CHAPTER 1
INTRODUCTION

1.1. BACKGROUND

Fills and embankment structures resting on a soft soil of low bearing capacity frequently suffer from excessive settlement and instability problems caused by imposed self-weight. Faced with these problems, the geotechnical engineer traditionally employs one or more of a number of the ground improvement techniques to stiffen and strengthen the subsoil such as: soil stabilization, soil replacement, pre-consolidation and vertical drains remedial methods.

Yet in a theoretical sense, a patently simple solution appears as follows: reduce the impact of self-loading until it is negligibly small but at the same time retain adequate strength to support the structure. Until Expanded Polystyrene (EPS) geofoam was invented, however, it was almost impossible to translate this concept into practice.

EPS geofoam is an extremely lightweight foam material made from expanded polystyrene beads and can be used in a geotechnical-related application to replace the conventional mineral fill. Its effectiveness derives from the fact that it has a density only about 1% to 2% of the density of soil and rock but with the strength to support the loading commonly encountered in geotechnical-related applications. The EPS approach can best be exemplified by the illustrations of Figure 1.1 where a traditional mineral fill used in a conventional embankment yields a net loading intensity which far exceeds the in-situ stress levels resulting in foundation instability and settlement problems. The replacement EPS solution produces negligible increase in the stress level at the foundation level because of its extreme lightweight, therefore the associated problems do not appear.
A cost efficient solution, EPS geofoam expedites the construction program whereas the traditional methods invariably delay the construction program because of the time required to perform the soil remedial works. Realising the benefits of EPS geofoam, the Norwegian engineers were the first to instigate a number of geotechnical applications such as in road embankments traversing peat bogs, reconstruction of slide areas, special low lateral stress fill behind bridge abutments and rehabilitation of bridge approaches suffering differential settlements. New, innovative and more efficient geotechnical applications await a more thorough, definitive understanding of the geofoam behaviour.
1.2. IMPETUS FOR THIS RESEARCH

Use of the EPS geofoam is not new. The material has been successfully employed in geotechnical-related applications by the Norwegian engineers since at least the mid 1960s. Even so, it has been in service for less than 40 years. In Australia the use is more recent where the first and only reported application was in 1992 in a Melbourne bridge approach (McDonald and Brown, 1993). A man-made material, the EPS geofoam does not appear enduring to the uninitiated. There has therefore been a lingering doubt, even among some engineers, if the function of the EPS geofoam can really be sustained in the long-term commensurate with design life of a civil engineering structure. This unquestionably remains as a major technical hurdle to wider acceptance and future use of the EPS geofoam in geotechnical applications, especially in Australia.

The performance of the EPS geofoam is temperature- and environment-sensitive, and where the insights are mostly derived from the field experience in Europe and to a much lesser extent, North America and Japan. It is quite extraordinary that even in this computer age, the current use of the EPS geofoam continues to rely heavily on extrapolations of these case histories. It has been questioned if the same European experience can be transposed onto the Australian landscape. The fact that the EPS geofoam is used so infrequently in Australia does not help but the fact that more Australian-relevant research could be done to address this issue will.

Whilst impressive gains have been made in the understanding of the EPS geofoam over the last forty years, specific areas of interest and concern for geotechnical-related application of EPS geofoam requiring attention include the following.

An evaluation of the suitability of current laboratory test procedures for the EPS geofoam. The design for the testing should be driven by geotechnical requirements, consistent with geotechnical laboratory methods and with soil mechanics theory. The methods that are designed for non-geotechnical applications, where inappropriate, should be revised and amended.

The long-term behaviour of the EPS geofoam under creep conditions awaits a more thorough investigation. In load-bearing applications, it is inevitable that creep deformation will result with elapsed time and the question of material stability in the long-term is of paramount importance. In Australia where the field temperature would be warmer than in
the cooler climate of Europe, study of the creep characteristics of the EPS geofoam is critical to its successful implementation.

An investigation of the behaviour of EPS geofoam under repeated dynamic loading caused by such forces as vehicular traffic, wind and temperature. While the material may be susceptible to strength and stiffness degradation under repeated loading, only a few studies had been done to shed light on the resilience properties of the EPS geofoam. The findings will add to the body of knowledge as well as give guidance to the design of EPS geofoam in dynamic load-bearing applications including pavements.

A fundamental question for the EPS geofoam deals with the characterization of its behaviour, whether it behaves like the frictional mineral fill material which it replaces or otherwise. Despite the current body of laboratory test data, there is no definitive answer of the fundamental behaviour available at present. This answer is needed to construct the appropriate constitutive models for the material.

Methods of geofoam analysis should be developed for geotechnical applications. This may include a range of techniques from the simple analytical solutions to the more sophisticated numerical methods. It will be useful to incorporate the constitutive models into the solution methods, which have advanced rapidly in recent years. A more cost-efficient and safer geotechnical design would result from the research.

1.3. THESIS ORGANISATION

This thesis is divided into 10 chapters.
Chapter 1 introduces the background and the impetus for this research.
Chapter 2 reviews the relevant literature, historical research and development of the EPS applications in geotechnical engineering. The various standard laboratory test methods used in this thesis are also reviewed and discussed.

The behaviour of EPS under uniaxial and triaxial compression is summarized in Chapter 3. The mechanical behaviour, strength parameters and various elastic and plastic moduli are characterized. The factors that affect the behaviour of EPS are also discussed.

The shear behaviour under direct shear loading and triaxial compression are discussed in Chapter 4. The internal shear resistance of EPS and the interface friction between the EPS blocks and soils are studied.
Chapter 5 summarizes the results from creep tests. The long-term behaviour of EPS under different sustained loading conditions is investigated. The creep tests are also conducted at elevated temperatures to study the behaviour of EPS under warm climatic conditions.

In Chapter 6, the results from unconfined cyclic compression tests are presented and the behaviour of EPS under cyclic loading has been studied, and the test results are also used to validate the creep model established in Chapter 5 under repeated type loading.

Chapter 7 describes the test on an EPS scale model embankment to study the behaviour of EPS used as a replacement subgrade material under flexible pavement. The influence of the size of the EPS geofoam on the deformation of the model pavement and the comparison of the deformation of EPS geofoam with that of conventional fill materials are also studied.

In Chapter 8, a semi-analytic model is developed for an EPS embankment resting on consolidating soft soil based on the principles of visco-elasticity. The model is used to model the creep and consolidation of EPS embankment and underlying soft soil. An illustrative example has shown the relative merits of using this model in a typical EPS fill where the foundation soil is very soft.

A non-linear variable moduli constitutive model for EPS geofoam is developed in Chapter 9. The parameter values of the constitutive model are identified based on the laboratory tests results. The Finite Element Method (FEM) analyses are performed for case studies of typical EPS embankment resting on soft soil foundation, and the advantages of EPS fill technique are discussed.

Finally, the conclusions and main findings of this research are summarized in Chapter 10. Discussions and recommendations for future research are also presented.
CHAPTER 2
LITERATURE REVIEW

2.1 INTRODUCTION

A survey of the existing literature on the research and development for EPS geofoam in geotechnical-related applications has been carried out for this thesis. In attempting to make the thesis more coherent, the specific literature review relating to the topics covered in each chapter will be presented within the chapter. The purpose of this chapter is to provide a background review of the EPS geofoam. A historical summary of the research and development of EPS geofoam, and its current applications in geotechnical engineering are given. In a later section, the current standard test methods and the test methods of the geotechnical laboratory, which may be used, for EPS geofoam are discussed.

Appendix A contains a review of the manufacture and the general properties (i.e. physical and chemical) of EPS.

2.2 HISTORICAL RESEARCH AND DEVELOPMENT FOR EPS APPLICATIONS IN GEOTECHNICAL ENGINEERING

EPS was invented at BASF in 1950 and introduced commercially at the Dusseldorf Trade Fair in 1952. Since at least the 1960s, the EPS block has been used as a frost protection material for highway and railroads to avoid the deeply penetrating ground frosts. Then the technique of “EPS geofoam method” was developed for building roads on unstable subsoils (BASF 1998). The current definition of geofoam was coined by Horvath (1995) to refer to any manufactured material created by some expansion process that results in a material with a texture of numerous, closed, gas-filled cells.
In 1972, EPS geofoam was first successfully used as an ultra-lightweight fill on NR 159 road at Flom, near Oslo by the Norwegian Road Research Laboratory (NRRL) (Refsdal 1985). The road was built over a peat bog where the in situ soil consists of 3 m thick of peat and 10 m of soft sensitive clay. The settlement rate of the road was as high as 30 cm a year in 1972 – 1973 during which the road surface dropped to 600 mm below the bridge level. A replacement layer of EPS (1100 mm thick) was used to raise the road level and a new 500 mm pavement was laid on the top of the EPS fill. The cross section of the project is shown in Figure 2.1. The road level had subsided 80 mm during the following 12 years and since then virtually no settlement has occurred.

Following this successful experience, the EPS method has been used increasingly as an alternative and cost-effective solution for the problems of soft soil. Up to 1996, more than 300 road projects involving EPS fill had been completed in Norway with volume of EPS-blocks totally some 400,000 m$^3$ (Frydenlund and Aaboe 1996).
In addition to Norway, the EPS method has also been used increasingly in other European countries such as France, The Netherlands, Sweden, Finland, UK, Ireland, Spain and Germany.

In France, EPS has been used since 1983, when a trial embankment was constructed for a motorway widening project in Provence (Magnan et al. 1990).

The EPS method was first used in UK in 1985 on the A47 Great Yarmouth Western Bypass in Norfolk. In the period of 1990 to 1995, around 35 EPS embankments have been constructed in the UK with a total volume of 75,000 m$^3$ (Williams and Snowdon 1990).

The first project of EPS used in Netherlands was in 1984 in a dike approach road close to existing piled buildings over very compressible peaty sub-soil. Up to 1996, more than 400,000 m$^3$ EPS geofoam had been used (Teun van Dorp 1996).

In Germany, a “Code of practice for the use of rigid EPS foams in the construction of highway embankments” was published in 1995. Since then, the EPS method has also been increasingly used (BASF 1998).

EPS has also been successfully used for embankment fill in North America and Asia, primarily in areas where poor soil conditions exist. However, in countries such as U.S.A. where there was, until recent years, a chronic indifference to geofoam technology the use of geofoam as lightweight fill has languished because of the lack of meaningful promotion and technology transfer (Horvath 1996).

On the other hand, the greatest interest and potential seems to be in Asia, where Japan is the major user. The Philippines, South Korea, Malaysia and Thailand have completed some initial projects (Frydenlund and Aaboe 1996).

The EPS method was introduced for the first time into Japan in 1985 and an EPS Development Organization was established in 1986 to promote its application (EDO 1989, 1991). In the following 10 years, Japan has become the largest user of EPS for construction works in the world (Gosaburo 1996).

Closer to home, the one and only reported use of the EPS geofoam in Australia was made on a highway bridge in Melbourne, Australia (McDonald and Brown 1993). The sub-soil conditions at the site consisted of up to 10m of soft to very soft silty clays overlying alluvial sequences of sands, gravels and stiff clays to depths of around 35m. Conclusions made at the end of the project were that the use of the EPS geofoam to
minimize the subsoil stress increases can be cost-effective and allowed construction programs to proceed without delays of the kind normally associated with ground treatment and pre-consolidation.

2.3 CURRENT APPLICATIONS OF EPS IN GEOTECHNICAL ENGINEERING

There are many kinds of EPS, supplied in many forms, but the material found useful in road construction commonly has a density of at least 20 kg/m$^3$ and is shaped as a rectangular block. EPS of lower density may be used for the cores of noise barriers. The applications of EPS geofoam in geotechnical engineering, summarized by BASF (1995), include the following.

2.3.1 Construction of Embankments

Subsoil improvement by one or more of the traditional methods are often time consuming and, therefore, costly. By using the ultra-lightweight EPS geofoam, the weight of the road embankment is reduced to the extent that the self-loading imparts negligible stress on the subsoil foundation (Figure 1.1). In this application, the geofoam may produce one or more of the following benefits:

- Adjust the loads to be within the bearing capacities of soils;
- Avoid interference settlement of adjacent structures;
- Allow roads to traverse difficult ground;
- Less or no soil replacement;
- Less material transportation;
- Construction work can be completed sooner.
2.3.2 Construction of Bridge Abutment Slope

Because of the heights that may have to be reached, foundations of slopes leading to bridges may present difficult technical problems. Usually pre-consolidation is not an option when the material has to be piled too high. The EPS method offers an effective solution to avoid the differential settlement of bridge and ramp (Figure 2.2).

2.3.3 Widening Embankments

Differential settlement is a common problem when existing embankments have been widened. Consolidation due to the weight of the new fill may lead to considerable deformation, possibly causing changes in crossfall or longitudinal cracking, or both.

If the additional fill is made up partly of EPS block, subsequent defects can be minimized or avoided altogether. The advantage of using this extremely light material is especially obvious when the site offers little space, since the method of construction lends itself well to the use of gabions. It is possible that the embankment can be constructed with steep slopes (Figure 2.3).
2.3.4 Repairing Defects Caused by Settlement

Differential settlement or subsidence is a frequent cause of damage to roads. Repairing the road with additional mineral fill is not the solution as it simply adds to the load and renews the settlement. If the mineral fill was replaced by EPS geofoam, and the loading intensity is reduced sufficiently, no further settlement of significance will take place.

2.3.5 Other Applications

Other applications of EPS geofoam have been reported successfully around the world (Duskov 1991, and Skuggedal and Aaboe 1991) and include:

- repair of damage due to slip,
- raising the height of the existing embankment,
- construction of subgrade for extensive pavement on soils of low bearing capacity,
- construction of noise barriers on ground prone to excessive settlement,
- bridging culverts, conduits, etc.,
- backfill adjacent to construction works,
- construction of embankments near structures that would be adversely affected by interference settlement.
2.4 TEST METHODS FOR ENGINEERING PROPERTIES OF EPS GEOFOAM

2.4.1 General

The adage that the accuracy of an analysis is only as good as the input parameters that go into the analyses applies equally well for the EPS geofoam. It is unfortunate that the current standard test methods used to determine the properties of the EPS geofoam are not driven by the requirements for geotechnical applications. This shortcoming is partly historical as the dominant use for the EPS material has been, and still is, in packaging and thermal insulation. As geotechnical applications increase, the need to standardize the geotechnical test methods is obviously required.

This section first reviews the current standard test methods for EPS geofoam. It next reviews some of the standard tests for soils and geomaterials: triaxial, shear, creep and repeated loading tests, and suggests how some of the practices could be revised for EPS geofoam.

2.4.2 Current Standard Test Methods

*Standard Test Conditions*

The environment conditions for laboratory tests are comparable around the world, which are at approximately room temperature of 23 ± 2°C and 50 ± 5 % relative humidity. The behaviour of the tested material in 'short-term' compression is generally determined by testing of small specimens under strain-controlled uniaxial loading. The geometry and dimensions of the specimens vary from standard to standard worldwide.

In Australia, the methods for determination of compressive strength of rigid cellular plastics is specified in standard AS 2498.3. The test specimens are cubes having sides of 50 ± 1mm. The rate of compression is specified as 10% of the original thickness of the specimen per minute. The compressive stress at 10% of relative deformation is defined as the compressive strength.

Similarly, in Germany, the compressive test specified in DIN 53421 for rigid cellular materials is preferably performed on specimen cubes having sides measuring 50
mm. The rate of compression is 5 mm (10% of the original height of the specimen) per minute. The compressive stress at 10% of compressive strain is defined as the compressive strength.

However, according to ASTM D 1621 – 94, specimens shall be square or circular in cross section with a minimum area of 25.8 cm² (4 in.²) and a maximum area of 232 cm² (36 in.²). The minimum height shall be 25.4 mm (1 in.) and the maximum height shall be no greater than the width or diameter of the specimen. The rate of compression is 10% of the sample thickness per minute. The compressive stress at 10% compression is quoted.

In all the above standards, the standard size of EPS specimen is cylindrical or rectangular with an aspect ratio of 1:1. The compressive stress at 10% compressive strain is quoted as the compressive strength. This strain value, however, already lies in the post yield range of the stress – strain curves, and is therefore information only of value for quality control (BASF 1998). In geotechnical application of the EPS geofoam, the definition of yield stress (Preber et al. 1994, and Horvath 1995) is a more important parameter than compressive strength.

**Standard Test Method of Triaxial Compression**

The AS 1289.6.4.1 – 1998 and the American equivalent ASTM Standard D2850 – 95e1, describe the basic test procedure applicable to cohesive soil as well as setting out a procedure for determining the compressive strength and deformation properties of cohesive soil in the triaxial compression apparatus under conditions in which the cell pressure is maintained constant.

The ASTM D5407 – 93 covers the determination of elastic moduli of intact rock core specimens in undrained triaxial compression. This is closely applicable, it is viewed, to the EPS material since no other relevant standards exist. The standard specifies the apparatus, instrumentation, and procedures for determining the stress-axial strain and the stress-lateral strain curves, as well as Young’s modulus and Poisson’s ratio.

In order to test the EPS specimens under triaxial loading condition, the cylindrical specimens are needed to fit into the triaxial cell. It was decided, after a review of standards that, the standard cylindrical specimens prepared for this thesis should have a diameter of 50 mm and a height of 50 mm, which is compatible with the specification of AS 2498.3 as well as other major standards. The cylindrical shaped specimens were employed for
unconfined compression, triaxial compression and creep tests.

Because the particles of soil specimen are loosely bounded, unlike the rigid rock specimen, the effect of confining pressure is much more significant. One of the most notable characteristics of clay is their susceptibility to slow volume changes, which can occur independently of loading due to swelling or shrinkage. The behavior of the EPS specimen under compression is much closer to that of rock specimen. Therefore, the rock specifications are more appropriate for EPS tests. In this thesis, the triaxial compression test method followed the procedures described in ASTM D5407 – 93, but reference is also made to ASTM Standard D2850 – 95e1, and AS 1289.6.4.1 – 1998 where necessary. A standard loading rate of compression was chosen at 1% of the original thickness of the specimen per minute.

**Standard Test Method of Uniaxial Compression**

The uniaxial compressive test is given in the AS 2498.3 and the ASTM D1621 – 94. The standard test method describes a procedure for determining the compressive properties of rigid cellular materials, particularly expanded plastics, based on the test machine cross head motion.

In addition to cylindrical shaped specimens, specimen cubes having sides measuring 50 mm and specimens of different sizes were also employed in uniaxial compressive tests to study the effects of specimen geometry in this thesis.

**Standard Test Method of Shear Box Test**

Since no relevant standards exist for interface shear resistance of EPS and soil, the shear box test (direct shear test) is conducted in accordance with the Australian Standard AS 1289.6.2.2 - 1998, and the ASTM D5321 – 97. The standard, AS 1289.6.2.2-1998, sets out a method for determining the shear strength of a soil by direct shearing in a shear box. The ASTM D5321 – 97 covers a procedure for determining the shear resistance of a geosynthetic against soil, another geosynthetic, or a soil and geosynthetic in any combination.

The same equipment was used for both internal and interface shear strength tests to ensure consistency in the testing. In the shear box test, rectangular specimens having an
area of 36 cm$^2$ (60mm $\times$ 60 mm) and 40 mm height were used.

*Standard Test Method of Creep Test*
Creep tests in this thesis generally follow the procedures in ASTM D 5202 – 91 (1997) that covers the determination of long term strength and deformation of cylindrical specimens in compression under a constant sustained load.

*Standard Test Method of Repeated Loading Test*
The procedures for repeated loading tests follow the Australian Standard AS 1289.6.8.1-1995 – determination of the resilient modulus and permanent deformation of granular unbound pavement material. However, because a confining pressure was not applied, a uniaxial compression test equipment was used instead of the triaxial test equipment.
CHAPTER 3
BEHAVIOUR UNDER MONOTONIC COMPRESSIVE LOAD

3.1. INTRODUCTION

Uniaxial and triaxial compression tests are commonly used to determine the compressive strength and stiffness of materials. This chapter describes the behaviour of the EPS geofoam under monotonic compressive loading.

Although compression and limited triaxial test results of EPS geofoam have been published in the literature (e.g. Hamada and Yamanouchi 1987, Eriksson and Trank 1991, Preber et al. 1994, Horvath 1995 and BASF 1990) such factors as the geometry, size of the specimen, confining pressure, strain rate (loading speed), and temperature that influence the behaviour of EPS geofoam remain to be investigated. The tests in this chapter were conducted to obtain a body of raw data to help characterize the behaviour of EPS geofoam, to verify the results of previous studies, and to investigate the factors which may influence the EPS compression test results.

As the geofoam is regarded in this context as a geomaterial, a deliberate attempt was made to carry out the testing in accordance with the AS and ASTM standards for soil and rock testing described in Chapter 2, where possible. It is not the first time compressive and triaxial tests have been done for EPS material but a standard triaxial test procedure specific to the EPS geofoam used in geotechnical application has not yet been written. This chapter describes and also explains why some decisions were taken in respect of the testing, which may deviate from normal soil and rock testing procedure.

3.2. TEST PROGRAM

3.2.1. Test Condition

The compressive behaviour of EPS geofoam was investigated through a series of unconfined compression and undrained triaxial tests that were performed on a triaxial apparatus and a uniaxial test system (INSTRON). The tests carried out on the triaxial...
machine were used to study the behaviour of EPS geofoam under various confining stress conditions. The uniaxial system tests were used to investigate the effects of loading rate, specimen size and geometry.

The laboratory test environment was approximately +23°C and 50% relative humidity. Three types of specimen with a nominal density of 13kg/m³, 20 kg/m³ and 27kg/m³ respectively were used. In the temperature controlled triaxial tests where the temperature spanned +23 and +50°C, only specimens with a nominal density of 20kg/m³ were used.

3.2.2. Test Specimen

All specimens used in the testing program of this thesis were prepared from material supplied by RMAX Rigid Cellular Plastics, an Australian manufacturer of EPS foam. The standard cylindrical specimen was cut from a cylindrical EPS prism of 50mm diameter, and the ratio of the specimen height to diameter was 1:1. The stock cylindrical specimen used in a soil laboratory with a 2:1 (height:diameter) aspect ratio was not adopted after signs of buckling were observed at this aspect ratio following yielding of the EPS specimen. Since the 2:1 ratio (2.5:1 for rock) has been recommended to minimise end effects, some comparative tests were therefore performed to assess the significance of end effects in EPS specimens. Tests done using 2:1 and 1:1 specimens produced virtually identical strength and elastic modulus values, hence it was concluded that the end effects of the 1:1 specimens were not significantly different to those of the 2:1 specimens. Also, the mode of failure of the EPS specimens occurred by compression in which no shear failure plane was observed to develop. This again appears to suggest that the end effects may not be as critical as in soils and rocks.

Cubic specimens having sides measuring 50 mm and rectangular prismatic specimens of different sizes were also tested to study the effects of specimen geometry. The types and the total numbers of the EPS specimen used in all tests are listed in Table 3.1.
Table 3.1 Specimen condition in triaxial and uniaxial compression tests

<table>
<thead>
<tr>
<th></th>
<th>Triaxial Test</th>
<th>Uniaxial Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Confining stress</td>
<td>0, 10, 15, 20 kPa</td>
<td>N/A</td>
</tr>
<tr>
<td>Geometry</td>
<td>Cylindrical Ø50×50mm; Rectangular 50mm × 50mm × 50mm, 76mm × 76mm × 36mm, 76mm × 76mm × 76mm, 76mm × 76mm × 110mm, 76mm × 76mm × 142mm</td>
<td>Cylindrical Ø50×50mm;</td>
</tr>
<tr>
<td>Density, kg/m³</td>
<td>13, 20, 27</td>
<td>13, 20, 27</td>
</tr>
<tr>
<td>Displacement rate</td>
<td>0.5 mm/min</td>
<td>0.5, 1, 2.5, 5, and 10 mm/min</td>
</tr>
<tr>
<td>Temperature, °C</td>
<td>23,30,35,40,45,50</td>
<td>23 (room temperature)</td>
</tr>
<tr>
<td>Total number</td>
<td>120</td>
<td>60</td>
</tr>
</tbody>
</table>

3.3. UNCONFINED COMPRESSION TEST

3.3.1. General

The unconfined compression test is a simple and rapid method to obtain the compressive strength of the tested material. This test may be deemed a special case of triaxial compression test, carried out at zero cell pressure, i.e. \( \sigma_3 = 0 \). The test may be carried out in the laboratory using a standard triaxial apparatus or a uniaxial loading machine. In this thesis, the triaxial apparatus and the uniaxial loading machine have been used since both apparatus were available.

3.3.2. INSTRON Loading Machine

The uniaxial loading machine used was an INSTRON 6027-R5500 Electro-mechanical Universal Testing Machine equipped with high-speed digital control and data acquisition (Figure 3.1). Axial loading and unloading were effected by moving the cross-head on which the base of specimen was seated up and down. The magnitude of loading was measured by a calibrated load cell and the test system was monitored using a proprietary program called Merlin.
3.1. TRIAXIAL COMPRESSION TEST

3.1.1. General

In an undrained triaxial test (dry specimen), the EPS specimen was completely sealed from the cell fluid. The term “undrained” was coined previously by Preber et al. (1994) for a similar triaxial test on EPS specimens. The ends of the EPS specimen were further smoothed by sand-papering to reduce end friction. With the cell disassembled, the specimen was mounted on the pedestal and enclosed by the rubber membrane. One end of the rubber membrane was stretched over the pedestal and the other end stretched over a loading cap. The ends were held in place with rubber rings. The cell was then reassembled and filled with de-aerated distilled water to give confining pressure during the test.
CHAPTER 3: BEHAVIOUR UNDER MONOTONIC COMPRESSIVE LOAD

Figure 3.2 Triaxial test apparatus

Figure 3.3 Triaxial test system schematic diagram
3.4.2. Test Apparatus

The triaxial test setup used in this thesis is shown in Figure 3.2, and Figure 3.3 shows the schematic diagram of the test system, which has the components described below:

A 50kN TRITECH digital loading frame, which has been designed for undrained or drained triaxial test of soil specimens up to 100 mm diameter for compression or CBR tests by conventional means. The loading speed ranges from 0.0 to 6.0 mm per min. The platen speed rate was set through a precise thumb wheel selector with an accuracy controlled to better than 1% (Wykeham Farrance 1995).

The test specimen was set up inside a Wykeham Farrance triaxial cell (WF 10201), which can withstand internal pressure up to 1700kPa.

The readings of applied load and the vertical displacement during the tests can be read manually by a proving ring and a dial gauge, or automatically recorded by a load cell and an LVDT transducer via in-house developed computer programs. The measurement resolutions of the load cell and the LVDT are 1 Newton and 0.01mm respectively. In this thesis, both manual data reading and automatic data acquisition were employed.

A GDS digit pressure/volume controller accurate to volume measurement of 1 mm$^3$ was used to control the cell pressure constant at a pre-set value and to measure any volume change in the cell fluid during the test.

The recorded volume change is the sum total of the displaced fluid due to the loading plunger and the volume change in the specimen itself as it is loaded axially. The volume change in the specimen was determined by subtracting the displaced fluid due to the loading plunger from the total volume change reading recorded by the controller.

3.4.3. Confining Pressure

Confining pressure or lateral stress occurs in the soil mass or fill material. Unlike the tests carried out by previous investigators (Preber et al. 1994), the triaxial tests in this thesis were performed at a confining pressure not more than 20kPa. The reason is because the confining or lateral stresses which result from a vertical loading is very small and it is not realistic in terms of geotechnical applications to impose higher confining pressure in the triaxial experiments. The explanation is given here:
In general, the lateral stress ($\sigma_{xx}$) was developed due to the lateral and vertical deformation and can be written as

$$\sigma_{xx} = \frac{E\varepsilon_{xx}}{(1+\nu)(1-2\nu)} + \frac{\nu E\varepsilon_{zz}}{(1+\nu)(1-2\nu)}$$

where the vertical strain can be written as

$$\varepsilon_{zz} = \frac{\sigma_{zz} - 2\nu\sigma_{xx}}{E}$$

In confined condition, the $\varepsilon_{zz}$ is zero and because the Poisson’s ratio is close to zero, then

$$\sigma_{xx} = \frac{\nu E\varepsilon_{xx}}{(1+\nu)(1-2\nu)} = \frac{\nu(\sigma_{zz} - 2\nu\sigma_{xx})}{(1+\nu)(1-2\nu)} = \nu\sigma_{zz}$$

The EPS material has been shown to have very low or almost zero Poisson ratio (Horvath 1995, and Eriksson and Trank 1991). Let the Poisson’s ratio takes 0.02 to 0.2 and the vertical stress takes 100 kPa, the resulted lateral stress (confining stress) will be 2 to 20 kPa.

Therefore it may be concluded that the confining or lateral stresses which results from a vertical loading is very small. This is also in accord with the recommendation of the Norwegian Road Research Laboratory which adopts a nominal 10 kPa for EPS lateral pressure acting on the retaining structures for design purposes (Aaboe 1987). In general, it is argued that it is not realistic in terms of geotechnical applications to impose a higher confining pressure than 20 kPa in the triaxial experiments.

### 3.4.4. Cross-sectional Area of Specimen

To calculate the vertical stress on the tested specimen in a normal triaxial undrained test, corrections should normally be made for the average cross-sectional area of the specimen for a given strain in the specimen. Because the Poisson ratio of EPS is so low, no correction was made for the cross-sectional area of the specimen. After unloading, the dimensions of each specimen were carefully measured and the results clearly indicate that there was negligible change in the specimen’s diameter.
3.5. DEFINITION

Figure 3.4 shows typical a stress – strain response in undrained triaxial tests. The stress-strain curves are typically bilinear. The compressive strength, initial and plastic Young’s modulus, and yielding stress, as shown in Figure 3.5, are defined below (Horvath 1995, and Preber et al. 1994).

Compressive strength, $\sigma_c$. The compressive stress or deviator stress (in triaxial tests) measured at a 10% axial strain, which corresponds approximately to the end of the yielding range.

Initial tangent Young’s modulus, $E_i$. The slope of the initial linear-elastic portion of the stress-strain curve.

Plastic tangent Young’s modulus, $E_p$. The slope of the post-yielding linear portion of the stress-strain curve.

Yielding stress, $\sigma_y$. The yielding stress is determined graphically by forward extrapolation of the initial linear portion and backward extrapolation of the post-yield linear portion of the stress-strain curve. The stress at which the two extrapolated lines cross is defined as the yielding stress.
Poisson’s ratio, $v$.

The ratio of the lateral strain and axial strain is defined as

$$v = \frac{\varepsilon_l}{\varepsilon_a}$$  \hspace{1cm} (3.1)

where

$\varepsilon_a$ = axial strain (compressive strain is positive),

$\varepsilon_l$ = lateral strain (expansion is positive).

3.6. TEST RESULTS

3.6.1. Stress-strain Response

In general, as shown in Figure 3.4, the monotonic stress-strain response of EPS geofoam over the strain range is typically elasto-plastic and the following general observations have been made:

1. The initial linear-elastic behaviour occurs up to values of strain of between 1% and 2%. The elastic limit increases with increasing EPS density.

