material Type	Cu/Po'		
	Active	**Direct**	**Passive**
Quick clay	0.27	0.16	0.03
Clay	0.29	0.17	0.07

B) ADP method

ADP stands for active, direct and passive shear strength of the soil. Test results from triaxial and direct shear test are used here. Emperical relations are coined between them.

$S_A = (a_A + Po')\tan\theta_{A;}$ $\tan\theta_A$=0,3 ;$\alpha \geq$+15o

$S_D = (a_D + Po')\tan\theta_{D;}$ $\tan\theta_D$=0,2; $-15^o \leq \alpha \leq +15^o$

$S_P = (a_P + Po')\tan\theta_{P,}$; $\tan\theta_P$=0,1; $\alpha \leq -15^o$

$S_D = 0{,}5(S_A + S_P)$……………………………..(43)

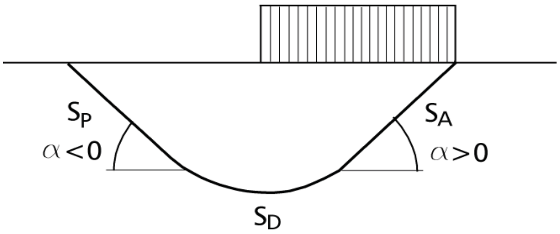


Figure 40: ADP method- parameter application

C) a D φ method

Strength parameters a(attraction) and φ(friction angle) from triaxial test and porepressure parameter D is used here. The D parameter helps us to determine the stress path while the others give strength.

D) a - φ analysis

This effective stress analysis is computed for long term stability analysis of embankment under drained condition. The safety value calculated by drained analysis is higher than the undrained.

$$\gamma_M = f_0 \cdot \frac{\Sigma \dfrac{(a + p_0') \cdot \Delta B \cdot \tan\varphi}{(1 + \tan\alpha \cdot \tan\varphi_d)\cos^2\alpha}}{\Sigma F_v \cdot \tan\alpha + \Sigma F_h} \qquad \tau_k = \frac{(a + p_0') \cdot \tan\varphi}{1 + \tan\alpha \cdot \dfrac{\tan\varphi}{\gamma_M}}$$
……………………..………..(44)

E) Direct method

Graph methods for a- φ and Cu analysis are provided on Hb016, ch04. The graph is for 2D problems, circular failure mode, degree of mobilization is the same throughout the failure circle and homogeneous soil. Design graphs (Janbu, 1954) are provided in Hb016, ch04.

F) Finite element method (PLAXIS)

Plaxis software is the most widely used software in stability analysis here in Norway. The stability analysis is performed by strength reduction techniques(c- φ) reduction. It consists of different built in models in it, starting from the simplest MC model to soil hardening models, ADP model, cam clay model and soft soil models.

4.5 Hydrostatic uplift

Because of the lightness of EPS fill embankment, it must be checked that the safety against buoyancy is sufficient. EPS blocks shall normally be drained and above the normal water level and fills must be protected against flooding, both during the construction phase and later. Design weight density of EPS for safety against uplift calculation shall be used as γ_d = 0.2 kN/m3. Factor of safety provision for buoyancy of γ_m = 1.3 based on the highest probable water level within a 200-year period. The factor of safety is calculated as the ratio between fill weights to the bottom of EPS divided by the buoyancy force.

Assuming that the EPS blocks are to be regarded as closed cells, where only negligible amounts of water will penetrate through them, and then the sudden immersion of the fill creates a buoyancy force per. unit volume. It's magnitude is calculated as the difference in weight between the density of EPS block and weight density of water, i.e:

$$F_{UP} = \gamma_{EPS} - \gamma_{water} \quad (45)$$

$$= (0,2 - 9,8) KN/m^3$$

$$= \mathbf{-9,6\ KN/m^3}$$

$$\boldsymbol{\gamma_M = q/q_{up} \geq 1,3}$$

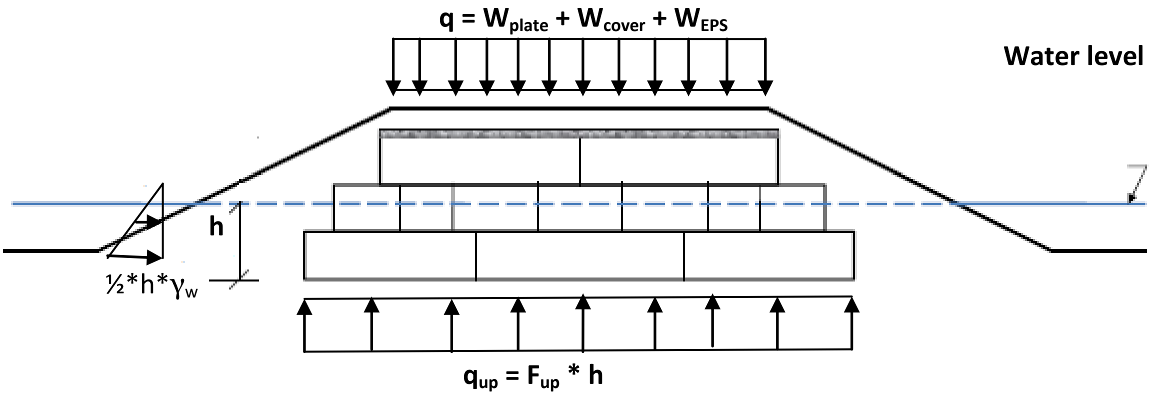


Figure 41: Example of hydrostatic uplift consideration

4.6 Translation and overturning due to water

Because of the weight of the embankment, it is necessary to check for possible translation and overturning failures. Specially, if the height of the embankment is too high.

4.6.1 Translation due to water

Translation happens if there is water only on one side of the fill and the counter acting friction is not adequate. According to the Norwegian standard, HB016 – ch02, the friction angle taken within the EPS blocks and the screed on the foundation soil and the geofoam is considered to be μ = 0,7.

$$\gamma_M = (\sum Wvertical * \mu)/(1/2*h*\gamma_w) \quad (46)$$

4.6.2 Overturning due to water

For embankments which have high fillings, it is advised in the handbook to consider the use of anchoring to prevent overturning and translation and give better stability.

Even if explicit calculation procedures for overturning is not documented for EPS fill, it is important to compute it based on figure 41 and compare the factor of safety with the recommended value from Eurocode 7.

4.7 Translation and overturning due to wind

In the design manuals, it is recommended that EPS fillings should be safe against wind forces both during construction and after.

5. Internal Stability Evaluation

5.1 Introduction

Failures inside the embankment may be one way of failure; connection between pavement and EPS, sliding between EPS blocks and effects of solvents should be considered. The main failure in this situation rises from the property of the fill materials. Different kinds of loads causing for internal failure will be discussed a follows.

5.2 Translation due to water

As described in 4.6.1, the friction coefficient between EPS blocks are taken to be $\mu = 0{,}7$ in Norwegian design manuals. Hydrostatic forces coming from the accumulated water should be checked by the counteracting friction force from the fill above the point in consideration. Factor of safety as presented in ch0 of Hb016 and Eurocode 7.

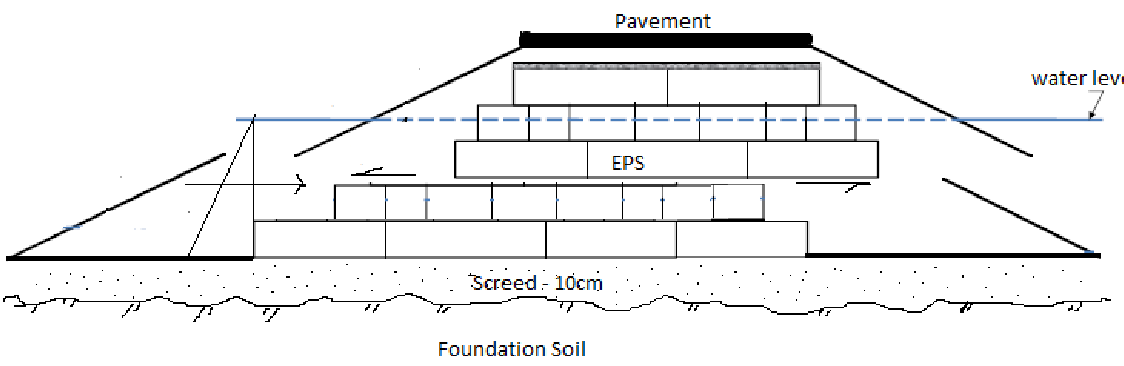


Figure 42: Internal stability consideration for water

5.3 Translation due to wind

In the design manuals it is described that EPS fills should be considered both during and after construction. Detail calculation procedures are not mentioned.

5.4 Load bearing

The main target is to select appropriate EPS block that supports the desired load. From top to bottom of the embankment, appropriate EPS block can be selected using material specification provided in European standard.

From the requirements for all applications the declaration of compressive stress at 10% is the most important issue. Determination has to be following EN 826. Depending on the type of application and the performance requirements the standard is set. For specific applications like in roads and railroads, Norway and other Scandinavian countries use compressibility at 2% or 5%.

Table 14: European standard for compressive strength of EPS (ref.10)

Compressive stress at 2 %	Compressive stress at 5 %	**Designation value, stress at 10 %**
40 kPa	50 kPa	**60 kPa**
60 kPa	90 kPa	**100 kPa**
100 kPa	120 kPa	**150 kPa**
150 kPa	200 kPa	**250 kPa**

EPS-Products are divided into types based on compressive strength and bending strength and the naming is given from the strength at 10% deformation.

Table 15: Overview of stress and E-modulus for different EPS types from European standard (ref.10)

Type of EPS	**EPS 60**	**EPS 100**	**EPS 150**	**EPS 250**	**kPa**
Long term strength	18	30	45	75	kPa
Bending strength	100	150	200	350	kPa
E- modulus	4000	6000	8000	12000	kPa

On design perspectives, EPS blocks are selected at each depth so that the maximum stress at that point be carried by EPS.

6. Seismic analysis

There is no comphrensive document on seismic analysis of EPS embankment. But, Norway has a national application document to Eurocode 8, which is earthquake engineering design. Many codes, including Eurocode 8, use the following principles for earthquake loads (ref.11)

- Specify PGA (ag)
- Specify Sa (Acceleration Response Spectrum) by taking account of soil conditions
- Introduce effect of structural type (ductility properties) for ULS design
- Compute total lateral earthquake force (most codes call it base shear)
- Distribute the load along the height of structure (and compute internal forces, etc. for design)

A) Specification of peak ground acceleration

Norway has an Earth quake zonation map based on acceleration measurement at 40Hz (ag,40Hz) and this value will be the basis for determination of the peak ground acceleration.

$a_g = \gamma I\ a_{gR} = \gamma I\ (0.8\ a_{g,40HZ})$

Where: γI is importance class.

Table 16: Seismic class

Seismisk klasse	γ_I
I	0,7
II	1,0
III	1,4
IV	2,0

B) Selection of Acceleration Response Spectrum

This is achieved according to ground type classification. Generally five ground types:

A – rock

B – very dense sand or gravel or very stiff

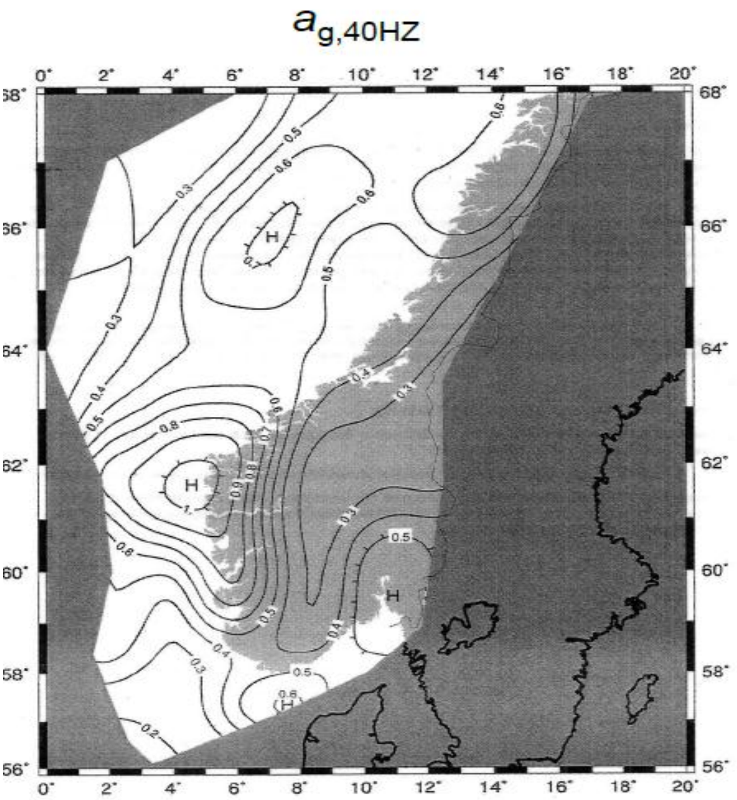


Figure 43: Earthquake zonation, Norway (ref.12)

Clay

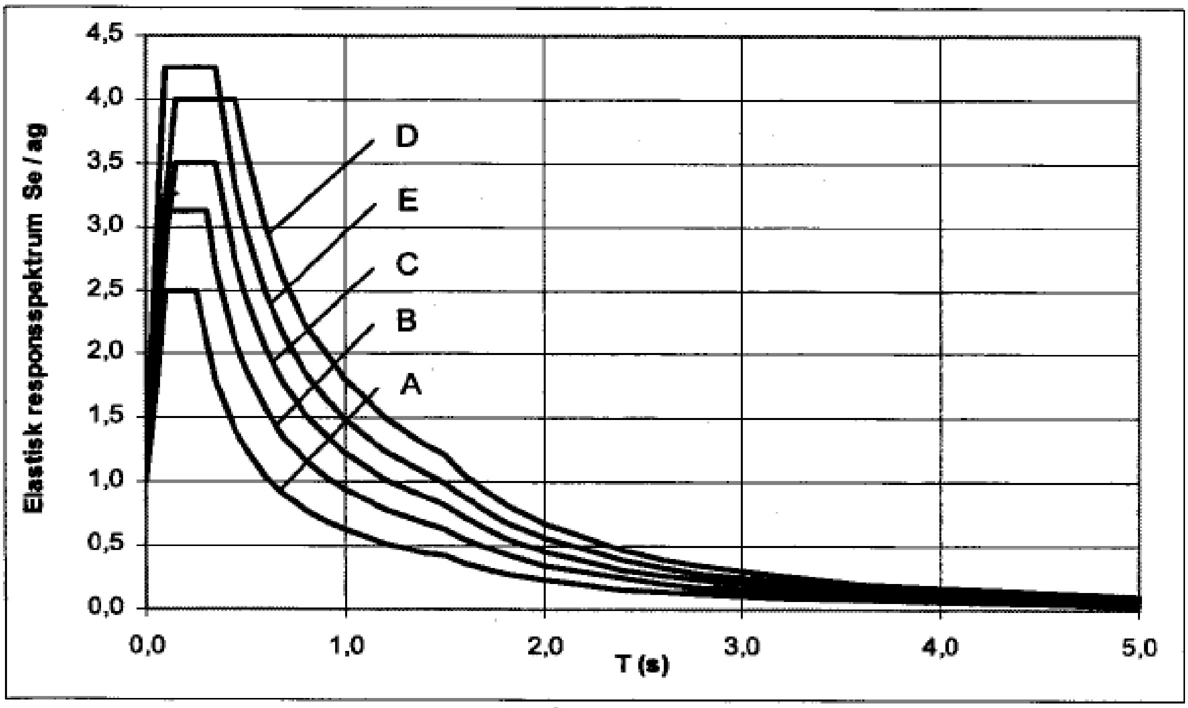
C – dense sand or gravel or stiff clay

D – loose to cohesiolesssoil or soft to firm cohesive soil

E – Surface alluvium layer C or D, 5 to 20m thick, over a much stiffer material

2 special grounds S1 and S2 requiring special studies

Depending from our ground type, we can select Sa from the provided charts.



Grunntype	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,10	0,25	1,5
B	1,25	0,10	0,30	1,5
C	1,4	0,15	0,35	1,5
D	1,6	0,15	0,45	1,5
E	1,7	0,10	0,35	1,5

Figure 44: Response spectra for different ground types

Based on the peak acceleration we have selected, we can calculte the horizontal force on our embankment.

Chapter 4. France

1. Introduction

From its first use as a thermal insulation layer in pavement structures in 1972 Norway, expanded polystyrene has been used in more 25 countries in the world, primarily in Norway, France, Canada, Netherlands, Sweden, Japan, Great Britain, Belgium.

In France, more than 200 projects have been carried out in different regions of the country in different aspects. Experience gained in the field, in parallel with detailed studies of the mechanical properties of EPS in the laboratory, recommendations has been given on the use of EPS as alternative lightweight material in embankment construction. These recommendations include:

- The choice of material;
- The design rules of the lightweight embankments;
- The implementation of expanded polystyrene;
- Precautions to ensure sustainability;
- Control procedures;

In this chapter design experience of only EPS embankment in France is briefly described. But other uses and its design requirements are also described.

2. Design guideline

2.1 Components of EPS fill embankment

Before the EPS fill, the subgrade should be analyzed its stability both transversally and longitudinally. The subgrade then can be adjusted and leveled using either 20/40 size crushed rock with thickness 10 – 20cm or 0/6 size sand can be used as a substitute. Ballast with 40/70 can be used if horizontal force is expected to be high.

The first layout of the ES block is closely monitored and the positions can be alternatively changed until it fits the design or the situation. The blocks are arranged in such a way that the spacing between them is less than 5cm and each block is arranged relative to the other. There is no need to glue the blocks together but if safety is in high note one can use metal connectors.

On top of the EPS fill, a concrete slab will be applied before the pavement structure continues. The typical dimension of the slab will be determined from traffic load based on number of traffic, tangential stress and slope, but the minimum thickness allowed is 10cm for low traffic loads and because of creep reasons, the soil cover on top of the slab should have a less stress than 0.02 MPa or max 1 meter cover.

Pavement structure on top of the fill would be applied on the same normative approach as before. There is several choice of pavement structure that can be applied and they are explained in the next topic and detail construction aspect of the embankment is established in French committee for road engineering report on utilization of EPS embankment (ref.3).

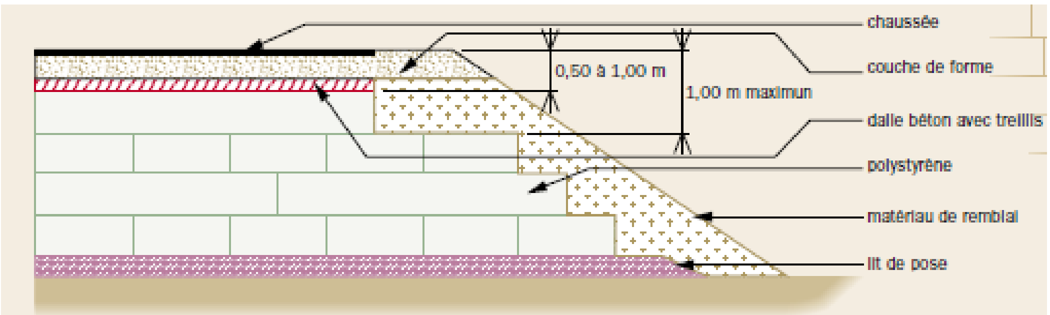


Figure 45: Typical EPS fill embankment

2.2 Design Principles

Special attentions for water uplift and horizontal loads coming from traffic impact are given because of the lightness of the fill. Other typical design follows classical geotechnical procedures.

3. Pavement system design

In France, concrete (C) and asphalt (A) pavements are two main kinds of road binder layer materials, using respectively cement and bitumen as well as natural aggregates as raw materials. Their mechanical properties, especially damage phenomena under cyclic loading induced by the traffic are very different. Therefore, once traffic is defined as the main parameter for road design, different initial thickness layers have to be used for road construction. General outline of thickness calculation is indicated in figure 46 shown below (Ref. 19).