2. Yielding occurs over a strain range which extends approximately between 3% and 5%. The radius of curvature within the yielding zone decreases with increasing EPS density.

3. The stress-strain behaviour immediately after yielding is linear and work-hardening in nature.
Figure 3.7 Triaxial compression tests (20kg/m³)

It is found, as shown in a photo (Figure 3.6) of the EPS specimen and from measurements post testing, that the EPS specimen in unconfined and/or confined triaxial compression tests at failure is compressed in both the direction of loading and perpendicular to the direction of loading. This indicates that the EPS geofoam may have a negative Poisson's ratio.
Figure 3.7 shows the typical stress-strain curves of EPS specimens from triaxial compression tests. Even though the data is somewhat scattered for each material type, the test results generally indicate that, with increasing confining pressure, the deviator stress value remained at approximately the same magnitude. The compressive strength and the initial Young’s modulus were found to be essentially independent of the confining pressure.

Density variations were found in EPS geofoam of the same density classification. Variations in density of ±10% about the mean value were recorded. As all the tests were carefully conducted in the identical conditions (height, diameter, density, temperature, confining pressure and displacement rates), the difference in compressive strength appears to be due to the variation in bulk density and the in homogeneity of the material. The study of Eriksson and Trank (1991) showed that the variation of the density in the same block of EPS geofoam was found to be up to 40%. The inhomogeneity of the EPS material was also reported and discussed by Horvath (1995).

The average values of compressive strength, yield stress, initial and plastic tangent Young’s moduli are shown in Table 3.2. It is found that, as the density of the EPS geofoam material increases, the compressive strength, yield stress and initial Young’s modulus increase correspondingly.

Table 3.2. Average values with different EPS geofoam densities.

<table>
<thead>
<tr>
<th>$\rho$, kg/m$^3$</th>
<th>$\sigma_c$, kPa</th>
<th>$\sigma_y$, kPa</th>
<th>$E_t$, MPa</th>
<th>$E_p$, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>48</td>
<td>36</td>
<td>2.13</td>
<td>141</td>
</tr>
<tr>
<td>20</td>
<td>98</td>
<td>84</td>
<td>4.62</td>
<td>145</td>
</tr>
<tr>
<td>27</td>
<td>136</td>
<td>124</td>
<td>6.46</td>
<td>166</td>
</tr>
</tbody>
</table>

Note: triaxial tests done at room temperature and at a displacement rate of 0.5mm/min.

3.6.2. Compressive Strength

Test results in Figure 3.8 show that the overall compressive strength of EPS geofoam increases linearly with increasing EPS geofoam density. Because of the variation in bulk density, the compressive strength of EPS geofoam material is normally around ±10% of the average value.
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Figure 3.8 Compressive strength of EPS vs. density

Figure 3.9 Compressive strength of EPS vs. confining pressure
In order to investigate the effect of confining pressure on the behaviour of EPS, regression analyses were carried out on the triaxial test data. Linear regression in terms of density and confining stress (Figure 3.9) gives the following relationship,

\[ \sigma_c = (0.50 - 0.03 \rho) \sigma_3 + 6.4 \rho - 35.3 \]  \hspace{1cm} (3.2)

where \( \sigma_c \) is compressive strength in kPa, \( \rho \) is EPS geofoam density in kg/m\(^3\) and \( \sigma_3 \) is the confining stress between 0 to 20 kPa.

For a given density, the maximum variation of \( \sigma_c \), between the mean value of Table 3.2 and the calculated value from the equation (3.2) was found to be \( \pm 0.3\% \). This result indicates that the influence of confining pressure on the EPS geofoam behaviour is very small and, for the range of confining stress level tested, this influence can be neglected.

![Graph showing yield stress of EPS vs. density](image)

Figure 3.10 Yield stress of EPS vs. density

### 3.6.3. Yielding Strength

Test results in Figure 3.10 and 3.11 show the yielding stress for different density and different confining stress. It was found that the yielding stress of EPS geofoam increases linearly with increasing density. The confining pressure also shows a small influence on the yielding stress, which decreases with increasing confining stress. Linear regression in
terms of density and confining stress gives the following relationship,

$$\sigma_y = (0.38 - 0.04\rho)\sigma_0 + 6.5\rho - 45.6$$  \hspace{1cm} (3.3)

where $\sigma_y$ is yielding stress in kPa, $\rho$ is in kg/m$^3$ and $\sigma_0$ in kPa.

For a given density, the maximum deviation of $\sigma_y$ between the mean value and calculated value from the equation (3.3) was found to be $\pm 0.5\%$. The yielding stress is found to have a small dependency on the confining pressure.

3.6.4. Stiffness Moduli of EPS Geofoam

3.6.4.1. Initial Tangent Young’s Modulus, $E_i$

Test results in Figure 3.12 and 3.13 show the initial Young’s modulus, $E_i$, for different density and different confining stress. It was found that the $E_i$ of EPS geofoam increases linearly with increasing density. However, the confining pressure only shows a small influence on the value of $E_i$. Linear regression in terms of density and confining stress gives the following relationship,
Figure 3.12 Variation of initial Young's modulus of EPS vs. density

Figure 3.13 Variation of initial Young's modulus of EPS vs. confining pressure
$E_t = (0.0013 - 0.0001\rho)\sigma_3 + 3.2\rho - 1.9 \quad (3.4)$

where $E_t$ is in MPa, $\rho$ in kg/m$^3$ and $\sigma_3$ in kPa.

For a given density, the maximum variation of $E_t$, calculated from the equation (3.4), was found to be ±0.8% of the average value.

3.6.4.2. Plastic Tangent Young’s Modulus, $E_p$

The slope of the post-yielding linear portion of the stress-strain curve is defined as the plastic tangent Young’s modulus, $E_p$. Figures 3.14 and 3.15 show the variation of $E_p$ with density as well as the confining pressure. The results indicate that the variation in $E_p$ values are generally not affected by either density or confining pressure. For this reason, it is useful to take an overall average $E_p$ value for EPS geofoam. This value is found to be 150 kPa.

![Figure 3.14 Variation of plastic Young's modulus of EPS vs. density](image-url)
Figure 3.15 Variation of plastic Young’s modulus of EPS vs. confining pressure

3.7. POISSON’S RATIO

3.7.1. General

A traditional method of determining the Poisson’s ratio for a soil is by measuring the volume change of the soil sample during the triaxial test. Such a method was adopted by Preber et al. (1994) to determine the Poisson’s ratio of the EPS geofoam in which the lateral strain was indirectly derived from the volumetric and axial strain values as:

\[ \varepsilon_r = 0.5(\varepsilon_v - \varepsilon_a) \]

However as the measured volume change of the cell fluid was not precisely equal to the volume change of the EPS geofoam specimen, it was not possible to obtain the volumetric strain accurately, especially at large strains. Also, if the lateral strain is very small (close to zero in the case of EPS), the Poisson’s ratio is very sensitive to any error in the value of the lateral strain. These errors made this method of determining the Poisson’s ratio unsatisfactory.

A second method considered was by strain gauges, which is commonly used in measuring the Poisson’s ratio of solid materials such as rock and concrete. The most common type of strain gauges consists of a grid of fine wire or a constant metal foil grid encapsulated in a thin resin backing. This gauge is normally glued to a carefully prepared
test specimen by a thin layer of epoxy, which acts as the carrier matrix to transfer the strain in the specimen to the strain gauge. However, normal epoxy dissolves the EPS geofoam and so was deemed unsuitable in this case. Therefore, instead of epoxy, a thin layer of "liquid nail" that does not dissolve the EPS geofoam was tried. Although the adhesion worked well, the flexible EPS surface was unable to transfer the strain in the specimen to the strain gauge properly.

Both of the methods described above were found to be inappropriate for the EPS and a third method described below, which eventually worked, was used.

3.7.2. Test Method

The third method was a simplified 'LVDT' method. As shown in Figure 3.16, two LVDT transducers were used to measure the horizontal deformation. The change in diameter of the specimen, hence the lateral strains, can be obtained directly during the compression test. Then the Poisson's ratio is calculated by equation (3.1).

The test results verified that the LVDT method is an acceptable method for estimating the Poisson's ratio of EPS geofoam.

Figure 3.16 LVDT set up for Poisson's ratio test
3.7.3. Test Results

Figure 3.17 shows the Poisson's ratio of EPS geofoam during compression loading obtained by the LVDT method. The test results indicate that, in general, the Poisson's ratio of EPS is very small in magnitude, approximately ranging from -0.07 to +0.02. The initial value of the Poisson's ratio is about -0.03 to -0.01. It increases slightly with increasing axial strain up to 0.02. When yielding occurs, it decreases and reaches the minimum value at the end of the yielding range. After yield, the Poisson’s ratio remains an approximately constant negative value. The average Poisson’s ratio of EPS geofoam based on the direct measurement of the dimensions of the EPS geofoam specimen after the unloading was found to be about -0.02. An explanation for the negative Poisson's ratio could be that the air-filled cells of EPS begin to burst at about 2% of axial strain, which leads to an inward movement of the EPS cellular particles and a decrease of Poisson’s ratio. The photo of the EPS specimen (Figure 3.18), which was taken for different stages during the loading, shows interesting lateral deformation under compression. It clearly shows that the Poisson’s ratio is negative towards the later part of the compression loading.
3.8. **BULK AND SHEAR STRENGTH PARAMETERS OF EPS**

In soil mechanics the shear and bulk moduli, $G$ and $K$, are often preferable to the Young’s modulus $E$ and Poisson’s ratio $\nu$ because it is important to consider shearing or change of shape separately from compression or change of size (Atkinson 1993).

In the same way that the Young’s modulus was obtained from a plot of deviator stress versus axial strain, the bulk modulus, $K$, and shear modulus, $G$, can also be determined from the plots of mean stress, $p$ versus volumetric strain, $\varepsilon_v$ (compressive is positive), and shear stress, $\tau$, versus equivalent shear strain, $(\varepsilon_a - \varepsilon_v)$, respectively. During the triaxial test, $\varepsilon_v$ and $p$, and $(\varepsilon_a - \varepsilon_v)$ and $G$ are related through

\[
p = \frac{(\sigma_1 + 2\sigma_3)}{3} = K \varepsilon_v \tag{3.5}
\]

\[
\tau = \frac{(\sigma_1 - \sigma_3)}{2} = G (\varepsilon_a - \varepsilon_v) \tag{3.6}
\]

\[
\varepsilon_v = \varepsilon_a + 2\varepsilon_v. \tag{3.7}
\]

In general, the volume change of a soil sample can be determined by measuring the amount of water flowing into or out of the triaxial chamber during the test. As discussed in section 3.7, this method was unable to accurately estimate the volumetric strain for the EPS material. However, because the lateral strain of the EPS is very small compared to its axial strain, the volumetric strain and the equivalent shear strain are approximately equal.
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Figure 3.19 Determination of shear modulus in triaxial test for EPS

to the axial strain with an acceptable accuracy. For example, given a Poisson’s ratio of ±0.02, the volumetric strain will range between 0.96ε_a and 1.04ε_a, and the equivalent shear strain will be vary from 0.98ε_a to 1.02ε_a.

Moreover, for isotropic materials (Jean-Pierre Bardet 1997), the $K$, $G$, $E$ and $\nu$ are related as follows,

$$G = \frac{E}{2(1+\nu)} \quad (3.8)$$

$$K = \frac{E}{3(1-2\nu)} \quad (3.9)$$

The above equations show that, when the Poisson’s ratio is close to zero, both the bulk modulus and shear modulus are not very sensitive to the changes of the Poisson’s ratio. Therefore, the shear modulus, $G$ could be approximately determined from the plot of the shear stress, $\frac{1}{2}(\sigma_1 - \sigma_3)$ against the axial strain, $\varepsilon_a$ (Figure 3.19) where,

$$G = \frac{(\sigma_1 - \sigma_3)}{2\varepsilon_a} \quad (3.10)$$
Figure 3.20 Determination of bulk modulus in triaxial test for EPS

Also, the bulk modulus, $K$ was obtained by plotting the mean stress $p$ against the axial strain, $\varepsilon_a$ (Figure 3.20), where

$$ K = \frac{(\sigma_1 + 2\sigma_3)}{3\varepsilon_a} $$

(3.11)

Using the approach described above, the approximate values of initial and plastic tangent moduli obtained from the results of the triaxial compressive tests are summarized in Table 3.3.

Table 3.3. Average $K$ and $G$ parameters of EPS from triaxial tests at room temperature

<table>
<thead>
<tr>
<th>$\rho$ kg/m$^3$</th>
<th>$K_i$, MPa</th>
<th>$G_i$, MPa</th>
<th>$K_p$, kPa</th>
<th>$G_p$, kPa</th>
</tr>
</thead>
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<tr>
<td>13</td>
<td>0.71</td>
<td>1.07</td>
<td>47</td>
<td>71</td>
</tr>
<tr>
<td>20</td>
<td>1.54</td>
<td>2.31</td>
<td>48</td>
<td>72</td>
</tr>
<tr>
<td>27</td>
<td>2.15</td>
<td>3.23</td>
<td>55</td>
<td>83</td>
</tr>
</tbody>
</table>
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3.9. TEMPERATURE CONTROLLED TRIAXIAL TESTS

3.9.1. Introduction

In many areas of the world, including large parts of Australia, it is not uncommon for the ground temperature to rise above 40°C. As the EPS geofoam is a thermo-viscoelasto-plastic material and its mechanical properties are much influenced by temperature, there is a significant interest to understand the behaviour of EPS geofoam under the higher temperature regimes. In view of this interest, temperature controlled triaxial tests were carried out to assess temperature effects on the compressive strength, stiffness and behaviour of EPS geofoam, which are of relevance geotechnically.

In this thesis, the temperature-controlled triaxial tests were performed in the temperature range from 23°C (room temperature) to 50°C. Isothermal conditions at an elevated temperature were achieved by setting up the test equipment in an oven whereby the temperature was automatically adjusted and controlled to a pre-set temperature.

The temperature controlled triaxial test use a similar system and method as the ones described earlier in Section 3.4 except for this case, the entire triaxial test load frame and the ancillary equipment were installed inside a Thermoline oven (Figure 3.21). The oven temperature was digitally set by means of the 3 term P.I.D. temperature controls and a safety thermostat adjusted to the maximum safe allowable temperature (Thermoline, 1995). The load cell, LVDT transducer and temperature sensor were connected to an A/D converter and the triaxial cell was connected to a GDS pressure/volume controller. The load, deformation, and temperature data were electronically recorded during the test, through the A/D converter, by a personal computer. The same computer also controlled the GDS controller as well as logged the volume change of EPS specimen. The entire win32 programs used for data logging and GDS control were developed in-house. A thermometer was also placed in the oven and the water temperature was measured before and after the test to ensure the temperature remained constant throughout the test. A schematic diagram of the test system is shown in Figure 3.22. The details of the temperature controlled test program are listed in Table 3.4.
Figure 3.21 Set up of triaxial test in oven

<table>
<thead>
<tr>
<th>Temperature, °C</th>
<th>Confining pressure, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>23 (room temperature)</td>
<td>0, 5, 10, 15, 20</td>
</tr>
<tr>
<td>30</td>
<td>0, 20</td>
</tr>
<tr>
<td>35</td>
<td>0, 20</td>
</tr>
<tr>
<td>40</td>
<td>0, 20</td>
</tr>
<tr>
<td>45</td>
<td>0, 20</td>
</tr>
<tr>
<td>50</td>
<td>0, 5, 10, 15, 20</td>
</tr>
</tbody>
</table>
3.9.2. Calibration of the Test System

In order to compare the triaxial test results at different temperature, care was taken to ensure that the test system worked consistently at the different temperatures. It was possible that the test system may have needed to be re-calibrated at each temperature level before use. To investigate the temperature sensitivity of the triaxial test system, the load cell and LVDT were calibrated at the limits of the temperature range 23°C (room temperature) and 50°C.

The calibration data of the test system under both room temperature and 50°C are shown in Figure 3.23 and 3.24. It was found that the temperature did not ultimately affect the calibrations for the test system and the system was stable in the temperature range 23 to 50°C. In view of this finding, the same set of calibration parameters was applied for the entire temperature range of the testing.
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Figure 3.23 Calibration of load cell

Figure 3.24 Calibration of LVDT
3.9.3. Test Results

The stress-strain response of the EPS specimen under different temperatures is shown in Figure 3.25. In general, the stress-strain curves are also bilinear and the EPS geofoam yields at 1% to 2% of the axial strain. It also shows that the initial and post-yielding slopes of the stress-strain curves decrease with temperature, albeit only slightly. However, with rising temperature, the EPS geofoam quite obviously yields at a lower stress level, and the compressive strength decreases.

Figure 3.26 shows the compressive strength under the different temperatures at zero confining pressure. The test results indicated that the compressive strength ($\sigma_c$) decreases approximately linearly with increasing temperature. The strength decreases from 105 kPa at room temperature (23°C) to 85 kPa at 50°C, or a decrement of approximately 20%. The rate of decrement is approximately 0.63 kPa per degree Celsius. Linear regression gives the following relationship between the temperature and the compressive strength as,

$$\sigma_c = -0.63T + 118.1$$  \hspace{1cm} (3.12)

where $\sigma_c$ is in kPa and $T$ is in °C.

![Figure 3.25 Strain stress curve in triaxial test ($\sigma_3 = 0$kPa)](image-url)
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\[ \sigma_c = -0.63T + 118.1 \]

Zero confining pressure \((\sigma_3 = 0 \text{ kPa})\)

Figure 3.26 Compressive strength of EPS in triaxial test

\[ E_I = -0.027T + 5.3 \]

\((\sigma_3 = 0 \text{ kPa})\)

Figure 3.27 Initial Young's modulus of EPS in triaxial test
As shown in Figure 3.27, the initial Young's modulus also decreases linearly from 4.9 MPa at room temperature (23°C) to 4.3 MPa at 50°C, where the rate of decrement is approximately 0.02 MPa per degree Celsius. Considering the scatter of the measured results, this rate may be deemed small as pointed out previously. Similarly, the initial bulk and shear moduli of EPS under different temperatures are obtained and the average values of \(E_i, K_i\), and \(G_i\) are given in Table 3.5.

Figure 3.28 shows the compressive strength under different confining pressures at 45°C. Linear regression of the test data gives the following relationship between the confining pressure and the compressive strength at a temperature of 45°C as,

\[
\sigma_c = 0.062\sigma_3 + 87.52
\]  
(3.13)

where the units of \(\sigma_c\) and \(\sigma_3\) are in kPa.

It shows that, at a higher temperature of 45°C, the confining pressure has only a very small effect on the compressive strength of EPS geofoam as evidenced by the small value of the slope of the relationship.

<table>
<thead>
<tr>
<th>Temperature, °C</th>
<th>(E_i), MPa</th>
<th>(K_i), MPa</th>
<th>(G_i), MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>23</td>
<td>4.86</td>
<td>1.62</td>
<td>2.43</td>
</tr>
<tr>
<td>30</td>
<td>4.52</td>
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<td>2.26</td>
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<td>35</td>
<td>4.35</td>
<td>1.45</td>
<td>2.18</td>
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<td>4.33</td>
<td>1.44</td>
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<td>4.26</td>
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<tr>
<td>50</td>
<td>4.25</td>
<td>1.42</td>
<td>2.13</td>
</tr>
</tbody>
</table>
Figure 3.28 Effect of confining pressure on the compressive strength of EPS

Figure 3.29 Monotonic unconfined compression tests on EPS blocks
3.10. OTHER FACTORS INFLUENCING THE RESULTS OF COMPRESSION TESTS

3.10.1. Geometry

As anticipated previously, the mechanical behaviour of EPS geofoam is sensitive to its bulk density. It is likely that the geometry of the specimen may also influence the compression test results. In this thesis, the compression tests were also carried out for a number of specimen sizes including cubic specimens of $50 \times 50 \times 50$ mm and $76 \times 76 \times 76$ mm. The influence of geometry on the stress-strain response, strength and moduli has been investigated where the displacement rate was kept at a constant value of 5 mm per minute. Tests were also conducted for the rectangular prism specimen with different aspect ratios in the following sizes: $76 \times 76 \times 36$ mm, $76 \times 76 \times 76$ mm and $76 \times 76 \times 142$ mm.

Figure 3.29 shows the stress strain curves for different sizes. The compressive strength, initial and plastic Young’s moduli are listed in Table 3.6. The results show that the height of the specimen, which ranges from 37 to 142 mm, has little influence on the strength and stiffness of EPS. However, as the cross sectional area was increased from 1954 to 5868 mm$^2$, the compressive strength and plastic modulus increased, but the initial Young’s modulus remained constant. Figure 3.30 shows the effect of cross sectional area...
on the compressive strength. This result may explain why the compressive strength value from a test on a cylindrical specimen is smaller than that from cubic specimen (Horvath, 1996) since the cylindrical specimen has a smaller cross sectional area.

<table>
<thead>
<tr>
<th>Cross section</th>
<th>Height, mm</th>
<th>Area, mm²</th>
<th>σc, kPa</th>
<th>Eᵢ, MPa</th>
<th>Eₚ, kPa</th>
</tr>
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<tbody>
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<td>Ø49.88</td>
<td>50.1</td>
<td>1955</td>
<td>108</td>
<td>5.85</td>
<td>202</td>
</tr>
<tr>
<td>Ø49.90</td>
<td>51.5</td>
<td>1956</td>
<td>112</td>
<td>6.10</td>
<td>194</td>
</tr>
<tr>
<td>50.50×50.80</td>
<td>50.2</td>
<td>2565</td>
<td>123</td>
<td>5.84</td>
<td>276</td>
</tr>
<tr>
<td>50.66×51.20</td>
<td>50.1</td>
<td>2594</td>
<td>126</td>
<td>6.04</td>
<td>291</td>
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<tr>
<td>76.79×76.27</td>
<td>78.4</td>
<td>5856</td>
<td>136</td>
<td>6.00</td>
<td>299</td>
</tr>
<tr>
<td>76.70×76.97</td>
<td>78.5</td>
<td>5903</td>
<td>144</td>
<td>6.30</td>
<td>314</td>
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<tr>
<td>76.79×76.48</td>
<td>142.7</td>
<td>5873</td>
<td>119</td>
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<tr>
<td>76.69×76.24</td>
<td>141.6</td>
<td>5847</td>
<td>125</td>
<td>6.12</td>
<td>302</td>
</tr>
<tr>
<td>76.72×76.36</td>
<td>110.6</td>
<td>5858</td>
<td>129</td>
<td>5.85</td>
<td>278</td>
</tr>
<tr>
<td>76.69×76.53</td>
<td>110.4</td>
<td>5868</td>
<td>132</td>
<td>5.28</td>
<td>321</td>
</tr>
<tr>
<td>76.77×76.40</td>
<td>37.1</td>
<td>5865</td>
<td>120</td>
<td>5.92</td>
<td>281</td>
</tr>
<tr>
<td>76.59×76.61</td>
<td>37.0</td>
<td>5867</td>
<td>131</td>
<td>5.45</td>
<td>308</td>
</tr>
</tbody>
</table>

### 3.10.2. Displacement Rate

Another factor, which could influence the test results, is the loading rate or speed. Previous studies indicated that the higher the strain rate, the higher the compressive strength (Eriksson and Trank, 1991). The EPS specimens were loaded under displacement rates of 0.5, 1, 2.5, 5 and 10 mm per minute, respectively to determine the loading rate effects. Figure 3.31 shows the stress-strain curves for the different displacement rates. The compressive strength and Young’s moduli are listed in Table 3.7 and shown in Figure 3.32 to 3.34. The results show that the initial Young’s modulus was not affected by the loading rate. On the other hand, the compressive strength and plastic Young’s modulus increased with increasing loading rate. This is especially noticeable at a lower displacement rate. The cause is probably due to the time effect of creep of the EPS specimen. For example, for a displacement rate of 1mm per minute, the loading time up to 10% axial strain is 5 minutes, which is long enough for the creep to have some influence (see later in Chapter 5). For the displacement rate of 5 mm per minute, however, it takes only 1 minute to load to 10% deformation, thus the influence would be negligible.
Figure 3.31 Effect of displacement rate

Figure 3.32 EPS compressive strength vs displacement rate
Figure 3.33 EPS initial Young's modulus versus displacement rate

Figure 3.34 EPS plastic Young's modulus vs displacement rate
Table 3.7 Displacement rate effects on EPS material (ρ = 20 kg/m³)

<table>
<thead>
<tr>
<th>Displacement rate, mm/min</th>
<th>0.5</th>
<th>1.0</th>
<th>2.5</th>
<th>5.0</th>
<th>10.0</th>
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<td>107</td>
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Figure 3.35 EPS specimen under different loading direction (20 kg/m³)

Figure 3.35 shows the stress – strain response of the EPS specimen loaded in two orthogonal directions. The responses in both directions are almost identical suggesting that the EPS can be regarded as an isotropic material.
3.11. SUMMARY AND RECOMMENDATION

3.11.1. Summary

By performing the triaxial and unconfined compression tests on EPS geofoam specimen, the mechanical properties of EPS geofoam under compressive load have been obtained. The following were observed and merit mentioning:

1. The density is an index property of EPS geofoam material. The mechanical properties such as compressive strength, yield stress and initial Young’s modulus of EPS geofoam are primarily dependent and proportional to its density.

2. There was some variability in the density value within EPS geofoam of each density class. Variations in density of ±10% about the mean value were recorded. The difference in compressive strength values can generally be attributed to the variation in bulk density and the inhomogeneity of the material.

3. In a triaxial loading condition, the influence of confining pressure on the EPS geofoam behaviour is very small. The test results for 0 to 20 kPa confining stress indicated that, with increasing confining pressure, the deviator stress remains approximately constant. The compressive strength and the initial Young’s modulus were found to be essentially independent of the confining pressure.

4. The study indicates that the Poisson’s ratio of EPS geofoam is very small in magnitude and ranges from −0.07 to +0.02. After yielding, the Poisson’s ratio becomes approximately constant with an average negative value of −0.02.

5. Generally, in circumstances of quick (short-term) compressive loading within the elastic limit, the temperature does not have a significant influence on the initial Young’s modulus of EPS geofoam. With rising temperature, however, the compressive strength and the yield stress of the EPS geofoam decrease approximately linearly. The decrease rate is approximately 0.63 kPa per degree Celsius.

6. The test results indicated that even at a relatively high temperature of 45°C, the confining pressure has only a very small influence on the compressive strength of the EPS geofoam.

7. The test results for different geometry sizes indicate that the compressive strength increases with an increasing cross sectional area.
8. The loading rate also influences the behaviour of EPS geofoam. The results of testing on EPS specimen at different displacement rates show that the compressive strength and plastic Young’s modulus increase with increasing displacement rate, especially, at the lower displacement rate (lower than 2 mm per minute). For displacement rates of 2.5 to 10 mm per minute, the influence will be negligible.

3.11.2. Recommendations

As the EPS geofoam is used widely in geotechnical engineering applications, it is necessary that the relevant test methods should be standardized in order to determine the properties of EPS for design and analysis. For carrying out a standard triaxial test on the EPS specimen, the following factors should be taken into consideration and standardized:

*The size and geometry of the EPS specimen.* It is recommended that the cylindrical EPS specimen with a diameter of 50 mm and a height of 50 mm be adopted as the standard for soil laboratory testing. It must be noted that the 50 by 50 cylinders are recommended for testing EPS in conventional soil laboratory test because the suitable standard equipment for soil laboratory test requires the cylindrical specimen.

*The loading rate.* From the geotechnical viewpoint, a rate of 5 to 10% of the initial height of the EPS specimen per minute is considered compatible to similar tests for foundation soil and subgrade material.

*The confining pressure.* For the EPS specimen, it is recommended that the confining pressure be limited to not more than 20kPa since higher confining pressures are not realistic in terms of the EPS geotechnical applications.

The method of evaluating the Poisson’s ratio of EPS also should be standardized. The method adopted in this thesis is suggested as one which could be considered as a standard test.
CHAPTER 4

SHEAR BEHAVIOUR OF EPS GEOFOAM

4.1. INTRODUCTION

Evaluation of the shear strength parameters of the associated geotechnical materials constitutes an important and necessary part of the analysis and design process of foundations, retaining walls and earth slopes. The most commonly used geotechnical material is soil. Because all soils are essentially frictional materials, the shear strength within a soil mass is due significantly to the development of frictional resistance between adjacent particles. For clay soils, due to the pore fluid being in tension and so creating a compressive effective stress between the soil particles, apparent cohesion rather than frictional resistance tends to dominate the shear resistance. Prior to employing EPS geofoam in the earlier stated applications, it is essential to understand the relative contributions of the frictional resistance and cohesion to the shear strength of the material. Also, as the geofoam blocks are packed together in applications, the interfacial joint between two blocks may form a natural plane of shear weakness. Shear tests were therefore performed in this thesis to elicit a better understanding of the shear behaviour of the foam in geotechnical applications.

The shear strength parameters, as prescribed by consideration of the principles of soil mechanics, can be obtained from either the triaxial test and the shear box test. In this chapter, the shear behaviour of EPS was investigated by means of the shear box test. The internal shear strength parameters: friction angle and cohesion were evaluated based on the shear box test results and a comparison made with those derived from the triaxial test. In an earlier chapter, the triaxial test of the EPS geofoam had already been described and the test results obtained were employed in this chapter to deduce the shear characteristics. The interface shear resistances of EPS-EPS and EPS-soil were studied as well.
4.2. DEFINITION OF THE SHEAR STRENGTH OF EPS

The ASTM standard D732 – 93, Standard Test Method for Shear Strength of Plastic by Punch Tool, covers the punch-type of shear test. In this standard, the shear strength is defined as the maximum load required to shear the specimen in such a manner that the moving portion has completely cleared the stationary portion.

In such a standard test for 20 kg/m$^3$, it was found that the shear strength of the EPS geofoam is much higher than the compressive strength in magnitude, as shown in a typical result of Figure 4.1 (Preber et al. 1994). Furthermore, the maximum shear stress defined in this test typically corresponds to a shear strain reaching 70%. In geotechnical application, such a high shear strain is unacceptable. Therefore, the shear strength defined by the standard method for shear strength by a punch tool appears not to be suitable for geotechnical applications.

From a geotechnical viewpoint, it is argued that it would be useful and practicable to define the shear strength as the shear stress at some level of shear deformation depending on the application and the interests of the designer. This approach is similar to
that of the compressive strength which is not defined as the maximum stress the EPS can support but the stress at the 10% deformation of the EPS specimen. Thus, it is suggested that the shear strength may also be defined as the shear stress at the 10% shear displacement.

![Graph showing shear strength vs shear displacement](image)

**Figure 4.2 Typical Shear Box Test on EPS specimen ($\rho = 20\text{kg/m}^3$)**

In standard shear box test (direct shear), AS1289.6.2.2 1998 and ASTM D5321 – 92, the shear of the specimen is represented by shear displacement. In this thesis, in order to obtain compatible results to those of EPS in the triaxial loading condition, the shear strength of EPS is defined as the maximum shear stress within 10% shear displacement of the EPS specimen. Because there is not a real “failure” of EPS specimen under compression or shear loading, the 10% shear displacement has been selected, in the same way of 10% compressive deformation in uniaxial and triaxial tests, to define the strength of this material. As observed in Figure 4.2 which shows a typical shear stress – shear displacement response from the shear box test results on the EPS specimen, the EPS geofoam does not have a characteristic peak shear stress value up to the displacement limit of the shear box test. The similarity of triaxial and shear box results indicated this definition is reasonable.
4.3. **STANDARD TEST METHODS**

In this thesis, the shear box test was conducted based on the Australian Standard AS 1289.6.2.2 – 1998, ‘Determination of shear strength of a soil - direct shear test using a shear box’, and the ASTM D5321 – 97, ‘Standard test method for determining the coefficient of soil and geosynthetic or geosynthetic and geosynthetic friction by the direct shear method’. For determining the shear strength of the EPS specimen, the test procedures prescribed in AS 1289.6.2.2-1998 were followed, and for determining the shear resistance of the EPS against the soil and another EPS block, the procedures of the ASTM D5321 – 97 were employed.

4.4. **APPARATUS**

The shear box consisted of two separate halves which move in a horizontal plane relative to each other, thus shearing a test specimen along a predetermined horizontal plane. The apparatus is instrumented so that the vertical movement of the upper loading plate and the relative horizontal movement of the top and bottom halves of the box (the shear displacement) can be measured.

A typical standard shear box test setup is shown in Figure 4.3, where the internal dimension of the box is 60 mm × 60 mm × 40 mm (height). The upper half box measures 60 mm × 60 mm × 19 mm (height), and the lower half box is 60 mm × 60 mm × 21 mm (height) (Wykeham Farrance, 1995).

The device for applying the normal force is a hanger loaded with the appropriate dead weights. The hanger applies the normal load centrally to the upper loading plate of the shear box through a ball bearing in a spherical seating.

The device measuring the shear force applied to the specimen consists of a proving ring mounted between the loading frame and the upper half of the shear box, with a dial gauge (precision of ±0.002 mm) measuring the compression of the ring. The load is obtained from the measured ring compression after factoring by a calibration constant.

A dial gauge instrument is attached to measure the horizontal displacement of the lower half of the box with respect to the upper half of the shear box to a precision of ±0.01 mm and has a travel of at least 15 mm. No correction was allowed in the measurement of
the horizontal displacement for the component due to the compression of the proving ring. A dial gauge is employed for reading the vertical displacement of the upper loading plate of the shear box to a precision of ±0.002 mm and it has a travel of at least 12 mm.

Figure 4.3 Standard shear box test apparatus

Figure 4.4 Schematic diagram of shear box test
CHAPTER 4: SHEAR BEHAVIOUR OF EPS MATERIAL

4.5. TEST PROGRAM

4.5.1. General

In this study, the following four types of shear tests were conducted to determine:

1. Internal shear resistance of EPS material
2. Interface friction between EPS blocks
3. Interface friction between EPS block and compacted sand
4. The internal friction of dense sand (baseline case for the purpose of comparison)

4.5.2. Internal Shear Resistance

To determine the internal shear strength of 20kg/m$^3$ EPS geofoam, a 60 mm $\times$ 60 mm $\times$ 40 mm rectangular prismatic specimen was carefully cut from a large EPS block and fitted into the shear box. With the halves of the box held together, the specimen was sandwiched within the box between rigid metal plates and a pressure pad was placed on the top (Figure 4.4). This box was placed in a larger outer box, which runs horizontally on roller bearings. A vertical load was applied to the specimen by means of a static weight hanger carrying the designated load as described previously. The specimen was sheared by applying a horizontal force with a screw jack at a constant strain rate of 1.0 mm per minute. All the tests continue until the 12 mm shear displacement which corresponds to approximately 20% shear displacement. The shear stress at 10% shear displacement (i.e. 6 mm shear displacement in sample length of 60 mm) was taken as the shear strength.

The shear box tests were repeated for several different values of normal loads by adding weights to give the equivalent initial normal stresses of approximately 5, 10, 20, 30 and 40 kPa. No allowance was made for the weight of the upper half of the shear box when calculating the normal stress. The values of the initial normal stress are within the expected normal stress range of application as well as the elastic range of this density type. A typical calculation data sheet is shown in Table 4.1.
Table 4.1 Typical calculation data sheet of Shear Box Test

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<th>Shear Box Test on EPS material</th>
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*modified area = initial area – horizontal displacement × initial width
4.5.3. Interface Shear Resistance

The test proceeded in the same manner as the internal shear test. However as the specimen preparation was more difficult, care was needed to run the test.

*EPS – EPS Test*

The interface resistance between EPS blocks was determined by fitting an EPS specimen block into the lower box and another specimen into the upper half shear box. Two 60 mm × 60 mm × 20 mm rectangular prismatic specimens were carefully cut from a large EPS block. The test interfaces of the two specimens were smooth and parallel to each other. The lower EPS specimen was placed in the lower half shear box. This specimen protruded above the upper face of the lower half box. The upper EPS specimen was placed over the previous lower specimen so that both specimens were in complete contact within the test area. The upper box was then positioned so that the EPS specimen protruded below the lower face of upper half box such that only the two specimens were in contact over the interface area.

*EPS – Sand Test*

For the EPS–Sand test, the lower half box was replaced with sand. The sand specimen selected in this study was Nepean sand (silica sand). It has a uniformity coefficient of 1.5 where sizes of $D_{10}$ and $D_{60}$ are approximately 0.25mm and 0.38mm respectively. Nepean sand is classified as a well graded sand with less than 1% of fines. The dry density of the loose sample was 1.44 t/m$^3$, while the dry density of the compacted sample was 1.67 t/m$^3$.

The loose sand specimen was placed in the lower half box and compacted to the desired density. The lower box was aligned so that the surface of the soil specimen protruded about 1mm above the upper face of lower box. The upper EPS specimen was placed over the previously prepared sand specimen so that both specimens were in complete contact within the sheared area. The upper box was positioned to allow the EPS specimen to protrude below the lower face of the upper half box and such that only the two specimens were in contact over the interface area.
Figure 4.5 Typical shear box test on EPS block ($\rho = 20\text{kg/m}^3$)

Figure 4.6 Internal shear strength of EPS geofoam ($\rho = 20\text{kg/m}^3$)
4.6. TEST RESULTS

4.6.1. General

The shear box tests were carried out to determine the internal shear parameters and interface friction coefficient of EPS material with a specified density of 20kg/m³. The shear behaviour of EPS material under various normal stress conditions within the expected stress range of application was investigated.

4.6.2. Internal Shear Resistance Test for EPS

The typical shear box test results are shown in Figure 4.5 and the shear strength and corresponding normal stress are plotted in Figure 4.6 in which the shear strength is defined as the maximum shear stress at 10% of shear displacement. The shear strength envelopes are fitted as the best straight lines in a shear strength - normal stress plot. Linear regression for the data obtained in the shear box test on EPS internal friction yielded the following Mohr-Coulomb relation:

\[ \tau = 0.11 \sigma_n + 42.6 \]  \hspace{1cm} (4.1)

where,\n
\[ \tau = \text{shear strength (kPa)} \]
\[ \sigma_n = \text{normal stress corresponding to } \tau \text{ (kPa)} \]

The slope of this straight line is the internal friction and the \( \tau \)-intercept is the bonding adhesion of the EPS particles. Thus, the shear strength of EPS can be expressed by internal friction angle, \( \phi \), and cohesion, \( c \). For 20 kg/m³ EPS material, it gives:

\[ \phi = \tan^{-1}(0.11) = 6.4^\circ \] \hspace{1cm} (4.2)
\[ c = 42.6 \text{ kPa} \] \hspace{1cm} (4.3)

This result indicated that the internal shear resistance of the EPS mostly depends on the internal cohesion of the bonded particles, and the internal friction angle of the EPS is very small (6.4°) i.e. the normal stress has only a small influence on the shear resistance.

The vertical displacements are also plotted against the shear displacement (Figure 4.7). It is found that the vertical displacement depends on the normal stress levels. Under a small vertical stress, the thickness of the EPS specimen increases slightly with increasing shear displacement (dilation), and at a high normal stress, the thickness of the EPS
specimen decreases slightly (compression). This behaviour is likely similar to that of soil. The explanation proposed is that during shearing, the EPS may slightly dilate due to the particles moving apart. However, at a lower normal stress the compression of the specimen is smaller than the dilation thus a net dilation results. On the other hand, at a higher normal stress level, the compression of the EPS is higher than the dilation of the specimen, thus a net compression occurs. Overall, the variation of the height of EPS during the shearing is negligible.

4.6.3. Interface Shear Friction of EPS Blocks (EPS – EPS)

The frictional resistance on the interface of the two specimen is represented by the interface friction or coefficient of friction, which is defined as ratio of shear stress to normal stress, $\tau/\sigma_n$ (AS1289.6.2.2 1998).

The shear stress of EPS – EPS interface versus shear displacement are shown in Figure 4.8. The shear strength $\tau$ is again defined as the maximum shear stress up to 10% shear displacement. The shear strength values versus corresponding normal stress are shown in Figure 4.9. It is found that the shear strength of the EPS-EPS interface increases nonlinearly with increasing normal stress. Regression analysis of the data yields the following relationship between $\tau$ and $\sigma_n$: 

- 65 -
\[ \tau = -0.013\sigma_n^2 + 1.15\sigma_n \]  

(4.4)

where

\(\tau\) = shear strength of EPS – EPS interface (kPa)

\(\sigma_n\) = normal stress (kPa)

Figure 4.8 Interface shear stress of EPS blocks versus shear displacement

Figure 4.9 EPS-EPS interface shear strength envelope in shear
The interface friction of the EPS-EPS is found to be stress dependent which decrease with increasing normal stress. The interface friction angle of EPS-EPS is

\[ \tan \phi = \frac{d\tau}{d\sigma_n} = 1.15 - 0.013\sigma_n \quad (4.5) \]

However, for normal stress levels lower than 20 kPa, an approximately linear relationship may be given by

\[ \tau = 1.28\sigma_n \quad (4.6) \]

Then, the interface friction angle of EPS-EPS is obtained as,

\[ \phi = \tan^{-1}(1.28) = 52.1^\circ \]

4.6.4. Interface Shear Friction of EPS and Compacted Nepean Sand (EPS – Sand)

The shear stress of EPS – sand interface versus shear displacement are shown in Figure 4.10, and the shear strength values versus the corresponding normal stress are shown in Figure 4.11. It was found that the shear strength of the EPS-sand interface increases linearly with increasing normal stress. The regressed line for the shear strength envelope gives the following relationship between \( \tau \) and \( \sigma_n \):

\[ \tau = 0.58\sigma_n \quad (4.7) \]

where

\( \tau = \) shear strength of EPS – sand interface (kPa)

\( \sigma_n = \) normal stress corresponding to \( \tau \) (kPa)

The friction angle of EPS – sand interface was found as:

\[ \phi = \tan^{-1}(0.58) = 30.1^\circ \]
Figure 4.10 Shear box test on 20kg/m$^3$ EPS and sand interface

Figure 4.11 The envelope of EPS-Sand interface shear strength

\[ \tau = 0.58 \sigma_0 \]
\[ \phi = 30.1 \text{ (degree)} \]
Figure 4.12 Shear Box Test on compacted sand ($\rho = 16.2\text{kN/m}^3$)

Figure 4.13 Shear strength envelopes of EPS, EPS-EPS, EPS-compacted sand and compacted sand
Table 4.2 summary of shear box test results

<table>
<thead>
<tr>
<th>Material</th>
<th>Internal angle of friction</th>
<th>Interface friction angle</th>
<th>Adhesion, kPa</th>
<th>Interface friction coefficient</th>
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<td>6.4°</td>
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<td>42.6</td>
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<tr>
<td>EPS – EPS</td>
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<td>—</td>
<td>1.28</td>
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<tr>
<td>(σn &lt; 20kPa)</td>
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<tr>
<td>Nepean Sand</td>
<td>40°</td>
<td>—</td>
<td>0</td>
<td>—</td>
</tr>
<tr>
<td>(peak)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nepean Sand</td>
<td>33°</td>
<td>—</td>
<td>0</td>
<td>—</td>
</tr>
<tr>
<td>(residual)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**4.6.5. Shear Behaviour of EPS and Compacted Nepean Sand**

For the purpose of comparison, the shear box test was also performed to obtain the internal friction of compacted sand. The shear stress versus shear displacement, Figure 4.12, shows that the compacted sand has a peak and residual shear strength which is unlike the shear behaviour between compacted sand and EPS.

The summary of shear box test results is listed in Table 4.2. In order to compare the shear resistance of EPS and compacted sand, a combined plot Figure 4.13 shows the shear strength envelopes of the EPS, the compacted sand, the interface of EPS and EPS, and the interface of EPS and sand for the normal stress range of 0 to 40 kPa. It shows that the internal shear resistance of EPS is highest and increases from 42.6 to 47.0 kPa. The interface shear resistance of EPS-EPS increases from 0 to 41.2 kPa with increasing normal stress. The internal shear resistance of the compacted Nepean Sand, and the interface shear resistance of EPS-Sand increase linearly with increasing normal stress, and the maximum shear resistances are 34.0 kPa and 23.2 kPa respectively.

These results indicate that in geotechnical application of EPS geofoam under shear loading, the EPS itself has a fairly high resistance to shear failure. It appears that the interfacial joint between the blocks, especially the joint between EPS and sand or other soil forming a shear weakness plane, would more likely give rise to shear failure.
4.6.6. Shear Strength Parameters from Triaxial Test

The shear strength parameters determined in this section are based on the triaxial test discussed in an earlier chapter.

In the triaxial loading condition, the stress state can be represented by the Mohr circle and this can also be related to a failure criterion (Whitlow 1995). The coordinates of the maximum shear stress point of a Mohr circle are given by:

\[
 s' = \frac{1}{2}(\sigma_1 + \sigma_3) \\
 t' = \frac{1}{2}(\sigma_1 - \sigma_3)
\]  

(4.8)

At the point of failure, the Mohr circle touches the Mohr-Coulomb failure envelope and thus an alternative failure criterion gives

\[
 t' = a' + s' \tan \alpha'
\]  

(4.9)

The parameters of this stress point failure envelope, \(a'\) and \(\alpha'\), are related to those of the Mohr-Coulomb criterion as follows:

\[
 \sin \phi = \tan \alpha' \\
 c \cos \alpha' = a'
\]  

(4.10)

Figure 4.14 shows the stress points at failure (10% axial strain) of the triaxial test results of 20kg/m\(^3\) EPS. The best fit line gives the alternative failure envelope and the parameters of shear failure are given in Table 4.3.

The internal shear strength envelopes of EPS based on the test results of shear box and triaxial tests are plotted in shear stress–normal stress space as shown in Figure 4.15. It is observed that in the normal stress range of 0 to 40 kPa, the shear resistance results are approximately the same. This indicates that compatible results are obtained from triaxial and shear box tests.

| Table 4.3. Shear strength parameters of EPS material from triaxial tests |
|---|---|
| Triaxial test (\(t', s'\)) | Shear box |
| \(\alpha^c\) | \(a'\) | \(\phi\) | \(c\) (kPa) |
| 8.5 | 40.0 | 6.4 | 42.6 |
| \(\phi\) | \(c\) (kPa) | 8.6 | 40.5 |
CHAPTER 4: SHEAR BEHAVIOUR OF EPS MATERIAL

Figure 4.14 Stress Point ($t', s'$) shear strength envelope of EPS in triaxial test

\[ t' = 0.15s' + 40.0 \]

\[ \rho = 20 \text{ kg/m}^3 \]

\[ s = (\sigma_1 + \sigma_3)/2, \text{ kPa} \]

Figure 4.15 Shear resistance of EPS from shear box and triaxial tests
4.7. SUMMARY

The behaviour of EPS geofoam under shear load has been studied based on the shear box test and the triaxial test. Like those in uniaxial and triaxial tests, the definition of failure in shear box is not a real “failure”. The shear strength parameters are terms used to represent the shear resistance (or shear strength) of the material when the specimen was sheared to 10% deformation. The results are summarized as follow:

1. The test results, from both the shear box and the triaxial test, indicate that the shear strength of EPS geofoam is derived mainly from cohesion or adhesion of the EPS particles (about 42 kPa for 20kg/m³ EPS).

2. The internal friction angle of 20kg/m³ EPS is very small with an average value of 8.6° from triaxial test and 6.4° from shear box test. This result indicated that the internal shear friction seems not to be significantly influenced by the normal stress.

3. The friction angle of the interface of the EPS blocks is relatively high and decreases with increasing normal stress. Test results indicated that the interface friction angles of the 20kg/m³ EPS blocks are 52.1° for normal stress lower than 20 kPa. For normal stress of 20 to 40 kPa, the interface friction angle is approximately 33.7°, which is equivalent to a friction coefficient of 0.67.

4. The interface friction angle of the EPS and compacted Nepean sand is 30.1°, which is equivalent to a friction coefficient of 0.52.

5. Under shear loading, it seems that for the EPS itself, shear failure in geotechnical applications will not be a problem. However, the interfacial joint between the blocks, especially the joint between EPS and sand or other soil, may form a shear weakness plane which could give rise to a problem of shear failure.
CHAPTER 5
CREEP BEHAVIOUR OF EPS UNDER COMressive LOADING

5.1. INTRODUCTION

EPS materials exhibit a viscous behaviour and therefore suffer from permanent deformation under sustained load (static) and repeated loading (dynamic) even at relatively small stress levels. When EPS geofoam is used as an ultra-lightweight filling material in embankments and as a load-bearing subgrade layer on very soft soils, it is subjected to sustained surcharge load imposed by the self-weight of the overlying material. The geofoam will creep with elapsing time and the extent to which it occurs will affect the performance of the geotechnical use of the geofoam.

In this chapter, a series of creep tests were carried out under a conventional laboratory environment to study the time-dependent behaviour under compressive loading. Several creep models were evaluated to select the appropriate ones which are applicable to EPS applications in geotechnical engineering.

A main consideration was to characterize the creep behaviour of EPS in the elastic range of applied loading, and to find proper creep models for EPS geofoam within the working load of geotechnical applications, which is normally less than 40kPa. However, the higher stress levels of 70 and 90 kPa were also investigated to obtain a more complete insight into the creep behaviour of this material.

The creep tests were carried out at room temperature and at 40°C, the latter in order to examine the creep response of EPS geofoam under temperature conditions that are higher than room temperature.
5.2. TEST METHODS AND APPARATUS

ASTM D 5202 – 91, Standard Test Method for Determining Triaxial Compression Creep Strength of Chemical Grouted Soils, covers the determination of the long term strength and deformation of cylindrical specimen in undrained compression under constant sustained load. The general method and procedures of the creep tests in this study follow the above ASTM standard where applicable.

The tests, however, were not performed in a triaxial cell primarily because, as discussed in Chapter 3, the influence of confining pressure on the compressive behaviour of EPS is negligible. The setup of the creep test is shown in Figure 5.1. The specimen was situated on a rigid beam and topped by a loading cap. The hanger laden with the appropriate weights applied the normal load centrally to the loading cap through a ball bearing in a spherical seating. A dial gauge attached to the cap measured the vertical displacement to a precision of ±0.01 mm and it had a travel of at least 15 mm.
The creep test at a temperature above the room temperature was carried out under controlled conditions in a oven using a slightly different arrangement as shown in Figure 5.2. The cast-iron weights were used to apply dead load which rests on a top cap to ensure the load is evenly distributed over the area of the test specimen. A dial gauge is securely fixed to a stand and is positioned to measure the deflection of the center of the top cap. The EPS specimen was set inside a steel casing that is capped at the bottom to avoid overbalancing the weights.

5.3. **TEST PROGRAM**

The series of creep tests was carried out on cylindrical samples of 20kg/m³ EPS geofoam with dimension of Ø50 × 50 mm. The EPS specimens in the creep test were subjected to compressive loading for a range of vertical stresses: 20, 30, 40, 50, 60, 70, and 90 kPa. Stress values from 20 to 60 kPa are considered to be within the elastic range of EPS of this density. The stress of 70 kPa is judged to be the beginning of yielding and 90 kPa to be about the end of yielding.
CHAPTER 5: CREEP BEHAVIOUR OF EPS UNDER COMPRESSIVE LOADING

The displacement reading was taken at the following time intervals after the start of the loading: 0.25, 0.5, 1, 2, 4, 8, 16, 30, 60 minutes, then every hour for four hours, every day for the first ten days, then every three to four days till 90 days. After that, a reading was taken every 15 days or when there was a significant change in displacement. The strain reading at 0.25 minutes was regarded as the immediate strain.

Being a thermo-viscoelastoplastic material, the mechanical properties of EPS geofoam are affected by temperature. In this thesis, the creep tests were also carried out at a temperature of 40°C to investigate the creep behaviour of EPS geofoam under a temperature condition higher than the room temperature.

The duration of the creep tests ranges from 4 months to 15 months. The details of the creep test program are tabulated in Table 5.1.

<table>
<thead>
<tr>
<th>Applied vertical stress, kPa</th>
<th>Temperature, °C</th>
<th>Duration, hours</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>+23</td>
<td>7344</td>
</tr>
<tr>
<td>30</td>
<td>+23</td>
<td>2954</td>
</tr>
<tr>
<td>40</td>
<td>+23</td>
<td>9912</td>
</tr>
<tr>
<td>50</td>
<td>+23</td>
<td>8904</td>
</tr>
<tr>
<td>60</td>
<td>+23</td>
<td>11304</td>
</tr>
<tr>
<td>70</td>
<td>+23</td>
<td>3576</td>
</tr>
<tr>
<td>90</td>
<td>+23</td>
<td>2954</td>
</tr>
<tr>
<td>30</td>
<td>+40</td>
<td>10560</td>
</tr>
<tr>
<td>40</td>
<td>+40</td>
<td>10560</td>
</tr>
<tr>
<td>50</td>
<td>+40</td>
<td>10560</td>
</tr>
</tbody>
</table>
5.4. TEST RESULTS

In general, the time-dependent deformation can be represented by two separate parts, i.e., the immediate response and the time-dependent component. Thus the total strain of a specimen under a given load can be written as:

\[ \varepsilon = \varepsilon_0 + \varepsilon_c(t) \]  \hspace{1cm} (5.1)

where

- \( \varepsilon \) = total strain at time \( t \) after a stress application
- \( \varepsilon_0 \) = the immediate strain upon a stress application
- \( \varepsilon_c(t) \) = creep strain at time \( t \)

Figure 5.3 shows typical creep test results of the total strain, \( \varepsilon \), versus elapsed time, \( t \). In general, the total strain increases sharply in the first few hours after a stress application (Figure 5.4), but increases more gradually at large values of elapsed time. For stress levels
Figure 5.4 Creep of EPS in the first few hours after load application

within the elastic range, i.e. stress level less than 60 kPa, an approximately linear relationship exists between \( \log(\varepsilon) \) and \( \log(t) \) as shown in Figure 5.3. When the stress level increases above the yield stress, i.e. 90 kPa in this case, the variation of total strain with time is non-linear even in a log-log plot.

Figure 5.5 shows the time-dependent (creep) component of the strain, \( \varepsilon_c = \varepsilon - \varepsilon_0 \), versus time using a log-log plot, and the creep strain of the EPS specimen under different temperature conditions is shown in Figure 5.6. The experimental evidence indicates that the creep strain of EPS increases with increasing temperature. The effect of temperature increases as the level of applied stress increases, and becomes quite significant when the applied stress level is higher than 40 kPa.
Figure 5.5 Creep strain of EPS ($\rho = 20$ kg/m$^3$)

Figure 5.6 Total strain of EPS under different temperature
CHAPTER 5: CREEP BEHAVIOUR OF EPS UNDER COMPRESSIVE LOADING

Figure 5.7 Sherby-Dorn plot of EPS at 23°C ($\rho = 20$ kg/m$^3$)

Figure 5.8 Sherby-Dorn plot for different temperature (for 14-month creep)
CHAPTER 5: CREEP BEHAVIOUR OF EPS UNDER COMPRESSIVE LOADING

Figure 5.7 shows the strain rate on a log scale against the creep strain on a normal scale (Sherby-Dorn plot), and a comparison at different temperature levels is shown in Figure 5.8. In a Sherby-Dorn plot, a stress level would be considered to be stable in the long term if the strain rate decreases with time and to be long term unstable if the strain rate increases with time. A transitional state would be between the stable and unstable where the strain rate will be constant with time (Horvath 1995). From all the creep data in Sherby-dorn plot, it is found that, at room temperature, the EPS material is generally long-term stable when the stress levels are low, where the creep rate continues to decrease with time. For stress level less than 40 kPa, the magnitude of the strain rates are very small and decrease with elapsed time. However, as the stress level increases above 50 kPa, the strain rate increases notably with increasing applied stress. The curves of 60 kPa and 70 kPa suggest a transitional but long term stable condition where the creep rate eventually becomes constant with time.

At a temperature of 40°C, for stress levels less than 40 kPa, the creep rate of EPS material still continues to decrease with time, which indicates long-term stable creep behaviour. However, for stress levels of 50kPa at 40°C, the strain rate decreases initially and then stabilizes suggesting a transitional but long term stable condition.

5.5. CREEP MODELS

5.5.1. Overview

A primary objective of the laboratory creep tests is to establish appropriate constitutive models for prediction and analysis of the time-dependent behaviour of EPS material. Theoretically, the immediate strain component, $\epsilon_0$, can include both elastic (recoverable) and plastic (non-recoverable) parts (Horvath 1998). With regard to the creep component, $\epsilon_c$, previous investigators (Findley and Khosla 1956, and Findley 1960) have used arithmetic functions of time to model the time-dependent strain behaviour. These functions are generally categorized in the following manner (Findley et al., 1976):

(i) an arbitrary mathematical curve fitting to a given set of data
(ii) the mathematical description of a relatively simple, abstract physical model composed of various combinations of mechanical elements such as springs and dashpots
(iii) a rigorous theoretical treatment deriving from observed physical and assumed rheological behaviour of a given material.

The well known power law – Findley equation belongs in category (i) and the class of such rheological mechanical models as proposed by Kelvin, Maxwell, Burger and others belong in category (ii), according to the above classifications. Models derived on the basis of micromechanics approach where the material behaviour is investigated at two different scale levels, the micro-scale and the macro-scale level, would belong in the last category.

Models in the last category are deemed outside the scope of this thesis. Instead, the thesis will focus on the functions of category (i) and (ii). To commence the discussion, a selection of those functions, which will be pursued in this thesis, are produced for elaboration below. More specifically, the mechanical model of category (ii) will be developed within the ambit of the visco-elastic theory concerning materials such as EPS which exhibit strain rate effects in the material deformation.

5.5.2. Power Law – Findley Equation

Findley (Findley and Khosla, 1956) assumed the creep strain is given by,

\[
\varepsilon_c = m \left( \frac{t}{t_0} \right)^{n_F}
\]  
(5.2)

where, \( m \) = stress dependent dimensionless material parameter from the test data, 
\( n_F \) = Findley material parameter from the test data (dimensionless),
\( t \) = elapsed time after stress application in hours, and 
\( t_0 \) = one hour (to normalize the elapsed time)

Therefore, the total strain can be simply represented by:

\[
\varepsilon = \varepsilon_0 + m \left( \frac{t}{t_0} \right)^{n_F}
\]  
(5.3)

The parameter \( m \) and \( n_F \) are determined from the creep test results. In a log-log plot of creep strain (\( \varepsilon_c = \varepsilon - \varepsilon_0 \)) versus time (\( t \)), the parameter \( m \) is the magnitude of creep strain at time \( 10^0 \) (=1) hour and \( n_F \) is the slope of the best-fit line. Findley’s equation was reported to be an excellent model of the time-dependent behaviour of a wide range of plastics and composite materials (Chambers 1984).
Findley (Findley and Khosla 1956) further generalized the relationships for \( \varepsilon_0 \) and \( m \) as hyperbolic sine (sinh) functions related to applied stress. The complete Findley's equation gives:

\[
\varepsilon = \varepsilon_{0F} \sinh \left( \frac{\sigma}{\sigma_{eF}} \right) + m_F \sinh \left( \frac{\sigma}{\sigma_{nF}} \right) \left( \frac{t}{t_0} \right)^{n_F}
\]

(5.4)

where

\( m_F \) = a dimensionless Findley material parameter,
\( \varepsilon_{0F} \) = a dimensionless Findley material parameter,
\( \sigma_{eF} \) = a Findley material parameter with dimension of stress,
\( \sigma_{nF} \) = a Findley material parameter with dimension of stress, and
\( \sigma \) = the applied stress

All Findley material parameters (\( n_F, m_F, \varepsilon_{0F}, \sigma_{eF}, \) and \( \sigma_{nF} \)) were assumed to be:

1. material dependent;
2. stress, strain, and time independent (for the stress-strain range over which the material parameters are determined); and
3. dependent on various environmental factors such as temperature and water content of the material. It is therefore necessary to specify the prevailing environmental conditions, particularly the temperature, under which the data were obtained.

In the application of this equation, it may be observed that the sinh term approaches the value of its argument if the magnitude of the argument does not exceed one (Findley 1960). Therefore, in the specialized case where \( \sigma / \sigma_{eF} \leq 1 \) and \( \sigma / \sigma_{nF} \leq 1 \), Findley's equation reduces to a linear function of the applied stress:

\[
\varepsilon = \varepsilon_{0F} \sigma + m_F' \sigma \left( \frac{t}{t_0} \right)^{n_F}
\]

(5.5)

where

\[
\varepsilon_{0F} = \frac{\varepsilon_{0F}}{\sigma_{eF}}
\]

\[
m_F' = \frac{m_F}{\sigma_{nF} t_0^{n_F}}
\]

The use of equation (5.5) has the advantage of the material parameter values being significantly easier to estimate because of the linear relationship.
5.5.3. Viscoelastic Mechanical Models

Experimental evidence suggests that EPS geofoam deforms in a way common to viscoelastic materials which is characterized by:

(i) Instantaneous elasticity
(ii) Creep under constant stress
(iii) Stress relaxation under constant strain
(iv) Instantaneous and delayed recovery
(v) Permanent set of deformation resulting from applied stress.

In a general form, the viscoelastic constitutive relationship of EPS material may be represented by a convolution integral relationship (e.g. Findley et al. 1976) gives,

\[ \varepsilon(t) = \varepsilon_0(t) + \int_0^t J(t-\tau) \frac{\partial \sigma(\tau)}{\partial \tau} d\tau \]  \hspace{1cm} (5.6a)

i.e.

\[ \varepsilon_0(t) = \int_0^t J(t-\tau) \frac{\partial \sigma(\tau)}{\partial \tau} d\tau \]  \hspace{1cm} (5.6b)

where

- \( \varepsilon_0(t) \) = \( J_0 \sigma(t) \) is the initial instantaneous elastic strain,
- \( J_0 \) = time-independent (elastic) compliance
- \( J(t) \) = the time-dependent creep compliance

The application of this constitutive equation is of course limited to one dimensional deformations or simple shear. The generalization of the equation to two and three spatial dimensions can be approached on the basis of similarity between viscoelasticity and elasticity. However, it resides outside the scope of research of this thesis and will be left as a consideration for future research.