The pavement design method consists in a rational approach based on the knowledge of the mechanical characteristics of the materials employed (normative stages), their manufacturing processes (control) and implementation. It allows adjustment of the thickness of the structures to the local context of bearing capacity of the roadbed and of traffic, according to the materials used and the maintenance policy adopted. Six families of structures are encountered in France :

(1) flexible pavements,

(2) thick bituminous pavements,

(3) pavements with base layers treated with hydraulic blinders,

(4) rigid pavements,
(5) composite and
(6) inverted pavements.

The most common asphalt pavements mainly used in France could be ranked among four types of pavement structures (SETRA-LCPC, 1997). These structures are listed on figure 48 (ref. 20)

The frost design method consists in a "verification" with respect to freeze/thaw phenomena, making sure that the roadway design as determined from mechanical calculations can withstand, without notable damage, a given winter chosen as a reference. With the exception of very large construction projects or special cases, pavement design in France is not carried out case by case. Each road owner has published a document which describes its policy and offers a number of recalculated mechanical and thermal solutions for its network (CFTR, 2003) (DRCR, LCPC, SETRA, 1998), (LCPC-SETRA, 1994 & 1997) (Ref. 21) .For EPS road embankments we can use the above usual method except when we select the pavement we need to take care of the vibration of the rollers.

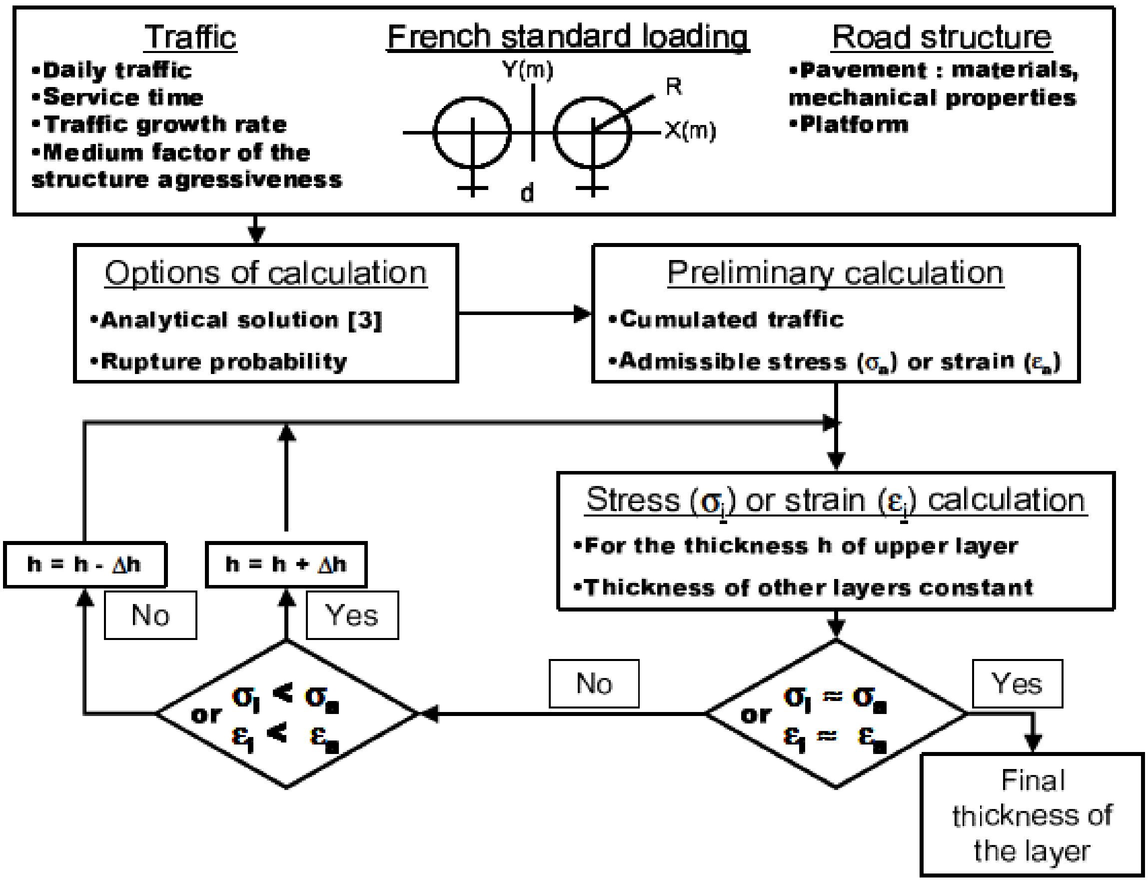


Figure 46: Typical dimensioning procedure for thickness calculation of pavement,functional scheme of the Alizé Win dimensioning software

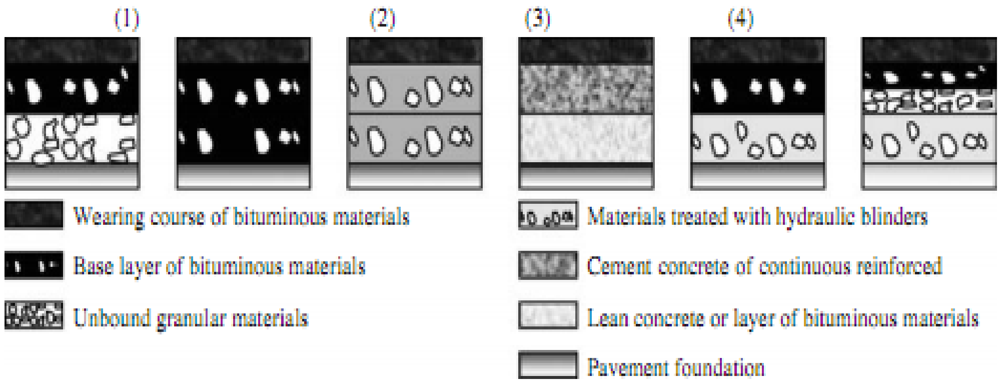


Figure 47: The six families of pavement structure

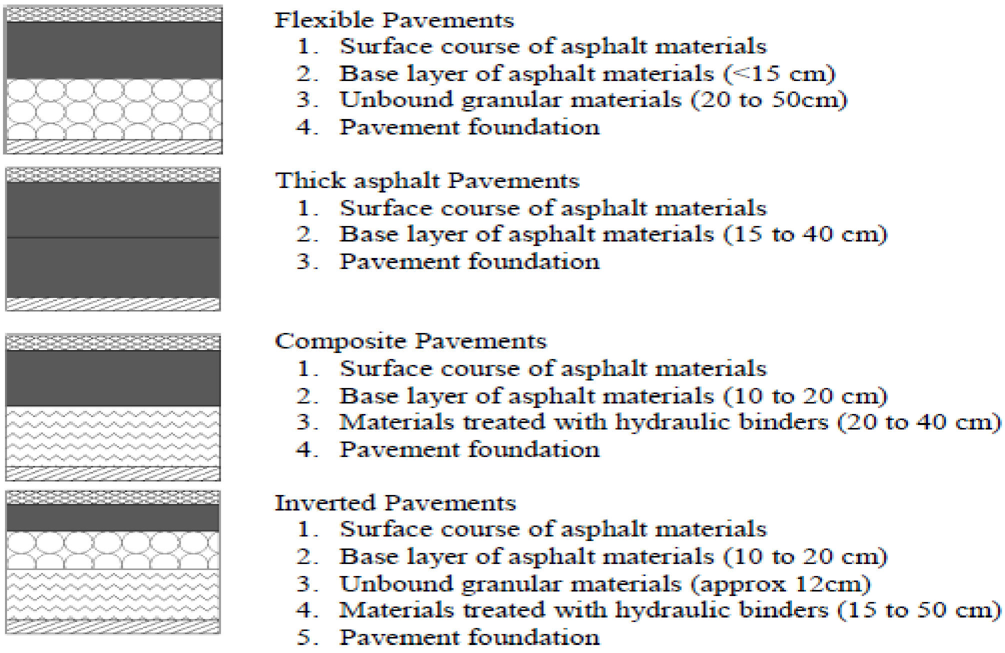


Figure 48: The different French asphalt pavements (SETRA-LCPC, 1997)

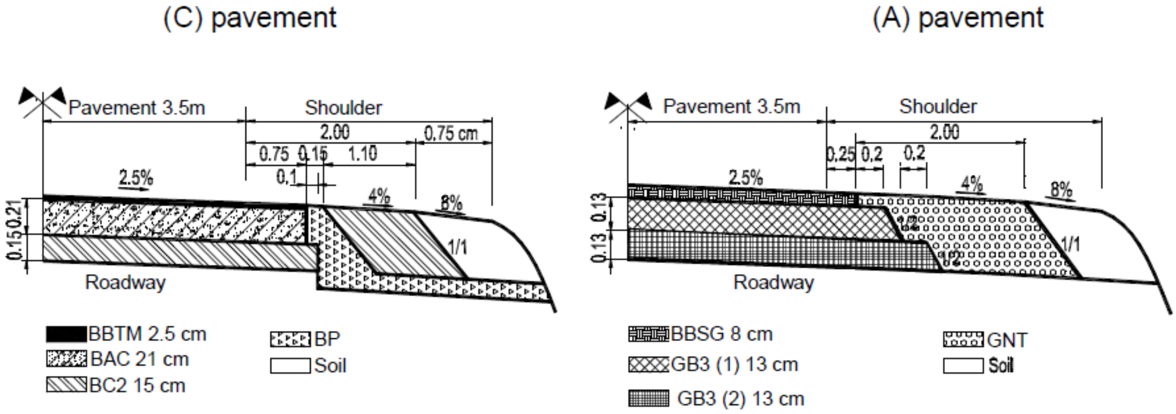
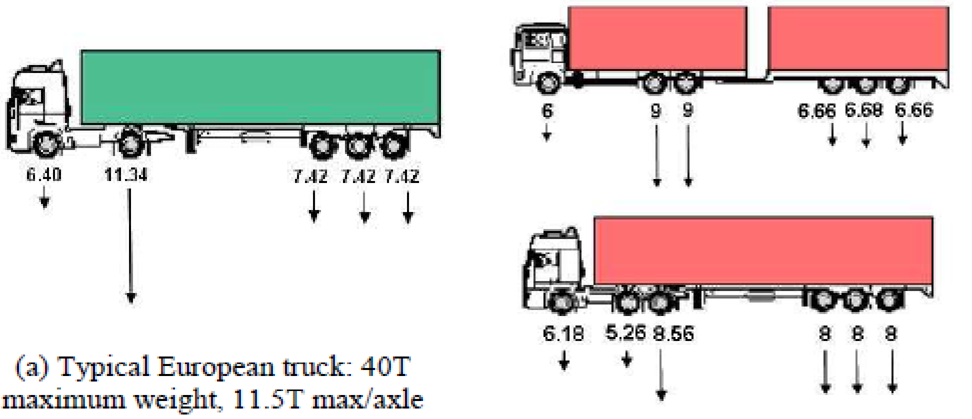


Figure 49: schematic structures of pavements

This types of typical road cross-sections are designed for a standard heavy vehicle of 40 tons (T), which is the French and Europe maximum admissible weight. However, since traffic is increasing, Europe has the desire to increase the total tonnage of freight carried without increasing the maximum weight per axle (11.5T maximum for Europe).



(a) Typical European truck: 40T maximum weight, 11.5T max/axle

(b) 44T combinations, 11.5T max/axle

Figure 50: Typical load configuration of European Truck (http://www.ilpga.ie/public/HGVWeights.pdf)

4. External (Global) Stability Evaluation

4.1 Introduction

This topic gives some indication of the calculations that can be brought to realize when design of an EPS embankment. For calculation we use classical soil mechanics.

4.2 Settlement of Embankment

The replacement of part of an existing embankment by blocks of expanded polystyrene is a reliable and effective method in reducing settlements of embankments on soft soils. This technique is thus used to relief heavy loaded neighboring places from undergoing to a continuation of settlement until it is unacceptable (places near heavy structures on piles, transition from bedrock to soil very compressible, etc.). In some cases, we can limit the long-term settlement by replacing some of preloading embankment by polystyrene.

4.2.1 Settlement of the base soil

To determine the thickness of expanded polystyrene needed for fill, it is necessary at first to estimate the settlement of the base foundation of the soil after relief, taking into account the slowdown in creep, which can be temporarily interrupted due to the decrease in effective stress, to return later with a lower speed, corresponding to the new state of effective stress and over consolidation of the soil. For this we use classical soil mechanics as explained in the earlier chapters and we assumed that the polystyrene has a density of 100 kg per cubic meter.

4.2.2 Deformation of EPS block

In order to determine the deformation of EPS block, first we have to identify the mechanical characteristics of the block. The compression curve of the polystyrene used for road embankments is shown in Figure 51.

It is characterized by:

- The curve shows an initial tangent modulus of linear and reversible, corresponding to a Young's modulus E and a Poisson's ratio close to 0;
- A threshold of irreversible deformation (or yield) σp is noted;
- A strain of ε = 10% depending on the speed of deformation.

The secant Young's modulus is the slope of the line OP joining the origin O of the curve constraint axial / strain at a given point P of the curve corresponding to a fixed percentage of resistance peak (of the standard: 10%).

The cyclic loading does not produce permanent deformation, while they remain below the limits and experience shows that when immersed in water, expanded polystyrene that absorbs very small quantities (less than 1% by volume or from 4 to 9% if continuous immersion for long periods).This may be neglected. However, for safety reasons, a density of 100 kg/m3 is chosen for the calculation of stresses in the blocks (This density was reached in blocks immersed for nine years).

After identifying the stress coming to each block from traffic load, vehicle impact load and pavement load by using stress distribution, we can estimate the deformation easily.

4.2.3 Creep of EPS block

The creep is limited as the stresses applied polystyrene are low and no adequate data on the creep rate available. For an expanded polystyrene of density 19 kg/m3, the creep rate at a pressure of 0.4 σp is about 0.2% per year, at a constant temperature 20 ° C. The creep rate increases significantly with the applied load. Experience shows that to be safe against to this phenomenon, we do not exceed 25% of σp while we load. For lower creep rate, up to 40% of the threshold plasticization is possible.

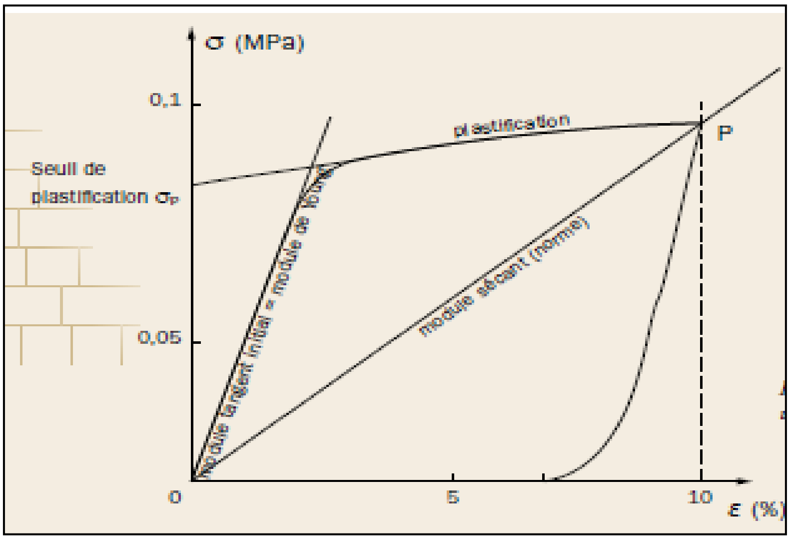


Figure 51 : Stress - strain curve of EPS mass(ρ=19kg/m3)

Total settlement then would be the settlement accompanied by the base soil as well as the total deformation coming from the EPS fill including creep as described above.

4.3 External Bearing Capacity of Embankment

This is done by classical geotechnical calculation methods as described in the other countries experience.

4.4 External Slope Stability of Embankment

For each EPS fill embankment subjected to considerable horizontal forces it is required to calculate stability analysis on possible slip surfaces. Even if no extra detail for calculation of the stability is mentioned in the manuals, it is mentioned that all possible failure modes should be investigated.

If the slip surface is assumed to cut the blocks, it is recommended to take a friction value of 0.5 or 27^{o}. In tall embankments analysis of possible anchor failure should be checked as well.

4.5 Hydrostatic Uplift (Flotation)

In all sites where the fill may be partially or totally submerged, check its stability under the influence of weight and thrust by Archimedes principle. For this calculation, we use the nominal weight of the expanded polystyrene blocks.

This calculation should be performed under the most pessimistic assumption about the water level. We must ensure stability of the embankment in the event of complete submersion. In case of accident, when an EPS fill was not designed to withstand submersion, it should be preserved by loading the surface of the embankment with heavy materials throughout the period of submersion.

5. Internal Stability Evaluation

5.1 Introduction

In this topic forces causing possible failure in EPS blocks or between EPS and the pavement structure mentioned in French guideline are mentioned. The methods of calculating the effect of wind or impact vehicles is inspired by the Norwegian practices.

5.2 Translation Due to Water (Hydrostatic Sliding)

It is clearly indicated in the guideline that possible full submergence of the EPS embankment during flooding time should be checked in the design. According to the height of the fill, we can calculate the force coming to the embankment and compare it to the respective resistance friction force from the blocks. Nothing has been mentioned on the factor of safety provision regarding this condition.

5.3 Wind Resistance

The analysis of the stability of a solid expanded polystyrene subjected to lateral wind (Figure 52) can be analyzed on very simplified method as follows.The forces exerted on such a work, per meter in length, are:

- R is the force corresponding to the direct pressure of wind on the exposed face of the embankment (Pr);
- S is the result of suction of the wind on the opposite face (Ps);
- P is the weight of the embankment and pavement layers;
- F is the result of friction mobilized to solid base of expanded polystyrene.

Pressures Ps and Pr are related to the wind velocity V by the formulas:

$Pr = 0.75.\ V^2.\ \mathrm{Sin}\ \theta.$

$Ps = 0.50.\ V^2.\ \mathrm{Sin}\ \theta.$

Where: V = Wind velocity in m/s

Ps and Pr = Pressures in Pa

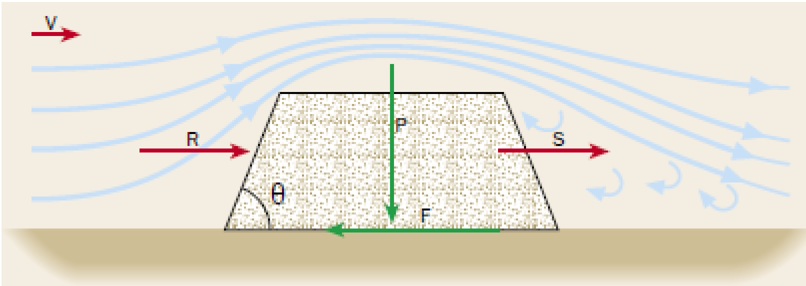


Figure 52: Effect of wind on an embankment of expanded polystyrene

$R = Pr * H_{fill}$

$S = Pr * H_{fill}$

$F = \rho_{EPS} * H_{fill} * Width * f$; $\rho_{EPS} = 0.2\ Kg/m^3$ and $f = \tan\varphi = 0.5$

$F > R + S$,for this calculation, it simply checks the balance of embankment with a safety factor of 1.

For this case it is also important to check the capacity of the joint, binding.

5.4 Load Bearing

For the construction of road embankments alleviated, the essential properties of EPS are:

- Its low density;
- characteristics of deformability
- Durability (over 30 years experience currently).

The density of polystyrene determines most of its physical and mechanical properties. Despite its short comings the AFNOR standard NF T 56-201 is used to characterize the material. The standard defines thirteen references of expanded polystyrene, but in Road construction only EM and FM standards are used.

Table 17: qualities of EPS produced in France (from the standard NF T 56-201)

Reference	AM	BM	CM	DM	EM	FM	GM
Stress (Kpa)at ϵ = 10%	/	30	50	70	90	140	190
Minimum density (Kg/m3)	7	10	13	15	19	24	29

We can check the load bearing capacity of the fill by distributing the load to each level of EPS floor and compare the value with lab test established stress and E values.

5.5 Translation due to vehicle impact

A very simplified method of verifying the internal stability of solid polystyrene under horizontal forces due to braking is illustrated by the diagram in Figure 53. It is assumed that slip occurs on a plane separating two layers of polystyrene with a friction coefficient of 0.5. Efforts to assess are:

- braking force exerted by S on the vehicle floor (product of the mass of the vehicle by its acceleration)
- P the weight of the vehicle, the roadway and EPS;
- F friction mobilized on the surface of Slip studied.