In observing equation (5.6), it is also evident that the creep compliance, \( J(t) \), represented in this general form, should be able to accommodate a function of choice which is determined from experimental data.

It is also useful to express the integral relationship (5.6) in an alternate form by performing the integration by parts thus giving,

\[ \varepsilon(t) = \varepsilon_0(t) - \int_0^t J(t-\tau) \frac{\partial \sigma(\tau)}{\partial \tau} d\tau \]  \hspace{1cm} (5.7)

In the case where a constant stress \( \sigma_0 \) is applied, it thus follows from the above equation (5.7) that,
where $\varepsilon_0$ is now the instantaneous (elastic) stress arising from the application of $\sigma_0$. Evaluating and selecting the appropriate creep compliance functions $J(t)$ from some given sets of experimental creep data can best proceed on the basis of equation (5.8).

An evaluation of the viscoelastic mechanical models has been made to select a suitable compliance relationship that is considered applicable to EPS geofoam. Mechanical models of various forms are already widely used to model a number of material and has been described as being suitable for polystyrene material. These models are made up of combinations of (Hookean) springs and (Newtonian) dashpots as exemplified by the well known Maxwell and Kelvin models.

The three-element model (Figure 5.9), where a Maxwell model and a Kelvin model are connected in series, has the following strain response relationship under a constant applied stress $\sigma_0$: 

\[ \varepsilon(t) = \varepsilon_0 + \sigma_0 J(t) \]  

(5.8)
\[ \varepsilon(t) = \frac{\sigma_0}{E_1} + \frac{\sigma_0}{E_2} \left( 1 - e^{-\frac{E_2 t}{\eta_2}} \right) \] (5.9)

and the compliance equation for creep gives,

\[ J(t) = \frac{1}{E_2} \left( 1 - e^{-\frac{E_2 t}{\eta_2}} \right) \] (5.10)

where

\( \sigma_0 \) = constant stress level,
\( E_1 \) = spring constant 1,
\( E_2 \) = spring constant 2,
\( \eta_2 \) = viscous constant 2.

The four-element or Burgers model (Figure 5.10) is simply the three-element model in series with a dashpot. The strain response for a constant applied stress \( \sigma_0 \) is given by,

\[ \varepsilon(t) = \frac{\sigma_0}{E_1} + \frac{\sigma_0}{\eta_1} \frac{t}{\eta_1} + \frac{\sigma_0}{E_2} \left( 1 - e^{-\frac{E_2 t}{\eta_2}} \right) \] (5.11)

and the compliance equation for creep gives,

\[ J(t) = \frac{1}{\eta_1} t + \frac{1}{E_2} \left( 1 - e^{-\frac{E_2 t}{\eta_2}} \right) \] (5.12)

where \( \eta_1 \) = viscous constant 1.

The modified four-element model, where the strain response to a step function stress input is shown in Figure 5.11, is a promising model for capturing the creep characteristics of EPS geofoam. The modification to the four-element model is that the dashpot 1 (\( \eta_1 \)) is assumed to be nonlinearly related to the time \( t \) (i.e. \( \eta_1(t) \)). The strain equation for a constant stress \( \sigma_0 \) applied at \( t = 0 \) is (Taylor et al, 1997),

\[ \varepsilon_z(t) = \frac{\sigma_0}{E_1} + \frac{\sigma_0}{\eta_1} t^n + \frac{\sigma_0}{E_2} \left( 1 - e^{-\frac{E_2 t}{\eta_2}} \right) \] (5.13)

where \( n \) is a creep parameter and the compliance equation for creep gives,

\[ J(t) = \frac{1}{\eta_1} t^n + \frac{1}{E_2} \left( 1 - e^{-\frac{E_2 t}{\eta_2}} \right) \] (5.14)

The dashed lines are asymptotes of the strain equations. In Fig 5.9, the asymptote is a constant strain, in Fig 5.10, it is a straight line and in Fig 5.11, it is a power law curve.
Figure 5.10 Four Element Model

Figure 5.11 Strain response to a step function input for a modified four-element model
5.5.4. Nonlinear Regression Program

In this thesis, the creep model is developed for geofoam applications where the stress loading is less than 40 kPa. This stress range typically occurs in representative conditions in a road sub-base and in embankments where EPS fills are being used. Creep data from the laboratory creep tests were normalized with respect to the applied stress, i.e. $\varepsilon/\sigma_0$, and are shown in Figure 5.12. In determining the value of the material parameters of the compliance function, the techniques of nonlinear regression analysis were employed.

The creep strain parts in sections 5.5.2 and 5.5.3 may be rewritten in the following forms for convenience in programming:

Power law model:  
$$\varepsilon_c(t) = \theta_1 (t)^{\theta_4}$$  \hspace{1cm} (5.15)

3-element model: 
$$\varepsilon_c(t) = \theta_1 [1 - \exp(-\theta_3 t)]$$  \hspace{1cm} (5.16)

4-element model: 
$$\varepsilon_c(t) = \theta_1 [1 - \exp(-\theta_3 t)] + \theta_4 t$$  \hspace{1cm} (5.17)

Modified 4-element model: 
$$\varepsilon_c(t) = \theta_1 [1 - \exp(-\theta_3 t)] + \theta_4 t^\theta_6$$  \hspace{1cm} (5.18)

where for Findley’s equation, $\theta_3 = m'F$, $\theta_4 = n_F$ as in equation (5.5).

For 3, 4 and modified 4 element models, $\theta_1 = 1/E_2$, $\theta_2 = E_2/\eta_2$, $\theta_3 = 1/\eta_1$, $\theta_4 = n$.

A nonlinear regression program was developed using the least squares minimization routine RNLIN from the IMSL library to analyze the experimental data of all creep tests. RNLIN is based on the MINPACK routines LMDIF and LMDER by More et al. (1980), and uses the modified Levenberg-Marquardt method (Levenberg 1944, Marquardt 1963, and Dennis and Schnabel 1983) to generate a sequence of approximations to a minimum point. Routine RNLIN does not actually store the Jacobian but uses fast Givens transformations to construct an orthogonal reduction of the Jacobian to upper triangular form (Golub and Van Loan 1983, and Gentleman 1974).
5.5.5. Regression Results

*General*

The duration of the creep tests on EPS geofoam ranges from four to fifteen months. The regression analysis was based on four months data and this was used to calibrate the creep models. The experimental data at room temperature (+23°C), used to determine the model parameters, are given in Table 5.2 and Table 5.3 summarizes the identified model parameters of each creep model based on four month data regressions.

The Figures 5.13 to 5.15 show the regression results and experimental creep data for stress levels of 20 to 40kPa. The regression results of different models are shown in Table 5.4 and discussed thereafter.
Table 5.2 Experimental creep data (used to determine the model parameters)*

<table>
<thead>
<tr>
<th>Time (hours)</th>
<th>20kPa</th>
<th>30kPa</th>
<th>40kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\varepsilon$, %</td>
<td>$\varepsilon$, %</td>
<td>$\varepsilon$, %</td>
</tr>
<tr>
<td>0.00</td>
<td>0.334</td>
<td>0.000</td>
<td>0.512</td>
</tr>
<tr>
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<td>0.344</td>
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</tr>
<tr>
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<td>0.350</td>
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<td>0.542</td>
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<td>0.562</td>
</tr>
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<td>0.375</td>
<td>0.041</td>
<td>0.571</td>
</tr>
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<td>0.5</td>
<td>0.380</td>
<td>0.046</td>
<td>0.582</td>
</tr>
<tr>
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<td>0.388</td>
<td>0.054</td>
<td>0.590</td>
</tr>
<tr>
<td>2.0</td>
<td>0.397</td>
<td>0.063</td>
<td>0.598</td>
</tr>
<tr>
<td>4.0</td>
<td>0.406</td>
<td>0.072</td>
<td>0.610</td>
</tr>
<tr>
<td>8.0</td>
<td>0.417</td>
<td>0.083</td>
<td>0.624</td>
</tr>
<tr>
<td>24.0</td>
<td>0.436</td>
<td>0.102</td>
<td>0.644</td>
</tr>
<tr>
<td>48.0</td>
<td>0.449</td>
<td>0.115</td>
<td>0.652</td>
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<tr>
<td>72.0</td>
<td>0.454</td>
<td>0.120</td>
<td>0.660</td>
</tr>
<tr>
<td>144.0</td>
<td>0.469</td>
<td>0.135</td>
<td>0.666</td>
</tr>
<tr>
<td>288.0</td>
<td>0.482</td>
<td>0.146</td>
<td>0.686</td>
</tr>
<tr>
<td>480.0</td>
<td>0.489</td>
<td>0.155</td>
<td>0.697</td>
</tr>
<tr>
<td>720.0</td>
<td>0.496</td>
<td>0.162</td>
<td>0.706</td>
</tr>
<tr>
<td>960.0</td>
<td>0.501</td>
<td>0.166</td>
<td>0.714</td>
</tr>
<tr>
<td>1200.0</td>
<td>0.506</td>
<td>0.172</td>
<td>0.727</td>
</tr>
<tr>
<td>1440.0</td>
<td>0.511</td>
<td>0.177</td>
<td>0.738</td>
</tr>
<tr>
<td>1800.0</td>
<td>0.513</td>
<td>0.179</td>
<td>0.743</td>
</tr>
<tr>
<td>2160.0</td>
<td>0.515</td>
<td>0.181</td>
<td>0.748</td>
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<tr>
<td>2592.0</td>
<td>0.517</td>
<td>0.183</td>
<td>0.752</td>
</tr>
<tr>
<td>2952.0</td>
<td>0.518</td>
<td>0.184</td>
<td>0.755</td>
</tr>
</tbody>
</table>

*Ø50mm x 50mm EPS specimen with density of 20kg/m³ at room temperature (+23°C) condition

Table 5.3 model parameters based on 4 month creep data (+23°C)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>3 Element Model</th>
<th>4 Element Model</th>
<th>Modified 4 Element Model</th>
<th>Power Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_0/E_2$</td>
<td>6.24E-03</td>
<td>4.85E-03</td>
<td>1.26E-03</td>
<td>—</td>
</tr>
<tr>
<td>$E_0/\eta_2$</td>
<td>1.45E-01</td>
<td>6.18E-01</td>
<td>1.096E-00</td>
<td>—</td>
</tr>
<tr>
<td>$\sigma_0/\eta_1$</td>
<td>—</td>
<td>1.45E-06</td>
<td>1.68E-03</td>
<td>—</td>
</tr>
<tr>
<td>$N(n_F)$</td>
<td>—</td>
<td>—</td>
<td>0.174</td>
<td>0.139</td>
</tr>
<tr>
<td>$M_F$</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>2.63E-03</td>
</tr>
</tbody>
</table>
CHAPTER 5: CREEP BEHAVIOUR OF EPS UNDER COMPRESSIVE LOADING

Figure 5.13 measured and predicted total strain ($\sigma = 20$ kPa)

Figure 5.14 measured and predicted total strain ($\sigma = 30$ kPa)
Figure 5.15 measured and predicted total strain ($\sigma = 40$ kPa)

Table 5.4 Regression results of different models at four month elapsed time

<table>
<thead>
<tr>
<th>Stress, kPa</th>
<th>Measured $\varepsilon$ (%)</th>
<th>4element $\varepsilon$ (%)</th>
<th>$r^2$</th>
<th>3element $\varepsilon$ (%)</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.518</td>
<td>0.519</td>
<td>0.862</td>
<td>0.462</td>
<td>0.776</td>
</tr>
<tr>
<td>30</td>
<td>0.755</td>
<td>0.789</td>
<td>0.945</td>
<td>0.706</td>
<td>0.845</td>
</tr>
<tr>
<td>40</td>
<td>0.995</td>
<td>1.650</td>
<td>0.899</td>
<td>0.955</td>
<td>0.826</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stress, kPa</th>
<th>Measured $\varepsilon$ (%)</th>
<th>Modified 4element $\varepsilon$ (%)</th>
<th>$r^2$</th>
<th>Power law $\varepsilon$ (%)</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.518</td>
<td>0.499</td>
<td>0.912</td>
<td>0.498</td>
<td>0.895</td>
</tr>
<tr>
<td>30</td>
<td>0.755</td>
<td>0.759</td>
<td>0.995</td>
<td>0.759</td>
<td>0.982</td>
</tr>
<tr>
<td>40</td>
<td>0.995</td>
<td>1.026</td>
<td>0.921</td>
<td>1.026</td>
<td>0.888</td>
</tr>
</tbody>
</table>
Three Element Model

The three-element model underestimates the four month measured creep strain by up to 30.0% (0.00129 versus 0.00184, at stress of 20 kPa). The values of $r^2$ for four-month regression range from 0.776 to 0.845.

An inspection of the three-element model would reveal the regressed strain asymptoting to a constant value at later values of elapsed time. Owing to this mathematical constraint, the creep strain calculated by the three-element model becomes constant after the first few days. Therefore the three-element model was considered unacceptable for modeling the creep behaviour of EPS material.

Four Element Model

In contrast, an inspection of Figures 5.13 to 5.15 shows the four-element model tending to overestimate the creep strain of EPS geofoam. The $r^2$ values for the regression of four-month data vary from 0.862 to 0.945 (Table 5.4). The regressed total strains by the four-element model are higher than the measured values by up to 23.6% (0.366 versus 0.296, at stress of 40 kPa).

The mathematical function of the four-element model constrains the strain rate to approach a constant value at large values of elapsed time. Following this constraint, the strain calculated by the four-element model is found to increase linearly with time after the first few days. Therefore the four-element model was also considered unacceptable for modeling creep behaviour of EPS material.

Modified Four Element Model

The regressed total strain by the modified four-element model agrees well with the observed total strain of EPS geofoam for the applied stress of 20 to 40 kPa. The $r^2$ values for the regression of four-month data range from 0.912 to 0.995. For the four-month total strain, the error between the regressed values and the measured values are within ±10%.
**Power law (Simplified Findley) Model**

The regression results also show the calculated strain by the power law model, equation (5.5). It agrees well with the measured total strain of EPS geoflake in the stress level range from 20 to 40 kPa. The $r^2$ values for the regression of four-month data are in the range 0.888 to 0.982. For four-month creep strain, the error between the calculated values and measured values are within ±10%.

![Graph showing total strain over time](image)

**Figure 5.16 measured and predicted total strain at 23°C (13 months)**

### 5.5.6. Comparison of the Predicted and Measured Experimental Results

The calibrated modified four-element and power law models were subsequently used in predicting the creep strain of EPS beyond four months. The predicted results were compared against actual test data as a part of the process for evaluating and validating the creep models.

Figure 5.16 shows the predicted total strain and measured total strain at about 13 months at stress level of 40 kPa. The observed total strain at the 13-month elapsed time is 1.055%.
For the modified four-element model, the predicted total strain at the 13-month elapsed time is 1.092% (error = +3.5%). Therefore, the modified four-element model, based on four month creep data, adequately predicts the creep strain of EPS up to 13 months.

The predicted total strain of the power law model is 1.086% at the 13-month elapsed time (error = +3.0%). This also indicates that the power law model, based on four month data, adequately predicts the creep strain of EPS up to 13 months.

5.5.7. Complete Models for EPS Material

The analysis and discussions in the above section indicate that the two best performing creep models are the modified four-element and the power law models. These models have been developed and calibrated for a stress range of up to 40 kPa, which generally covers the stress loading applicable in geotechnical applications.

As well, for stress levels less than 40 kPa which are within the linear elastic limit of the material, the initial strain of EPS material, ε₀, can be obtained taking the quotient of the stress value and the initial Young's modulus averaged from the experimental results in Chapter 3:

\[
\varepsilon_0 = \frac{\sigma}{46.2} \quad (\sigma \leq 40 \text{ kPa}) \tag{5.19}
\]

Therefore, the complete strain model of EPS (at room temperature) can be written as a power law model,

\[
\varepsilon = 0.0216\sigma(t) + 0.00263\sigma(t)t^{0.174} \quad (\sigma \leq 40 \text{ kPa}) \tag{5.20}
\]

or a modified four-element viscoelastic model as

\[
\varepsilon = \frac{\sigma(t)}{46.2} + \int_0^t (t - \tau) \frac{d\sigma(\tau)}{d\tau} d\tau
\]

where \( J(t) = \frac{t^{0.174}}{592.24} + \frac{1}{793.65} \left[ 1 - e^{-1.1t} \right] \)
5.5.8. The Influence of Temperature on the Creep Model of EPS

The creep test results in section 5.4 indicate that the creep strain of EPS increases with increasing temperature, and in particular the effect of temperature becomes significant when the applied stress level is higher than 40 kPa. However, as the stress loading in geotechnical application of EPS geofoam is generally less than 40 kPa, the combination of high temperature and stress levels higher than 40 kPa is unlikely to be a major design consideration.

In order to investigate the effect of temperature, a similar nonlinear regression of the 4 month creep test data at temperature of 40°C was performed. The normalized creep strain, $\varepsilon_t / \sigma_t$, is shown in Figure 5.17. The identified model parameters of the modified four-element model and the power law model are given in Table 5.5. The predicted and experimental total strain of EPS for more than 14 months at stress levels of 30 and 40 kPa are shown in Figure 5.18. The results show that both the modified four-element model and the power law model are still able to model the creep of EPS at the elevated temperature of 40°C well.

For the modified four-element model, there are only slight changes in the values of the parameters $\sigma_0 E_2$ (a decrease of 12.7%) and $n$ (a decrease of 6.9%) as the temperature increases from 23°C to 40°C, but large changes occur in the parameters $E_2 / \eta_2$ (from 1.1 to 0.007) and $\sigma_0 / \eta_1$ (from 0.00168 to 0.0041). This indicates that the influence of temperature on the creep model affects mainly the viscous elements while the spring element is approximately independent of the temperature in the specified stress range.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Modified 4 Element Model</th>
<th>Power Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_0 E_2$</td>
<td>1.10E-03</td>
<td>—</td>
</tr>
<tr>
<td>$E_2 / \eta_2$</td>
<td>7.22E-03</td>
<td>—</td>
</tr>
<tr>
<td>$\sigma_0 / \eta_1$</td>
<td>4.10E-03</td>
<td>—</td>
</tr>
<tr>
<td>$n$ ($n_f$)</td>
<td>0.162</td>
<td>0.177</td>
</tr>
<tr>
<td>$m_f$</td>
<td>—</td>
<td>4.01E-03</td>
</tr>
</tbody>
</table>
Figure 5.17 Normalized creep strain in creep test (+40°C)

Figure 5.18 measured and predicted total strain at +40°C (14 months)
On the other hand, it is found that for the power law model, both parameters have experienced large increases in values. The parameter $m_F$ increases from $2.63\times10^{-3}$ to $4.01\times10^{-3}$ (an increase of 52%), and the $n_F$ increases from 0.139 to 0.177 (an increase of 27%). This indicates that the parameters of the power law model are temperature dependent.

5.6. SUMMARY

1. In general, creep deformation of EPS geofoam can be represented by two separate parts, i.e., an immediate component and a time-dependent component. The creep test results indicated that the creep behaviour of EPS geofoam is dependent on the applied stress levels and the temperature.

2. The Sherby-Dorn plot (strain rate against the creep strain) shows that at room temperature (23°C) conditions, the EPS material is generally long-term stable for a stress level less than 40kPa, where the creep rate continues to decrease with time. However, as the stress level increases to 60 kPa, a transitional though long-term stable condition exists where the creep rate eventually becomes constant with time. The threshold stress level beyond which the applied loading must never exceed to ensure long-term stability at room temperature appears to be about 60 kPa.

3. The creep strain of EPS increases with increasing temperature. At a temperature of 40°C, and for stress levels less than 40 kPa, the EPS material is long-term stable. However, for a stress level of 50kPa at 40°C, EPS material enters the transitional stage between long term stable and unstable behaviour. Hence 60 kPa and 50 kPa may be considered as threshold stress values for design purposes in room temperature (23°C) and 40°C environments, respectively.

4. As many parts of Australia experience extreme temperatures (well above 40 degrees Celsius) for prolonged period, it is recommended that the application of EPS in Australia should be under 40 kPa applying load to avoid high creep deformation.

5. A modified four-element viscoelastic model and a power law model have been developed based on four month data for room temperature (23°C) and 40°C. These
models are able to adequately predict the creep strain of EPS geofoam beyond the four months elapsed time.

6. Due to a lack of available data, no multiaxial creep model was developed. For the same reason, no development work was undertaken for a coupled temperature dependent creep model. These investigations will be left for a future research.

7. The models have been validated against constant levels of applied stress. There remains a need, however, to test the validity of these models in a case where the applied stress varies with time and this work will be carried out in Chapter 6.
CHAPTER 6
THE BEHAVIOUR OF EPS UNDER REPEATED LOADING

6.1. INTRODUCTION

The behaviour of EPS geofoam under repeated loading is of major engineering interest in geotechnical and pavement-related applications of this material. Although EPS geofoam has been reported to perform well as a subgrade replacement material in pavement construction over poor quality soil, these observations do not yield detailed information of the material response. There have been few, if any, reports of controlled laboratory studies carried out to rigorously investigate the EPS geofoam response under controlled repeated loading. Such information, if available, would not only give useful insight into the material behaviour subjected to repeated loading but it would also give guidance to pavement design and construction where the use of the EPS geofoam is being considered.

The design procedures for pavements presented in the AUSTROAD (Austroads 1992) and AASHTO Guide for Design of Pavement Structures (AASHTO Guide 1993) utilize the mechanical properties of the asphalt concrete, base course and soil subgrade. The standard method of test for resilient response accounts for the repetitive nature of traffic loading. The property that describes this behaviour in materials is called the Resilient Modulus of Elasticity, which is defined as the deviator dynamic stress (due to the moving vehicular traffic) divided by the resilient (recoverable) strain (AS 1289.6.8.1 – 1995). This is considered to be a required input for determining the stress-strain characteristics of pavement structures subjected to traffic loading.

Another required input is the failure criteria of the pavement materials subjected to repeated loading. The mechanistic design approach requires the failure criteria for the EPS geofoam to be established, in the same manner is required for any subgrade material used in the pavement construction.

In this chapter, the behaviour of the EPS geofoam under cyclic loading condition has been studied in two series of unconfined cyclic loading tests at various stress levels
and the following have been investigated:

1. The effects of the number of load repetitions, including the resilient modulus of elasticity;
2. The effects of the stress levels, including the static stress and dynamic stress;
3. The permanent deformation accumulation with number of load repetitions;
4. The number of cycles to “failure” at a given stress level or compressive strain which is termed the limiting stress or strain repetitions criteria (Austroads 1992).

In the cyclic loading test, on the other hand, the EPS specimen accumulates plastic strain with each cycle. In Chapter 5, a creep model was developed for EPS geofoam that is able to capture creep (plastic) deformation under time-dependent loading. The model in Chapter 5 was however confined only to a ramp-type loading. In this Chapter, the results from the cyclic loading test afford an opportunity to further validate the creep model under repeated type loading. The results from this validation are also discussed within.

6.2. DEFINITIONS

6.2.1 Definitions of Key Parameters

In cyclic loading tests described in this chapter, the key parameters of the stress-strain curve used are defined below. Figure 6.1 further illustrates these definitions.

Peak/maximum cyclic stress, $\sigma_n$: the maximum stress of the $n$th loading/unloading cycle
Static stress, $\sigma_i$: the sustained stress level of the dead load
Dynamic stress, $\sigma_d(n)$: magnitude of the cyclic deviator stress in the $n$th loading/unloading cycle
Peak strain, $\varepsilon_n$: total strain at $n$th cycle (corresponding to the maximum cyclic stress)
Accumulated permanent strain, $\varepsilon_{p(n)}$: accumulated permanent strain after unloading at the $n$th cycle (Note that the irrecoverable strain includes the strain due to the static load)

Resilient strain, $\varepsilon_{r(n)}$: the recoverable strain, equal to peak strain minus permanent strain after unloading at $n$th cycle, $\varepsilon_{r(n)} = \varepsilon_n - \varepsilon_{p(n)}$

Initial Young's modulus, $E_i$: slope of initial linear-elastic portion of stress-strain curve

Compressive strength, $\sigma_c$: compressive stress at 10% of axial strain on stress-strain curve of monotonic compression test

Stress ratio, $S$: the ratio of peak stress to compressive strength, $S = \sigma_p/\sigma_c$

6.2.2 Concept of Resilient Modulus

The concept of resilient modulus applies to a subgrade material which undergoes deformation as it is subjected to repeated loads due to moving vehicular traffic. A part of the deformation strain is resilient or recoverable, $\varepsilon_r$, and the remainder is permanent, $\varepsilon_p$. The resilient modulus ($E_{r(n)}$ at the $n$th cycle), which is equivalent to a cyclic Young's modulus, is defined according to the following equation (AS 1289.6.8.1) as:
\[ E_{r(n)} = \frac{\sigma_{d(n)}}{\varepsilon_{r(n)}} \times 10^{-1} \]  

(6.1)

where \( E_{r(n)} \) is in MPa, \( \sigma_{d(n)} \) in kPa and \( \varepsilon_{r(n)} \) in percent.

6.3. TEST PROGRAM AND PROCEDURES

The test program consisted of two series of repeated loading tests: the first is used mainly to establish the resilient properties of the EPS geofoam; and the second is to define the failure criteria for the EPS geofoam used as a replacement subgrade.

The cyclic compression tests were performed on cylindrical specimens of \( \Phi 50 \times 50 \) mm with a density of 20kg/m\(^3\). Test conditions are typically those of a laboratory environment (approximately +23°C and 50% relative humidity).

As the EPS geofoam is a cohesive material with a very small or negligible internal friction angle \(^{(1)}\), a confining pressure was not applied. Also, current evidence indicates that confining pressure has very little or no effect on the compressive behaviour of EPS (results in Chapter 3, Horvath 1995, Preber et al. 1994, and Eriksson and Trank 1991). Thus, an unconfined compression test equipment, INSTRON 6027-R5500 electromechanical Universal Testing Machine as described in Chapter 3, was used instead of the triaxial test equipment. The system is capable of sampling 200 samples per second, and the sampled data is fed to the D/A converter at the same rate.

6.3.1 Loading Waveform

In Australian Standard AS 1289.6.8.1 (1995), the suggested loading waveform (Figure 6.2) specifies a loading period of 3 seconds with a holding and rest period of 2 seconds. However, the test system in this thesis was unable to apply such a loading waveform, therefore, an alternative saw-tooth waveform was used as shown in Figure 6.3. The loading period was 2 seconds (0.5 Hz), and without the holding and rest period.

Note \(^{(1)}\): Some reports suggest that the internal friction angle might even be negative, contrary to normal behaviour of geological material (e.g. Hamada & Yamanouchi, 1987).
6.3.2 Test for Resilient Properties

According to the prescribed standard test procedures, preconditioning is to be performed for the resilient modulus test to allow the end caps to bed into the specimen and the resilient strain to stabilize under the imposed stress conditions. The preconditioning
stresses to be used will depend on the stresses to be applied during the resilient modulus test. For lower sub-base materials (below 200 mm in the pavement), a vertical stress, $\sigma_1$, of 0.1$p$ is recommended, where $p$ is the contact stress in the range 550 to 700 kPa.

However, the stresses recommended above are generally not suitable for the EPS subgrade. In subgrade applications, the EPS is normally covered with about 500mm of pavement material (about 10kPa static stress), and so a total (10 kPa static plus 10 kPa dynamic) vertical stress of 10 to 20 kPa would be applied in a loading cycle (Tompsonsett et al. 1995, and Aaboe 1987). Therefore, to investigate the resilient properties of EPS geofoam in applications of practical interest, a static vertical stress in the range of 5 to 30 kPa, and a dynamic vertical stress range of 10 to 30 kPa were applied. However, as it is of engineering interest to understand the resilience of the geofoam under very high dynamic loading which are above normal loading applications, it was decided to include a further sequence of dynamic stress loading from 40kPa to 110 kPa and zero static stress for testing. The details of the stress levels for determination of the resilient properties are summarized in Table 6.1

<table>
<thead>
<tr>
<th>Static vertical stress, kPa</th>
<th>Dynamic vertical stress, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>40,50,70,80,90,100,110</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>10</td>
<td>10, 20, 30</td>
</tr>
<tr>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>30</td>
<td>10</td>
</tr>
</tbody>
</table>

6.3.3 Procedure for Determining the Resilient Modulus of EPS Geofoam

Tests results obtained in this chapter typically show that about 500 to 1000 cycles of preconditioning is required to cause the end caps to bed into the EPS specimen and the resilient strain to stabilize under different stress conditions (Figure 6.4). Based on the prescribed standard procedures for soil subgrade, the following steps have been adopted
for determining the resilient modulus of the EPS geofoam:

![Resilient strains in typical cyclic loading test](image)

1. Apply and hold the selected static stress ($\sigma_s$).
2. Apply loading/unloading cycles of the specified dynamic stress ($\sigma_d$) to the specimen for at least 1000 loading cycles as preconditioning.
3. Continue the application of the loading/unloading cycles until at least 3000 cycles. The resilient modulus values were determined for every cycle greater than 1000 cycles.

### 6.3.4 Test for Failure of EPS Geofoam (Permanent Deformation)

Austroads (1992) defines the limiting strain criterion of a soil subgrade as the allowable number of repetitions of this strain until an unacceptable level of permanent deformation develops. These accumulated permanent deformations eventually manifest themselves as rutting along the wheel paths. In the same manner, the limiting stress criterion of a soil subgrade is defined as the allowable number of repetitions of this stress until permanent deformation failure develops or the stress level cannot be attained. With EPS geofoam, neither in the design guide nor elsewhere in literature has the permanent deformation threshold been specified. A few possibilities have been considered in this thesis.

The first two possibilities considered are the accumulated permanent strains from repeated loading corresponding to 5% and 10% where these considerations have been
based conveniently on the historical definitions of the compressive strength. These strain values are thought to be rather large and probably not appropriate.

On the grounds of safety, the BASF (1995) has specified 1.5 % as the limit compressive strain of 20 kg/m³ EPS. Assuming a dead load of 30 kPa (30% of the compressive strength), the expected strain will be approximately 0.5%. Therefore, the strain due to the dynamic load is restricted not to exceed 1.0%, which includes accumulated permanent strain and resilient strain. In this thesis, a threshold of 0.5% accumulated permanent strain for 20 kg/m³ EPS geofoam was adopted. Although the chosen threshold may still appear somewhat arbitrary, it at least provides a reasonable basis to achieve the object of this study and would serve as a precursor for more detailed research at a later stage, or until field or other experimental evidence suggest otherwise.

A second series of cyclic loading test was also performed within the whole stress range (up to 110 kPa, into the post yielding regime of this type of EPS material). The stress loadings of this test series are listed in Table 6.2.

<table>
<thead>
<tr>
<th>Peak stress, kPa</th>
<th>Stress ratio*</th>
<th>ε_{r(t)} **(microstrain)</th>
<th>No. of cycles to 0.5% permanent strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>110</td>
<td>1.12</td>
<td>51363</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>1.02</td>
<td>41200</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>0.92</td>
<td>30370</td>
<td>50</td>
</tr>
<tr>
<td>80</td>
<td>0.82</td>
<td>21842</td>
<td>1000</td>
</tr>
<tr>
<td>70</td>
<td>0.71</td>
<td>12241</td>
<td>3200</td>
</tr>
<tr>
<td>50</td>
<td>0.51</td>
<td>8360</td>
<td>7500</td>
</tr>
<tr>
<td>40</td>
<td>0.41</td>
<td>6721</td>
<td>22000</td>
</tr>
</tbody>
</table>

Note: * stress ratio = stress/compressive strength, where the compressive strength is 98 kPa.
** ε_{r(t)} = σd/Er
6.4. EXPERIMENTAL RESULTS

6.4.1 General

Figure 6.5 shows the typical stress – strain curves in a cyclic loading test. As might be expected, it is found that the higher the stress level, the higher the rate of increase of the accumulated permanent strain. The strains induced for all levels of loading of EPS are not fully recoverable upon unloading; this is true even for relatively small levels of peak stress (e.g. <10 kPa). The resilient modulus decreases with increasing stress levels. As the word ‘resilient’ suggests, the resilient modulus at a given stress level as seen from the slope of the loading and unloading cycles gradually tends to a constant value after the initial (500 – 1000) cycles.

6.4.2 Permanent and Resilient Deformation of EPS Geofoam

Because of the high compressibility of EPS, the deformation behaviour of this material is very sensitive to the magnitude of the applied stress whether the loading is static or dynamic. Typical plots of the increase in the accumulated permanent strain, $\varepsilon_p$, with
number of loading/unloading cycles, \( n \), for various static stress levels are presented in Figure 6.6, where the dynamic stress is a constant 10 kPa. After a short transient stage, the accumulated permanent strain of the EPS increases at a relatively very small rate. The accumulated permanent strain is also shown to increase, at a given stress cycle, as the static stress is increased.

However, Figure 6.7 shows that the dynamic stress has a relatively small influence on the magnitude of the accumulated permanent strain, suggesting that, in this stress range, the accumulated permanent strain is largely dependent on the level of the static stress, not the dynamic stress.

Figures 6.8 and 6.9 show the effects of dynamic and static stress on the resilient strain of EPS respectively. For static and dynamic stress of 0 to 30 kPa, it appears that an increase in the level of the dynamic stress now produces a notable and corresponding increase in the resilient strain, an indication that the resilient strain is affected significantly by the change in the dynamic stress. However, it also appears that the resilient strain is less influenced by the static stress as a change in the magnitude of static stress produces relatively small increases in the resilient strain.
Figure 6.7 Accumulated permanent strain of EPS under different dynamic loading

Figure 6.8 Resilient strain of EPS under different dynamic loading
Figure 6.9 Resilient strain of EPS under different static loading

Figure 6.10 Effect of different dynamic loading on resilient modulus of EPS
6.4.3 Resilient Modulus of EPS

Based on the results of the above tests, the resilient moduli of EPS under different loading conditions were obtained. Figure 6.10 shows the resilient modulus for the constant static stress condition (static stress $\sigma_s = 10$kPa) and the dynamic stress ($\sigma_d$) ranging from 10 to 30 kPa. Such a plot shows the behaviour of EPS under various dynamic loading, thus it can normally be used to evaluate the effects of dynamic loading on the resilient modulus of EPS.

In the same way, Figure 6.11, which shows the resilient modulus at a constant dynamic stress condition ($\sigma_d = 10$kPa) and the static stress ranging from 5 to 30 kPa, gives some indication of the effects that static loading has on the resilient modulus of the EPS.

In general, the resilient modulus of EPS reaches a stable value after preconditioning (500–1000 cycles) was completed. It changes marginally with the number of loading cycles beyond this.
6.4.4 Discussions on Resilient Modulus of EPS

Previous research into granular materials has demonstrated that the relationship between $E_r$ and $\theta$ can be expressed in the form (Boyce et al. 1976):

$$E_r = k_1 (\theta)^{k_2}$$

where $\theta$ is the sum of the principal stresses, and $k_1$, $k_2$ are constants which depend on the material and the test conditions.

For most fine-grained, or clay subgrade soils, $E_r$ decreases rapidly as the magnitude of cyclic stress increases. This decrease in $E_r$ with increasing deviator stress is often referred to as "stress-softening" behaviour (Drumm et al. 1996).

This Chapter intends, based on the experimental results, to find out the relationship between the resilient modulus of EPS and applied stresses (static or dynamic).

Table 6.3 summarizes the resilient modulus values at 5000 cycles obtained from the cyclic loading tests. It was found that the resilient behaviour of EPS material is different to the granular materials or the fine-grained soils discussed previously. Figures 6.12 and 6.13 indicate that both the dynamic and static stress levels have only a small effect on the resilient modulus of EPS because the stress levels examined are within the linear elastic range of the EPS material. The combined results, for the resilient modulus versus the total stress shown in Figure 6.14, show a linear and slightly decreasing relationship for stress levels less than 70 kPa.

<table>
<thead>
<tr>
<th>Static vertical stress, kPa</th>
<th>Dynamic vertical stress, kPa</th>
<th>Resilient Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>10</td>
<td>6.5</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>6.0</td>
</tr>
<tr>
<td>20</td>
<td>10</td>
<td>6.1</td>
</tr>
<tr>
<td>30</td>
<td>10</td>
<td>5.5</td>
</tr>
<tr>
<td>10</td>
<td>20</td>
<td>5.8</td>
</tr>
<tr>
<td>10</td>
<td>30</td>
<td>6.0</td>
</tr>
<tr>
<td>0</td>
<td>40</td>
<td>6.1</td>
</tr>
<tr>
<td>0</td>
<td>50</td>
<td>6.1</td>
</tr>
<tr>
<td>0</td>
<td>70</td>
<td>5.8</td>
</tr>
<tr>
<td>0</td>
<td>80</td>
<td>3.8</td>
</tr>
<tr>
<td>0</td>
<td>90</td>
<td>2.7(1)</td>
</tr>
<tr>
<td>0</td>
<td>100</td>
<td>2.5(1)</td>
</tr>
<tr>
<td>0</td>
<td>110</td>
<td>2.2(1)</td>
</tr>
</tbody>
</table>

Note (1): the value of resilient modulus was taken at end of cyclic test.
Beyond what appears to be a threshold value of the total stress, 70 kPa, it is observed that the resilient modulus reduces sharply and nonlinearly. This demarcation is both interesting and worth elaborating. It is as if there are two distinct resilient behaviours: below 70 kPa the geofoam remains resilient but above 70 kPa, the resilience diminishes quickly.

\[ E_r = 5.85 + 0.0035\sigma_t \]

\[ \sigma_t = 10\text{kPa} \]

**Figure 6.12** Resilient modulus of EPS Vs dynamic stress level (after 5000 cycles)

\[ E_r = 6.52 - 0.031\sigma_s \]

\[ \sigma_s = 10\text{kPa} \]

**Figure 6.13** Resilient modulus of EPS vs. static stress level (after 5000 cycles)
6.4.5 Failure of EPS under Cyclic Compressive Loading

To investigate the behaviour of EPS geofoam up to permanent deformation failure under different cyclic stresses, the cyclic tests were performed in the approximate stress range from 40 kPa to 110 kPa, while the static stress was set to zero. The test results show that a decrease in the stress level results in an approximately logarithmic increase in the number of cycles to failure (0.5 % accumulated permanent strain).

For the various stress levels, the accumulated permanent strain versus the number of load cycles is shown in Figure 6.15. The curves show an approximately linear relationship between \( \log \varepsilon_p \) and \( \log N \), but the rate appears to increase at a accumulated permanent strain of about 1%. This indicates that some form of "yielding" probably commences when the accumulated permanent strain reaches 1.0%. In this case, the results seem to vindicate the earlier selection of the 0.5 % accumulated permanent strain as the threshold value deemed as the permanent deformation failure point.

The results of the resilient strain of the first cycle, \( \varepsilon_{r(1)} \), and the number of cycles to permanent deformation failure for different dynamic stress levels are given in Table 6.2.
Beyond what appears to be a threshold value of the total stress, 70 kPa, it is observed that the resilient modulus reduces markedly and nonlinearly. This demarcation is both interesting and worth elaborating. It is as if there are two distinct resilient behaviours: below 70 kPa the geofoam remains resilient but above 70 kPa, the resilience diminishes quickly.

In general design, the dynamic loading and the resilient modulus are known. Therefore, the resilient strain of the first cycle, which is defined as response strain due to the dynamic load, can be obtained directly by Equation (6.1) or deduced from the numerical modeling results of layered pavement (e.g. CIRCLY).

On the basis of the cyclic test results obtained, and using the Austroad design guide (Austroad 1992) to give guidance, the limiting strain criterion of EPS geofoam was developed and the plot is shown in Figure 6.16. The equation of the best-fit line gives,

\[ N = \left( \frac{112000}{\mu \varepsilon} \right)^{0.57} \]  

(6.3)

where the \( \mu \varepsilon \) is the applied strain, which is defined as the resilient strain of first cycle (in units of micro-strain) and \( N \) is the allowable number of repetitions of this strain before an unacceptable level of deformation developed in EPS geofoam subgrade. The coefficient of
CHAPTER 6: THE BEHAVIOUR OF EPS UNDER REPEATED LOADING

![Graph showing the relationship between resilient strain of 1st cycle and number of load repetitions to failure.](image)

**Figure 6.16 Limiting strain criterion for EPS subgrade (20 kg/m³)**

The determination, \( r^2 \), from the regression is 0.97.

As an example, if the fatigue life of the EPS geofoam is assumed as \( 10^6 \) load repetitions, the allowable limiting strain of at the EPS subgrade may be calculated as follows:

Given \( N = 5 \times 10^6 \),

\[
\varepsilon_{r(0)} = \left( \frac{112000}{\mu e} \right)^{3.57} = 1491 \text{ (micro-strain)}
\]

This result implies that for a service life of \( 5 \times 10^6 \) load repetitions, the applied compressive strain (due to the dynamic loading) to the EPS geofoam should be limited to 0.15%.

The limiting stress criterion may also be used to predict the service life time during which the number of repetitions to failure occurs for a given stress level (Austroad 1992). The results are plotted in Figure 6.17, the best-fit relationship was obtained as:

\[
\log S = -0.126 \log N + 0.218 \quad (6.4)
\]

where \( S \) is the stress ratio. An alternate equation form giving \( N \) as a function of \( S \) is,

\[
N = \left( \frac{1.65}{S} \right)^{2.85} \quad (6.5)
\]

The coefficient of determination, \( r^2 \), of the regressed data is 0.94.
CHAPTER 6: THE BEHAVIOUR OF EPS UNDER REPEATED LOADING

![Graph showing applied stress ratio against number of load repetitions to failure](image)

Figure 6.17 limiting stress criterion for EPS material (20 kg/m$^3$)

In the same way, equation (6.5) may be used to predict the number of cycles of loading to failure for the EPS geofoam under a given stress level, or for a given service life, to predict the limiting stress level. For example,

Given $N = 5 \times 10^6$,

$$S = \frac{1.65}{N^{0.125}} = 0.237$$

Given a compressive strength of 98 kPa for the 20 kg/m$^3$ EPS geofoam (Chapter 3), the applied dynamic compressive stress should be limited to 23 kPa ($0.237 \times 98$) if the service life is $5 \times 10^6$ cycles.

6.5. CREEP BEHAVIOUR OF EPS GEOFOAM UNDER CYCLIC LOADING

The creep behaviour of EPS under constant loading has been studied in Chapter 5. In cyclic loading test, the EPS specimen was subjected to both static and dynamic loading and therefore undergoing creep and cyclic deformation. Even though the viscoelastic model in Chapter 5 was developed under constant stress loading, it may also be used for time-dependent loading condition.

Given a time-dependent stress $\sigma(t)$, the complete strain equation (5.21) for the
modified four-element model can be written as

$$\varepsilon(t) = \frac{\sigma(t)}{E_1} + \int_0^t \left( 1 - e^{-\eta_1 \tau} \right) \frac{\partial \sigma(\tau)}{\partial \tau} d\tau$$

(6.6)

where: $\varepsilon(t)$ = the total strain at time $t$ after the stress application, $E_1$, $E_2$, $\eta_1$, $\eta_2$, and $n$ are experimental model parameters and the compliance equation for the creep gives,

$$J(t) = \frac{1}{\eta_1} t^n + \frac{1}{E_2} \left( 1 - e^{-\frac{\eta_2}{E_2}} \right)$$

(6.7)

Based on the regression of the experimental tests discussed in Chapter 5, the identified parameters of the modified 4-element model for EPS geofoam of specified 20 kg/m$^3$ density are established as tabulated in Table 6.4.

<table>
<thead>
<tr>
<th>$n$</th>
<th>$E_1$ (kPa)</th>
<th>$\eta_1$ (kPa-sec)</th>
<th>$E_2$ (kPa)</th>
<th>$\eta_2$ (kPa-sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.174</td>
<td>4620</td>
<td>2475</td>
<td>793.6</td>
<td>2.61×10^6</td>
</tr>
</tbody>
</table>

Table 6.4. Identified parameter values of modified 4-element model for EPS

Figure 6.18 Cyclic Loading Function

The time-dependent loading, $q(t)$, consists of a ramp loading representing the construction sequences from beginning until completion ($t=t_0$) when a static load, $q_0$, is imposed and a series of cyclic loading, $q(t)$, (Figure 6.18). The cyclic loading function is given by,
The solution of the Equation (6.6) containing a convolution integral is significantly facilitated by introduction of a Laplace transform defined as,

\[
\bar{\varepsilon} = \int_0^t \varepsilon(\tau)e^{-\sigma \tau} d\tau
\]  \hspace{1cm} (6.9)

where the superior bar on the left-hand-side indicates the transformed Laplace variable. It thus follows from Equations (6.6), (6.7) and (6.9) that,

\[
\bar{\varepsilon} = \left( \frac{1}{\sigma} + s\bar{J} \right) \bar{\sigma}
\]  \hspace{1cm} (6.10)

where \(\sigma(0) = 0\). The compliance Equation (6.7) yields,

\[
\bar{J} = \frac{1}{\eta_1} \frac{\Gamma(n+1)}{\varepsilon^{n+1}} + \frac{1}{s(\eta_1 + E_2)}
\]  \hspace{1cm} (6.11)

where \(\Gamma\) is the Gamma function. The Laplace transform of the time dependent loading (Equation 6.8) is,
\[
\frac{q_0}{s^{T_0}T_0}  \quad 0 < t \leq T_0
\]
\[
\frac{q_0}{s^{T_0}}\left(1 - e^{-rT_0}\right) + \frac{q_1}{s^{T_0}}e^{-rT_0}  \quad T_0 < t \leq T_0 + t_0
\]
\[
\frac{q_0}{s^{T_0}}\left(1 - e^{-rT_0}\right) + \frac{q_1}{s^{T_0}}e^{-rT_0} - \frac{2q_1}{s^{T_0}}e^{-r(T_0 + t_0)}  \quad T_0 + t_0 < t \leq T_0 + 2t_0
\]
\[
\vdots
\]
\[
\vdots
\]
\[
\frac{q_0}{s^{T_0}}\left(1 - e^{-rT_0}\right) + \frac{q_1}{s^{T_0}}e^{-rT_0} - \frac{2q_1}{s^{T_0}}e^{-r(T_0 + 2t_0)} + \cdots - \frac{2q_1}{s^{T_0}}e^{-r(T_0 + (2n-3)t_0)} + \frac{2q_1}{s^{T_0}}e^{-r(T_0 + (2n-2)t_0)}
\]
\[
T_0 + (2n - 2)t_0 < t \leq T_0 + (2n - 1)t_0
\]
\[
\frac{q_0}{s^{T_0}}\left(1 - e^{-rT_0}\right) + \frac{q_1}{s^{T_0}}e^{-rT_0} - \frac{2q_1}{s^{T_0}}e^{-r(T_0 + (2n-2)t_0)} + \cdots + \frac{2q_1}{s^{T_0}}e^{-r(T_0 + (2n-1)t_0)}
\]
\[
T_0 + (2n - 1)t_0 < t \leq T_0 + 2nt_0
\]

(6.12)

Then, the Laplace transform solution of Equation (6.10) is inverted numerically to produce the real-time solutions by way of the inversion algorithm by Talbot (1979). More discussions of this method are given in Chapter 8.

Figure 6.19 and 6.20 show the modeling results and those measured from cyclic loading tests, which exclude the component strain due to the static loading. Figure 6.19 shows the effect of the static loading while the dynamic loading was kept at a constant level of 10 kPa. Figure 6.20 represents the results of constant static loading of 10 kPa, which shows the influence of dynamic loading. In general, the modeling results show the same manner as those measured. For constant dynamic stress, the higher the static stress, the higher the creep strain. For constant static stress, the higher the dynamic stress, the higher the creep strain. However, by comparing the modeling results with those measured from cyclic loading tests, it is also found that, for constant dynamic loading, the modeling results of creep strains are lower than that measured at static loading level of 10 kPa and 20 kPa, but higher than that measured at static stress of 30 kPa. On the other hand, for constant static loading, the creep strains are lower than those measured. It seems that the model parameters maybe slightly different for EPS under constant loading and time dependent dynamic loading condition, which indicates that the dynamic loading also has some contributions to the parameters of the stress-strain model of EPS. Therefore, further validations or amendments on the model parameters might be made when used to predict the creep strain under cyclic loading condition.
Figure 6.19 Permanent strain (excluded static component) under cyclic loading

Figure 6.20 Permanent strain (excluded static component) under cyclic load
6.6. SUMMARY

Cyclic tests on EPS at different static and dynamic stress levels have been carried out. The influence of the repeated loading on the behaviour of EPS has been investigated. The results are summarized as follow:

1. The permanent deformation of EPS is affected by the applied stress (or strain) level and the number of loading and unloading cycles. The accumulated permanent strain is increased as the number of cycles increase and the higher the stress level, the higher the rate of increase of the accumulated permanent strain.

2. The permanent deformation of EPS accumulates with the number of loading and unloading cycles even in the so-called elastic stress range of the stress-strain curve obtained from the monotonic compression test.

3. The accumulated permanent strain increases with an increase in the static stress. Conversely, the static stress level has very little influence on the resilient strain.

4. In unconfined compression tests, the resilient modulus of EPS reaches a stable value as pre-conditioning is completed. Above the preconditioning limit, the resilient modulus does not change significantly with an increase in the number of loading cycles.

5. For total stress levels less than 70kPa, both the dynamic and static stress levels have only a minor effect on the resilient modulus of EPS. The reason is due to the stress levels remaining within the linear elastic range of the EPS materials.

6. It seems that there are two distinct resilient behaviours: below 70 kPa the geofoam remains resilient but above 70 kPa, the material loses resilience very quickly.

7. The limiting stress or strain versus load repetition criterion for 20 kg/m³ EPS geofoam in pavement subgrade application developed in this thesis may be used predict the approximate load repetitions to failure (at accumulated permanent strain of 0.5%).
For a given service life, this criterion may also be used to calculate the limiting applied stress or applied strain level in pavement design.

Based on the limiting stress or strain criterion for dynamic stress level less than 23 kPa, or given a resilient strain less than 0.15% on the 20 kg/m$^3$ EPS geofoam, the accumulated permanent deformation will not exceed 0.5% and the effect of the loading cycles can be neglected.

The similar model may be used to predict the long-term behaviour of EPS under both static and dynamic stress conditions. However, the model parameters might be adjusted by long-term testing under different static and dynamic loading, and further validations of the creep model for longer time cyclic loading tests are necessary.
CHAPTER 7
EPS BEHAVIOUR IN MODEL PAVEMENT TEST

7.1. INTRODUCTION

Although some laboratory studies (e.g. Eriksson and Trank 1991, Horvath 1995, and Preber et al. 1994) have shown that EPS geofoam could be used successfully in geotechnical applications including pavements and embankments, however, the basic material behaviour is investigated by carrying out the appropriate standard laboratory tests under controlled conditions and then extrapolating the results to field applications. There is a limit, however, to the use of laboratory results for the interpretation of field performance because standard laboratory tests can never fully replicate actual conditions in the field. It has been shown in similar investigations (Sparks 1970, Taylor 1966, and Moghaddas-Nejad and Small, 1996) that some of the limitations of standard laboratory tests can be overcome using physical models, thereby providing better insights into the material performance under more realistic conditions.

This chapter will present the model tests simulating the application of repeated vehicular loading in realistic yet controlled conditions. EPS geofoam fill was used below the model pavement instead of normal conventional soil. The test facility instrumentation was capable of accurately measuring the deformations of the pavement and the EPS subgrade when subjected to repeated vehicular loading simulating live traffic.

In the series of model pavement tests, the behaviour of EPS under repeated vehicular loading was investigated. The influence of the EPS block size and the effects of lateral restraint on EPS blocks were studied. In addition, the performance of EPS geofoam blocks and conventional sand (the baseline case) as subgrade materials were compared and assessed.
7.2. SCALE MODEL TEST

The principle of similitude is now normally applied to experiments with models. The use of the "principle of similitude" in connection with scale models is great importance (Huntley 1967). The number of physical variables in certain problems is so large that it is not possible to write down the equations that relate them, not to mention solving them. But if the physical quantities involved are all known, then the dimensional analysis can find some of the necessary relations that exist between these quantities thereby, in effect, reducing the number of parameters. On the other hand, allowing the behaviour of a full-size problem to be inferred from that of a scale model, the measurements of the physical variables connected with model may, with the help of the principle of similitude, be made to yield useful information not economically obtainable by any other means.

More details of the Dimensional Analysis Methods and advantages of their applications can be found in Focken (1953), Pankhurst (1964), Huntley (1967) and others.

7.3. OVERVIEW OF TESTING FACILITY

The model test facility used in the current series of tests was first developed by Wong and Small (1994) at one-quarter scale (1:4) of the full dimensions in order to study the effect of orientation of approach slabs on pavement deformation. The facility was described in detail by Wong (1991), and a brief overview is given herein.

There are four main structural components in the facility namely, the overhead track, the test tank, running track and the loading carriage. A loading wheel is driven around an oval-shaped track by an overhead guided-rail system as shown in Figure 7.1.
The oval-shaped track is chosen because it offers some advantages over both the linear and circular tracks — typically, a linear track requires an elaborate mechanism (Sparks 1970, and Brown and Brodrick 1981) to control the loading wheel so that it can perform the following functions.

1. Accelerate from the initial starting position to the desired speed.
2. Pass over the test pavement at a constant speed.
3. Decelerate and stop at the end of travel.
4. Be lifted to the initial starting position to commence the next pass.

The disadvantage of a circular track is that it requires a large area and that the wheel does not follow a straight path across the pavement. The oval–shaped track at Sydney University by contrast has a relatively simple mechanism, is compact in size, and allows the use of a straight test section of pavement.

The test section of model pavement was constructed inside the test tank and positioned below one of the straight sections of the overhead track. The test tank consists of two hollow steel boxes bolted together on the flanges, and is 1.4m long by 0.5m wide by 0.8m deep internally. Several LVDTs were installed to measure subsurface displacement in the test pavement and the subgrade. A conductor rail system supplies power to the motor, which drives the test wheel. The wheel passes over the test section of pavement once during each revolution around the running track and triggers a microswitch that starts the recording data by a computer. Wheel load data from the loading carriage can also be sent back to the computer.
The loading carriage consists of two bogies, a spring loading system, a driven pneumatic tire, and a driving unit utilizing timing belts and grooved pulleys (Figure 7.2). The tire is supported by the rotating arm, which is pivoted at the top by means of roller bearings. Four identical springs with guide rods placed concentrically inside them make up the loading unit. By means of a hinged connection, the spring loading is connected to the bottom bogie plate. The compression in the springs can be changed by adjustment of the nuts above and below the rotating arm. A LVDT fitted between two reaction plates is used to monitor the variation of spring compression. The top of the loading system is connected to the bottom bogie plate of the trolley. Therefore, as the trolley runs along the test track, the wheel load is influenced by the levelness of the track.

The test facility is situated in a temperature-controlled room where temperature was maintained at approximately 23°C to minimize thermal effects that may have influenced the properties of the pavement and subgrade materials.

The maximum wheel load is limited to 1.4 kN and the maximum speed to 7.2 km/h for safety reasons.
Figure 7.3. A typical model pavement test section and the arrangement of the instrumentation
7.4. EXPERIMENTAL PROCEDURE

7.4.1. Pavement and Subgrade Material

The model pavement section consisted of a wearing course, a road base and a sand subbase, all of which overlaid the subgrade (which in this series of tests was either compacted sand or EPS blocks) and an uncompact sand layer used for leveling at the bottom of the test tank. A typical model pavement test section is presented in Figure 7.3.

Wearing Course

The wearing course used in road pavements varies widely, but typically the maximum stone size is about 20 to 25 mm. Thus a proprietary bituminous mix (known commercially as Pavefix) with a bitumen content of approximately 5% and a maximum aggregate size of 5 mm was chosen for the wearing course, based on the one-quarter scale of the models. Although the mix could be compacted at ambient temperature, as claimed by the manufacturer, it was heated for approximately 4 hours at 150°C before compaction because this practice resulted in a superior performance as shown in the test results.

Road Base

The base layer comprised crushed aggregate (basalt) with nominal size of 5mm and mainly subangular particles. The coefficient of uniformity is 2, $D_{10}$ being approximately 2.0 mm and $D_{60}$, 4.0 mm. It is classified as a uniform fine gravel with less than 1% of particles finer than 75 μm. The particle size was chosen so that it was roughly scaled in accordance to the model dimensions. The minimum and maximum densities of the base material were 1.38 t/m³ and 1.49 t/m³, respectively.

Sub-base and Subgrade

As discussed previously, two types of subgrade were used in this series of testing. For the base case, Sydney sand, which is a silica sand, was used as the conventional subgrade. It has a uniformity coefficient of 1.5 where sizes of $D_{10}$ and $D_{60}$ are approximately 0.25 mm and 0.37 mm respectively. Sydney sand is classified as a well graded sand with less than 1% of fines. The minimum density was 1.44 t/m³, while the
average maximum density was 1.69 t/m$^3$. The same sand was also used for the compacted sand sub-base.

The second subgrade material used was EPS geofoam of density 20 kg/m$^3$, the density most commonly used in geotechnical filling applications. The compressive strength (at 10% compressive strain) of this grade of EPS foam was about 98 kPa and the yielding stress was about 84 kPa (Chapter 3).

Compaction Methods
Due to the strong influence of the degree of compaction on the granular material deformation characteristics, the compaction procedure was chosen so as to give a uniform layer with similar densities and particle structures. To obtain a reproducible test result, close control of compaction processes was exercised.

A manual compaction method was selected for this series of tests. In order to ensure uniformity and repeatability, filling was completed in seven sections in any single layer as described by Wong (1991) and Moghaddas-Nejad (1996). The number of blows for all sections was specified to be 20 and the height of fall of the compacting tool was 50 mm at each section of a layer. After compaction, the average densities of wearing course, base layer and sand subgrade were found to be 1.75 t/m$^3$, 1.5 t/m$^3$ and 1.65 t/m$^3$ respectively.

Strength Parameters of Pavement and Subgrade Material
The strengths of pavement structure materials can vary and are influenced by many factors such as thickness, compaction efforts, moisture content, load level, temperature and methods of sample preparation. It is worth measuring the strength parameters of materials under the same conditions as they are tested. For this reason, the Standard California Bearing Ratio (CBR) Tests were carried out on the crushed aggregate and compacted sand while cone penetration tests were conducted on the wearing course, crushed aggregate, and compacted sand as well as the EPS blocks using a hand-held penetrometer. For the EPS material with a density of 20 kg/m$^3$, unconfined cyclic compression tests were also carried out on $\varnothing 50\text{mm} \times 50\text{mm}$ cylindrical specimens to measure the resilient modulus ($E_r$). The strength parameters of the pavement and subgrade materials are shown in Table 7.1.
Table 7.1 Strength and deformation parameters of pavement materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Resilient modulus Er (Pa)</th>
<th>Cone penetrometer resistance (MN/m$^2$)</th>
<th>CBR (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wearing course (Pavefix)</td>
<td>N/A</td>
<td>7.6</td>
<td></td>
</tr>
<tr>
<td>Crushed aggregate</td>
<td>$140 \times 10^9$*</td>
<td>3.8</td>
<td>14</td>
</tr>
<tr>
<td>Compacted sand</td>
<td>$50 \times 10^6$*</td>
<td>1.0</td>
<td>5</td>
</tr>
<tr>
<td>EPS (20 kg/m$^3$)</td>
<td>$5 \sim 6 \times 10^6$</td>
<td>0.5</td>
<td>1.5 $\sim$ 2.0</td>
</tr>
</tbody>
</table>

Notes: * The resilient modulus was estimated using the following relationship: $E, (\times 10^9$Pa) = 10 x CBR value (Austroad 1992). N/A = not available. $\rho_{EPS}$=20kg/m$^3$.

Wheel Loading

The pneumatic tire selected for the test facility had a diameter of 230 mm, which was about one quarter the size of a full-scale truck tire. In the current series of tests, the applied wheel load ranges from 490 to 690N. The tire pressure was kept at 210kPa. As described in an earlier section, the wheel loading was provided by four springs in compression, the magnitude of the load depending on the spring compression and stiffness. Based on the calibration data of Wong (1991), the relationship between wheel load and the spring compression, and wheel load and contact area were obtained as:

$$F = 26.9L + 4192$$

(7.1)

$$A = 0.027F + 14.433$$

(7.2)

where $F$ is the wheel load in Newtons, $L$ is the compression of the springs in millimeters and $A$ is the contact area of the tire at a pressure of 210kPa in square centimeters. Equation (7.1) was used to calculate the wheel load, which was then divided by the contact area in Equation (7.2) to give the contact stress.

The loading unit had a measure of difficulty keeping the wheel load constant during running of the test. As the deformation of the pavement is increased, it releases some of the spring compression in the loading system, thereby causing the wheel load to decrease. A 1mm deformation in the pavement surface results in a wheel load decrease about 13.5 N, which translates to about 2.0kPa in contact stress for the applied loading in the current experiments. During testing, the wheel load was regularly checked and, if necessary, the spring compression was adjusted so as to maintain the desired wheel load.
The speed of the wheel going around the test track was set to 2.7 km/h or about 11 km/h on the actual scale. This gave a loading frequency of about 241 passes per hour for the current series of tests.

7.4.2. Test Program

The current test series consisted of five tests using an EPS geofoam and one test using a sand subgrade (baseline case). Table 7.2 summarizes the parameters used in the test series. Tests 1, 2 and 3 used large blocks (the definition of “large” block is given in section 7.4.3) and all had identical experimental setups except for the magnitude of the wheel load. All of the tests were run to gather information on pavement and EPS geofoam response when subjected to increased wheel loads.