For example, a 30 T truck, undergoing 7 m/s2 deceleration on a floor equal to 50 cm of a material unit weight of 25 kN/m3, creates a braking force of 210 kN, while the frictional force mobilized is 270 kN.

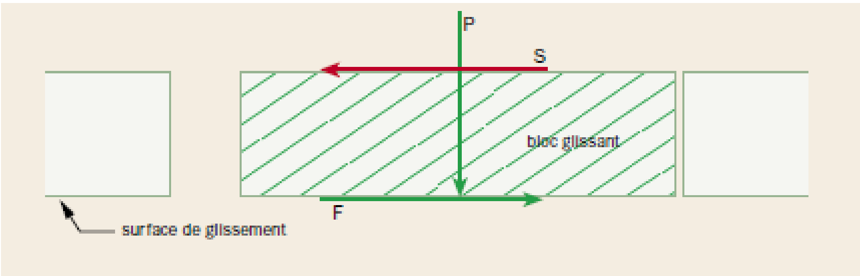


Figure 53: Force equilibrium due to vehicle breaking force

6. Seismic Analysis

France follow the basic guidelines presented in Eurocode 8. Neverthless, they have internal standards adpted to Eurocode 8.

According to France national code NF P06-013, the country is divided in to four seismic zones (zone 0. Zone l, zone ll and zone lll) and four structure classes (class0, class1, class2 and class3). Soils have been classified in to:

- sound rock
- Group a – good to very good mechanical resistance (eg. Compacted marls, heavily consolidated clays)
- Group b – Average mechanical resistance(eg. Weathered rock, medium dense sands and gravels)
- Group c – low mechanical resistance (eg. clays, gravel, mu, weathered chalk)

Accordingly construction sites are classified in to 4 classes:

- Sites S0 – bedrock sites, group a soils lessthan 15 thickness
- Site S1 – group a soils > 15m in thickness, group b < 15m
- Site S2 – group b soils 15 – 50m, group c < 10m
- Site S3 – group b > 50m, group c 10 – 100m

Elastic reponse spectra for a 5% damping is presented in the code and they correspond to a nominal acceleration value.

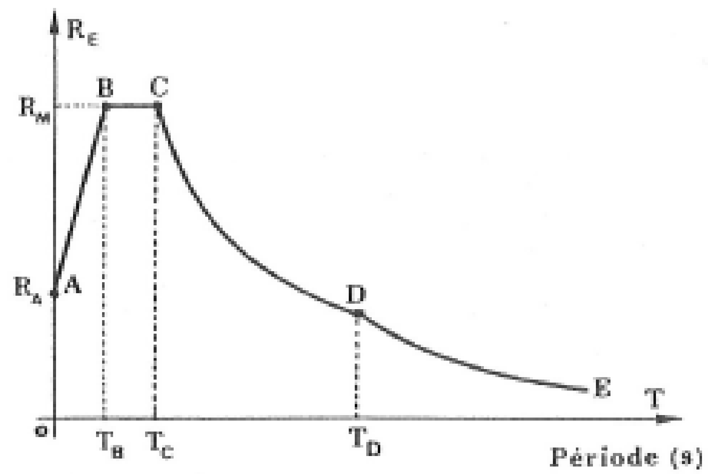


Figure54: Elastic response spectra (ref.13)

A) *Horizontal acceleration component*

Depending on the acceleration components we calculated from graphs, we can calculate the horizontal force that might apply to our embankment. The effect of the vertical acceleration should be checked, especially if the direction is upward.

SITE TYPES	T_B (s)	T_C (s)	T_D (s)	R_A	R_M
S0	0.15	0.30	2.67	1.0	2.5
S1	0.20	0.40	3.20	1.0	2.5
S2	0.30	0.60	3.85	0.9	2.25
S3	0.45	0.90	4.44	0.8	2.0

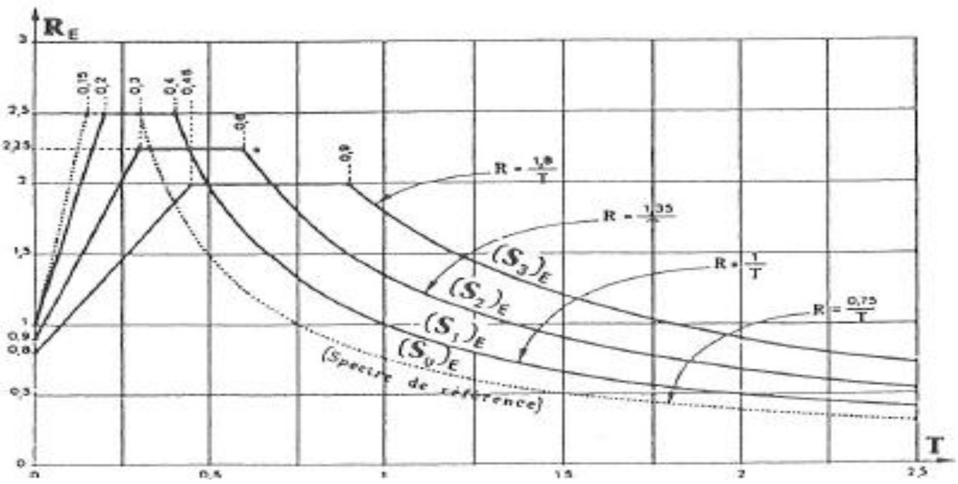


Figure 55: Horizontal acceleration component (ref.13)

A) *Vertical acceleration component*

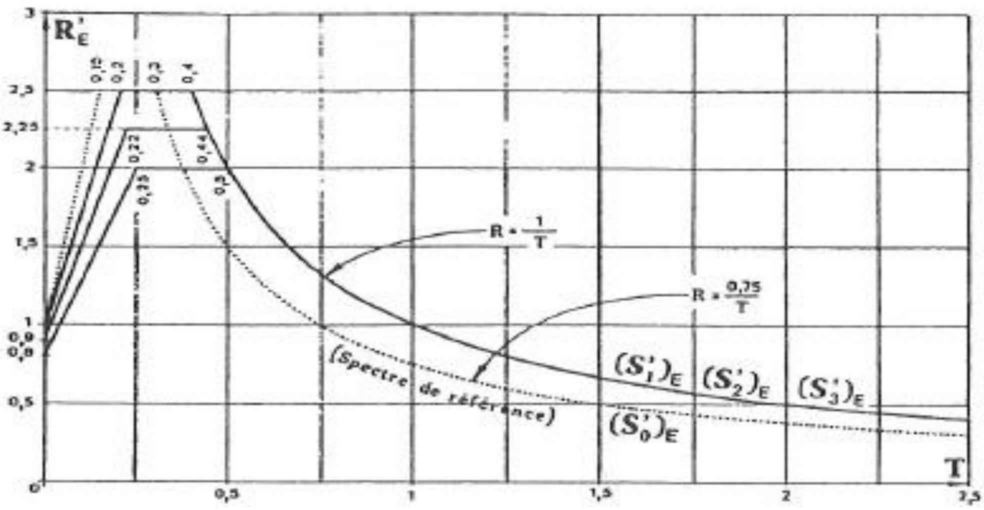


Figure 56: Vertical acceleration component (ref.13)

Chapter 5. Greece

1. Introduction

EPS is becoming popular in high fill embankments because of faster construction time, better quality and its applicability in solving geotechnical problems. The main source of design procedure mentioned in this chapter is from the information obtained on paper presented by Georgios Papacharalampous on EPS 2011 conference in the design of an embankment in a section of the Athens-Thessaloniki highway, very close to the historical site of Thermopylae (ref. 22).

2. Design guidline

2.1 Major Components of an EPS-Block Geofoam Embankment

Three major components are synchronized: Underlying subsoil, EPS fill and pavement. For a complete design, all the three have to be able to satisfy design requirements. Proposed scheme for construction was as follows:

- Complete removal of the existing fill and the top soil to a depth 0.75m from GL
- Placement of granular material at the base, of total thickness _50cm, comprising from bottom to top by: a) 25cm selected material serving as working platform, b) separation geotextile and tensile reinforcement geogrid, c) drainage-foundation and levelling granular layer of thickness varying from 25cm to 75cm, with a slightly inclined roof oriented parallel to road surface.

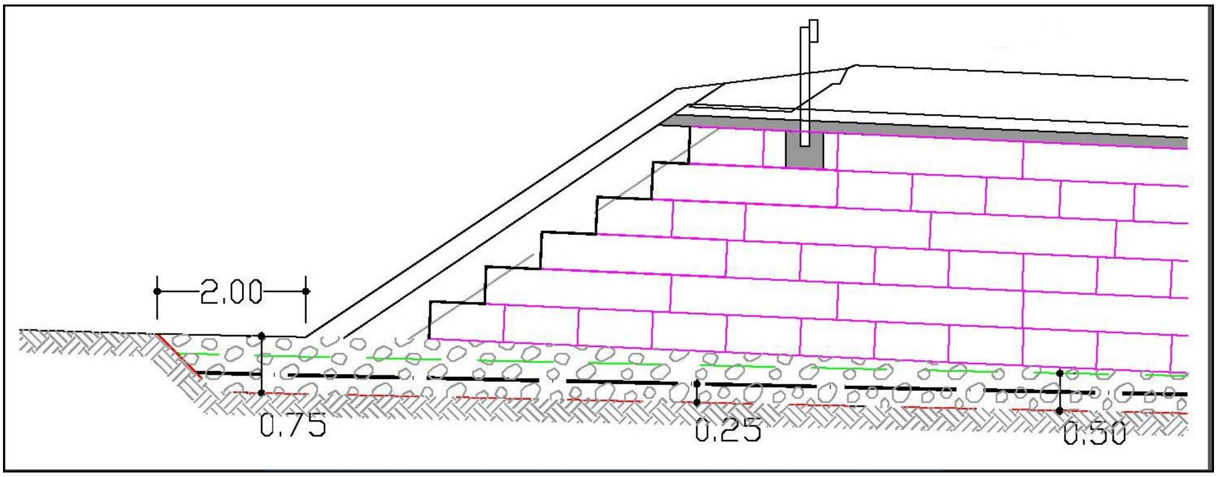


Figure 57: Material placement above subsoil in Athens-Thessaloniki highway

- Placement of EPS 100 type blocks, with minimum dimensions 0.5x1.0x2.0m, on top of the inclined leveling surface. Consecutive layers are placed at right angles to each other in order to avoid continuous vertical joints, with the upper row aligned transversal to the axis direction. A stepped side slope similar to embankment's final slope 2:3 (v:h) is formed. Steel fasteners prevent blocks from moving out of position during the construction process.

- Side slopes are protected by 1mm polyethylene membrane and a geotextile cloth covered with _0.5m lightly compacted fine granular soil and 0.3m top soil.

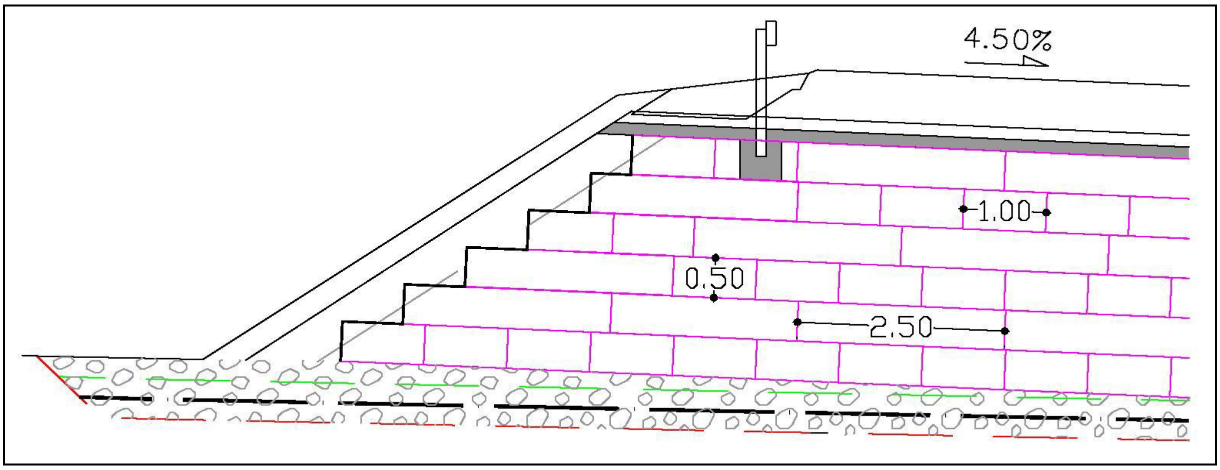


Figure 58: EPS block arrangement

- By casting a 15cm lightly reinforced concrete slab C20/25 on top of EPS fill, protection of fill against solvents and pavement's proper foundation and drainage is achieved.
- At the junction of lightweight and earth embankment a stepped surface is formed every 0.5m of the earth fill and a drainage pipe is placed at the base. The stepped interface is covered by granular material, geotextile and sealing membrane and EPS blocks are placed on top.
- Side slopes:
  •1mm PE membrane
  •geotextile cloth
  •≈0.5m lightly compacted fine granular soil
  •slope 2:3 (v:h)
  •0.3m top soil.
- Consecutive layers at right angles to each other to avoid continuous vertical joints, upper row aligned transversal to the axis direction.
- Steel fasteners prevent blocks from moving out of position during the construction process.

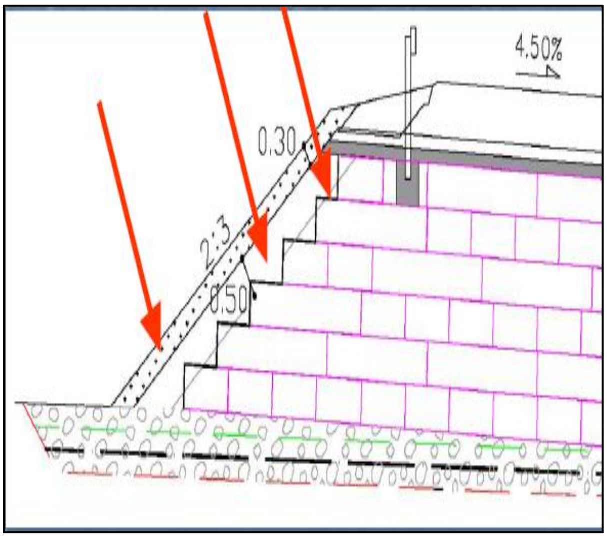


Figure 59 : Side slopes of an EPS fill

- Pavement is applied on top with drainage layer on top of inclined slab.

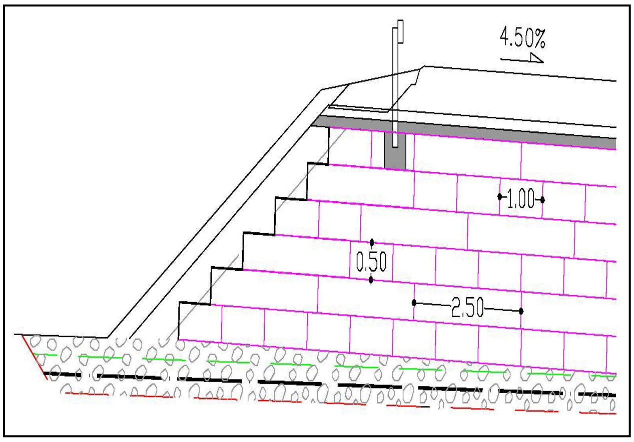


Figure 60: Usage of steel fasteners during construction

3. Pavement Design

3.1 Introduction

Typical cross-section of pavement structure used in Greece is presented in successful topics.

3.2 Flexible Pavement System Design Catalog

The typical cross-section of flexible pavement practiced in Greece is as shown below here in figure 61.

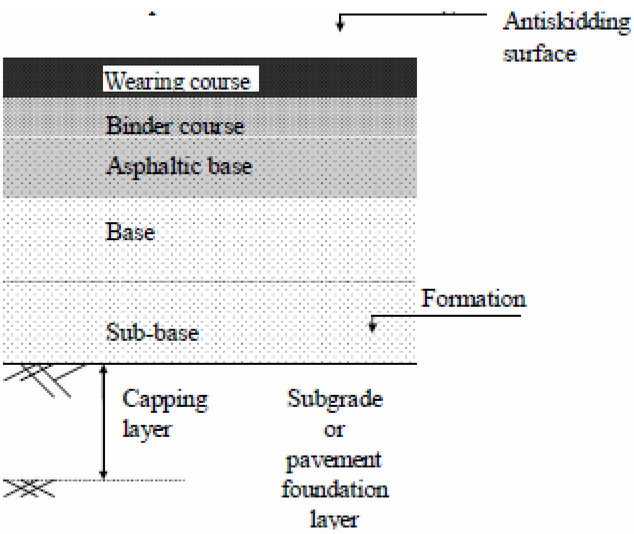


Figure 61: Typical cross section of a flexible pavement

The wearing course, the binder or base course and the asphaltic base are all bound with bituminous binder. The base and sub-base layers are normally unbound and in some cases weakly bound with cement granular or soil material.

The *wearing course* is the upper most bituminous layer of the pavement. Its function is to provide an even rolling surface with good antiskidding properties. The antiskidding properties are provided either by the gradation and the hardness and durability of the aggregates of the mixture that the wearing course is constructed of, or by a specially designed antiskidding mixture (layer). Apart from this, the wearing course should not deform under traffic and it should resist crack propagation. It is desirable to contribute to the strength of the pavement as well as to be almost impermeable to water and to reduce the noise generated by the moving wheel. If the first two properties mentioned in the last sentence do not coexist, they should be provided by the underlying asphaltic layers.

The *binder course or basecourse* provides the platform on which the wearing course is laid. Together with the underlying asphaltic layer it contributes to the strength of the pavement and forms the main structural layer of the flexible pavement. In case the wearing course is water permeable, the binder course must always be of a dense graded mixture (asphaltic concrete). Additionally, the binder course should not deform easily under traffic, should resist crack propagation and it should have good fatigue life.

The *asphaltic base* together with the binder course is the main structural layer of the flexible pavement required to distribute the applied traffic loading so that the underlying materials are not overstressed. It should sustain the stresses generated within itself without excessive or rapid deterioration. It is the layer which contributes the most to the overall stiffness of the pavement and to the pavement’s resistance to fatigue cracking. Due to its asphaltic nature and its greater thickness compared to other asphaltic layers, it contributes substantially to the permanent deformation of the pavement. It is therefore necessary, that the asphaltic base should have good dynamic and static stiffness modulus and good fatigue performance.

The *base/sub-base* layer is the unbound layer (sometimes cement bound) which may consist of two sub-layers: the base and the sub-base layer. Its function is to reduce further the vertical stresses induced by the traffic loading to the subgrade. It provides a good platform for laying and compacting the asphaltic base. During construction, it also provides a good platform to the construction vehicles. Indirectly, it also acts as a frost protection layer to the frost susceptible subgrade.

The *capping layer* is a subgrade improvement layer which protects weak subgrade from damage during the period of construction as well as throughout the pavement’s service life.
It may be constructed with relatively cheap suitable imported material or it may be a treated in-place material. It is placed between the native subgrade and the sub-base layer, as a replacement of the native subgrade (i.e. say 600mm of capping layer is required, remove 600mm of subgrade and replace it with better quality material). The base/sub-base layer together with the capping layer (when used) forms the foundation of the pavement.

Apart from the above mentioned pavement components there is also the drainage layer and, in some cases, the frost protection layer. Either layer or both together, when placed, they are not considered to contribute substantially to the strength of the pavement and therefore they are characterized as non structural layers.

The drainage layer is provided to protect the subgrade, the capping layer and the base/subbase layer, both during construction and during the service life of the pavement, from incoming surface rain water through the overlying layers. When the water table is high and the subgrade is moisture sensitive the drainage layer is also beneficial. The drainage layer is placed between the subgrade or capping layer and the sub-base layer, and is connected to the french or fin drain. The thickness of the drainage layer is usually 200mm, and never less than 150mm.
The drainage layer placed at the above mentioned position does not act, by any means, as drainage layer for lowering the water table. In case that lowering the water table is required and if drainage layer is selected to resolve this problem, the position of the drainage layer should be lower into the subgrade. In order to stop pore clogging by fines from other adjacent layers, geosynthetic materials acting as separators may be used when those layers are constructed of fine soil or fine capping material.