In Test 4, no lateral restraint was provided for the EPS blocks. As a result, a gap of approximately 10 mm-wide gap was left between the EPS blocks and the side walls of the test tank. Test 5 examined the effects of block size by using “small” blocks (the definition of “small” is in section 7.4.3) instead of the standard large blocks. Finally for the purposes of a comparative study, Test 6 (the baseline case) used conventional compacted sand instead of an EPS geofoam subgrade.

Table 7.2 Summary of test parameters

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Subgrade</th>
<th>Laterally Restrained</th>
<th>Max wheel load, (N)</th>
<th>Contact stress*, (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EPS (big)</td>
<td>Yes</td>
<td>690</td>
<td>209 × 10^3</td>
</tr>
<tr>
<td>2</td>
<td>EPS(big)</td>
<td>Yes</td>
<td>590</td>
<td>196 × 10^3</td>
</tr>
<tr>
<td>3</td>
<td>EPS(big)</td>
<td>Yes</td>
<td>490</td>
<td>179 × 10^3</td>
</tr>
<tr>
<td>4</td>
<td>EPS (big)</td>
<td>No</td>
<td>590</td>
<td>196 × 10^3</td>
</tr>
<tr>
<td>5</td>
<td>EPS (small)</td>
<td>Yes</td>
<td>590</td>
<td>196 × 10^3</td>
</tr>
<tr>
<td>6</td>
<td>Sand (baseline case)</td>
<td>Yes</td>
<td>540</td>
<td>188 × 10^3</td>
</tr>
</tbody>
</table>

Note: * Tire pressure = 210×10^3 (Pa) for all tests.
7.4.3. Pavement Thickness And EPS Blocks

Pavement Thickness. The pavement layers were made thick enough to diffuse the imposed loading so that the combination of pavement live and dead load did not stress the EPS geofoam beyond a given stress/strain level. This ensured that unacceptable deflections and permanent deformations of the pavement would be avoided. At full scale, typical thickness of a pavement wearing course, base layer and sand sub-base are: 80 mm, 200 mm and 200 mm respectively. Thus the corresponding model thickness of these layers at one-quarter scale (i.e. the model scale used in this thesis) were 20 mm, 50 mm and 50 mm, respectively.

EPS Geofoam Blocks. The block sizes in actual applications vary over a range of dimensions. Based on the size of the test tank and the possible block sizes in the field, the size of the EPS geofoam chosen for the model was 500×160×100 mm (this is referred to as the large block in current series of tests). Each is a rectangular parallelepiped block that was carefully cut using a hot wire. Since a one-quarter scale was used, a large block represents a geofoam block of size 2000mm×480mm×400 mm for the prototype.

The small blocks, with half the length of the large blocks (i.e. measuring 250mm×160mm×100 mm), were used in Test 5 to study the potential effects of block size. It should be noted that a small block arrangement results in more joints and block-to-block interfaces than for the large blocks.
In all the experiments involving EPS, four layers of geofoam each 100 mm thick were arranged inside the test tank. Each layer was placed horizontally and in a pattern that minimized the vertical continuity of joints between blocks. The longest dimension of the blocks in one layer was aligned orthogonal to the longest dimension of the blocks in the adjacent layers both above and below it. This arrangement is illustrated in Figure 7.5.

![Figure 7.5 The arrangement of EPS blocks](image)

7.4.4. Measurement of Surface and Subsurface Deformation

The displacement measurements were taken at two sections in the test tank using LVDTs. The plan and section views showing the positions of the transducers are given in Figure 7.3. Three LVDTs were placed in each section (T1, T2 and T3 in section 1; T4, T5 and T6 in section 2) along the longitudinal centerline of the pavement at three elevations, namely the wearing course-base interface, sub-base-EPS block interface and in the middle of the EPS blocks. Also two additional LVDTs were installed in each section (T7 and T8 in section 1; T9 and T10 in section 2) at the sub-base-EPS block interface beside the center LVDTs (T2, T5). In Test 6, the LVDTs were placed at the same positions and elevations
as the EPS geofoam. This LVDT setup was capable of measuring displacements to an accuracy of ±0.01 mm.

Several drawing pins were pushed into the surface of the pavement directly above the LVDTs, along a line perpendicular to the travel direction of the wheel, to act as settlement points. Displacement readings taken at the settlement points allowed construction of the surface deformation profile at the same cross-section as the LVDTs. The surface measurements were made using a micrometer for measuring deflections to an accuracy of ±0.01 mm with the help of a straight edge placed across the sides of the test tank as reference.

As the driving wheel passed over the test tank, the subsurface displacements were recorded at every second pass for the first 20 passes, every fifth pass up to 100 passes, every tenth pass up to 200, and fiftieth pass between 200 to 1000, and every hundredth pass after 1000 passes. The surface displacement was measured at 5, 10, 20, 50, 100, 200, 500 and 1000 passes, and after that, at every thousandth pass. A typical displacement curve for the first three passes is shown in Figure 7.6 for LVDT T2 at the top of subgrade. The maximum displacement was achieved as the driving wheel passed directly above the LVDT and then decreased as the wheel load is released. For the pavement system as a whole, some permanent displacement was clearly recorded with each pass of repeated loading. It was found that there is a small gap between the end of the first pass and the beginning of the second pass, which indicated a slow recovery between the two passes.

Referring to Figure 7.6, the maximum displacement of each pass is defined in this thesis as the maximum instantaneous displacement, the initial displacement of the nth pass is taken as the cumulative permanent (plastic) displacement of the (n-1)th pass, and the resilient displacement is the recoverable part of the total deformation. The typical maximum and cumulative permanent displacement curves are shown in Figure 7.7. The largest incremental plastic deformation occurs in the first few hundred passes of the driving wheel, after which the rate of increase is negligible.
Figure 7.6 Typical displacement behaviour at the top of subgrade

Note: Readings by LVDT T2 at the top of subgrade under centerline (section 1)

Figure 7.7 Typical maximum and cumulative permanent displacement curves
CHAPTER 7: EPS BEHAVIOUR IN MODEL PAVEMENT TEST

Figure 7.8 Cumulative permanent displacement at the top of subgrade under the center of the wheel load

7.5. TEST RESULTS

7.5.1. Deformation of Subgrade

The results for subgrade deformation have been categorized into permanent (plastic) and resilient (recoverable) displacements as measured by the LVDTs T1 and T4, which were installed in the middle of the subgrade (depth of 320mm).

Figure 7.8 shows the typical cumulative permanent displacement curve at the top of the EPS subgrade under the center of the wheel load for Tests 1, 2 and 3 at maximum wheel loads of 690N, 590N and 490N respectively. For the same wheel load, cumulative permanent deformation increases rapidly in the first approximately 500 passes and more gradually after that. Differences in deformation readings were recorded at sections 1 and 2 even though these were two identical sections in the test tank. It was therefore decided that, for the sake of consistency, comparison of test results should be made only for the same test section.

Figure 7.9 shows that increasing the wheel load from 590 N to 690 N results in significantly higher cumulative permanent deformation than for the same wheel load increment from 490 N to 590 N. Therefore, permanent (plastic) displacement appears to
increase (in a highly nonlinear fashion) as the magnitude of the wheel load increases even though the stress levels in the EPS geofoam are fairly small, falling within the linear range of the stress-strain relationships. The previous remark is based on finite element analysis which revealed that the maximum stress at the top of the blocks, for the imposed loading in this series of tests (estimated to range between 10 to 15 kPa), is well below the linear limit of the stress-strain. The reason is that, as a plastic material, the plastic deformation of the EPS can be ignored only if the applied stress is very small.

Figure 7.9 Cumulative permanent (a) and resilient deformation (b) at the top of the EPS subgrade after 5000 cycles

Figure 7.10 Cumulative permanent displacement at the top of the EPS subgrade
In Figure 7.10, the cumulative permanent displacements at the top surface of the EPS layer after 5000 passes for Tests 2, 4, and 5 are shown (maximum wheel load = 590 N). It may be recalled that the large blocks were used in Test 2, large blocks with no lateral restraint were used in Test 4, and small blocks were used in Test 5. After the first 1000 passes, the deformation curve flattens becoming fairly uniform and displaying a small rate of increase per pass.

It is worth noting that the length of a large block (500mm) is twice the length of a small block (250mm), which results in a block-block interface area for Test 5 that is approximately 1.4 times that of Test 2. The test data provided no evidence to suggest that there was any significant change in deformation behaviour of the EPS blocks when the block length was halved.

Data from Test 2 and Test 5 also suggest that the presence or lack of lateral confinement to the EPS blocks did not result in any significantly different behaviour. On the contrary, a higher deformation was measured in Test 2 (lateral restraint on the blocks) when compared to Test 4 (no lateral restraint). This behaviour is vastly different in comparison to granular material, which derives much of its shear strength and stiffness from the presence of lateral confinement.

Also shown in Figure 7.8 are the results for Test 6 representing the baseline case (maximum wheel load = 540 N) where the subgrade was compacted sand instead of EPS geofoam. For sand, the increment of the cumulative permanent deformation was more gradual over the first 3000 passes but also flattens out to a more gradual rate of increase. After 5000 passes, the deformation reached approximately the same magnitude, if not marginally higher than the EPS geofoam cases. Test 2 and 3, which had wheel loads of 490 N and 590 N, respectively, provided the lower and upper loading bounds for comparison with the baseline case.

Unlike the permanent deformation, when resilient deformation after 5000 passes is plotted against the wheel load in Figure 7.9b (for Tests 1, 2 and 3), the resulting curve is approximately linear because the stress levels are small and fall within the initial linear section of the stress-strain curve.
The data for the resilient deformation per pass are shown in Figure 7.11. For Tests 2, 3, 4 and 5, the curve flattens after approximately 1000 passes while for Test 6, this only occurred after approximately 3000 passes. While the magnitude of the cumulative permanent deformation, especially after 5000 passes, did not differ significantly between EPS blocks and compacted sand for the same approximate wheel load, the same did not apply for resilient deformation. The results show that after 5000 passes, the magnitude of the resilient deformation for the baseline case is about 25 μm while for the EPS cases it is between 65 μm and 110 μm or approximately 2.5 to 4.5 times greater.

The breakdown of the total deformation (i.e. the previously defined maximum instantaneous deformation) during the load cycle into its component permanent and resilient parts after 5000 passes is presented in Table 7.3. The experimental evidence suggests that, at the subgrade level, the EPS geofoam performs just as well as compacted sand in terms of the permanent deformation, but fared worse for resilient deformation during a load cycle. It is quite clear that resilient deformation dominates over the permanent deformation during a load cycle, for readings up to 5000 passes, although the cumulative permanent deformation is much higher than the resilient deformation.

The effects of resilient deformation on surface deformation are discussed further in section 7.6.2.
Table 7.3 Cumulative permanent and resilient displacement after 5000 Passes

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Subgrade</th>
<th>Cumulative permanent displacement, (mm)</th>
<th>Resilient displacement, (mm)</th>
<th>Max wheel load, (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EPS (large blocks)</td>
<td>0.66</td>
<td>0.16</td>
<td>690</td>
</tr>
<tr>
<td>2</td>
<td>EPS (large blocks)</td>
<td>0.31</td>
<td>0.11</td>
<td>590</td>
</tr>
<tr>
<td>3</td>
<td>EPS (large blocks)</td>
<td>0.27</td>
<td>0.07</td>
<td>490</td>
</tr>
<tr>
<td>4</td>
<td>EPS (large blocks)</td>
<td>0.30</td>
<td>0.10</td>
<td>590</td>
</tr>
<tr>
<td>5</td>
<td>EPS (small blocks)</td>
<td>0.29</td>
<td>0.12</td>
<td>590</td>
</tr>
<tr>
<td>6</td>
<td>Sand (baseline case)</td>
<td>0.31</td>
<td>0.03</td>
<td>540</td>
</tr>
</tbody>
</table>

7.5.2. Deformation of Pavement Surface

Figure 7.12 shows the cross-sectional view of the cumulative permanent displacements of the pavement surface after 5000 passes for the current series of tests. A rut developed at the center of the pavement where the wheel load passed whereas heaving occurred on both sides of the wheel path. The rutting depth is the smallest for Test 6 (baseline case) and the largest for Test 1 (the greatest wheel load, 690 N, of all the tests was used). A permanent deformation profile similar to the one at the pavement surface was observed for the top surface of the EPS blocks (i.e. at the subbase-EPS interface), but at significantly reduced amplitude. These results may seem paradoxical when it is observed that, at the subgrade level (see Figure 7.10), there is little difference between the recorded cumulative permanent deformation at 5000 passes for sand (Test 6) and the EPS geofoam at approximately similar wheel load (Tests 2, 3, 4, and 5).
Figure 7.12. Deformation of the pavement surface after 5000 passes

Figure 7.13. Cumulative permanent displacement of the pavement surface and the top of subgrade
Figure 7.13 presents the curves for cumulative permanent deformation at the pavement surface and at the top of the subgrade of both compacted sand (Test 6) and EPS material (Tests 2, 4 and 5). After the initial rapid increment, the deformation curve flattens to a gradual rate of increase for both the pavement surface as well as the subgrade. It clearly shows that most of the deformation is taking place in the pavement layers. This figure again suggests that while the permanent deformation of EPS and compacted sand are not significantly different at subgrade level, the difference is however very much accentuated on the pavement surface. Compacted sand clearly performs better than EPS geofoam in relation to pavement surface deformation. Further discussion pertaining to this is developed in section 7.6 below.

7.6. DISCUSSION

7.6.1. Extrapolation of Results (Permanent Deformation)

It was observed (Figure 7.8 and 7.10) that, the cumulative permanent deformation versus pass number curve after the first 1000 passes for EPS geofoam is not only very gradual but also approximately linear. Based on this observation, permanent deformation data from Tests 2, 4 and 5 can be plotted on a common chart after excluding the first 1000 passes (i.e. the permanent deformation after the 1000th pass deduct the permanent deformation at the 1000th pass). The data in Figure 7.14 shows that the contact stress has a small influence on the permanent deformation after 1000 passes. By using linear regression analysis, an approximate relationship between the cumulative permanent deformation and cumulative number of load cycles (passes), $N$, is established as:

$$d_{EPS} = 6.05 (N - 1000) \quad N \geq 1000$$

(7.3)

where $d_{EPS} = \text{cumulative permanent deformation after 1000 load cycles or passes (\mu m)}$; and $N = \text{cumulative number of load cycles (passes) in kilocycles}$. The correlation coefficient of the linear regression is 0.97 and standard error is 0.002. Equation (7.3) yields a permanent deformation rate of approximately 6.05 \(\mu m\) per kilocycle. The relationship between the cumulative permanent deformation and cumulative number of load passes after 3000 passes (Figure 7.15) for compacted sand was established as:
Figure 7.14. Cumulative permanent deformation at the top of EPS blocks (depth of 120mm)

Figure 7.15. Cumulative permanent deformation at the depth of 120mm in compacted sand subgrade
\[ d_{\text{sand}} = 7.36 \left( N - 3000 \right) \quad N \geq 3000 \quad (7.4) \]

where \( d_{\text{sand}} \) = cumulative permanent deformation after 3000 load cycles or passes (\( \mu m \)); and \( N \) = cumulative number of load cycles (passes) in kilocycles. The cumulative permanent deformation rate is marginally higher than that for EPS geofoam. This is another clear indication that at the subgrade level, EPS geofoam did not incur more permanent deformation than compacted sand. Extrapolating to a hypothetical \( 1 \times 10^6 \) passes yields a deformation of 6.05 mm for EPS geofoam and 7.36 for compacted sand. At full scale, these represent cumulative permanent deflections of approximately 24 mm and 29 mm respectively, which normally would be considered as acceptable. As discussed below, within each load cycle or pass, it is the resilient rather than the permanent component of the deformation of the subgrade that appears to govern the rutting deformation at the pavement surface.

7.6.2. Resilient Deformation and Pavement Rutting

It is generally thought that rutting in the pavement is related to the relative stiffness of the pavement material and its subgrade (Austroads, 1992). It should be noted that the pavement structures for all of the tests were constructed identically and, therefore, had identical stiffness. Based on the experimental data, any difference in pavement performance and behaviour among the tests could only have been a result of the wheel load and/or subgrade (Table 7.2) after ruling out the effects of lateral restraint. As well, any permanent deformation at the subgrade would have propagated upward and been manifested as a part of the surface rut. However, experimental data show that the cumulative permanent deformation in either EPS geofoam or compacted sand at the subbase-subgrade interface is significantly small (approximately 0.3 mm after 5000 passes) compared to the surface rut depth (15 to 20 or 8 mm, respectively). Therefore, while permanent subgrade deformation can account for a small amount of the rut depth, it cannot clearly account for all the rutting that occurred. This raises the question: If the permanent deformations of the compacted sand and the EPS are approximately the same at the subgrade level, why then is the difference magnified at the pavement surface (EPS to compacted sand rut depth ratio ranges from 2 to 3)?
CHAPTER 7: EPS BEHAVIOUR IN MODEL PAVEMENT TEST

The answer could possibly be found by observing the deformation within each cycle or pass. After the initial passes, the permanent deformation per pass becomes constant at approximately $6.05 \times 10^{-6}$ mm for EPS, and $7.36 \times 10^{-6}$ mm for compacted sand (from equations 7.3 and 7.4, respectively), whereas the resilient deformations are 65 to $110 \times 10^{-3}$ mm and $25 \times 10^{-3}$ mm, respectively. Apart from the fact that resilient deformation dominates over permanent deformation for each pass, resilient deformation is approximately 2.5 to 4.5 times higher for the EPS than for the compacted sand as previously mentioned. It is suggested that for the current series of tests, the large difference in resilient deformation at the subgrade is ultimately manifested as the large difference in deformation at the pavement surface. EPS is a soft material (as the strength and estimated modulus values in Table 7.1 demonstrate), particularly when compared to compacted sand, indicating that the results could have been predicted.

7.6.3. Implications for Pavement Design

The EPS geofoam block size and lateral restraints did not appear to significantly influence pavement performance. The latter is in general agreement with previous laboratory studies indicating that there is very little dependency between lateral confining pressure and the EPS geofoam strength and stiffness. (e.g. results in Chapter 3, and Preber et al. 1994). Therefore, the results in this study present engineers with added flexibility in design.

The model testing demonstrates that resilient modulus of EPS geofoam is a most important factor influencing pavement deformation at the surface. Correlation to the resilient modulus of EPS or its equivalent would be useful for pavement design. In pavement structure design, the EPS can be considered as normal subgrade material with low resilient modulus.

It is important not to lose sight of the fact that EPS geofoam is typically used to address the problems of soft soil because of its low compressibility and bearing capacity. Also, the low resilient modulus of EPS geofoam can easily be overcome by designing a suitable pavement structure, i.e. by increasing the pavement thickness or using stiffer EPS geofoam.

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7.7. SUMMARY

The behaviour of EPS geofoam subgrade under pavement subjected to repeated wheel loads in a model testing facility was investigated and resulted in the following conclusions:

1. In terms of permanent (plastic) deformation at subgrade level, EPS geofoam performs as well as, if not better than, compacted sand under the repeated traffic loading.

2. The permanent (plastic) deformation occurs even at small stress levels. The magnitude of cumulative permanent deformation increases nonlinearly with the loading.

3. For an EPS geofoam subgrade (and a compacted sand subgrade), the resilient deformation (or by extension, the resilient modulus) will have a significant influence on the rut depth of the pavement surface.

4. Resilient deformation of EPS at the subgrade level is much higher than that for compacted sand. The resilient deformation manifests as a deeper rut on the pavement surface of the EPS geofoam subgrade test section than on the compacted sand subgrade test section, even when both have the same pavement structure. The rut depth could be reduced, however, by using an appropriately designed pavement structure (e.g. increase the pavement thickness or use a stiffer pavement).

5. Test results showed that block size and lateral restraint did not significantly affect the performance of the EPS geofoam blocks. These findings will give design engineers added flexibility in design.
CHAPTER 8
A ONE-DIMENSIONAL MODEL OF EPS FILL ON SOFT SOILS

8.1 Introduction

The long-term behaviour of the EPS geofoam embankment and foundation clay reveal important details of the performance of the construction that are sought by the geotechnical engineer.

Long-term behaviour of a material is defined by various aspects of the material response under stress and strain loading and includes the time-dependent phenomena of creep and stress relaxation. In fill applications, it is the creep response in the geofoam that will be a major design consideration. The viscoelastic model of the EPS geofoam has been developed in this thesis (Chapter 5) which includes creep effects. In this chapter, this viscoelastic model is combined with the time-dependent consolidation of soft soil to produce a model that is capable of simulating the long-term settlement behaviour of a typical EPS fill application.

It is worth noting from the onset that EPS fill and the underlying soft soils are constitutively quite different materials since the former does not deform in the same manner as a consolidating soil where excess pore water are being expelled under effective stress. Thus the EPS geofoam is treated as a viscoelastic material whereas the consolidating soil is considered a poro-viscoelastic material.

In this chapter, a semi-analytic model is developed for an EPS embankment resting on consolidating soft soil based on the principles of visco-elasticity. The model is then used in an illustrative example to show the relative merits of using the EPS fill where the foundation soil is very soft.
8.2 MODEL DEVELOPMENT

The proposed model considers the long term deformation of two material types: EPS fill and the underlying consolidating soft soil. In many geotechnical application such as in a pavement construction, the extensive surface area of the applied loading is very large relative to the thickness of structure. The problem, in this case, can then be idealized as a one-dimensional problem. A schematic of this is illustrated in Figure 8.1, where it is shown that the EPS blocks rest on a soft soil foundation separated by a thin layer of sand used both for leveling and as a drainage layer.

A FORTRAN program was developed to solve for the creep and consolidation using a matrix transfer method (this method is further elaborated in section 8.3). The settlement arising from the deformation of the EPS layer is obtained as the product of the strain and the depth of EPS fill layer. This result is superimposed on to the settlement from the consolidating soft soil (Leo and Xie 2001) to obtain the total settlement of the system.

8.2.1 Viscoelastic Creep Model for EPS Geofoam

With few exceptions, the fundamental equation for creep models has the following qualitative form (Findley 1976):

\[ \varepsilon(t) = \varepsilon_0 + \varepsilon_c(t) \]  

(8.1)
where: $\varepsilon(t) =$ the total strain at time $t$ after the stress application; $\varepsilon_0 =$ the instantaneous strain upon a stress application and $\varepsilon_0(t) =$ the time-dependent creep component of strain at some time $t$ after the stress application. For the small stress ranges considered in the present case and which occur under representative conditions in pavement and embankments, the instantaneous strain is given by,

$$\varepsilon_0 = \frac{\sigma}{E_1}$$  \hspace{1cm} (8.2)

where $\sigma$ is the applied stress, $E_1$ is the linear spring constant (or the Young's modulus) of the geofoam.

If the stress history is piecewise differentiable, then the creep component of strain under any given stress history may be expressed as a convolution integral,

$$\varepsilon_c(t) = \int_0^t J(t - \tau) \frac{\partial \sigma(\tau)}{\partial \tau} d\tau$$  \hspace{1cm} (8.3)

where the kernel $J(t)$ is the compliance for the creep component of strain whose form depends on the model adopted. Equation (8.3) forms the basis of hereditary creep in that the material "remembers" the stress history it has been subjected to.

As experimental data yields typically parabolic creep curves (Chapter 5), the modified 4 element model was selected as the model that best represented the creep characteristics of EPS geofoam since it enables the creep strain rate to asymptote to a parabola as $t$ tends to infinity. Similar results were also presented by Taylor et al. (1997), in applying mechanical models to flexural creep deflection of a structural insulated panel.

The strain equation for the modified four-element model for a constant stress $\sigma_0$ applied at $t = 0$ is

$$\varepsilon(t) = \frac{\sigma_0}{E_1} + \frac{\sigma_0}{\eta_1} t^n + \frac{\sigma_0}{E_2} \left(1 - e^{-\frac{E_{2t}}{\eta_2}}\right)$$  \hspace{1cm} (8.4)

where $E_1, E_2, \eta_1, \eta_2,$ and $n$ are experimental model parameters and the compliance equation for creep gives,

$$J(t) = \frac{1}{\eta_1} t^n + \frac{1}{E_2} \left(1 - e^{-\frac{E_{2t}}{\eta_2}}\right)$$  \hspace{1cm} (8.5)

This model is developed for a geofoam material supplied by an Australian firm, with a small stress range (0-40 kPa) in mind. This stress range has practical interest since it typically occurs in representative conditions in road sub-base and embankments where
EPS fills are being used. In Chapter 5, it has been described how the creep data from laboratory creep tests were collated and normalised with respect to the applied stress i.e. $\varepsilon/\sigma_0$, and then the creep parameters were determined using a Levenberg-Marquardt nonlinear least squares regression procedure (Levenberg 1944, Marquardt 1963, and Dennis and Schnabel 1983). The identified parameters of the modified 4-element model for EPS foam of specified 20 kg/m$^3$ density are tabulated in Table 8.1.

<table>
<thead>
<tr>
<th>$n$</th>
<th>$E_1$ (kPa)</th>
<th>$\eta_1$ (kPa-yr)</th>
<th>$E_2$ (kPa)</th>
<th>$\eta_2$ (kPa-yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.165</td>
<td>4620</td>
<td>14000</td>
<td>77520</td>
<td>0.131</td>
</tr>
</tbody>
</table>

### 8.2.2 Creep Model of Soft Soil

In field applications, the EPS blocks rest on a soft soil foundation commonly separated by a thin layer of sand used both for leveling and as a drainage layer. The consolidation of the soil with time effects can be described by the constitutive equation:

$$\varepsilon(t) = \frac{\sigma'(t)}{E_1} + \frac{1}{\eta_1} \int_0^t J(t-\tau) \frac{\partial \sigma'(\tau)}{\partial \tau} d\tau$$  \hspace{1cm} (8.6)

where $\sigma'$ = effective vertical stress, the kernel $J(t)$ is the compliance for the creep component of strain whose form again depends on the model adopted. If a 4-element (Burgers) model had been use for the consolidating soil in this case, the compliance equation for soil creep gives

$$J(t) = \frac{1}{\eta_1} + \frac{1}{E_2} \left( 1 - e^{-\frac{E_2 t}{\eta_2}} \right)$$  \hspace{1cm} (8.7)

so that

$$\varepsilon = \frac{\sigma'}{E_1} + \frac{1}{\eta_1} \int_0^t \sigma'(\tau) d\tau + \frac{1}{E_2} \frac{\sigma'(t)}{\eta_2} e^{\frac{E_2 t}{\eta_2}} d\tau$$ \hspace{1cm} (8.8)

The constitutive equation (8.8) has to be solved in combination with the continuity equation describing drainage of the pore fluid in the soil,
\[
\frac{\partial \varepsilon_v}{\partial t} = \frac{\partial \varepsilon}{\partial t} = -\frac{k_v}{\gamma_w} \frac{\partial^2 u}{\partial z^2}
\]  

(8.9)

where \( \varepsilon_v \) = volumetric strain; \( k_v \) = coefficient of vertical permeability; \( \gamma_w \) = unit weight of fluid; \( u \) = excess pore pressure, to give the 1D equation consolidation with time effects,

\[
c_v \frac{\partial^2 u(z,t)}{\partial z^2} = \frac{\partial u}{\partial t} + \frac{E_1}{\eta_1} u(z,t) + \frac{E_1}{\eta_2} \frac{\partial u(z,t)}{\partial \tau} e^{\frac{E_2}{\eta_2} (t-\tau)} d\tau + f_q(t)
\]  

(8.10)

in which \( c_v = k_v E_1 / \gamma_w \),

\[
f_q(t) = -\frac{\partial q(t)}{\partial t} - \frac{E_1}{\eta_1} q(t) - q(0) e^{\frac{E_2}{\eta_2} \int_0^t \frac{\partial q}{\partial \tau} e^{\frac{E_2}{\eta_2} (t-\tau)} d\tau}
\]

and \( q(t) \) = applied loading.