The frost protection layer is constructed, if needed, in order to protect the frost susceptible subgrade. This layer is an extension of the sub-base layer, and its thickness should be such that the total thickness of the pavement equals or is greater than the depth of frost penetration.

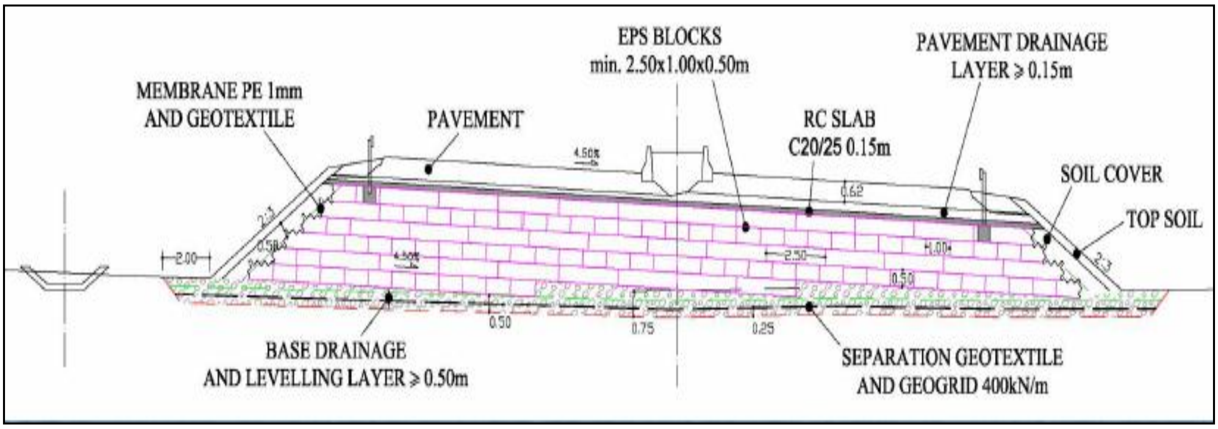


Figure 62: Interaction between pavement and fill mass

4. External (global) stability evaluation

4.1 Introduction

The total composite EPS fill embankment should have to fulfill geotechnical engineering parameters like settlement, bearing capacity and slope stability. On the following topics these terms are explained.

4.2 Settlement of Embankment

4.2.1 Introduction

Total settlement of the EPS fill embankment is the summation of the deformation contribution of the EPS blocks and the settlement undergone by the subsoil. Settlement calculation of the subsoil is based on geotechnical parameters of the subsoil acquired through borehole investigation or back analysis. Deformation manipulation of an EPS block during different loading is described in subsequent topics.

4.2.2 Immediate settlement

Elastic deformation, immediate deformation, takes place mainly during construction. It can be estimated using the formula:

$S_{EPS} = P \text{ x } H_{EPS}/ E_{EPS}$ assuming a

E_{EPS} =5 MPa assumed deformation modulus based on TRB

P= load on top of EPS block (traffic load, earth fill and concrete block)

H_{EPS} = Height of EPS fill

Provided settlements based on this calculation gives negligible compared to the settlement of the subsoil.

4.2.3 Deformation under repetitive loading

Deformations under repetitive traffic loads should be within acceptable elastic limits. Traffic loading (q) on top of EPS was roughly estimated by considering a load distribution coefficient of $I_c = 1$ for the pavement of thickness $T_{p,min}$ (thickness from pavement design) and $I_c = 2$ for the concrete slab of thickness Tc = 0,15cm (widely used). Calculated traffic load, q, on top of EPS block should be less than 35%*σ_{c10} and total load q+p < σ_e =50kPa, where σ_e minimum elastic limit stress (recommended by TRB).

4.2.4 Creep deformation

Permanent loads should be within allowable limits in order to avoid considerable creep deformations. Pavement and concrete slab loads (p) acting on top of EPS do not exceed 20-25kPa. For stresses of that order, which correspond to p < 30% σ_{c10} = 30kPa (recommended by Frydenlund) and to <1% immediate strain (recommended by TRB), tolerable creep deformations are expected.

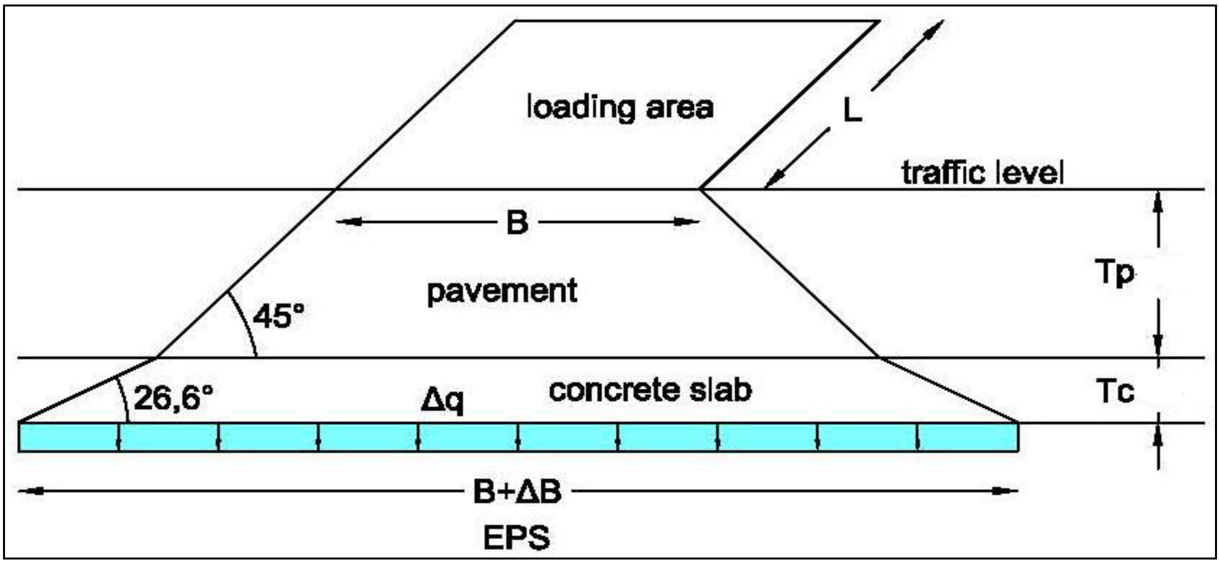


Figure 63: Traffic load distribution up to top of EPS

4.2.5 Allowable Settlement

According to Greece guidelines for highways, post construction settlement for highway should be less than 10cm (PC settlement ≤10cm).

4.3 External Bearing Capacity of Embankment

Classical geotechnics is used to assess the bearing capacity as described in earlier experiences.

4.4 External Slope Stability of Embankment

The global stability of the lightweight embankment interacting with the foundation soil, was checked on typical cross-sections, for circular sliding surfaces through the very soft clay layer (sub soil layer).Both short term, undrained conditions, long term effective stress analysis and earthquake loading according to Greek Seismic Code is examined by Bishop's slices method and the use of numerical software like SLIDE.

4.5 Hydrostatic Uplift (Flotation)

Hydrostatic uplift of EPS embankement is a concern due to the weight of the EPS. Base drainage layer and side ditches protect lightweight fill from the incident of flooding and uplift failure due to buoyancy forces. Further to the protection measures, the case was checked by the following calculations, based on NRRL (Norwegian road research laboratory) and TRB (traffic research board, USA) recommendations:

Global stability: F.S. = $\{W_p + W_{sl} + W_{eps}\}/ A > 1{,}3$

Local slope uplift: F.S. = (H_{sl})/ [γ_w x (D+HWL)] > 1,2
W_P = pavement and slab weight,
W_{sl} = slope earth cover weight,
γ = soil unit weight,
W_{eps} = EPS fill weigth, considering unit weight and γ_{EPS} =0,2kN/m3,
A = buoyancy force = γ_w x (D+HWL) x B_w (B_w = bottom embankment width),
H_{sl} = minimum slope cover = 0,8m,
D = maximum depth of EPS below ground level,
HWL = maximum probable water level above GL, γ_w = water unit weigth = 10kN/m3

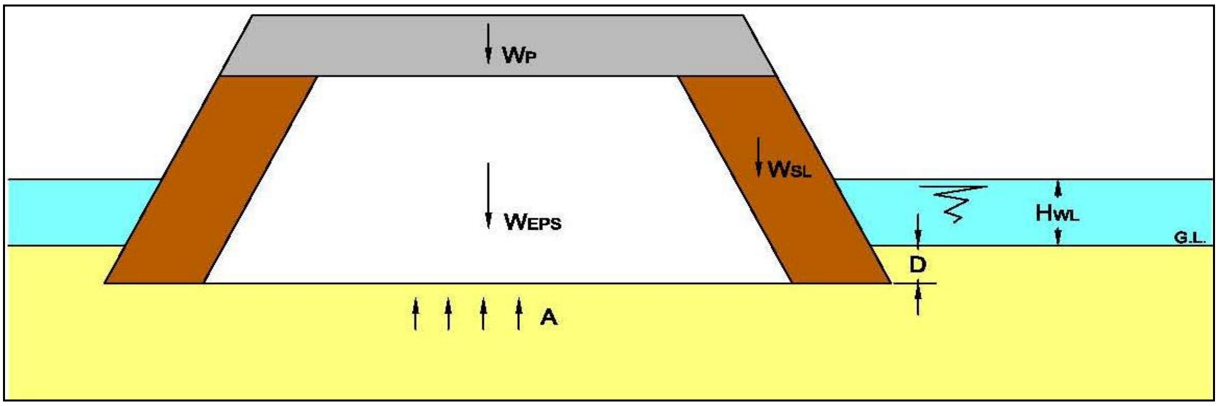


Figure 64: Schematics for uplift calculation

4.6 Translation and Overturning Due to Water

(Hydrostatic Sliding and Overturning)

Horizontal equilibrium equations can be formulated based on the above figure 64 and manipulate factor of safety based on that. Similarly, moment at the right corner of the embankment can be taken to calculate the same for overturning.

4.7 Translation and Overturning Due to Wind

Due to inconsistency in the methodology and lack of failure record, calculations for wind is not mentioned for the particular project mentioned at the beginning.

5. Internal stability evaluation

5.1 Introduction

Sine EPS blocks are an external element of construction, attention should be given to the internal stability of the embankment. In this aspect sliding between EPS blocks, load bearing capacity of the block is checked.

Sliding between two consecutive EPS layers has to be checked. The critical surface lies between the upper layers, where both minimum overburden weight and friction resistance and utmost seismic action occur. On the basis of TRB recommendations three loading cases were examined:

- static (due to inclined layers)
- wind force
- seismic (see section 6.2)

5.2 Translation Due to static (due to inclined layers)

This is to check the free falling of the EPS blocks (if layered at an inclination) sideways based on the weight on top of it.

$$F.S.s = Wp \times \tan(\delta)/Wt \geq 2{,}0$$

Where:

$W_{p,t}$ = overburden pressure (weight of pavement and concrete slab acting in direction perpendicular and parallel to sliding surface respectively,

δ = friction angle between EPS layers = 30^0,

5.3 Translation Due to Wind

If there is a consistent data for wind for the construction site, the effect can be analysed using:

$$\text{wind force: } F.S.w = Wp \times b \times \tan(\delta)/(Pxh+Wtxb) \geq 1{,}2$$

Where:

$W_{p,t}$ = overburden pressure (weight of pavement and concrete slab acting in direction perpendicular and parallel to sliding surface respectively,

δ = friction angle between EPS layers = 30^0,

P = wind force at fill crown, perpendicular to slope,

b, h = width and height of EPS layer,

5.4 Load Bearing

Inorder to avoid any undesired excessive deformation on the EPS block, the load on top of the EPS block should be limited according to the elastic limit stress.

For the specific highway project mentioned (Athens-Thessaloniki highway), the same principle as that of TRB is used. The load coming to the EPS block should be within the elastic stress limit of the block, σe. Refer table 7 of ch-2 to get σe for different EPS.

•$p < 30\% * \sigma_{c10} = 30kPa$ (Frydenlund, Aaboe, 2001)

•$\varepsilon i < 1\%$ immediate strain (NCHRP65, 2004)

Load bearing tolerance based on tolerable creep deformations

6. Seismic Analysis

6.1 Introduction

For this topic, Greece code for seismic resistence structures is used (ref.23).

6.1.1 General requirements – Adequacy of the foundation soil

The subsoil, the topography and the general geology of the area of civil engineering structures should ensure, with sufficient probability, that there will be no risk of soil rupture, slope instability or extended liquefaction during an earthquake vibration compatible with the intensity and spectral characteristics of the design earthquake provided in this Code.

6.1.2 Slope Stability

It is obligatory to check the general stability against sliding of the slope on which a structure will be founded, as well as slopes located uphill or downhill relative to the structure when their failure may affect the structure. The stability analysis may be performed according to the provisions in subtopic 6.2. The check shall be based on a suitable geotechnical survey, and on a geological survey if deemed necessary by the former.

6.1.3 Liquefaction Hazard

Even if they are considered to have sufficient resistance against liquefaction, the need for reduction of the design value of the effective angle of friction due to excessive pore pressure build-up during the cyclic design seismic action must be investigated (see annex F.5).

6.1.4 Shear settlement of the soil due to cyclic loading

Loose non-saturated sandy formations may be subject to a dynamic volume reduction (settlement), resulting in permanent settlement and deformations. Similar effects may occur in very soft and sensitive clays due to the gradual reduction of their shear resistance during cyclic loading of long duration. The possibility of occurrence of such phenomena must be investigated by means of well-established geotechnical methods by studies compiled on the basis of the results of in situ or laboratory tests. Soils of this type are characterized as seismically sensitive and their existence must be pointed out in the geotechnical design.

6.2 External Seismic Stability of Embankment

6.2.1 Slopes

Stability of natural or artificial slopes during a seismic action shall be verified using the following additional effective accelerations acting on the soil mass.

Horizontal: $\alpha_h = \alpha_\pi$

Vertical: $\alpha_V = \pm 0.50\ \alpha_\pi$

Where α_π is the design seismic acceleration of the slope, taken equal to 0.5a for natural slopes or equal to $(\alpha B + \alpha K)/2$ for the slopes of embankments according to 6.1.2.

In soil type Γ, seismicity zones III or IV and when the structure under design is of importance Σ3 or Σ4 or when the general stability of the area is affected, the estimation of the shear strength parameters must be based on suitable in situ and/or laboratory tests under cyclic loading. For clay soils the residual strength (after considerable deformation) shall be used (see table 17 for classification).

Table 18: soil class according to Greek seismic code

CLASS	DESCRIPTION
A	Rock or semi-rock formations extending in wide area and large depth provided that they are not strongly weathered. Layers of dense granular material with little percentage of silt-clay mixtures having thickness less than 70 m. Layers of stiff over consolidated clay with thickness less than 70 m.
B	Strongly weathered rocks or soils which can be considered as granular materials in terms of their mechanical properties. Layers of granular material of medium density with thickness larger than 5 m or of high density with thickness over 70 m. Layers of stiff over consolidated clay with thickness over 70 m.
Γ	Layers of granular material of low relative density with thickness over 5 m or of medium density with thickness over 70 m. Silt-clay soils of low strength with thickness over 5 m.
Δ	Soft clays of high plasticity index (I_p > 60) with total thickness over 12 m.
X	Loose fine-grained silt-sand soils under the water table which may liquefy(unless a specific study proves that such a hazard can be excluded or their mechanical characteristics will be improved). Soils which are close to apparent tectonic faults. Steep slopes covered with loose debris. Loose granular soils or soft silty-clayey soils which have been proved hazardous in terms of dynamic compaction or loss of strength. Recent loose backfills. Organic soils. Soils of class Γ with excessively steep inclination

Table 19: Importance factor

Importance Category		γ_I
Σ1	Buildings of small importance for public safety e.g. agricultural buildings, sheds, stables etc.	0.85
Σ2	Ordinary residential and office buildings, industrial buildings, hotels, etc.	1.00
Σ3	School buildings, public assembly buildings, airport terminals and generally buildings where a large number of people gather during the greater part of the day. Buildings that house installation of great economic value (e.g. buildings that house computer centres, special industries, etc).	1.15
Σ4	Buildings whose operation both during an earthquake and after the event is of vital importance, such as telecommunication buildings, power stations, hospitals, fire stations, public services buildings. Buildings that house works of unique artistic value (e.g. museums)	1.30

6.2.2 Embankments

Stability of embankments up to 15.0m height shall be checked using additional horizontal active accelerations on their mass which vary from $\alpha B = 0.5\alpha$ at the base up to $\alpha K = \alpha B\ \beta(T)$ at the top of the embankment,

where,

α is the normalized seismic ground acceleration, and

$\beta(T)$ is the spectral magnification that corresponds to the fundamental period of the structure.

In the absence of a more accurate analysis, $T = 2.5\ (H/Vs)$ may be used.

where,

Vs is the average velocity of shear wave in the embankment.

The design of embankments of height larger than 15m, embankments bearing important structures and dams, is not covered by the present Code. These cases require a special geotechnical and seismic design.

6.2.3 Stability Check

Stability shall be checked using the most unfavourable sliding surface and ensuring a safety factor γ at least equal to 1.0.

6.3 Internal Seismic Stability

Translation between EPS block layers due to seismic

seismic: $F.S.e = [W_p – E_h\sin(\alpha) - E_v\cos(\alpha)]\ b \times \tan(\delta) / (W_t + E_h\cos(\alpha) + E_v\cos(\alpha)) \geq 1.0$

Where:

$W_{p,t}$ = overburden pressure (weight of pavement and concrete slab acting in direction perpendicular and parallel to sliding surface respectively,

δ = friction angle between EPS layers = 30^{0},

α= inclination (cross fall) of EPS layers

P = wind force at fill crown, perpendicular to slope,

b, h = width and height of EPS layer,

$E_{v,h}$ = vertical and horizontal earthquake force according to Greek Seismic Code

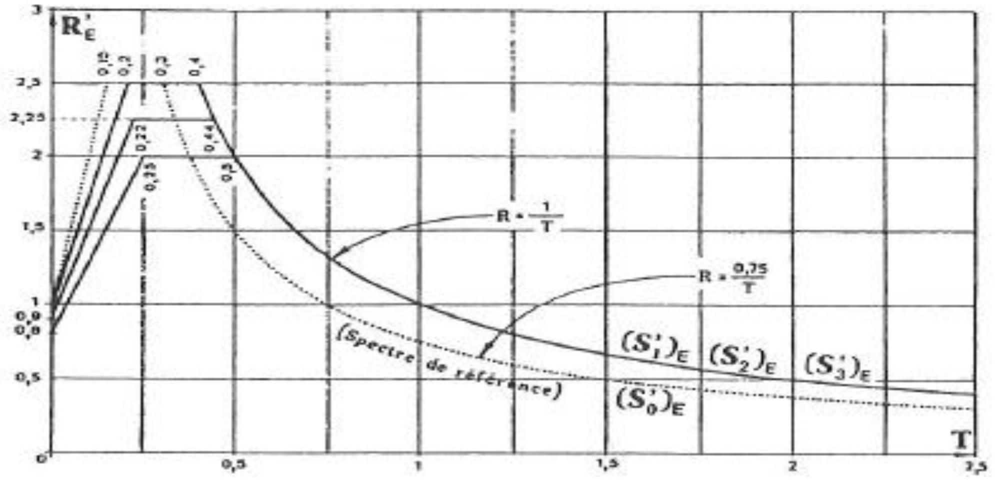


Figure 65: Vertical acceleration component (ref.13)

Chapter 6. Summary Properties of EPS material

6.1 Introduction

EPS has been used to solve settlement and bearing capacity problems for road construction in soft grounds for several years. Satisfactory results have been achieved in practice because of the unique material behavior that provide for us. Since its first use in road construction, good performance has been recorded also in reducing horizontal forces while using EPS as a fill material.

Several monitoring programmes, large field tests and laboratory tests have been done to establish some of the basic material behaviors' of EPS. In this chapter summary of parameters gathered from different literature are presented.

6.2 Unit density

EPS densities for practical civil applications range between 11 and 30 kg/m3 but because of the tendency of EPS to absorb water, the unit weight can vary depending on the amount of water absorbed in the EPS. For design purposes, we have to identify the situation whether the EPS is expected to be in drained or submerged condition.

A) *Drained condition*

Under drained condition EPS can absorb approximately of 1% by volume of water. We can use density value of less than 30Kg/m3 for such cases.

B) *Undrained condition*

An average value of 4% water absorption is obtained in a test undergone in Canada (ref. 6). But some tests done on old used EPS blocks in Norway suggests up to a maximum of 10% water absorption (ref.7). For this kind of undrained fill situation we can use 90 -95 kg/m3.

6.3 Compressive strength

6.3.1 Stress strain curves

Figure 66 shows the uniaxial compression stress strain curve of EPS geofoam for two different densities. The two densities shown are considered the extreme values for most engineering applications done so far. Specimens are 0.05m cubes tested at a displacement rate of 0.005m/min. From the figure the stress strain curve can be simply divided into two main straight lines connected with a curved portion. The slope of the straight-line portion increases with density.The stress at any strain level increases also with the density.

There is no defined shear rupture for EPS geofoam under compression. As will be shown later in chapter six, more than 70 % strains are reached without any break point and the tests were stopped because the maximum travel of the machine head was reached. The 1%, the 5%, and the 10% strains are common reference strain level, at which the stress is considered as the strength of the material (ref.8).

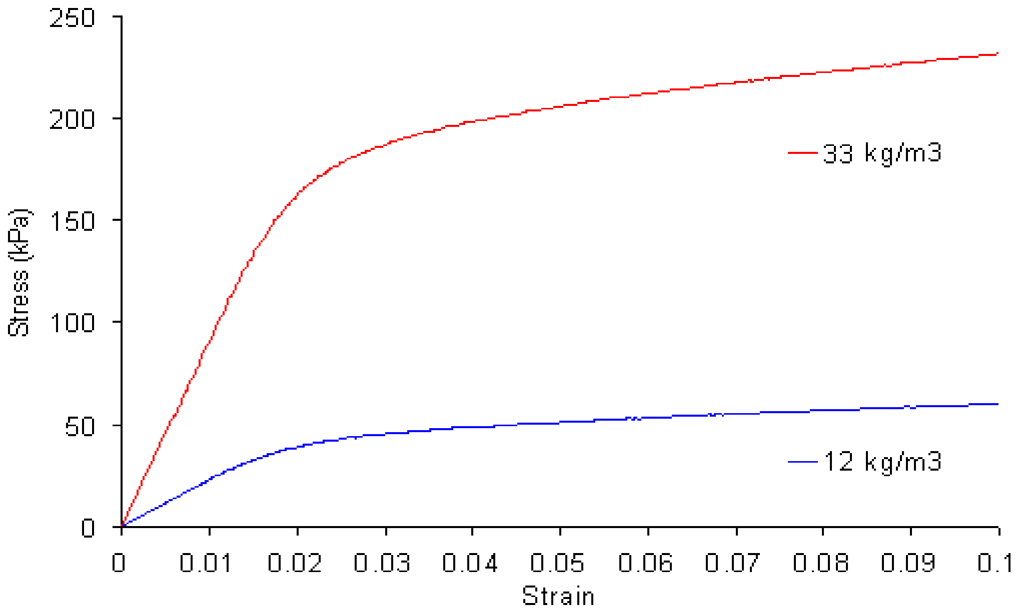


Figure 66: EPS Uniaxial Compression Stress Strain Curves (after Ne-gussey and Elragi, 2000b)

Table 20: EPS Types in United Kingdom (after Sanders, 1996)

Density (kg/m^3)	12	15	18	22	29
Compressive Strength at 10% Strain (kPa)	35	69	90	104	173

6.3.2 Initial Elastic Modulus

The initial tangent modulus or Young's modulus of EPS varies, some say it is linearly varying and some suggest non-linearity.

6.3.3 Poisson's ratio

A value ranging from 0.05 to 0.5 registered from different tests and recent test have been showing a negative poison's value as well.

6.3.4 Cyclic Loading

EPS geofoam may experience cyclic loading in a number of situations. This can include traffic loading and dynamic loading. The majority of laboratory testing and field observations suggest that the cyclic load behavior of block molded EPS geofoam is linear elastic provided that the strains are no greater than approximately 1%.

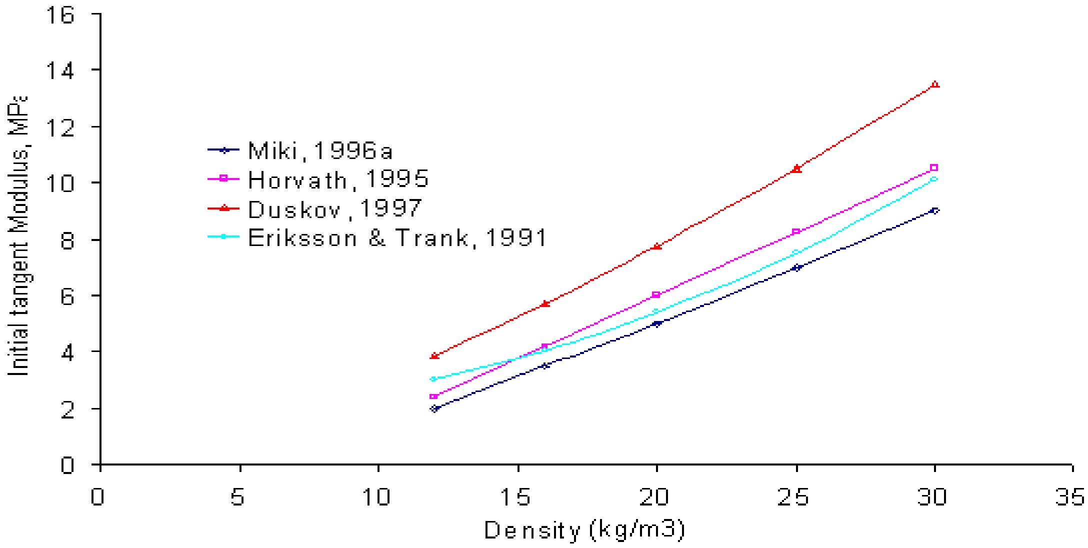


Figure 67: Initial Young's modulus of EPS geofoam (Ref.8)

Table 21: Values of poisons ratio

Reference	Yamanaka, et al. (1991)	Negussey and Sun (1996)	GeoTech (1999a)	Duskov et al. (1998)	Ooe, et al. (1996)	Sanders (1996)	Momoi and Kokusyo (1996)
Poisson's Ratio	.075	.09 and 0.33	0.05	0.1	0.08	.05 up to 0.2	0.5

For three loading cycle tests, the initial tangent modulus in the second and third cycles is much less than those for the first cycle, when the three cycles are loaded to 10% strain (Eriksson and Trank, 1991). Flaate (1987) reported that cyclic load tests show that EPS geofoam will stand up to an unlimited number of load cycles provided the repetitive loads are kept below 80% of the compressive strength (ref.8).

6.4 Tension

Tensile strength of EPS material can be an indication of the quality of fusion of the prepuffs and any recycled EPS geofoam used in the process (Horvath, 1995b). From figure 68 it can be seen that the tension strength increases with the density (ref.8).

6.5 Flexural strength

Typical flexural strength of EPS is presented down here.

Table 22: ASTM C 578-95 EPS Flexural Strength

Density (kg/m^3)	12	15	18	22	29
Minimum Flexural Strength (kPa)	70	173	208	276	345

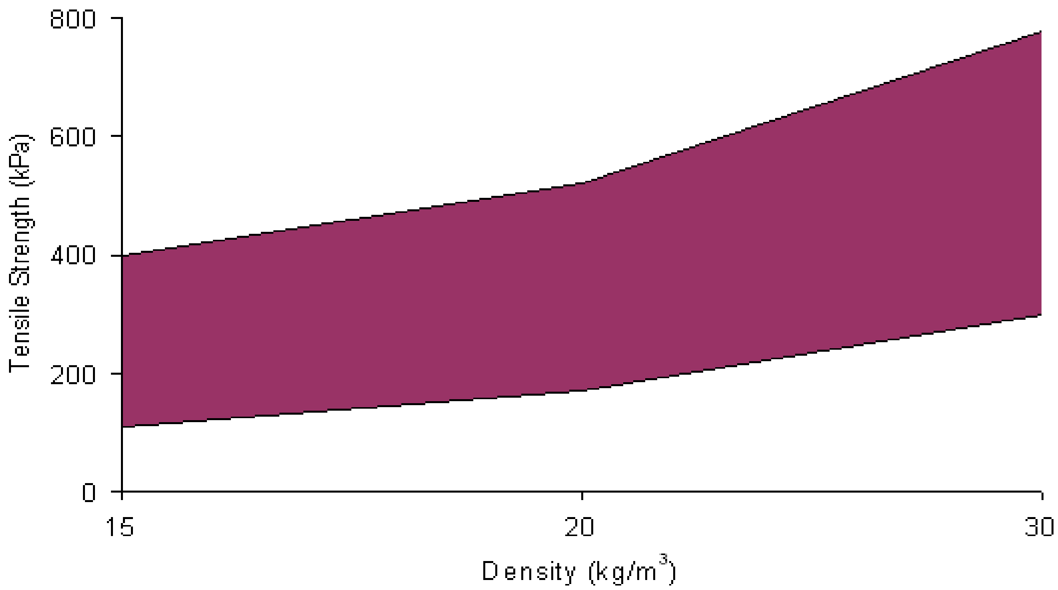


Figure 68: EPS geofoam tensile strength (after BASF, Corp., 1997)

6.6 Settlement

6.6.1 Deformation

A small strain is achieved when EPS is loaded within its compressive strength. Field monitoring programme at Interstate 15 (I-15) , a part of the Eisenhower National Highway System linking Montana, Idaho, Utah, Arizona, Nevada and California. Reconstruction project showed around 14% strain during construction (ref.9).

6.6.2 Creep

EPS geofoam is susceptible to time dependent creep deformation when a constant stress level is applied. A number of parameters affect the creep behavior of EPS geofoam, among which is the density. Creep deformations decrease with density increase (Sun, 1997). Figure 70 represents the results of three 0.05m cube specimens each are sub-jected to an unconfined axial stress for a period of over 500 days. The stresses are 30%, 50% and 70% of the strength of the material. The three specimens are of type VIII and minimum density of 18kg/m3. It can be seen from the figure that the creep behavior is stress level dependent. For the lower stresses, very little creep deformation occurred after 500 days.

Both full scale and laboratory creep tests have been performed at Norwegian road research laboratory (Aabøe, 2000) A test was done with 2m height of geofoam loaded to 52.5% of its compressive strength. Results observed in a three year period show continuous deformation with time. The strain after the three years was about 1% and slightly increasing with time. The full-scale test was for an EPS bridge abutment at Løkkeberg. Stresses in the geofoam abutment ranged between 25 and 60% of EPS strength at 5% strain. Observed deformation after 10 years in operation shows negligible creep.

Apart from this there are two time dependent creep empirical formulas developed: the general power- law equation and Findley equation.

The general power rule equation: $\varepsilon = \varepsilon_o + \varepsilon_c$

Where: ε_o – immediate deformation

ε_c – time dependent strain

ε – total strain

$$\varepsilon = \left(\frac{\sigma}{E_{ti}}\right) + 0.00209\left(\frac{\sigma}{\sigma_p}\right)^{2.47}\left(t^{\left\{-0.9\log_{10}\left[1-\left(\frac{\sigma}{\sigma_p}\right)\right]\right\}}\right)$$

$$\sigma_p = 6.41\rho - 35.2 \qquad E_{ti} = 450\,\rho - 3000.$$
$$E_{ti} = 479\rho - 2875$$

Where:

σ and σ_p are applied and plastic stress in KPa

E_{ti} is the initial modulus at 1%

ρ is EPS density

Findley equation:

$$\varepsilon = 1.1\sinh\left(\frac{\sigma}{54.2}\right) + 0.0305\sinh\left(\frac{\sigma}{33.0}\right)(t)^{0.20}$$

Where: σ is applied stress and t is time in hrs.

6.7 Stress distribution

A general value for horizontal stress reduction could be of in the range of 10 – 30% while vertically it falls within the range of 1:2(ref.7).

6.8 Durability

No deficiency effects are to be expected from EPS fills placed in the ground for a normal life cycle of 100 years, Aabøe (2000). Aabøe added that this should hold true provided possible buoyancy forces resulting from fluctuating water levels are properly accounted for, the blocks are properly protected from accidental spills of dissolving agents and the applied stress level from dead loads is kept below 30-50% of the material strength (ref.8).

6.9 EU design parameters for EPS

The product standard for EPS in Civil Engineering Applications (EN 14933) came into force in March 2009. After years of application EU came up with naming EPS by their strength or grade. See table 22 for summary of design parameters for EPS under EU.

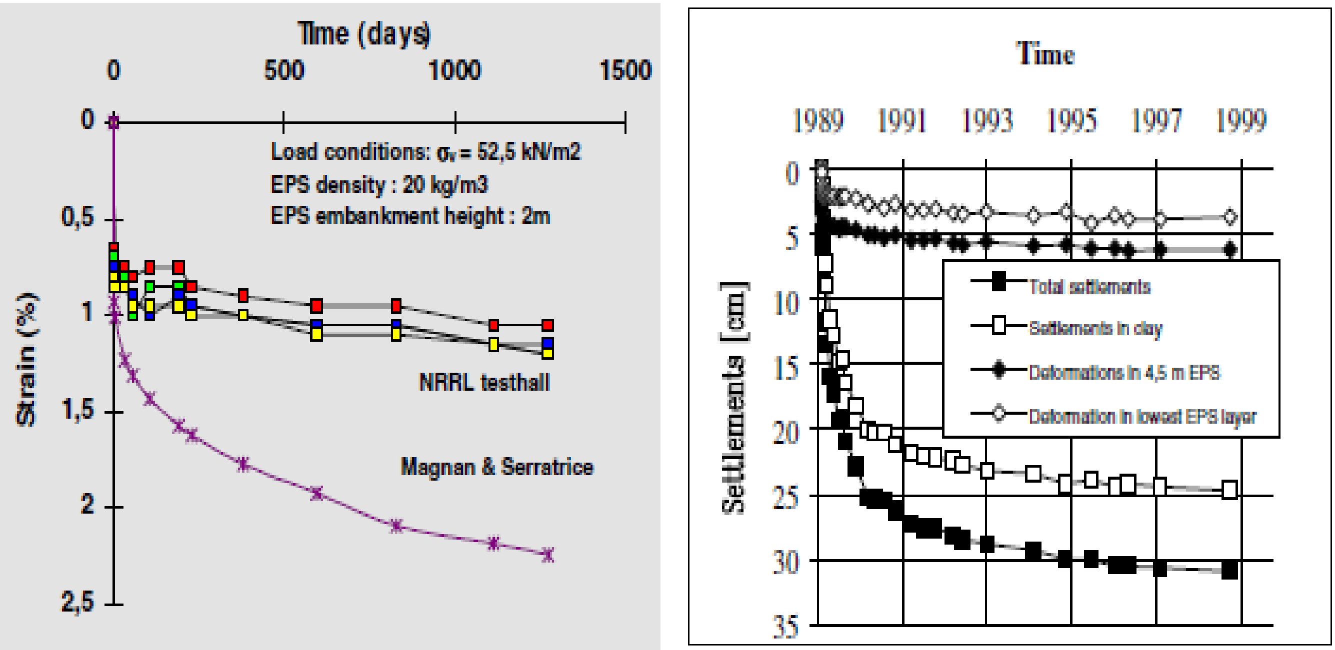


Figure 69: Creep deformation at Løkkerberg and at the test fill in NRRL(Ref.7)

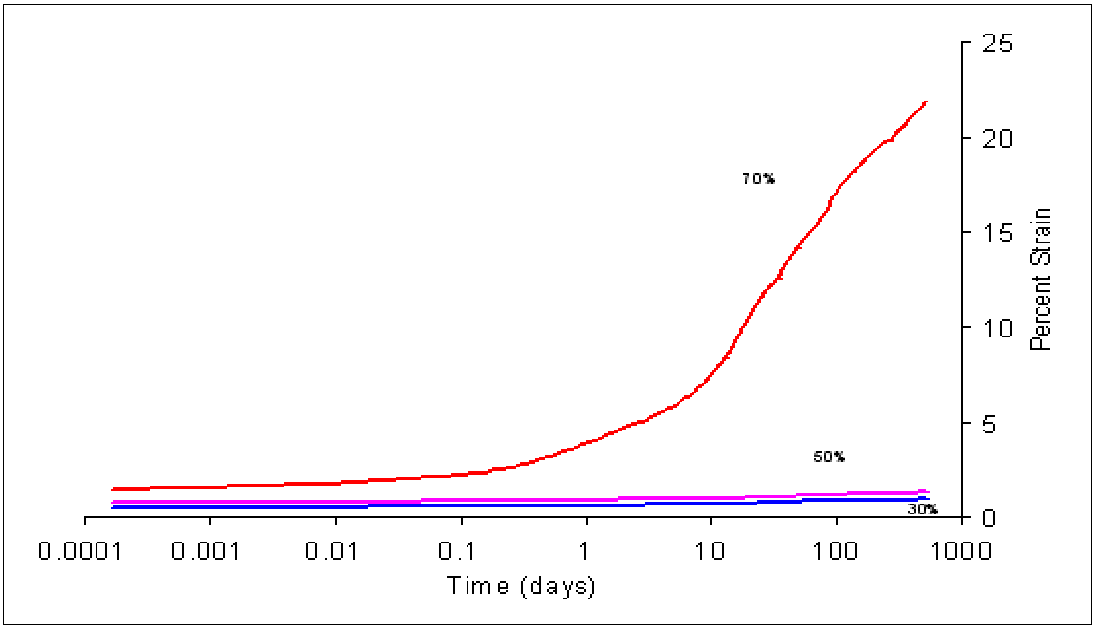


Figure 70 : EPS Creep Behavior for Different Stress Levels (after Sheeley, 2000)

Table 23: EU design parameters for EPS (ref.10)

Property			**EPS product type**				
Description	Symbol	Unit	EPS60	EPS100	EPS150	EPS200	EPS250
Declared value short-term compressive strength	σ_{10}	kPa	60	100	150	200	250
Design value short-term compressive strength	$\sigma_{10;d}$	kPa	48	80	120	160	200
Modulus of elasticity	E_t; E_{dyn}	kPa	4000	6000	8000	10000	12000
Declared value permanent compressive strength	$\sigma_{10;perm}$	kPa	18	30	45	60	75
Design value permanent compressive strength	$\sigma_{10;perm;d}$	kPa	14,4	24	36	48	60
Declared value compressive strength under cyclic load	$\sigma_{10;cycl}$	kPa	21	35	52,5	70	87,5
Design value compressive strength under cyclic load	$\sigma_{10;cycl;d}$	kPa	17	28	42	56	70

Chapter 7. Recommendation

1. Numerical Modelling

1.1 Introduction

There are two main categories of numerical methods: Continuous modelling, where the mass is treated as a continuous medium and only a limited number of discontinuities may be included, and discontinuous modelling, where the mass is modelled as a system of individual blocks interacting along their boundaries.

For EPS fill embankment, the tradition has been to use PLAXIS software, which is one of the continuous modeling techniques, as a tool for finite element analysis. But from consideration of the embankment, we might take experience from rock mass modeling in which each EPS blocks in the fill mass can be modelled in a similarly manner as done for rock blocks in the rock mass.

PLAXIS and Discontinuous Deformation Analysis (DDA)/ Distinct Element Model (DEM) can be used as representative tools for continuous and discontinuous modelling, respectively. DDA is both a theory and a computer program. The modelling procedure is similar to the distinct element modelling, while it more closely parallels the finite element method with respect to: i) Minimizing the total potential energy to establish equilibrium equations, ii) Choosing displacements as unknowns of the simultaneous equations and iii) Adding stiffness, mass and loading submatrices to the coefficient matrix of the simultaneous equation (ref.24).

The disadvantage of discontinuous model is finding out the input data is very difficult and challenging compared to the continuous models.