### 8.2.3 Surface Loading

![Ramp loading](image)

Figure 8.2. Ramp loading

The time loading, \( q(t) \), simulated by a ramp loading representing the construction sequences from beginning until completion \( (t = t_0) \) when a steady load, \( q_0 \), is imposed on the surface (Figure 8.2). The ramp loading is given by,

\[
q(t) = \begin{cases} 
\frac{q_0}{t_0} t & 0 < t \leq t_0 \\
q_0 & t > t_0
\end{cases}
\]  

(8.11)
8.3 SOLUTION OF EPS GEOFOAM ON SOFT SOILS

8.3.1 Strain in Geofoam Layer

The solution of the Equation (8.8) containing a convolution integral is significantly facilitated by introduction of a Laplace transform defined as,

\[ \tilde{\varepsilon} = \int_0^\infty \varepsilon(t)e^{-st} \, dt \]  

(8.12)

where the superior bar on the left-hand-side indicates the transformed Laplace variable. It thus follows from Equations (8.1), (8.2) and (8.3) that,

\[ \tilde{\varepsilon} = \left( \frac{1}{E_2} + s\tilde{\sigma} \right) \tilde{\sigma} \]  

(8.13)

where \( \sigma(0) = 0 \). The compliance Equation (8.5) yields,

\[ \tilde{J} = \frac{1}{\eta_1} \Gamma(n+1) \frac{1}{s^{n+1}} + \frac{1}{s(\eta_2 + E_2)} \]  

(8.14)

where \( \Gamma \) is the Gamma function. The Laplace transform of the surcharge (Equation 8.11) is,

\[ \tilde{q} = \begin{cases} \frac{q_0}{s^2 t_0} & 0 < t \leq t_0 \\ \frac{q_0}{s^2 t_0} (1 - e^{-t_0}) + \frac{q_0}{s} e^{-n_0} & t > t_0 \end{cases} \]  

(8.15)

8.3.2 Consolidation and Excess Pore Pressure in Soil Layers

The consolidating soil is often layered as a result of natural deposition process and so can be represented approximately as a finite number of distinct homogenous layers 1,2,..,J (Figure 8.3). It is possible each soil layer may have its distinct consolidating as well as creep characteristics governed by Equation (8.10). The boundary conditions for the system are: \( z = 0; u(0,t) = 0; z = H; u(H,t) = 0 \) or \( \partial u(H,t)/\partial z = 0 \).
A Laplace transformation is now introduced to eliminate the time variable in the governing equations and so facilitate the solution of the consolidating problem with time effects. Invoking the Laplace transform of the consolidation Equation (8.10) gives,

\[
c_i \frac{\partial^2 \tilde{u}}{\partial z^2} = \tilde{\phi} \tilde{u} - u(z,0) \frac{u(z,0)E_{ul}}{s\eta_{2i} + E_{2i}} + \tilde{f}_{q,i} \tag{8.16}
\]

where the subscript ‘i’ is introduced for sake of clarity to denote that the parameters belong to a typical i\textsuperscript{th} soil layer and,

\[
\tilde{\phi}_i = s + \frac{sE_{ul}}{s\eta_{2i} + E_{2i}} \frac{E_{ul}}{\eta_{1i}} \tag{8.17a}
\]

\[
\tilde{f}_{q,i} = \begin{cases} 
- \frac{q_0}{t_0} \left[ \frac{1}{s} + \frac{E_{li}}{s^2 \eta_{li}} + \frac{E_{1i}}{s(\eta_{2i} + E_{2i})} \right] & 0 < t \leq t_0 \\
\left(1 - e^{-t_0}\right) \frac{q_0}{t_0} \left[ \frac{1}{s} + \frac{E_{li}}{s^2 \eta_{li}} + \frac{E_{1i}}{s(\eta_{2i} + E_{2i})} \right] & t > t_0 
\end{cases} \tag{8.17b}
\]

In the case of the ramp loading represented by Equation (8.11), \(u(z,0)\) vanishes immediately. It is worth noting here that other complex time loading may also be considered where necessary provided it is possible to implement the Laplace transform.
procedures for the desired loading. It is not difficult to show that the solution to Equation (8.16) is given by,

\[ \bar{u} = Ae^{\alpha z} + Be^{-\alpha z} - \frac{\bar{f}_q}{\bar{\phi}_i} \]  \hspace{1cm} (8.18a)

\[ \alpha_i = \sqrt{\frac{\bar{\phi}_i}{c_{vi}}} \]  \hspace{1cm} (8.18b)

where the coefficients \( A, B \) are to be determined. Using Equation (8.18), it is found that for the layer \( i \) the flow (\( v = -k_i / \gamma_w \partial u / \partial z \)) is given by,

\[ v = -\frac{\alpha_i k_{vi}}{\gamma_w} \left[ Ae^{\alpha z} - Be^{-\alpha z} \right] \]  \hspace{1cm} (8.19)

so that at the interlayer boundary \( z = z_{i-1} \),

\[ \bar{u}_{i-1} + \frac{\bar{f}_q}{\bar{\phi}_i} = Ae^{\alpha \eta_{i-1}} + Be^{-\alpha \eta_{i-1}} \]

\[ \bar{v}_{i-1} = -\frac{\alpha_i k_{vi}}{\gamma_w} \left[ Ae^{\alpha \eta_{i-1}} - Be^{-\alpha \eta_{i-1}} \right] \]

yielding,

\[ \begin{bmatrix} \bar{u}_{i-1} + \frac{\bar{f}_q}{\bar{\phi}_i} \\ \bar{v}_{i-1} \end{bmatrix} = \begin{bmatrix} e^{\alpha \eta_{i-1}} & e^{-\alpha \eta_{i-1}} \\ -\frac{\alpha_i k_{vi}}{\gamma_w} e^{\alpha \eta_{i-1}} & -\frac{\alpha_i k_{vi}}{\gamma_w} e^{-\alpha \eta_{i-1}} \end{bmatrix} \begin{bmatrix} A \\ B \end{bmatrix} \]  \hspace{1cm} (8.20)

Similarly at the interlayer boundary \( z = z_i \), thus yielding,

\[ w_{i-1} = H_i w_i + \Lambda_i \]  \hspace{1cm} (8.21)

in which,

\[ w_{i-1} = \begin{bmatrix} \bar{u}_{i-1} \\ \bar{v}_{i-1} \end{bmatrix}, \quad w_i = \begin{bmatrix} \bar{u}_i \\ \bar{v}_i \end{bmatrix}, \quad H_i = \begin{bmatrix} a_i & b_i \\ c_i & d_i \end{bmatrix}, \quad \Lambda_i = \begin{bmatrix} \Lambda_{i1} \\ \Lambda_{i2} \end{bmatrix} \]

where,

\[ a_i = d_i = \frac{1}{2} (e^{\alpha \Delta z_i} + e^{-\alpha \Delta z_i}), \quad b_i = \frac{1}{2} \left( \frac{\gamma_w}{\alpha_i k_{vi}} \right) (e^{\alpha \Delta z_i} - e^{-\alpha \Delta z_i}), \]

\[ c_i = \frac{1}{2} \left( \frac{\alpha_i k_{vi}}{\gamma_w} \right) (e^{\alpha \Delta z_i} - e^{-\alpha \Delta z_i}), \quad \Lambda_{i1} = \left( \frac{\bar{f}_q}{\bar{\phi}_i} \right) \left( \frac{1}{2} (e^{\alpha \Delta z_i} + e^{-\alpha \Delta z_i}) - 1 \right) \]
\[ \Lambda_{i2} = \frac{1}{2} \left( \frac{\alpha_i k_w}{\gamma_w} \right) \left( \frac{f_{q_i}}{\phi_i} \right) \left( e^{\alpha_i \Delta z_i} - e^{-\alpha_i \Delta z_i} \right), \quad \Delta z_i = z_i - z_{i-1} \]

If Equation (8.21) were to be applied recursively, it leads to the equation,

\[ w_0 = H_1 H_2 \cdots H_J w_J + H_1 H_2 \cdots H_{J-1} A_J + H_1 H_2 \cdots H_{J-2} A_{J-1} + \cdots + H_1 A_2 + A_1 \quad (8.22) \]

Equation (8.22) immediately allows the determination of the unknown boundary pore pressure or flow values of the layered system. For example if \( u_0 = 0 \) and \( u_J = 0 \), it enables \( v_0 \) and \( v_J \) to be obtained. Other boundary conditions may be dealt with in the same manner, if required. Then using Equation (8.21) recursively, the pore pressure and flow values at the interfaces of the layers can be determined accordingly (Leo and Xie 2001).

To obtain the strains of each soil layer, use is made of Equation (8.6) where its Laplace transform version is:

\[ \bar{e}_z = \bar{\sigma}_z \left[ \frac{1}{\tilde{E}_1} + \frac{1}{s \eta_1} + \frac{1}{(s \eta_2 + E_2)} \right] \quad (8.23) \]

in which \( \bar{\sigma}_z = \bar{q} - \bar{u} \). Commencing from the base \( (z = z_J) \), the strains are integrated over each layer and summed cumulatively to give the settlements at the various depths from the surface. Finally, the Laplace transform solutions are inverted numerically to produce the real-time solutions by way of the inversion algorithm by Talbot (1979).

Finally, it should be mentioned that the transform solution of the strain of the EPS fill is given directly by equation 8.13, which enable the fill deformation to be worked out. The settlement at the surface of the EPS fill is obtained simply by adding this deformation to the settlement obtained at the surface of the soft soil.
8.4 ILLUSTRATIVE EXAMPLE

8.4.1 General

In embankment construction, the settlement of the sub-soil at each construction stage is also enhanced by the creep component within the subsoil layer. Indraratna et al. (1993), for example, found in a study that the creep component accounts for up to 15% of the total settlement for embankments founded on soft marine clay. However, creep (time effects) is often not modeled in the Finite Element Method analysis of the geotechnical problem. Instead an empirical approach is normally adopted such as field deformation analysis (FDA) in which the creep component is related to the consolidation settlement based on the field measurements (Loganathan et al. 1993).

The EPS - soil model is developed as a semi-analytical method to predict the long-term settlement of EPS fill resting on very soft consolidating soil. One of the advantages of this model is that by properly selecting the model parameters, it is able to model both the creep (time effects of applied stress) and consolidation (time effect of pore water pressure) separately and/or in a coupled fashion.

The same model can also be used to gauge these results against the ‘no EPS’ case where conventional fill material are used and considerably higher self-weight would be imposed on the consolidating soil.

8.4.2 Embankment With and Without EPS Fill

The illustrative case considered in this example case has a 5 m high EPS (20 kg/m$^3$ density) embankment situated above 5 m of very soft clay. This in turn overlies fairly firm stratum. The values of the model parameters for the EPS fill are provided in Table 8.1 (from creep test data discussed in Chapter 5) while those for the very soft soil are presented in Table 8.2 below.

Table 8.2 Parameter values of 4-element model for soft subsoil

<table>
<thead>
<tr>
<th>$E_1$ (kPa)</th>
<th>$\eta_1$ (kPa-yr)</th>
<th>$E_2$ (kPa)</th>
<th>$\eta_2$ (kPa-yr)</th>
<th>$c_v$ (m$^3$/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td>$0.5 \times 10^6$</td>
<td>1000</td>
<td>25</td>
<td>5</td>
</tr>
</tbody>
</table>
A pavement was constructed directly above the EPS fill during a 6 months period, which imposes an ultimate surcharge of 20 kPa. The present model is applied to evaluate the expected surface settlement over the next 100 years. In the case where a conventional fill material has been used in the construction of the embankment, it is assumed that this would cause an ultimate surcharge of 100 kPa (instead of only 20 kPa) based on a unit weight of 16 kN/m$^3$ for the fill material. The expected settlement results for both these cases are plotted in Figure 8.4. The results indicate that the EPS fill performed quite effectively for the duration of the 100 years in that settlement is restricted to a value of about 0.24 m. In the conventional case, a total settlement of about 1.1 m would have resulted assuming no other soil improvement was done.

8.4.3 Effects of Consolidation and Creep

The creep effects can be switched off for both EPS and soft soil in the semi-analytical model. This can been simply done by setting the values of $1/\eta_1$, $1/E_2$ and $E_2/\eta_2$ in Equations (8.5) and (8.7) approach zero, such as by taking the model parameters of $\eta_1 = \eta_2 = 10^{20}$ and $E_2 = 10^{10}$. Then the constitutive equation of the soil or EPS simplifies to:
\[ \varepsilon(t) = \frac{\sigma(t)}{E_i} \]  \hspace{1cm} (8.25)

Equation (8.25), in combination with Equation (8.9), can be used to solve the consolidation problem of EPS embankment on soft sub-soil without the time effect of creep.

Figure 8.5 shows the total settlement at the bottom of EPS fill (original ground level) for the cases with creep and without creep effect. This is also the total deformation of the sub-soil. The creep deformation of the soft soil can be obtained by deducting the consolidation of the soil without creep from the total settlement with creep effects. The consolidation, the creep and the total deformation of sub-soil are given in Figure 8.6. It shows that in this assumed case \((E = 1000\text{kPa}, \nu=0.3)\), the creep of this very soft sub-soil increases with increasing time and the deformation of creep approaches the same level as the deformation of consolidation after about 20 years. This indicates that for the long-term behaviour of an embankment on very soft sub-soil, the creep of the sub-soil is as important as the consolidation deformation. However, it must be keep in mind that the magnitude of the creep deformation depends largely on the model parameters which should be obtained by experimental testing and/or from field measurement.
Figure 8.6. Deformation of the sub-soil

Figure 8.7. Deformation of EPS layer
The immediate deformation (without creep effect), the creep deformation and the total deformation (immediate + creep) of EPS fill are also given in Figure 8.7. The results indicated that the creep deformation of EPS increases continuously with increasing time. For a 100 years period of time, it accounts for approximately 50% of the total deformation of the EPS layer.

8.5 SUMMARY

1. A semi-analytical one-dimensional EPS – soft soil model based on visco-elastic principles has been developed to analyse the typical EPS embankment resting on soft soil. This model can be easily used to evaluate the long-term effectiveness of the EPS fill solution for overcoming settlement problems in soft soil.

2. The semi-analytical model is able to predict the consolidation and creep deformation of the soft sub-soil. However, the creep and consolidation deformations are largely dependent on the model parameters which should be obtained by laboratory and/or field measurements.

3. This EPS–soil model can also be extended to simulate other aspects of construction and geotechnical design considerations (such as installation of vertical drains) currently not provided for.
CHAPTER 9

CONSTITUTIVE MODELING AND FINITE ELEMENT ANALYSIS OF EPS GEOFOAM IN GEOTECHNICAL APPLICATIONS

9.1 INTRODUCTION

In general, the design of a geotechnical construction in soil and rock involves the analysis of the stress and strains responding to the loads acting on the components of the construction and the surrounding ground. To do this, it is essential to know the constitutive relationships of the construction and ground material. Hovarth (1996) quite appropriately pointed out that to make best use of the EPS material in geotechnical construction, there needs to be a consistent program for developing useful constitutive relationships of EPS geofoam. One of the main objectives of this thesis, therefore, has been to develop a simple and practical constitutive model for EPS geofoam for use in geotechnical engineering applications.

EPS geofoam, made from expanded polystyrene beads fused together under high temperature and pressure, can be regarded as a particulate material with strong interparticle bonding or cohesion. Tests have shown that EPS material is similar to soil in some ways but also dissimilar in others. Like soil, the strains induced on primary loading of EPS are not fully recoverable upon unloading. Its yield stress is dependent on the stress history (i.e. there is some work hardening). However, EPS response under stress is different from granular soil in that its stress-strain behaviour has negligible or no dependency on the confining pressure.

In this chapter, the development of a variable moduli constitutive model for EPS geofoam is discussed. Verification results are then presented showing that the model has generally been able to capture the essential and special characteristics of the EPS geofoam behaviour. This constitutive model has been incorporated into a finite element package ‘FEAPACK’ which was developed in-house and can be used to make predictions of the deformation of EPS embankments. Two case studies have been investigated.
9.2 A VARIABLE MODULI CONSTITUTIVE MODEL FOR EPS IN GEOTECHNICAL APPLICATION

From a practical engineering and a computational point of view, providing that the essential characteristics of EPS response are properly captured, a simple model is to be preferred to complex stress-strain relationships. In this thesis, a simple nonlinear (variable moduli) model of EPS geofoam has been developed. This model falls into the same category as the well-known hyperbolic (hypo-elastic) model of Duncan and Chang (1970) which provides a simple procedure for representing nonlinear, stress-dependent, inelastic stress-strain behaviour for soils. For the sake of completeness, the development of various constitutive models in soil mechanics is briefly reviewed in Appendix B. Despite certain shortcomings such as not being able to compute collapse load in the fully plastic range, the Duncan-Chang model is still very much favored by the geotechnical community. It may be argued that in many cases of EPS applications, the deformational rather than strength behaviour is the main subject of interest to geotechnical engineers. The proposed model fulfills the first requirement quite adequately. The model parameters are easily obtained from standard triaxial tests, and the procedures are discussed in this section. The use of the model for prediction of deformation in triaxial tests, as part of verification of the constitutive model, was also performed and the result is discussed.

9.2.1 Formulation of Model for EPS Geofoam

9.2.1.1. General

The proposed model for EPS geofoam may be described as a variable moduli model. Nelson and Baron (1971) described the variable moduli model as a special case of isotropic hypoelastic model in which the tensor relating stress and strain increments depends on the invariants, but not on the stress (or strain) tensor itself. The response of such a model is irreversible even in incremental loading and does not contain an explicit yield condition used in elasto-plastic approaches. Hence yield stress and the presence of strain hardening are not formally incorporated within the model.
9.2.1.2. Generalized Hooke's Law

The stress-strain relationship of the proposed model can be expressed incrementally by the generalized Hooke's law as:

\[ \Delta \sigma = D_t \Delta \epsilon \]  

(9.1)

where,

\[ \epsilon = \begin{bmatrix} \epsilon_{xx}, \epsilon_{yy}, \epsilon_{zz}, \gamma_{xy}, \gamma_{yz}, \gamma_{zx} \end{bmatrix}^T \] is the vector of the strain

\[ \sigma = \begin{bmatrix} \sigma_{xx}, \sigma_{yy}, \sigma_{zz}, \sigma_{xy}, \sigma_{yz}, \sigma_{zx} \end{bmatrix}^T \] is the vector of the stress

\[ D_t \] is the tangential material stiffness matrix

It is assumed that the EPS material is isotropic, therefore the tangential elasticity matrix is given by:

\[
D_t = \begin{bmatrix}
\frac{K_t}{3} + \frac{4}{3}G_t & \frac{K_t}{3} - \frac{2}{3}G_t & \frac{K_t}{3} - \frac{2}{3}G_t & 0 & 0 & 0 \\
\frac{K_t}{3} - \frac{2}{3}G_t & \frac{K_t}{3} + \frac{4}{3}G_t & \frac{K_t}{3} - \frac{2}{3}G_t & 0 & 0 & 0 \\
\frac{K_t}{3} - \frac{2}{3}G_t & \frac{K_t}{3} - \frac{2}{3}G_t & \frac{K_t}{3} + \frac{4}{3}G_t & 0 & 0 & 0 \\
0 & 0 & 0 & G_t & 0 & 0 \\
0 & 0 & 0 & 0 & G_t & 0 \\
0 & 0 & 0 & 0 & 0 & G_t
\end{bmatrix}
\]  

(9.2)

where \( K_t \) and \( G_t \) are the tangential bulk modulus and the tangential shear modulus, respectively. Relationships for the tangential moduli are given below.

9.2.1.3. Tangential Moduli in Initial Loading

The tangential moduli are functions of the stresses. In initial loading, the moduli are given by the following nonlinear relationships:

\[
K_t = \begin{cases} 
\frac{1}{2}(K_n + K_p) + \frac{1}{2}(K_n - K_p) \left( 1 - \exp \left( -\frac{p}{p_s} \right) \right) & \text{when } \frac{p}{p_y} \leq 1 \\
\frac{1}{2}(K_n + K_p) - \frac{1}{2}(K_n - K_p) \left( 1 - \exp \left( \frac{p}{p_s} \right) \right) & \text{when } \frac{p}{p_y} > 1 
\end{cases}
\]  

(9.3)
\[ G_t = \begin{cases} \frac{1}{2}(G_{tt} + G_{tp}) + \frac{1}{2}(G_{tt} - G_{tp}) \left( 1 - \exp \left( -\frac{\sqrt{J_2}}{\sqrt{J_{2y}}} \right) \right) & \text{when } \frac{\sqrt{J_2}}{\sqrt{J_{2y}}} \leq 1 \\ \frac{1}{2}(G_{tt} + G_{tp}) - \frac{1}{2}(G_{tt} - G_{tp}) \left( 1 - \exp \left( -\frac{\sqrt{J_2}}{\sqrt{J_{2y}}} \right) \right) & \text{when } \frac{\sqrt{J_2}}{\sqrt{J_{2y}}} > 1 \end{cases} \] (9.4)

where \( K_{tt}, G_{tt} \) are the initial tangential moduli and \( K_{tp}, G_{tp} \) are the tangential moduli beyond the reference stresses \( p_y \) and \( \sqrt{J_{2y}} \) respectively. The \( p \) and \( J_2 \) are mean stress and second stress invariant respectively. \( K_{tt}, G_{tt}, K_{tp}, G_{tp}, p_y, \sqrt{J_{2y}}, \alpha \) and \( \beta \) are parameters which can be obtained by conventional triaxial test.

Clearly for large values of \( \alpha \), equation (9.3) and (9.4) reduces to bilinear relationships given by:

\[ K_t = \begin{cases} K_{tt} & \text{when } p < p_y \\ K_{tp} & \text{when } p \geq p_y \end{cases} \] (9.5)

\[ G_t = \begin{cases} G_{tt} & \text{when } \frac{\sqrt{J_2}}{\sqrt{J_{2y}}} \leq 1 \\ G_{tp} & \text{when } \frac{\sqrt{J_2}}{\sqrt{J_{2y}}} > 1 \end{cases} \] (9.6)

It suffice to say that the simpler relationships given in (9.5) and (9.6) would probably be sufficiently accurate in most applications. However, in this age of computers, there is no distinct disadvantage in using the more relevant relationships given in (9.3) and (9.4).

9.2.1.4. Tangential Moduli in Unloading and Reloading

The tangential moduli in unloading is given by:

\[ K_t = K_{uu} \] (9.7)

\[ G_t = G_{uu} \] (9.8)

In reloading the expressions for the tangential moduli are similar to the equations (9.3) and (9.4), and given as:
\[
K_t = \begin{cases} \frac{1}{2} (K_u + K_p) + \frac{1}{2} (K_u - K_p) \left( 1 - \exp^{-\left( \frac{\gamma}{p_{\text{max}}} \right)} \right) & \text{when } \frac{\gamma}{p_{\text{max}}} \leq 1 \\ \frac{1}{2} (K_u + K_p) - \frac{1}{2} (K_u - K_p) \left( 1 - \exp^{-\left( \frac{\gamma}{p_{\text{max}}} \right)} \right) & \text{when } \frac{\gamma}{p_{\text{max}}} > 1 \end{cases}
\]

\[
G_t = \begin{cases} \frac{1}{2} (G_u + G_p) + \frac{1}{2} (G_u - G_p) \left( 1 - \exp^{-\left( \frac{\sqrt{J_2}}{J_{2,\text{max}}} \right)} \right) & \text{when } \frac{\sqrt{J_2}}{J_{2,\text{max}}} \leq 1 \\ \frac{1}{2} (G_u + G_p) - \frac{1}{2} (G_u - G_p) \left( 1 - \exp^{-\left( \frac{\sqrt{J_2}}{J_{2,\text{max}}} \right)} \right) & \text{when } \frac{\sqrt{J_2}}{J_{2,\text{max}}} > 1 \end{cases}
\]

where \( K_u \) and \( G_u \) are the reloading initial tangential moduli. \( p_{\text{max}} \) is the larger of \( p \), and the historical maximum value of the mean stress while \( \frac{\sqrt{J_2}}{J_{2,\text{max}}} \) is the larger of \( \sqrt{J_{2,y}} \) and the historical maximum value of the second stress invariant. The unloading and reloading moduli may also be obtained from laboratory tests. For no energy to be generated during infinitesimal stress cycles, the necessary conditions in this model are that (Chen and Saleeb 1994):

\[
K_u \geq K_0 \geq 0 \\
G_u \geq G_0 \geq 0
\]

9.2.1.5. Loading, Unloading and Reloading Criteria

The tangential moduli corresponding to loading, unloading and reloading may be written as:

- **Bulk modulus \( K \)**
  - Loading: when \( p = p_{\text{max}} \) and \( \dot{p} > 0 \)
  - Unloading: when \( p \leq p_{\text{max}} \) and \( \dot{p} < 0 \)
  - Reloading: when \( p < p_{\text{max}} \) and \( \dot{p} > 0 \)

- **Shear modulus \( G \)**
  - Loading: when \( J_2 = J_{2,\text{max}} \) and \( \dot{J}_2 > 0 \)
  - Unloading: when \( J_2 \leq J_{2,\text{max}} \) and \( \dot{J}_2 < 0 \)
  - Reloading: when \( J_2 < J_{2,\text{max}} \) and \( \dot{J}_2 > 0 \)

It is clear from above that the material could be loading in shear while unloading in mean stress simultaneously and vice versa.
9.2.2 Parameters of EPS Model

The proposed model in the previous section was developed on the basis of experimental data obtained from triaxial testing using cylindrical EPS specimen with density of 20 kg/m³.

In each of the experiments, as described in Chapter 3, the volume strain, $\varepsilon_v$, of the EPS specimen could be reasonably approximated to the axial strain, $\varepsilon_a$, as the radial strain, $\varepsilon_r$, is negligible. A plot of the shear stress $\frac{1}{2}(\sigma_1 - \sigma_3)$ against the axial strain, $\varepsilon_a$, enables the shear modulus to be approximately determined (Figure 9.1). Also, the bulk modulus was obtained by plotting the mean stress $p$ against the axial strain, $\varepsilon_a$ (Figure 9.2).
The tangential bulk and shear moduli are often expressed as functions of the stress invariants such as given by $K(I_1), G(I_1, \sqrt{J_2})$ where $I_1$ and $\sqrt{J_2}$ are the first and second stress invariants respectively. While many possible and theoretically sound relationships could be established in the case of EPS, it was found that both $K$ and $G$ could be reasonably approximated as functions of $I_1$ (or mean stress $p$) and $\sqrt{J_2}$ respectively on the basis of experimental data obtained. The typical experimental data of $K - I_1$ and $G - \sqrt{J_2}$ are shown in Figure 9.3 and 9.4 respectively. This has a major advantage of enabling a simple and neat characterization of the material. The model parameters $K_{ii}, K_{ip}, G_{ii}, G_{ip}$, are estimated by fitting the appropriate linear relationships in the respective plots as shown in Figures 9.1 and 9.2 respectively. The remaining parameters $P_y, a$ and $\sqrt{J_2}, b$ are determined by linear regression of the values of $K_i, G_i$ against the mean stress, $p$, and the second stress invariant, $\sqrt{J_2}$, using the following logarithmic relationships of Equations (9.3) and (9.4), respectively.
Figure 9.3 Bulk Modulus in triaxial test (ρ = 20kg/m³)

Figure 9.4 Shear Modulus in triaxial test (ρ = 20kg/m³)
Figure 9.5 An example of determining model parameters

\[
\log \left(1 - \frac{1}{2} \left( K_i - \frac{1}{2} \left( K_{ii} + K_{ip} \right) \right) \right) = \begin{cases} 
\log \left( \frac{p}{p_y} - 1 \right), & \text{when } \frac{p}{p_y} \leq 1 \\
\log \left( 1 - \frac{p}{p_y} \right), & \text{when } \frac{p}{p_y} > 1 
\end{cases} \quad (9.11)
\]

\[
\log \left(1 - \frac{1}{2} \left( G_i - \frac{1}{2} \left( G_{ii} + G_{ip} \right) \right) \right) = \begin{cases} 
\log \left( \sqrt{\frac{J_2}{J_{2y}}} - 1 \right), & \text{when } \sqrt{\frac{J_2}{J_{2y}}} \leq 1 \\
\log \left( 1 - \sqrt{\frac{J_2}{J_{2y}}} \right), & \text{when } \sqrt{\frac{J_2}{J_{2y}}} > 1 
\end{cases} \quad (9.12)
\]

A sample plot of such a relation between \( K_i \) and \( p \) is given in Figure 9.5 which shows how to obtain the values \( p_y \) and \( a \). The \( p_y \) is the \( x \) value at the maximum \( y \) value (or at \( y = 0 \)). Then, the value of \( a/p_y \) equals to the absolute value of the slope of the linear part of experimental data, or \( a \) equals to the absolute value of the intercept. In an ideal case, the absolute value of slope or intercept on both side of \( p_y \) should be the same. Therefore, the average value \( a \) of the experimental data was taken. A similar procedure was used to determine the \( \sqrt{J_{2y}} \) and \( b \) values. The values for the model parameters yielded by regression of the triaxial tests are summarized in Table 9.1.
Table 9.1 Model Parameters from Triaxial Tests

<table>
<thead>
<tr>
<th>$K_{ii}$, MPa</th>
<th>$K_{ip}$, kPa</th>
<th>$p_y$, kPa</th>
<th>$a$</th>
<th>$G_{ii}$, MPa</th>
<th>$G_{ip}$, kPa</th>
<th>$\sqrt{f_{2y}}$, kPa</th>
<th>$b$</th>
<th>$K_{is}$, MPa</th>
<th>$G_{is}$, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.54</td>
<td>36</td>
<td>38</td>
<td>11</td>
<td>2.31</td>
<td>60</td>
<td>45</td>
<td>10</td>
<td>2.80</td>
<td>4.00</td>
</tr>
</tbody>
</table>

9.2.3 Model Verification

It may be noted that a number of laboratory studies (Horvath 1994, Eriksson and Trank 1991, and this thesis) undertaken to study the behaviour of EPS concluded that EPS exhibits characteristics which include the following.

1. Response in the initial loading is nearly bilinear under uniaxial loading.
2. Deformation is irreversible even at small stress levels, therefore, the “elastic regime” does not strictly exist.
3. “Strain-hardening” occurs when the material is loaded beyond the “yield stress”.
4. Very small or virtually zero values of Poisson ratio until yielding.
5. For each cycle of loading-unloading, a hysteresis loop is observed.
6. Unlike a particulate material in a dense state of packing, this material does not experience significant volume dilation under shear loading.

A good model should attempt to capture the essential characteristics of the material behaviour in applications. Independent tests have been carried out to verify the above characteristics in the response of the proposed model for EPS.
Figure 9.6 shows the experimental and predicted results of a constrained loading-unloading-reloading uniaxial compression test on an EPS specimen. The predicted results are obtained through numerically integrating Equation (9.1) while maintaining the lateral strains at zero. In the initial loading stage, the almost bilinear type response in which a sharp curve joins the two linear sections could be reproduced fairly accurately by the model. The model is represented by a linear response during unloading which deviates slightly from the actual experimental results showing a distinct curve at the tail. Upon reloading, the actual experimental results show a slight curve, which joins to the virgin loading curve once the original maximum stress level is exceeded. The model response is linear in the first part with a very small curve joining to the virgin loading curve, exhibiting that a “strain hardening” effect has occurred. It may be concluded that overall the model is able to predict the response quite satisfactorily.
Figure 9.7 Stress-strain curve in triaxial test ($\rho = 20$ kg/m$^3$)

Figure 9.7 shows the deviator stress-strain results for a triaxial test with a confining pressure of 10 kPa and the predictions obtained by numerical integration from the model. In the loading sequence, the shape of the curve (the distinct linear sections joined by a curve in the middle) is clearly reproduced by the model. The first unloading is in the linear part of the curve but it produces a residual strain, which is also correctly predicted by the model. The reloading joins the linear part of the virgin curve and the loading continues until the second unloading takes place at a higher stress level beyond the yield value. The second unloading and reloading then occur until loading returns on the flat part of the virgin loading curve. The prediction could reproduce all these response quite satisfactorily (including the effects of strain hardening) except for the response in the second unloading/reloading cycle. As hysteresis in the stress-strain cycles increases with stress levels, the limitation of the model, in its current form, to reproduce these effects is apparent.
Figure 9.8 Volumetric strain versus axial strain in triaxial test ($\rho = 20 \text{ kg/m}^3$)

Figure 9.9 Volumetric strain versus deviator stress in triaxial test ($\rho = 20 \text{ kg/m}^3$)

Figure 9.8 and 9.9 show the volumetric strain against the axial strain and the volumetric strain against the deviator stress respectively for various confining stresses (0-20 kPa). Shear loading is imposed as the deviator stress/axial strain is increased. Clearly as the material is loaded to “failure”, the volumetric strain is decreased (compression). This phenomenon is also quite correctly predicted by the model.
The plot of the deviator stress against radial strain from the triaxial test of various confining stresses (0-20 kPa) is shown in Figure 9.10. One of the characteristics of the EPS material is the very low or sometimes almost zero value of Poisson's ratio which is manifested in the very low lateral strain produced in axial loading. The predicted results also quite clearly demonstrated this unique feature of the behaviour of EPS material.

Generally the model has been able to reproduce the important characteristics of the material as well as the stress-strain response satisfactorily.
9.3 FINITE ELEMENT ANALYSIS OF EPS FILL IN GEOTECHNICAL APPLICATIONS (CASE STUDIES)

The Finite Element Method (FEM) is a computer-based solution technique that discretizes the whole region to be analyzed into many small subregions or finite elements which are interconnected only at a discrete number of nodal points situated on their boundaries. It plays an important role evaluating the piecewise approximation of the unknown variables in the nonlinear analysis of engineering and, particularly in structural and geotechnical engineering, when their closed-form solutions are difficult to be found.

The FEM has been widely used in geotechnical engineering analysis and design. It is especially versatile when dealing with the nonlinear behaviour of soils, especially in the cases where field trials are limited by time and costs. Computer simulations can give an alternative low-cost method to assess various design options.

In the case studies in this chapter, the EPS embankment was considered as a design option. The constitutive model for EPS, developed in this thesis, has been incorporated into the in-house finite element program ‘FEAPACK’ and used in the embankment design analysis. The finite element analysis employed 8-noded Linear Strain Quadrilateral (LSQ) elements. The cases were studied as a two-dimensional problem. The details of analysis are given in the individual case study.