1.2 Proposal

There exists field insitu and laboratory test datas from large scale and small scale EPS fil embankments performed by Norwegian road research laboratory (NRRL). We can try to fit the models in the above mentioned finite element models and back calculate some parameters which can be used for the next discontinuous modelling. From outputs, we can compare to existing creep empirical formulas suggested by Findley and the general power rule.

2. Seismic Analysis

2.1 Introduction

The basic practice of handling seismic action to an embankment fill is to apply a horizontal force along the center of mass of the fill. As expresses in the Chapter 3, we can achieve representative values of horizontal acceleration components which can be used to calculate the equivalent horizontal force for the earthquake loading.

The main concern of such horizontal loading can be, the differential horizontal movement that can be generated by the force between the EPS fill and the pavement structure. Such forces might consequently cause excessive rutting or settlement and finally failure of pavement structure.

2.2 Proposal

For a typical cross-section of a road (we can take 2 lane, 3 lane or four lane road cross-section), we can model them for earthquake loads by applying equivalent horizontal forces.
Challenge – How to model the EPS – pavement interaction?
Conclusion – for a T years return earthquake, what will be the response of the fill with respect to the pavement deformation and whether the values are acceptable or not.

3. Field or laboratory test

The test can be performed related with finding the load distribution of EPS in embankment.
The test can be supported by numerical modeling. The aim of the test can be to suggest usage of different type EPS in relation with depth for different amounts of fill depth as well as width (related with number of lane).

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Appendices

Appendix 1: *Flow chart of design procedure for an EPS-block geofoam roadway embankment,USA*

Appendix 2 *: Static external slope stability design chart for trapezoidal embankments with a two-lane roadway with a total road width of 11 m, 23m and 34m.*

Appendix 3 *: Static external slope stability design chart for vertical embankments with a two-lane roadway with a total road width of 11 m, 23m and 34m.*

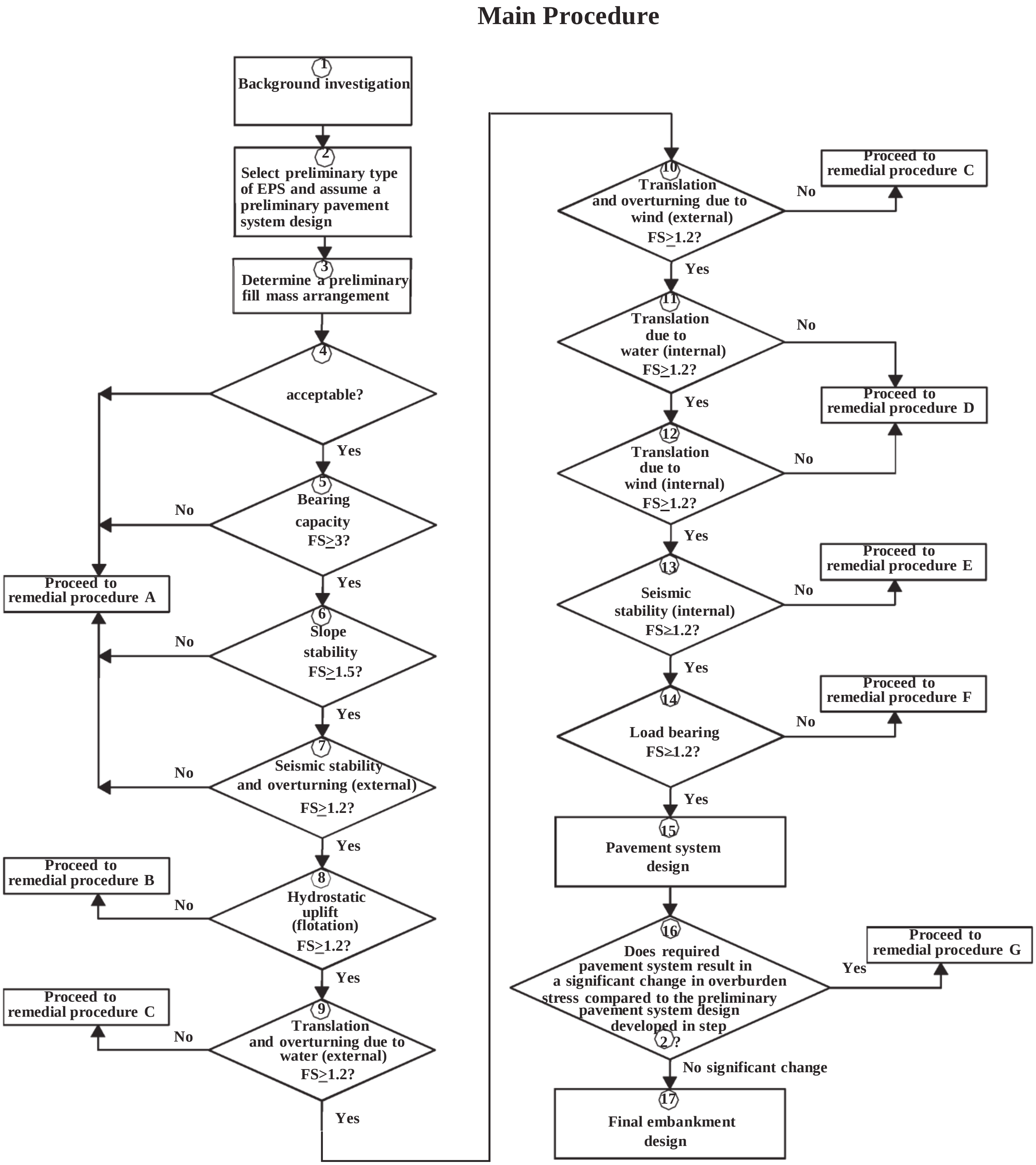
Appendix 4: *Hydrostatic uplift design chart*

Appendix 5: *Hydrostatic Sliding design chart*

Appendix 6: *Seismic external slope stability design chart for trapezoidal embankment*

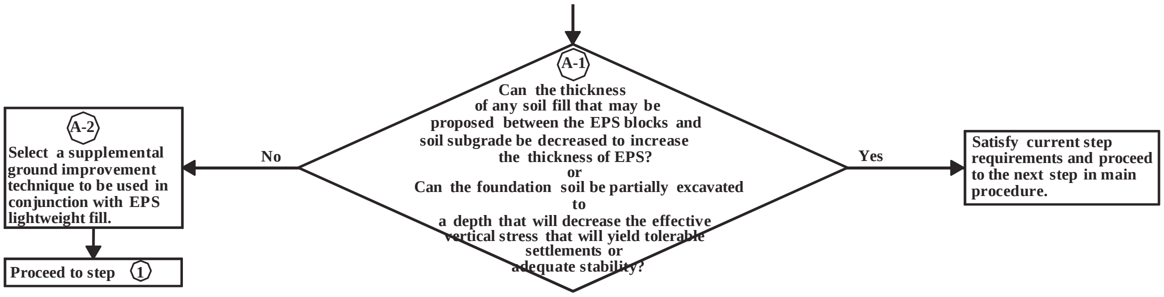
Appendix 7: *Seismic external slope stability design chart for vertical embankment*

Appendix 1: *Flow chart of design procedure for an EPS-block geofoam roadway embankment,USA*



FS = Factor of Safety.

Start of Remedial Procedure A



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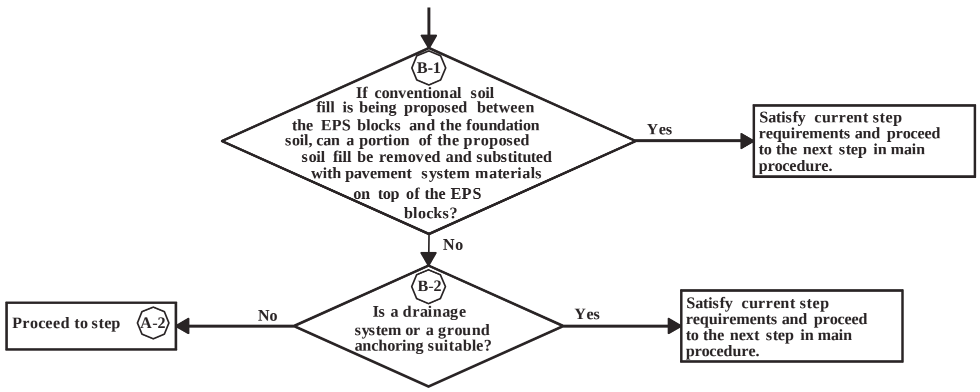
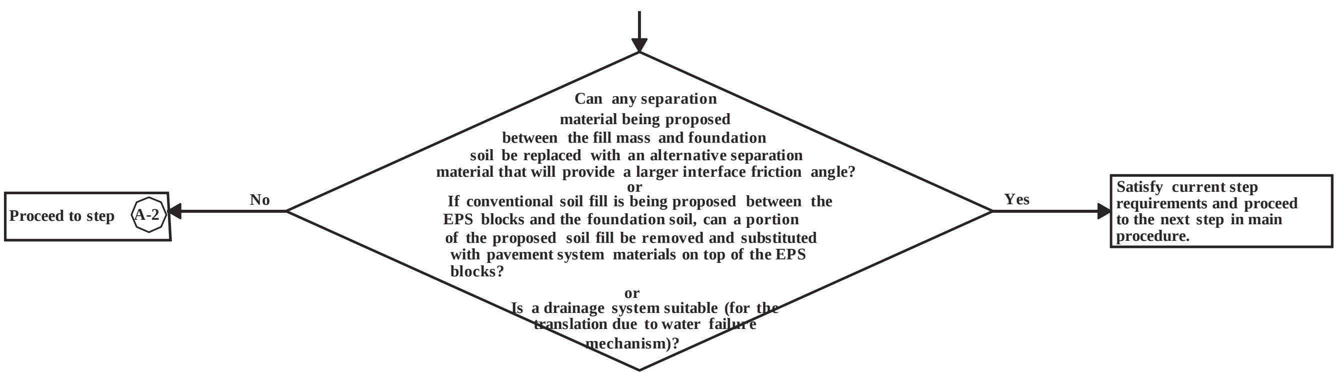
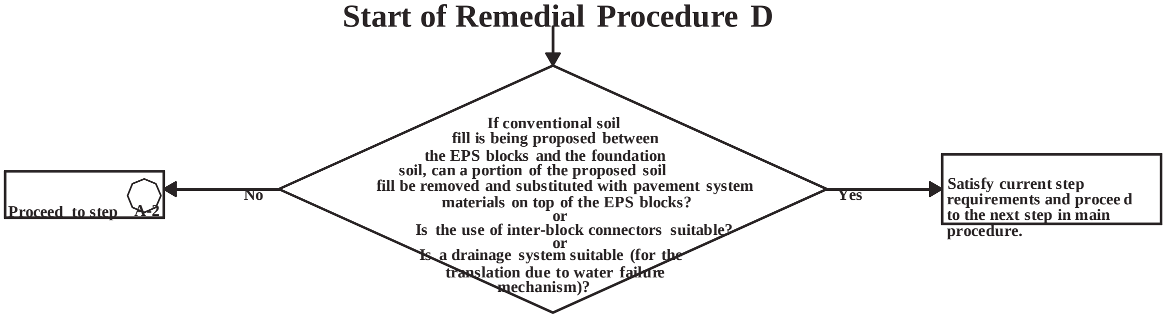


Start of Remedial Procedure C



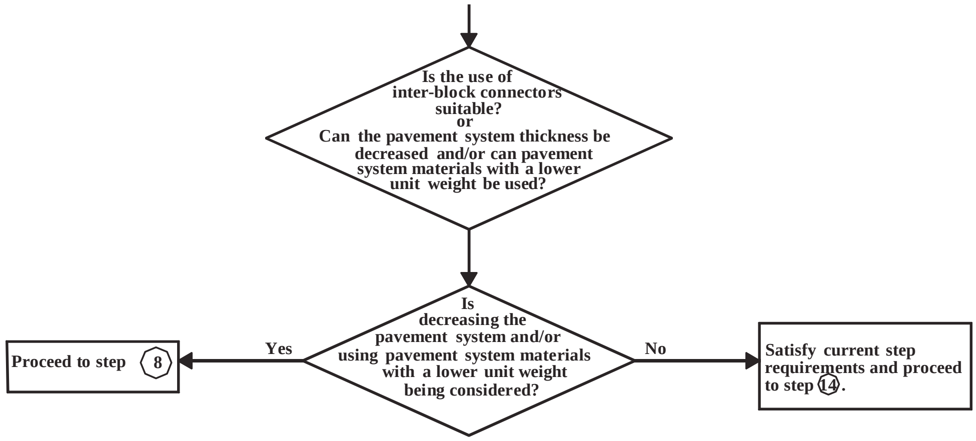
Start of Remedial Procedure D



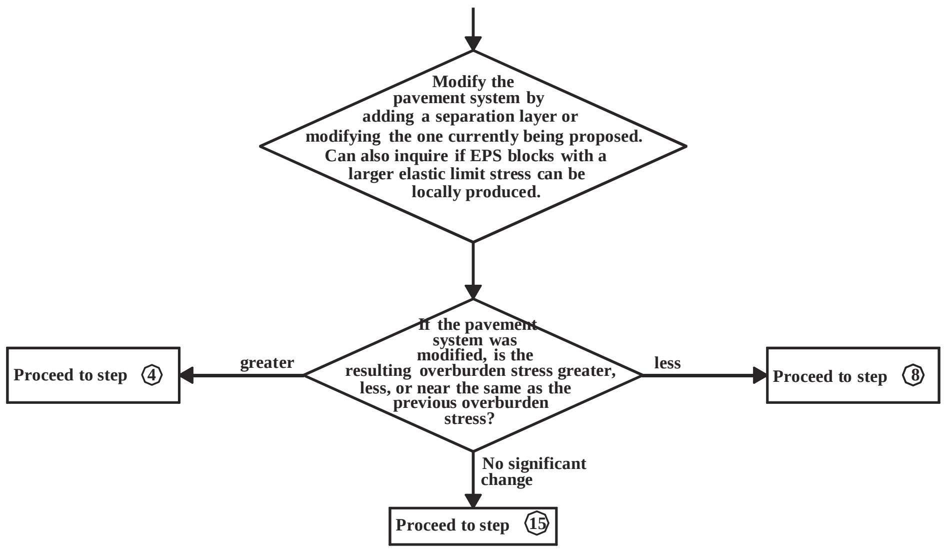
Note: These remedial procedures are not applicable to overturning of a vertical embankment about the toe of the embankment at the embankment and foundation soil interface. If the factor of safety against overturning of a vertical embankment is less than 1.2, consideration can be given to adjusting the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

(*Continued*)

Start of Remedial Procedure E



Start of Remedial Procedure F



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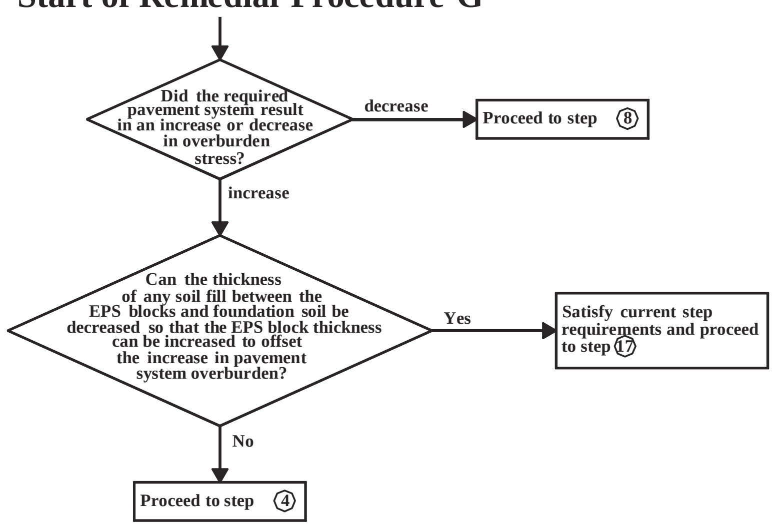

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(*Continued*)

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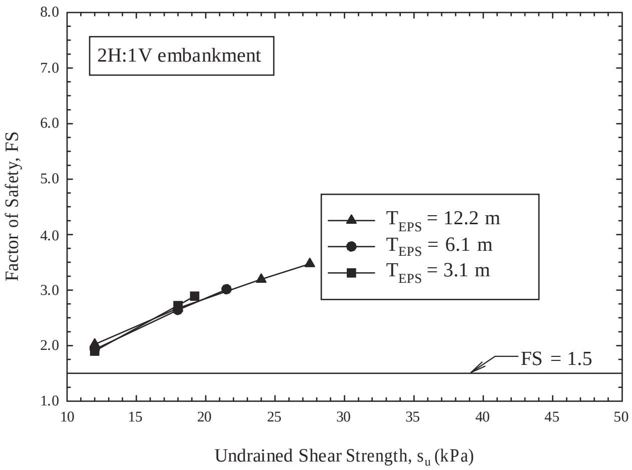
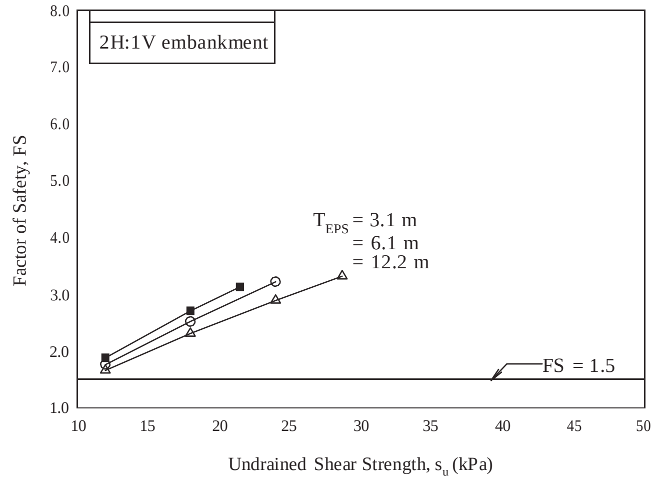
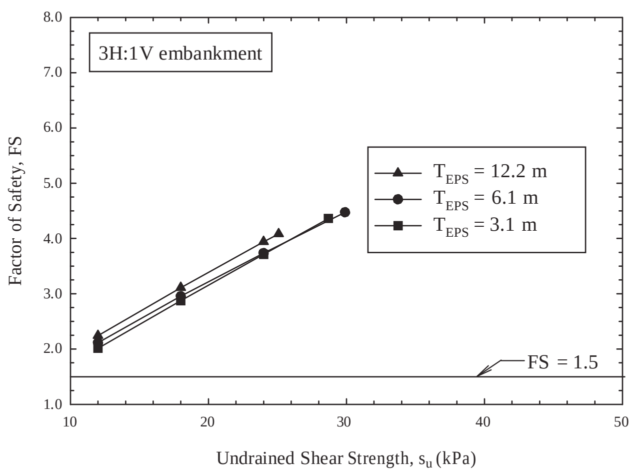
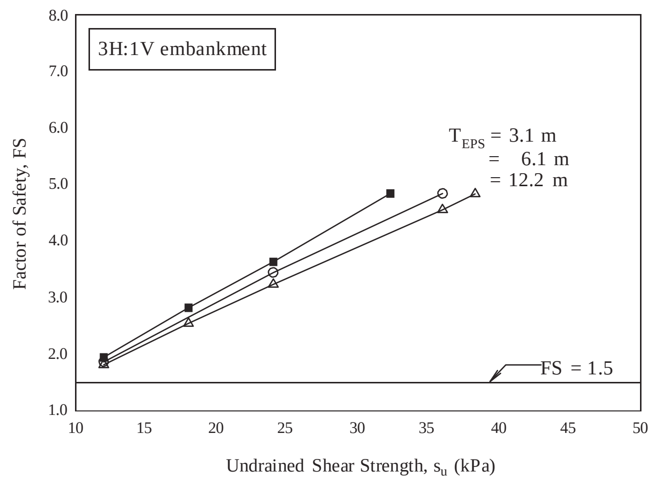
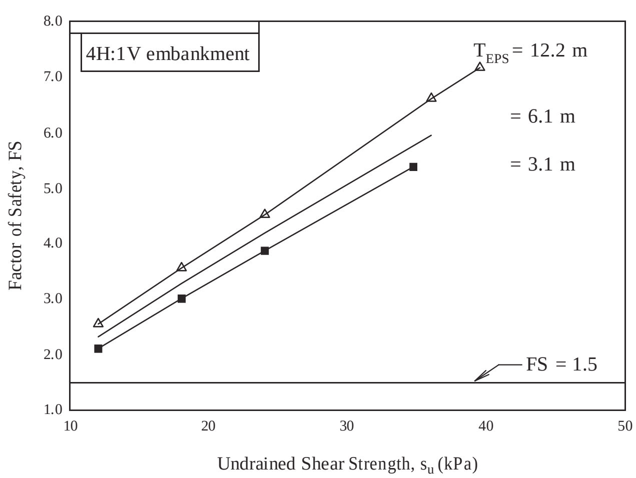
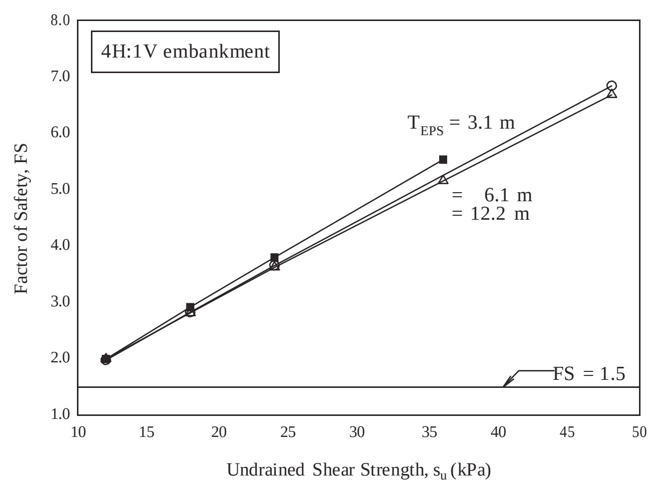


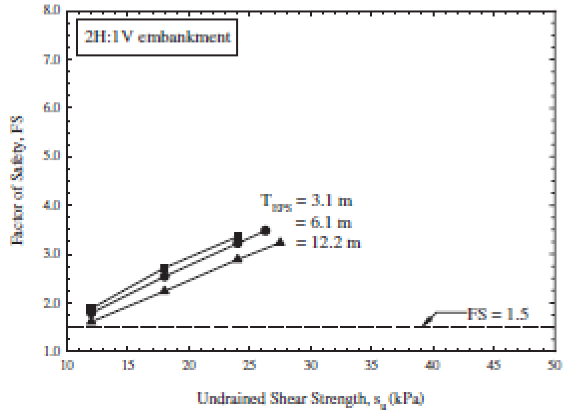


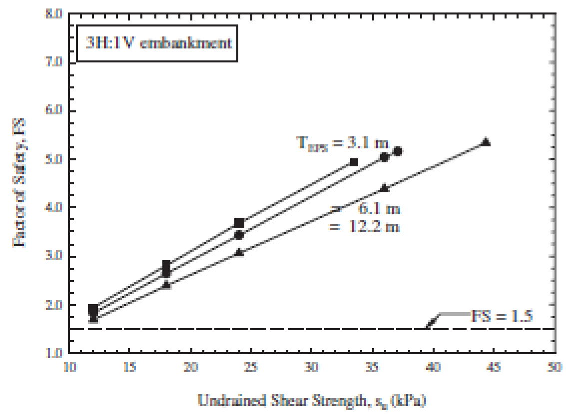


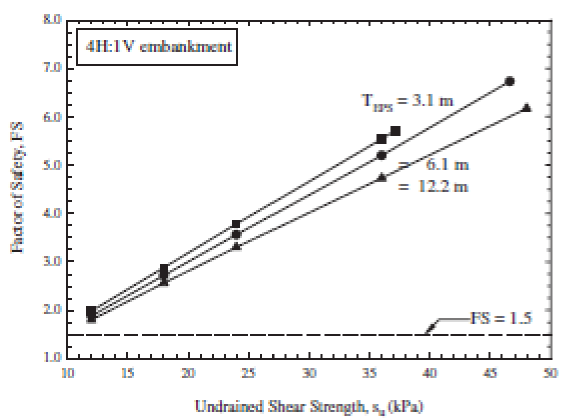




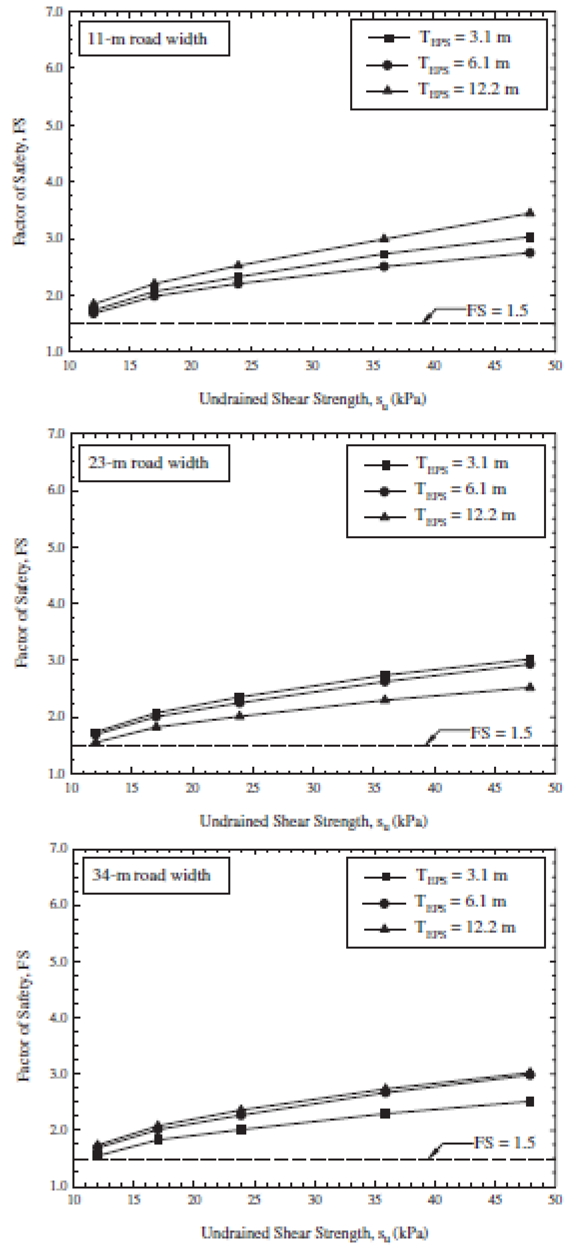



2H:1V embankment
T_{EPS} = 3.1 m
= 6.1 m
= 12.2 m
FS = 1.5
Factor of Safety, FS
Undrained Shear Strength, s_u (kPa)

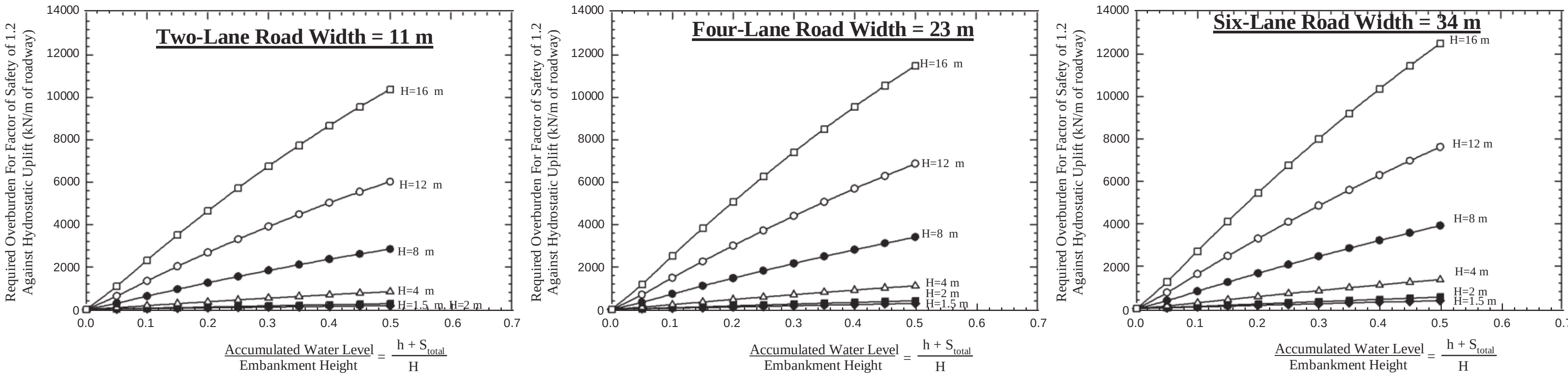

3H:1V embankment
T_{EPS} = 3.1 m
= 6.1 m
= 12.2 m
FS = 1.5
Factor of Safety, FS
Undrained Shear Strength, s_u (kPa)


4H:1V embankment
T_{EPS} = 3.1 m
= 6.1 m
= 12.2 m
FS = 1.5
Factor of Safety, FS
Undrained Shear Strength, s_u (kPa)

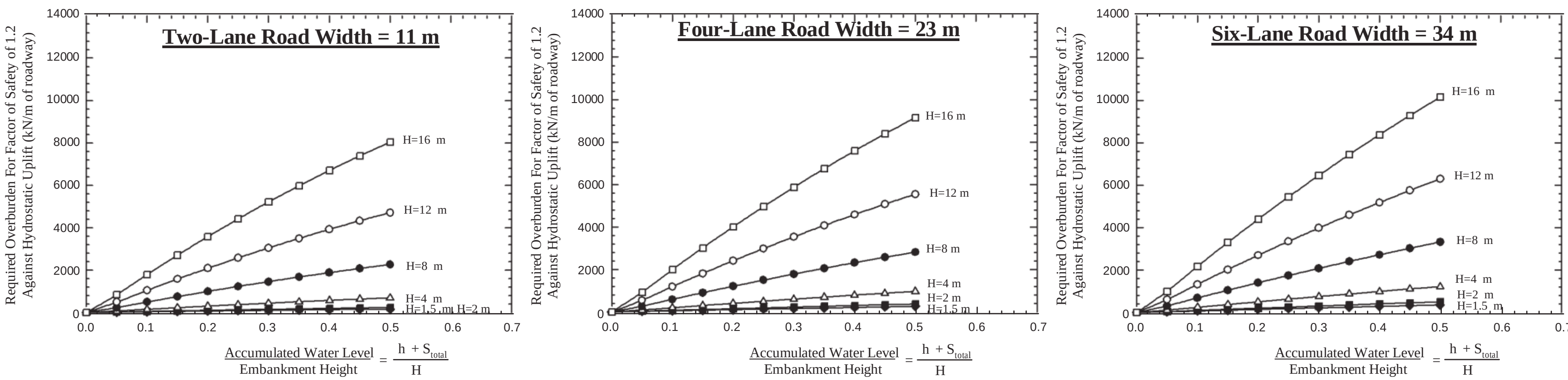
Appendix 3 *: Static external slope stability design chart for vertical embankments with a two-lane roadway with a total road width of 11 m, 23m and 34m.*



Appendix 4: *Hydrostatic uplift design chart*



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.



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.

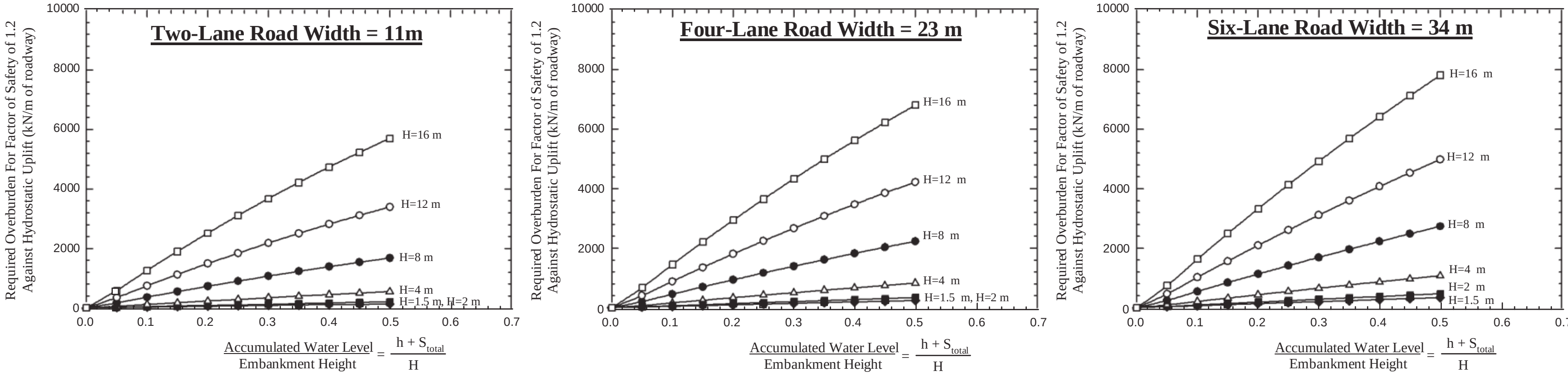
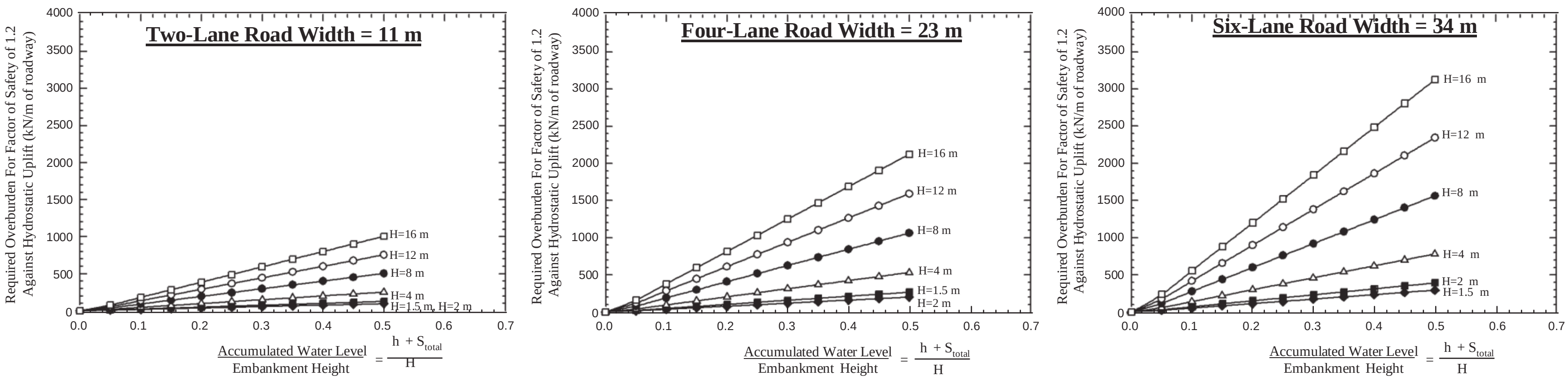
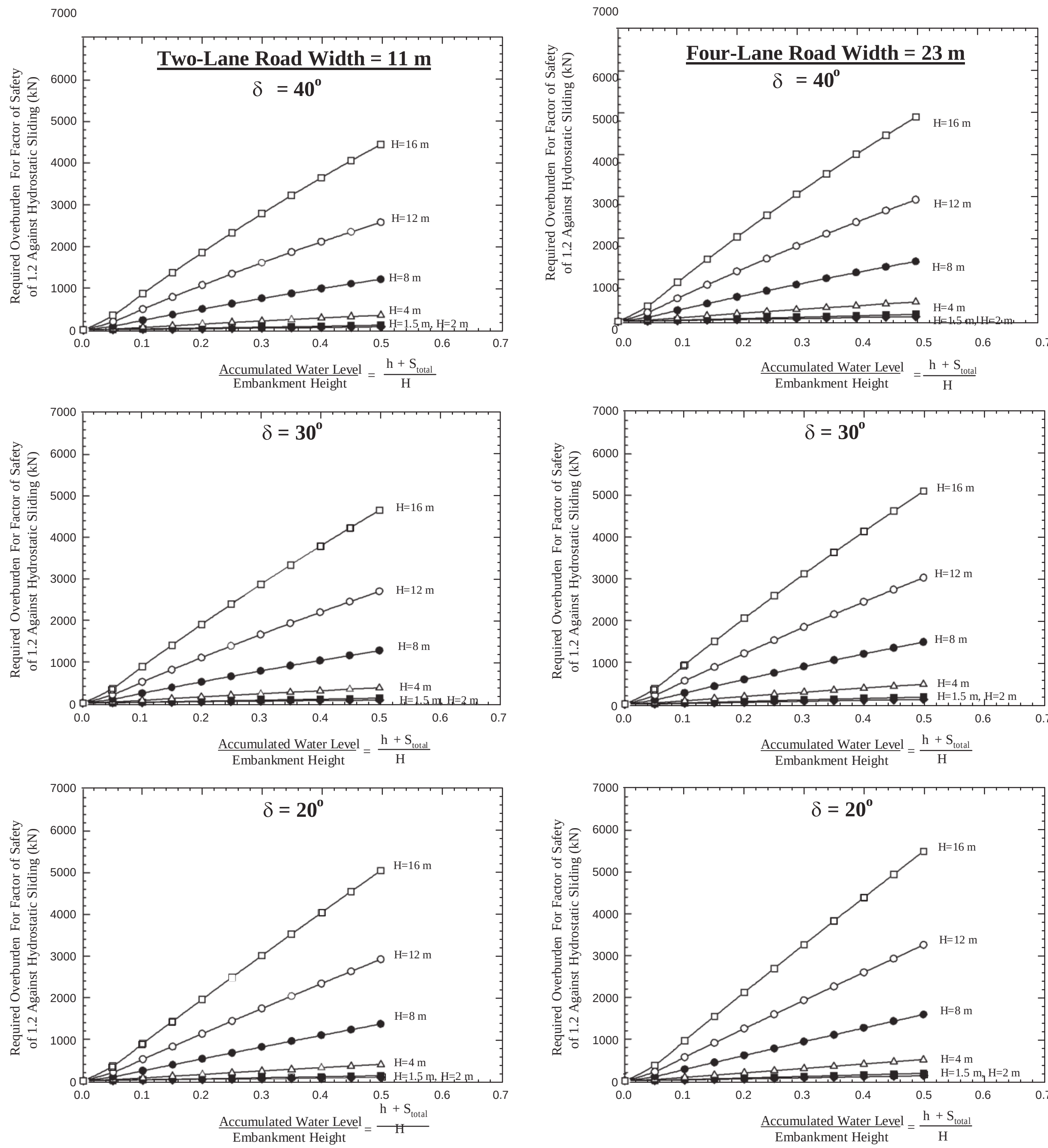


Figure 24. Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with tailwater level equal to upstream water level, 2H:1V embankment slope, and three road widths.

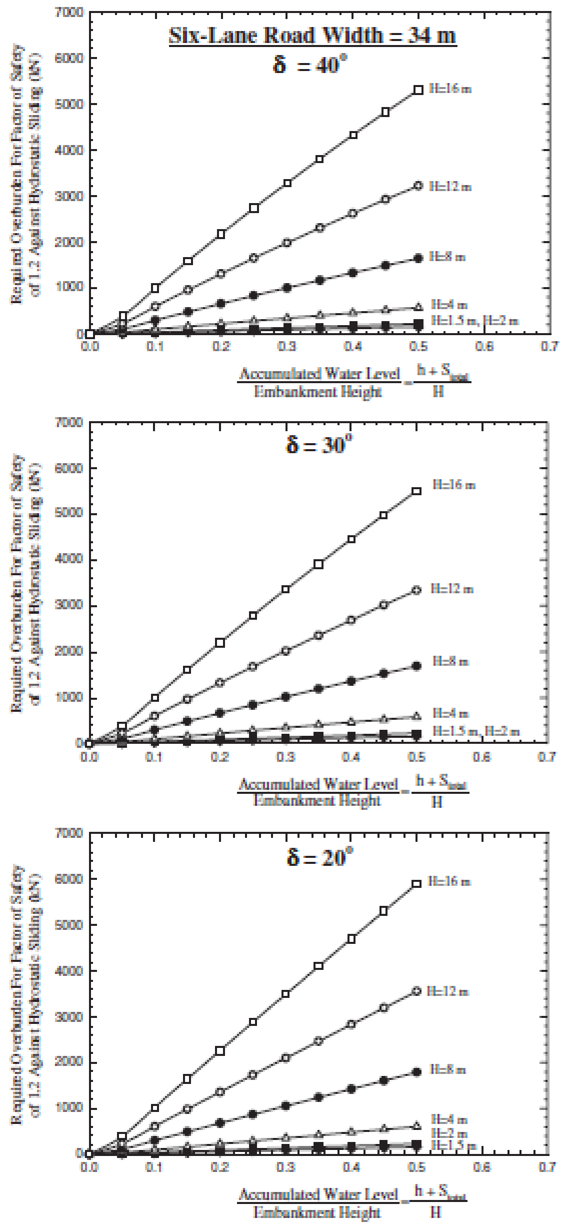


Hydrostatic uplift (flotation) design for a factor of safety of 1.2 with tailwater level equal to upstream water level, vertical embankment (0H:1V), and three road widths.

Appendix 5: *Hydrostatic Sliding design chart*

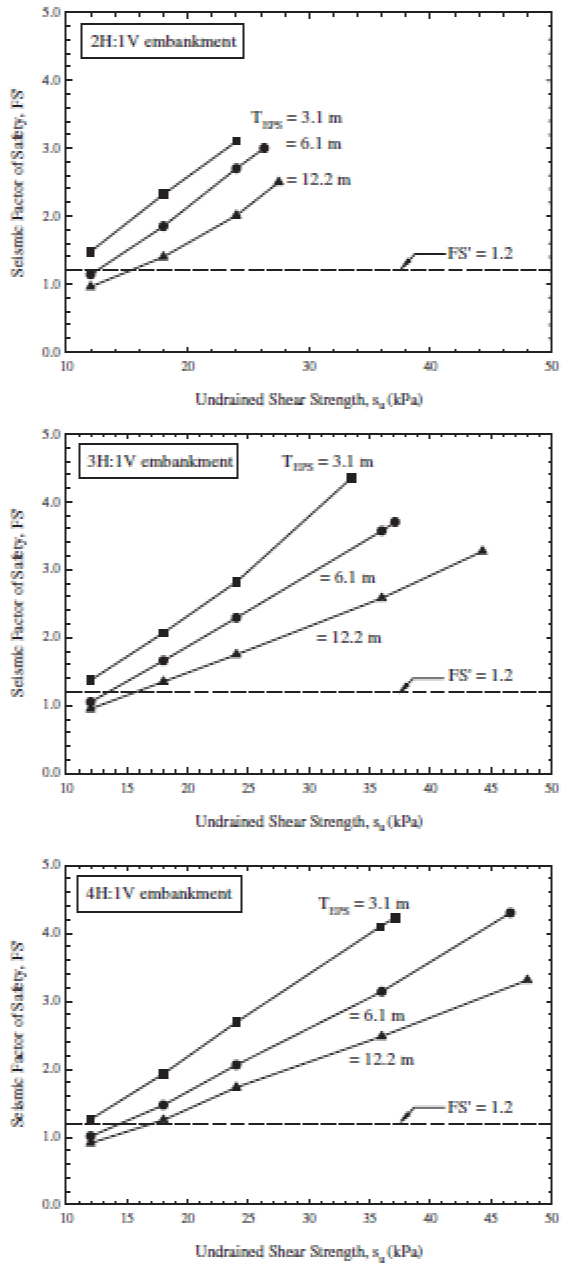


Hydrostatic sliding (translation due to water) design for a factor of safety of 1.2 with no tailwater, 4H:1V embankment slope, and three road widths for various interface friction angles.

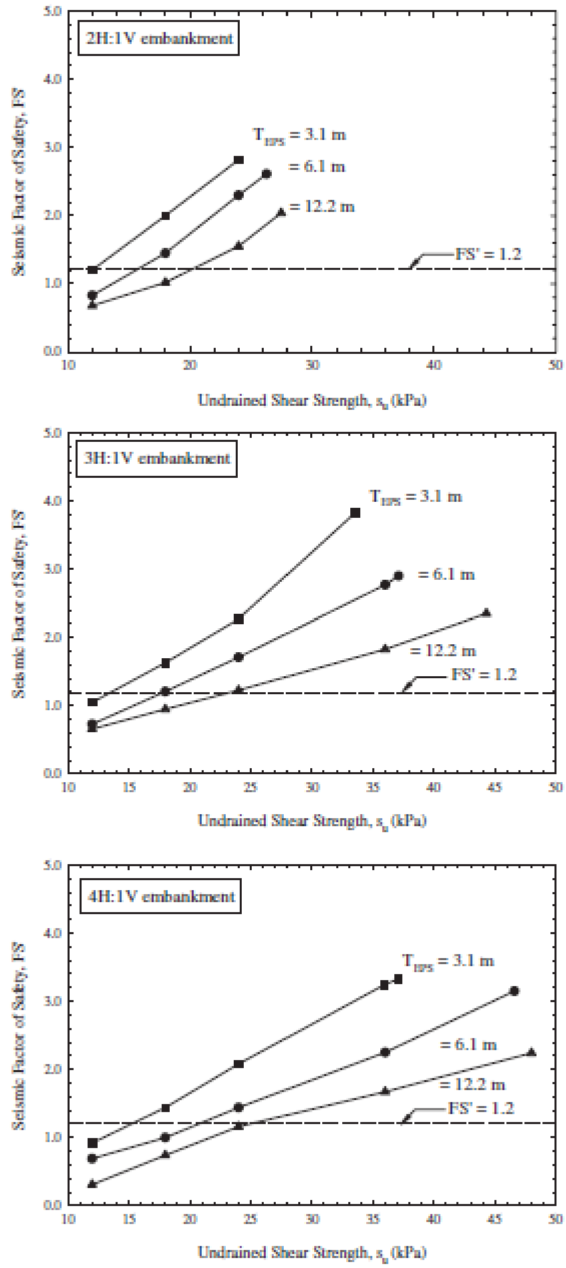


Six-Lane Road Width = 34 m
δ = 40°
H=16 m
H=12 m
H=8 m
H=4 m
H=1.5 m, H=2 m
Required Overburden For Factor of Safety of 1.2 Against Hydrostatic Sliding (kN)
Accumulated Water Level / Embankment Height = (h + S_total) / H
δ = 30°
H=16 m
H=12 m
H=8 m
H=4 m
H=1.5 m, H=2 m
Required Overburden For Factor of Safety of 1.2 Against Hydrostatic Sliding (kN)
Accumulated Water Level / Embankment Height = (h + S_total) / H
δ = 20°
H=16 m
H=12 m
H=8 m
H=4 m
H=2 m
H=1.5 m
Required Overburden For Factor of Safety of 1.2 Against Hydrostatic Sliding (kN)
Accumulated Water Level / Embankment Height = (h + S_total) / H

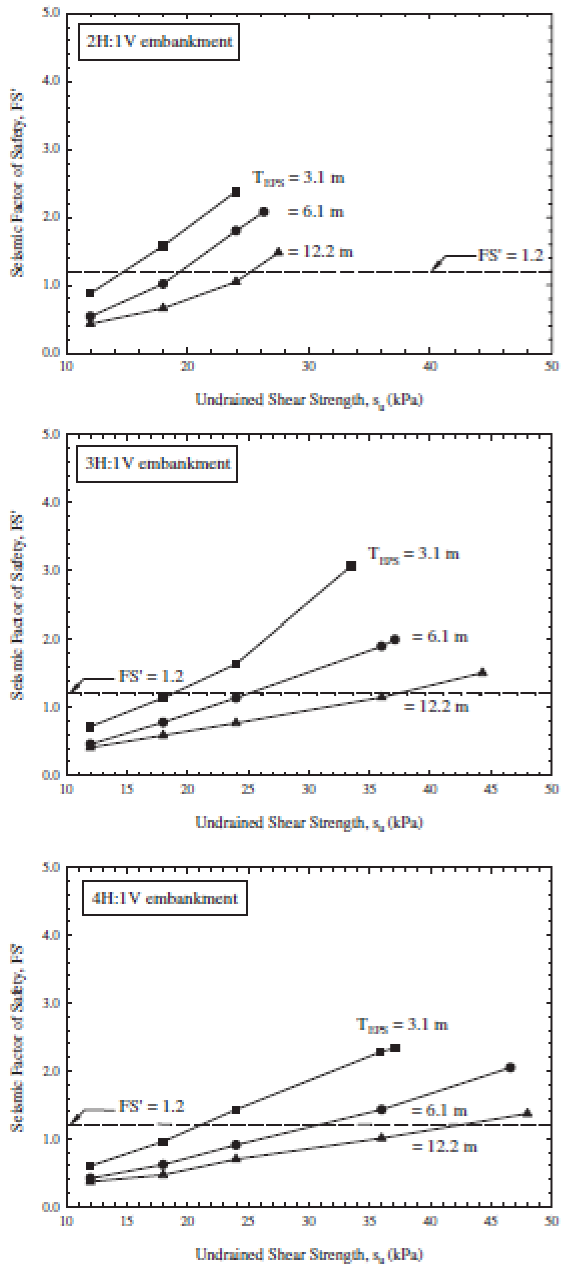
Appendix 6: *Seismic external slope stability design chart for trapezoidal embankment*



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.

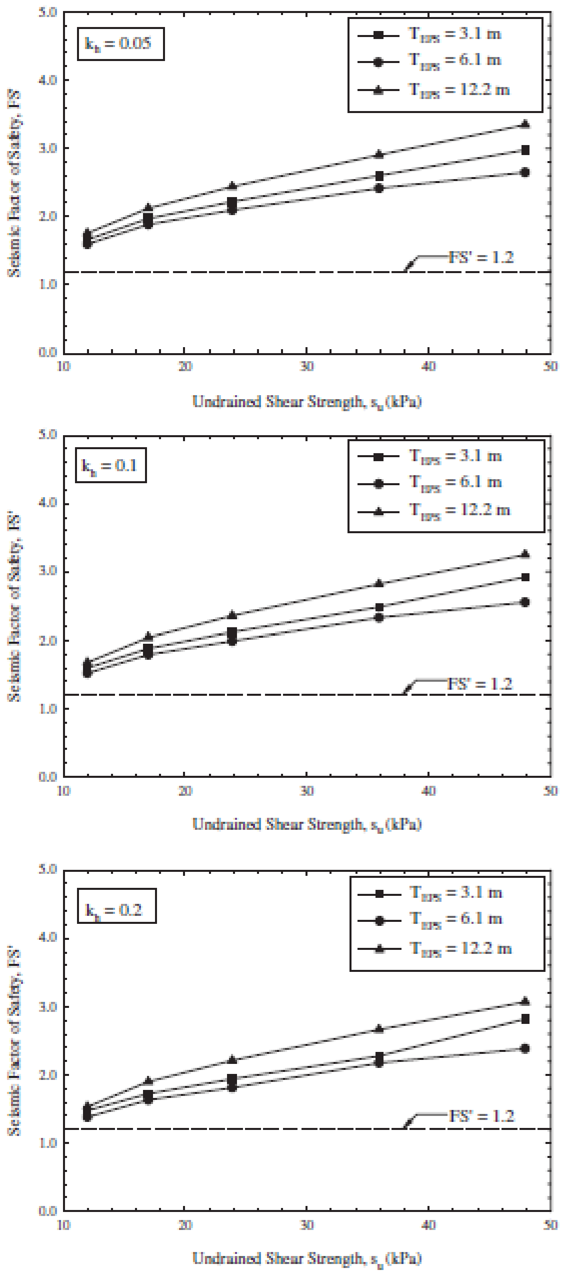


Seismic external slope stability design chart for trapezoidal embankments with a six-lane roadway with a total road width of 34 m (112 ft) and a k_h of 0.10.

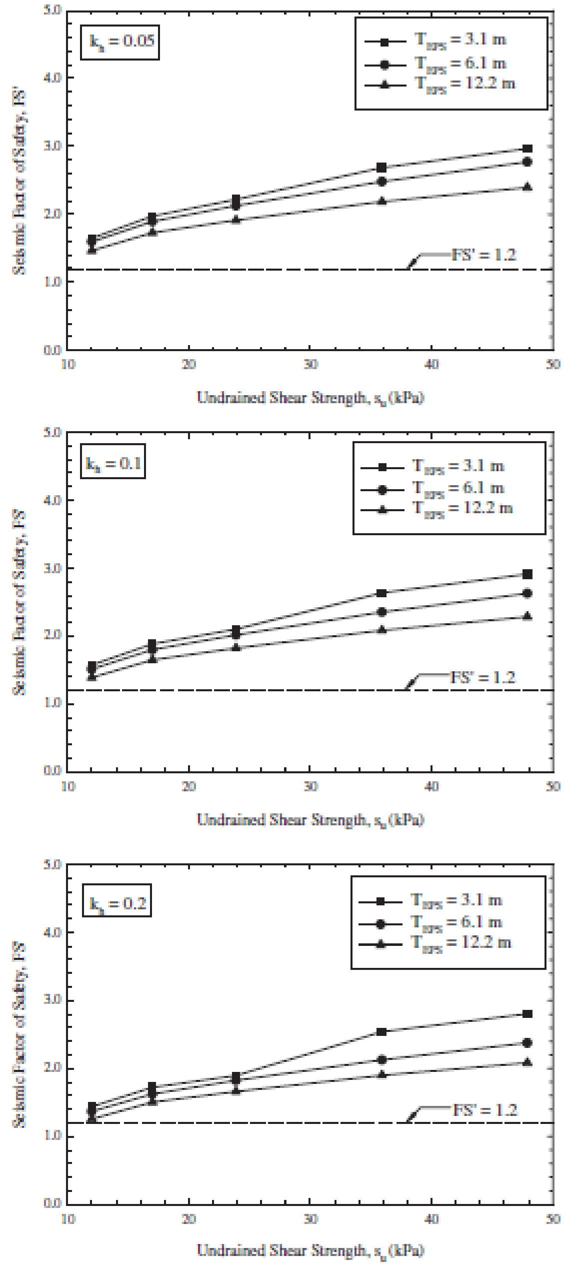


Seismic external slope stability design chart for trapezoidal embankments with a six-lane roadway with a total road width of 34 m (112 ft) and a k_h of 0.20.

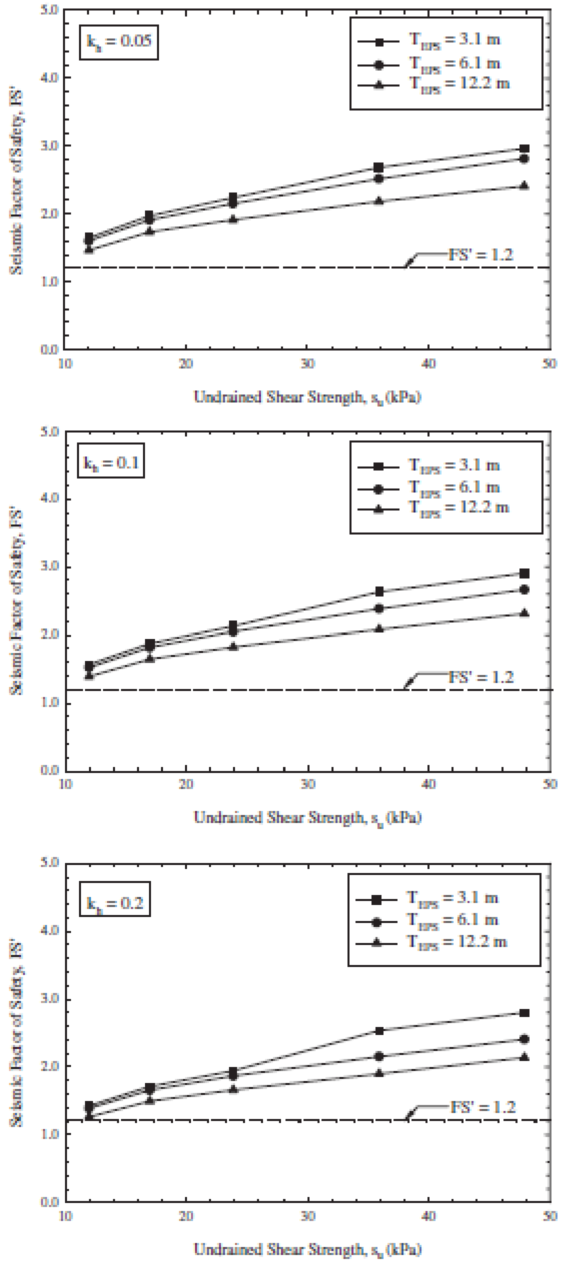
Appendix 7: *Seismic external slope stability design chart for vertical embankment*



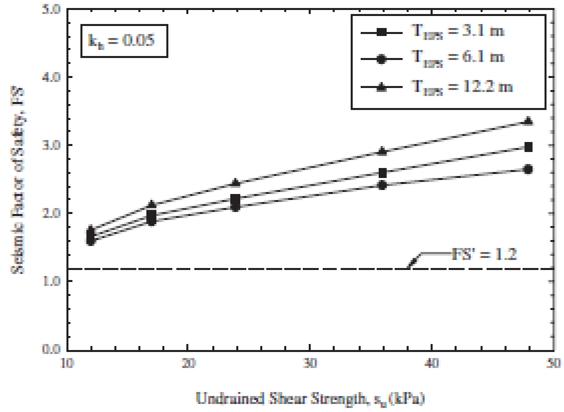
Seismic external stability design chart for a two-lane roadway vertical embankment and a total width of 11 m (36 ft).



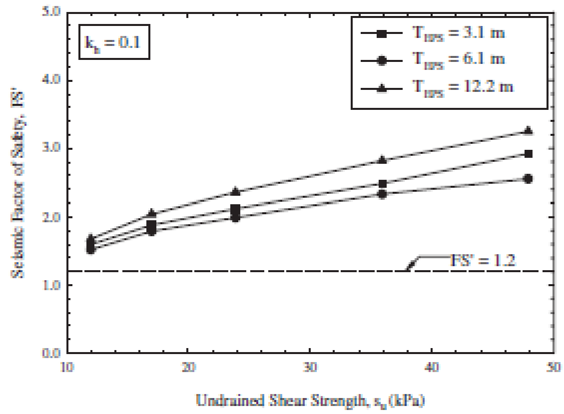
Seismic external stability design chart for a four-lane roadway vertical embankment and a total width of 23 m (76 ft).



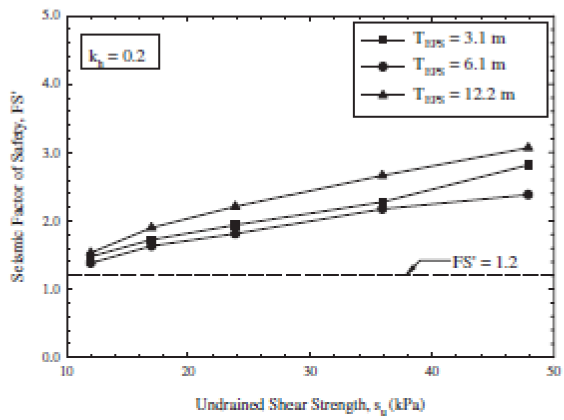
Seismic external stability design chart for a six-lane roadway vertical embankment and a total width of 34 m (112 ft).



k_h = 0.05
T_EPS = 3.1 m
T_EPS = 6.1 m
T_EPS = 12.2 m
FS' = 1.2
Seismic Factor of Safety, FS'
Undrained Shear Strength, s_u (kPa)
0.0
1.0
2.0
3.0
4.0
5.0
10
20
30
40
50



k_h = 0.1
T_EPS = 3.1 m
T_EPS = 6.1 m
T_EPS = 12.2 m
FS' = 1.2
Seismic Factor of Safety, FS'
Undrained Shear Strength, s_u (kPa)
0.0
1.0
2.0
3.0
4.0
5.0
10
20
30
40
50



k_h = 0.2
T_EPS = 3.1 m
T_EPS = 6.1 m
T_EPS = 12.2 m
FS' = 1.2
Seismic Factor of Safety, FS'
Undrained Shear Strength, s_u (kPa)
0.0
1.0
2.0
3.0
4.0
5.0
10
20
30
40
50