Comparisons have been made of the deformation behaviour, stress and strain conditions of the EPS embankment with those of normal soil fill with conventional ground improvement techniques.

9.3.1 CASE STUDY ONE

9.3.1.1. Description

A finite element analysis of an EPS road embankment built on slightly overconsolidated clay has been undertaken (similar to an example in Hamada and Yamanouchi 1987). The dimensions of the embankment and the loading consisting of concentrated wheel loads (4 x 19.6 kN/m) and its self weight are shown in Figure 9.11. In this study, the pavement and base layers were regarded as a single layer (1m) and modeled as an elastic material. A reinforced soil was used as the slope cover material, while the EPS fill was modeled by the
Figure 9.11 EPS embankment on overconsolidated clay

*K-G* model developed in this thesis. For the overconsolidated clay foundation, the Duncan-Chang (Hyperbolic) model was used. The embankment with normal soil fill has been analyzed as a baseline case and the effectiveness of the EPS fill in road embankment has been evaluated.

9.3.1.2. Model Embankment and Parameters

The model parameters of the EPS fill are as given in Table 9.1. The properties used for the non-EPS component parts of the embankment are taken from Hamada and Yamanouchi (1987) and tabulated in Table 9.2. The finite element mesh (and the deformed outline under the applied loading and self weight) is shown Figure 9.12.

Table 9.2 Material properties of non-EPS material

<table>
<thead>
<tr>
<th>Material</th>
<th>Cohesion (kPa)</th>
<th>Friction angle ((\phi^c))</th>
<th>(R_f)</th>
<th>(n)</th>
<th>Initial (E^') (MPa)</th>
<th>Poisson ratio ((\nu))</th>
</tr>
</thead>
<tbody>
<tr>
<td>OC clay</td>
<td>49</td>
<td>0</td>
<td>0.9</td>
<td>0</td>
<td>9.8</td>
<td>0.3</td>
</tr>
<tr>
<td>Reinforced soil</td>
<td>98</td>
<td>35°</td>
<td>0.9</td>
<td>0.55</td>
<td>49</td>
<td>0.25</td>
</tr>
<tr>
<td>Pavement</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>200</td>
<td>0.3</td>
</tr>
</tbody>
</table>
9.3.1.3. Modeling Results and Discussion

For road embankment design and analysis, it is important to study the stress state and the deformation within the embankment. The analysis results in Figure 9.13 show that large parts of the embankment itself are under some tension. For the most part, tensile (principal) stresses in the EPS fill are typically low (less than 10 kPa), except in the region next to the EPS-pavement interface. Stresses are within the tensile strength of the EPS material (where tensile stress is considered as positive). The reinforced covering soil would have assisted to resist some tensile stresses.
The highest compressive (principal) stress in the EPS exist in a small region at the centre of the fill just below the pavement, but even these stresses (about 30 kPa) are also well within the elastic limit of the EPS 20kg/m$^3$ (Figure 9.14). The clay remains overconsolidated after loading.

The Figure 9.15 shows the total vertical stress distribution within the foundation clay before and after embankment construction. By comparing the total stress distribution, it clearly shows that had the EPS fill not been used, a conventionally built embankment would have stressed the clay beyond the preconsolidation stress into the range of normal consolidation (the preconsolidation stress, $\sigma'_{pc}$, is 50 kPa at the surface of the foundation and increases linearly with depth as shown in Fig 9.15). It also shows that the EPS embankment significantly reduces the total vertical stress in foundation clay and almost does not change the in-situ stress state. The surcharge load due to the pavement construction and vehicular load ($4 \times 19.6$ kN) are approximately 20 kPa and 10 kPa, respectively. The soft soil remains overconsolidated relative to the loading from the EPS embankment.
Figure 9.15 Total vertical stress in foundation soil after embankment construction (centerline of embankment)
Figure 9.16 Settlement profile along the ground surface (a) EPS fill, (b) Soil fill
Figure 9.17 Lateral displacement profile at toe of embankment (a) EPS fill, (b) Soil fill
The maximum settlement which occurs at the center and top of the embankment is about 37 mm and the maximum differential settlement is about 13 mm. This is within acceptable tolerance which, in circumstances using normal fill, would not normally have been possible to achieve.

The settlement profile along the base of embankment and the horizontal displacement at the toe of embankment are shown in Figure 9.16 and 9.17 respectively. For the EPS embankment, the maximum settlement is 33 mm and the maximum lateral displacement is 4 mm, while the maximum settlement and lateral displacement are 880 mm and 750 mm, respectively for conventional soil fill embankment (assuming no other soil improvement was done). By comparing with conventional soil fill embankment, the EPS fill reduces the settlement and lateral displacements significantly in the foundation clay.

As discussed in Chapter 8, in embankment construction, the settlement of the foundation is also influenced by the creep of the sub-soil due to surcharge loading of the elevated fill. In order to account for the creep deformation of the EPS fill and foundation soil, the semi-analytical solution described in Chapter 8 is used to determine the approximate creep deformations which are then added to the finite element solution to give the total settlement with time effects. It is however important to emphasize that the stress \( \sigma_0 \) in equation (8.4) and \( q_0 \) in (8.17b) become layered variables \( \sigma(i) \) and \( q_0(i) \), respectively. An example is given hereafter to explain the procedure of the analysis.

Figure 9.18 shows the profile of the surcharge stress (equal to the total vertical stress minus the in situ stress as shown in Figure 9.15) distribution in EPS embankment and sub-soil from FEM analysis due to the construction of embankment (at the centerline of the embankment). It is noted that the surcharge stress distribution varies with depth. It is now necessary to divide the EPS and sub-soil into smaller sub-layers. Figure 9.19 also shows the piecewise constant approximation of the vertical surcharge stress distribution in each sub layer, and the average surcharge stress is taken as one of the layer variables of each layer to calculate the creep deformation. Then the creep deformation of each sub-layer is added onto the consolidation deformation and the total settlement with time effects are obtained. The thickness of the sub-layer is 1 m, and the creep model parameters of EPS and sub-soil are given in Table 9.3. The consolidation, the semi-analytical creep deformation and the total deformations of EPS and sub-soil are shown in Figure 9.20 and
9.21 respectively.

Figure 9.18 Surcharge load distribution (at the centerline of the embankment) from FEM analysis

Figure 9.19 Sub-layers and surcharge load in EPS fill and sub-soil
Figure 9.20 long-term deformation of the EPS layer due to the dead loads

Figure 9.21 long-term deformation of the sub-soil due to the dead loads

Table 9.3. Parameter values of creep models for EPS and sub-soil

<table>
<thead>
<tr>
<th>Material</th>
<th>(N)</th>
<th>(E_1) (kPa)</th>
<th>(\eta_1) (kPa-yr)</th>
<th>(E_2) (kPa)</th>
<th>(\eta_2) (kPa-yr)</th>
<th>(c_v) (m/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPS</td>
<td>0.165</td>
<td>4620</td>
<td>1.4\times10^4</td>
<td>77520</td>
<td>0.131</td>
<td>–</td>
</tr>
<tr>
<td>Sub-soil</td>
<td>–</td>
<td>9800</td>
<td>5.0\times10^7</td>
<td>50000</td>
<td>25</td>
<td>5</td>
</tr>
</tbody>
</table>
Table 9.4. The deformation of the EPS fill and sub-soil for 100 years

<table>
<thead>
<tr>
<th></th>
<th>1-D semi-analytical solution</th>
<th>Hybrid (FEM + semi-analytical) solution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Consolidation, mm</td>
<td>Creep, mm</td>
</tr>
<tr>
<td>EPS</td>
<td>16.8</td>
<td>17.5</td>
</tr>
<tr>
<td>% of total</td>
<td>49.0</td>
<td>51.0</td>
</tr>
<tr>
<td>Sub-soil</td>
<td>20.4</td>
<td>4.4</td>
</tr>
<tr>
<td>% of total</td>
<td>82.2</td>
<td>17.8</td>
</tr>
</tbody>
</table>

The results of consolidation, creep and total settlement of the one-dimensional semi-analytical solution and the hybrid method (FEM plus semi-analytical solution) are given in Table 9.4. The magnitudes of the creep of EPS and consolidation of subsoil calculated by the one-dimensional semi-analytical solution are larger than the hybrid method (FEM with semi-analytical solution). The reason is due to the fact that a uniform surcharge load (one-dimensional idealization) was used in the semi-analytical solution. The hybrid method as well as the semi-analytical method appear to provide reasonable solutions for the long-term problem of a typical EPS embankment resting on very soft subsoil. The results indicate that the creep of the EPS and sub-soil significantly influence the settlement of the embankment. The creep components of EPS and sub-soil account for about 42% and 24% of the total deformation, respectively. It also shows that the creep of the sub-soil is largely influenced by the model parameters, the value of which, therefore, should be carefully determined by laboratory or field measurements.

This case study clearly shows that the EPS fill technique would minimize the ground disturbance and significantly reduce the settlement and lateral displacement within foundation soil. The results of the analysis also show that the stress state in the EPS embankment is well within the elastic limit. The deformation of the EPS embankment was also very low (less than 1%) which indicates that the EPS embankment itself is very stable.
9.3.2 CASE STUDY TWO

9.3.2.1. Background

In order to evaluate the effectiveness of various ground improvement techniques, Indraratna et al. (1997) analyzed the settlement and lateral deformation of soft clay foundation beneath a series of full-scale trial embankments constructed on the Mura Plain, Malaysia in 1986. The soft marine clays in this area of test embankment are extremely soft and have a thickness of up to 20 m which give rise to serious construction problems. One of the trial embankments is composed of a compacted granitic residual soil fill. The foundation beneath this embankment was improved with geogrids and vertical drains. The maximum height of the embankment was 9.7 m, which was much greater than that of the critical height of a failed embankment (5.5 m) built on an untreated foundation in an adjoining area. The settlements of the foundation were monitored by settlement gauges and extensometers, while the lateral movements were monitored by inclinometers.

This embankment is selected here as a case study to evaluate the EPS replacement technique. Detailed field and laboratory tests have been conducted to estimate the soil properties with depth that are used in the finite element analysis. These tests include in-situ vane and cone penetration tests, CIU, Ck0, CD and constant ratio tests. Indraratna et al. (1997) established the soil parameters corresponding to pre-yield response, Mohr-Coulomb criterion and modified Cam-clay theory. The effectiveness of the EPS replacement technique was studied using finite element analysis and compared with those of the compacted granitic residual soil fill embankment built on ground improved by vertical drains and geogrids which is taken as the baseline case in this study.

9.3.2.2. Foundation Soil and Embankment Construction

The details of the embankment and foundation soil were presented by Indraratna et al. (1997). A brief description is given as follows.

The soft marine clays in this area of test embankment are extremely soft and have a thickness of up to 20 m. The uppermost layer of soil (1.5 to 2.0 m) is overconsolidated because of high suctions developed when dry in comparison with the underlying soft clay (normally consolidated or lightly over-consolidated). There is a thick dense sand layer deposit below 20 m.
The embankment fill (the baseline case constructed as a trial) consisted of a granitic residual soil compacted to an average unit weight of 20.5kN/m³. It was built above the preloaded foundation with geogrids and vertical drains and raised to a maximum height of 9.7 m above the original ground surface. The embankment and stabilizing methods are shown in Figure 9.22. A total number of 45 DESOL (a proprietary brand) prefabricated vertical drains were installed and their equivalent diameter was 70mm. Drainage at the upper boundary was facilitated by a layer of sand (0.5 m) and 50 mm diameter horizontal drains spaced at 2m above the ground level. A thin layer of fill was later removed from the top of the embankment to maintain the total height of the embankment as 9.2 m. Two layers of Tensar SR110 geogrids were placed as interface reinforcement between the embankment and the foundation.

For the EPS embankment, the case study assumes the same overall geometry of the embankment, the construction sequence, the ground profile and soil parameters of the foundation soft clay as those of Indraratna et al. (1997) which are given in Table 9.5 and 9.6. This study modeled the soft marine clay in the foundation as a modified Cam-clay material. The embankment construction had taken more than one year (about 380 days) to complete and the construction history is shown in Figure 9.23 where a three stage linear sequence was adopted for the numerical analysis. Two EPS design cases (partial and full replacement of the granitic soil) were considered for analysis. In addition, a fourth case which is the baseline case without vertical drains, was also analyzed to provide a further basis for comparison.
Figure 9.23 Three stages of embankment construction sequence

Table 9.5 Model parameters for foundation soil (Indraratna et al. 1997)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\kappa$</th>
<th>$\lambda$</th>
<th>$E_{cs}$</th>
<th>$M$</th>
<th>$\nu$</th>
<th>$\gamma_{bulk}$ (kN/m$^3$)</th>
<th>$K'_x$** (10$^{-6}$ m/s)</th>
<th>$K_x$ (10$^{-9}$ m/s)</th>
<th>$K_y$ (10$^{-9}$ m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>0.06</td>
<td>0.16*</td>
<td>3.10</td>
<td>1.19</td>
<td>0.29</td>
<td>16.5</td>
<td>2.20</td>
<td>6.40</td>
<td>3.00</td>
</tr>
<tr>
<td>2-6</td>
<td>0.06</td>
<td>0.16*</td>
<td>3.10</td>
<td>1.19</td>
<td>0.31</td>
<td>15.0</td>
<td>1.80</td>
<td>5.20</td>
<td>2.70</td>
</tr>
<tr>
<td>6-8</td>
<td>0.05</td>
<td>0.15</td>
<td>3.06</td>
<td>1.12</td>
<td>0.30</td>
<td>15.5</td>
<td>1.10</td>
<td>3.10</td>
<td>1.40</td>
</tr>
<tr>
<td>8-18</td>
<td>0.04</td>
<td>0.09</td>
<td>1.61</td>
<td>1.07</td>
<td>0.25</td>
<td>16.0</td>
<td>0.44</td>
<td>1.30</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Note: * The value of $\lambda$=0.36 was adopted for foundation clay of 0-6 m beneath the embankment.

** $K'_x$, horizontal permeability equivalent for vertical drain

Table 9.6 In-situ conditions in the vicinity of the embankment (Indraratna et al. 1997)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\sigma'_{xz}$, kPa</th>
<th>$\sigma'_{yz}$, kPa</th>
<th>$u$, kPa</th>
<th>$P_c$, kPa</th>
<th>Soil condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>110</td>
<td>Compacted crust</td>
</tr>
<tr>
<td>2</td>
<td>19.8</td>
<td>33</td>
<td>2.4</td>
<td>90</td>
<td>Very soft clay</td>
</tr>
<tr>
<td>6</td>
<td>30.8</td>
<td>51.3</td>
<td>41.6</td>
<td>40</td>
<td>followed by soft</td>
</tr>
<tr>
<td>8</td>
<td>37.6</td>
<td>62.6</td>
<td>61.3</td>
<td>60</td>
<td>silty clay</td>
</tr>
<tr>
<td>18</td>
<td>74.8</td>
<td>124.6</td>
<td>159.3</td>
<td>&gt;65</td>
<td>Dense silty clay (stiff)</td>
</tr>
</tbody>
</table>
9.3.2.3. Finite Element Analysis

The study of Indraratna et al. (1997) has shown that the coupled consolidation model was more realistic in representing the actual field conditions accommodating both the immediate deformation and the consolidation response. Thus, coupled analyses were conducted to compute the immediate and time-dependent deformation.

The idealized mesh configuration of the embankment is given in Figure 9.24 with the boundary and support conditions. It was possible to exploit symmetry and consider one half of the embankment in the analysis where the lateral flow was not permitted across the centerline of the embankment. The maximum horizontal length of the finite element mesh was made at least 5 times that of the vertical dimension so that the boundary effect could be minimized. Six layers of Linear Strain Quadrilateral elements were employed. The locations of the horizontal grid lines of the mesh were based on the different material zones identified in Table 9.5.

The rate of construction loading was simulated by successive addition of elements corresponding to each stage of construction. The vertical drain system was converted to an equivalent drainage wall and the ideal drain condition was assumed, i.e. the thickness of the equivalent drainage wall was negligible compared to the spacing of the drains (2.0 m). Vertical drainage walls were arranged according to their equivalent spacing and the corresponding equivalent permeability, as shown in Table 9.4, was assigned to the zone stabilized by the drains (Indraratna et al. 1997).
In the original study of Indraratna et al. (1997), the main consideration was the settlement and lateral deformation of the foundation clay. Therefore, the effect of the geogrids was simplified as a 0.5 m reinforced sand layer between the original ground level and embankment. The reinforced layer and the embankment soil fill were modeled as Mohr-Coulomb materials and the parameters are given in Table 9.7 (Indraratna et al. 1997). These assumptions were also followed accordingly in the present study.

<table>
<thead>
<tr>
<th>$E$ (kPa)</th>
<th>$\nu$</th>
<th>$c_0$ (kPa)</th>
<th>$\phi^o$</th>
<th>$\gamma_{bulk}$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5100</td>
<td>0.3</td>
<td>15</td>
<td>31</td>
<td>20.5</td>
</tr>
</tbody>
</table>

For EPS embankment, both the Mohr-Coulomb model and the variable $K$-$G$ model were tested, and similar results were obtained due to the stress state in the EPS embankment lying within the elastic limit of the EPS material. The parameters for EPS fill are refer to $K$, $G$ parameters in Table 9.1.

The embankments with different fills were illustrated in Figure 9.25. In the full EPS replacement embankment, the EPS blocks were constructed on top of the 0.5 m sand layer above the ground surface. The heights of the EPS fills were 7.5 m at the central embankment and 2.5 m at the side embankment respectively. The maximum construction height was 9.2 m, which equals the final height of the baseline case.

In the partial EPS replacement embankment, the central embankment was first filled with compacted soil and the side embankment with EPS blocks, up to 3.0 m. Then, the central embankment was filled by EPS blocks (height of 4.5 m) up to 7.5 m, and the side embankment and slope were covered by 1 m of compacted soil. At the last stage, the central embankment was covered by compacted soil of 1.2 m at the top of the embankment.
The variations of the total vertical stress and pore water pressure from the finite element analysis, at the bottom of the embankment (centerline), were analyzed for the different methods during the construction and are shown in Figure 9.26 and 9.27 respectively. As shown, the vertical drain does not affect the total vertical stress but it reduces the pore water pressure significantly, especially at later stages of construction. On the other hand, the results clearly show that the EPS fill significantly decreases the total vertical stress as well as the pore water pressure.
CHAPTER 9: FINITE ELEMENT ANALYSIS OF EPS GEOFOAM

Figure 9.26 Total vertical stress during construction period

Figure 9.27 Variation of pore water pressure during construction period
Figure 9.28 Settlement of the subsoil after embankment construction for different methods

Figure 9.29 Lateral displacement profile beneath the embankment toe (stage 3)

9.3.2.4. Prediction of Settlement and Lateral Deformations

The finite element settlement profiles after the construction along the base of the embankment and the lateral displacement under the toe of the embankment are shown in Figure 9.28 and 9.29, respectively. As expected, the maximum settlements occur beneath
the centerline, and decrease in magnitude towards the toe of the embankment. The results of the analysis clearly show the effectiveness of the different improvement techniques.

For the situation where the ground was improved by geogrids and preloading only, the original permeability of the foundation soil was used, and this case represented the embankment construction without vertical drains. At the time of completion of the construction, the maximum settlement of the foundation was 1.18 m, and the maximum lateral displacement was 0.62 m at the toe of the embankment. In comparison, in the baseline case (vertical drain + geogrids + preloading), the maximum settlement was 1.39 m, and maximum lateral displacement was 0.53. The results also indicated that the vertical drain speeded the consolidation and dissipation of the excess pore pressure of the foundation soil during the construction period. On the other hand, for the foundation without vertical drains, the embankment with EPS fill showed a significantly decrease in settlement and lateral movement of the foundation soil. For full replacement EPS fill, the maximum settlement and lateral displacement were 0.27 m and 0.018 m respectively. For partial replacement EPS fill, the maximum settlement and lateral displacement were 0.52 m and 0.12 m respectively.

In order to evaluate the long-term behaviour of the embankment, the FEM consolidation analysis was continued after the embankment construction was completed. Figure 9.30 shows the settlement with time up to 1800 days (approximately 5 years), the measured results are also included. Figure 9.31 further shows the settlement with time up to about 50 years.

It should be noted that the calculated settlement due to the consolidation of soil (Figure 9.30) does not include any correction, although a correction was suggested in the paper by Indraratna et al. (1997). In soil mechanics, the settlement of a foundation on a saturated soil may be considered as consisting of three different types (Simons 1987):

(i) Immediate, elastic, or initial settlement which occurs immediately upon load application under conditions of no change in volume.

(ii) The primary consolidation settlement develops as the volume changes as a consequence of the dissipation of excess pore water pressure.

(iii) The secondary settlement, which is a creep phenomenon and occurs under conditions of practically zero excess pore water pressure.
Figure 9.30 Settlement of embankment from FEM consolidation analysis (no creep) and measured data

Figure 9.31 Long-term settlement for about 50 years
In practice, the consolidation settlement of the foundation soil is generally based on the Terzaghi theory of one-dimensional consolidation. The consolidation parameters, $C_v$, $m$, or $C_c$ are determined by standard oedometer test. The theoretical results, however, need to be corrected based on the field situations such as the state of the stress and the pore pressure. Many empirical methods exist to do such corrections, and Skempton and Bjerrum (1957) introduced the correction factor $\mu$ on the calculated settlement, which is most often used in soil mechanics. The coefficient $\mu$ is a function of the pore pressure parameter $A$, and the shape of the foundation.

The analyses were also performed on EPS embankment with vertical drains. Figure 9.32 shows the results of settlement during construction period. The consolidation at the end of the embankment construction is 1.35 m and an additional 0.45 m settlement occurs at 50 year. The modeling results show that the consolidation at the end of construction is approximately 75%. It appears that, in this case, the design of the vertical drain is not as effective as the other cases.

For the case of partial EPS replacement, it is found that the vertical drains have a notable effect in speeding the consolidation during the construction and reducing lateral displacement. However, in the case of full replacement, the effect of vertical drains, as anticipated, is negligible. This is due to the very small surcharge load and negligible excess pore pressure. This result indicates that for full replacement EPS embankment, it may not necessary to undertake any ground treatment before construction which will save construction time as well as cost.

Based on the above analysis, the construction of EPS embankments may be completed in a relatively very short time without the waiting period between the construction stages.
Figure 9.32 Effect of vertical drains on EPS embankment: (a) settlement, (b) lateral displacement

9.3.2.5. Discussion

The results in the above section show that, in the baseline case, the primary consolidation (settlement = 1.35 m) is completed soon after the construction and the additional settlement is 0.45 m for the next 50 years. The modeling result shows that the consolidation at the end of construction is approximately 75%. It appears that, in this case, the design of the vertical drain is not as effective as other cases.
In the case of the foundation without vertical drains, the predicted settlement is 1.18m at the end of the construction and the consolidation continues with time for another 10 years approximately. The settlement at 50 years is 2.18m.

For full and partial EPS fill cases, the final total settlements are 0.29 m and 0.73 m, respectively (Figure 9.31).

By comparing the FEM results of the baseline case and the measured data, it is found that even though the predicted lateral displacement fits well with the measured data (Figure 9.29), a large difference exists between the predicted and measured settlement. The predicted consolidation deformation (without any correction) is approximately 66% of the total measured settlement (Figure 9.30). This indicated that the time dependent (creep) deformation of the very soft sub-soil (soft marine clay in this case) is significant and was influenced by the surcharge load of the elevated fill. The total vertical surcharge load after the embankment construction is about 200 kPa (Figure 9.26) which would have stressed the clay beyond the preconsolidation stress into the range of normal consolidation, and the creep component could be very high under such a sustained load. This problem may be solved by using EPS geofoam which significantly reduces the surcharge load and the large deformation due to the creep can be avoided.

This case study has shown the advantages of using EPS geofoam in an embankment fill. The most important advantages are time efficiency and flexibility in application. The project could be completed in very short time and the fill height of the EPS embankment can be adjusted depending on the ground condition to limit the settlement and lateral displacement specified for design, as well as the budget of the project.
9.4 SUMMARY

1. The variable moduli $K$-$G$ model has been developed in this thesis for predicting the deformation of EPS geofoam fill in geotechnical applications. In this model, the moduli $K$ and $G$ are functions of the current stress state, and the model parameters can be obtained by laboratory experiments.

2. Verification studies have indicated that the model is able to capture, for the most part, the essential characteristics of the EPS material behaviour, and reproduce the stress-strain response satisfactorily.

3. The finite element analysis has shown the important advantages of the EPS fill techniques in geotechnical applications such as minimizing the ground disturbance and significantly reducing the settlement and lateral displacement within foundation soil. Also, the stress in the EPS embankment was well within the elastic limit and the deformation of the EPS embankment was very low which indicates that the EPS embankment itself is very stable.

4. The hybrid method (FEM with semi-analytical solution) may provide a more reasonable solution for the long-term problem of an embankment resting on very soft sub-soil where the creep is expected to be significant.

5. For long-term behaviour of the embankment, the EPS fill is largely able to reduce the excess pore pressure and the total surcharge load on underlying layers, so that the foundation soil remains over-consolidated relative to the loading. This will assist to minimize the problems of consolidation and creep in the foundation soil.

6. FEM analysis show that another advantage of the EPS fill technique, probably the most important in geotechnical applications, is the time efficiency. This enables the construction work to be completed in a very short time.

7. EPS fill technique generally offers an appropriate solution for those soft grounds with problems of large subsidence, differential settlement, high excess pore pressure, consolidation and long-term stability.
CHAPTER 10
SUMMARY AND CONCLUSIONS

10.1. SUMMARY

The main aims of the present study are to gain an understanding of the mechanical properties and engineering behaviour of EPS geofoam in geotechnical application, to develop a constitutive model of EPS geofoam, and to develop useful methods of analysis for geotechnical applications where EPS geofoam are used. In this thesis, the research on geotechnical engineering applications of EPS geofoam around the world have been widely reviewed. Extensive laboratory experimental studies and numerical modeling have been conducted to achieve the above aims. The principal studies are summarized as follow:

First, the nature of the problem was outlined and the current gaps in the knowledge of application of EPS geofoam in geotechnical engineering were presented. The relevant methods for testing EPS geofoam under different loading and environmental conditions involved in various geotechnical applications were outlined.

In Chapter 2, the various applications of the EPS geofoam in geotechnical engineering have been summarized. The research and development of EPS geofoam techniques in geotechnical engineering applications have been reviewed. The various laboratory test methods involved with the laboratory studies in this thesis were also reviewed and discussed.

Under monotonic compressive loading condition, the deformation behaviour, strength parameters and elastic and plastic moduli of EPS were characterized by testing EPS specimens under various loading conditions. The factors, such as confining stress, temperature, strain rate and geometry, that may affect the behaviour of EPS were discussed and summarized in Chapter 3.

Shear behaviour of soil is one of the most important considerations in geotechnical engineering. In Chapter 4, the main mechanism of shearing in direct shear loading and
triaxial compression were studied. As well, the internal shear behaviour of EPS, EPS-EPS interface and EPS-sand interface were investigated.

The time-dependent or long-term performance and durability of EPS is of primary interest in geotechnical applications such as embankment fill. Chapter 5 studied the long-term behaviour of EPS under sustained loading conditions. The results from creep tests with different applied loads under normal and elevated temperature were obtained. Creep models for EPS were proposed and assessed.

The loading conditions imposed on embankment subgrade are the result of the dead loads of the pavement structure and the cyclic loads generated by traffic, earthquakes, winds and temperature. EPS geofoam, as a visco-elastoplastic material, is known to be susceptible to cyclic strength degradation where large resilient and permanent deformation may be induced by cyclic loading. In Chapter 6, the behaviour of EPS under cyclic loading has been studied and the results presented.

In Chapter 7, a scale-model study of the EPS geofoam used as a replacement subgrade material under flexible pavement was undertaken. This study provided a fresh perspective on which to draw insights of the behaviour of the geofoam as it was subjected to vehicular loading under realistic but controlled conditions not possible to attain under conventional laboratory tests.

In Chapter 8, a semi-analytical viscoelastic EPS–Soft soil model was proposed for analyzing the long-term behaviour (both consolidation and creep) of EPS embankment resting on the soft soil.

Finally, a variable moduli model was developed for FEM analysis of EPS geofoam under monotonic and cyclic loading conditions. The parameters of the constitutive model were derived from the laboratory test results conducted in this study. The variable moduli model has been verified by independent laboratory tests and numerical modeling. This model was subsequently applied in FEM analysis to analyze typical EPS embankments in some case studies.

10.2. CONCLUSIONS

A series of laboratory tests and numerical studies have been performed on EPS geofoam specimens to investigate the mechanical properties and response of EPS geofoam under
various geotechnical loading conditions. The main conclusions of this thesis derived from experimental and numerical studies may be summarized as follows.

10.2.1. Mechanical Properties and Compressibility of EPS Material

The density is an index property of EPS geofoam material. The mechanical properties such as compressive strength, yield stress and initial Young’s modulus of EPS geofoam are primarily dependent and proportional to its density.

In a triaxial loading condition, the influence of confining pressure on the EPS geofoam behaviour is very small. The test results indicated that, with increasing confining pressure, the deviator stress remains approximately constant. The compressive strength and the initial Young’s modulus were found to be essentially independent of the confining pressure.

The studies in this thesis indicate that the Poisson’s ratio of EPS geofoam is very small in magnitude and range from -0.07 to +0.02. After yielding, the Poisson’s ratio becomes approximately constant with an average negative value of -0.02.

Generally, in circumstances of quick (short-term) compressive loading within the elastic limit, the temperature does not have a significant influence on the stiffness (initial Young’s modulus) of EPS geofoam. With rising temperature, the compressive strength and the yield stress of the EPS geofoam decrease slightly and approximately linearly. The triaxial test results also indicated that, even at a relatively high temperature of 45°C, the confining pressure has only a very small influence on the compressive strength of the EPS geofoam. However, it is important to note that the temperature effect on the long-term behaviour of EPS is significant, as discussed in a later section 10.2.3.

The test results for different geometry sizes indicated that the compressive strength of the EPS increases with increasing cross sectional area. For this reason alone, it is necessary to standardize the size of the specimen used in the strength test of EPS geofoam.

The strain rate also influences the behaviour of EPS geofoam. The results of testing on EPS specimens at different strain rates show that the compressive strength and plastic Young’s modulus increase with increasing strain rate, especially, at the lower strain rate (lower than 4% per minute). For the strain rate of 5 to 10% (2.5 to 10 mm of deformation) per minute, the influence will be negligible. It would appear that the strain rate in a
standard test should be set within this range.

10.2.2. Shearing Resistance and Interface Friction
Some interesting findings have been obtained from the shear box test and triaxial test. The test results indicate that the shear strength of EPS geofoam is derived mainly from cohesion or adhesion of the EPS particles (about 42 kPa for 20kg/m³ EPS). The internal friction angle of 20kg/m³ EPS is very small with an average value of 8.2° from triaxial test and 6.4° from shear box test, which indicated that the internal shear friction seems not to be significantly influenced by the normal stress.

The friction angle of the interface of the EPS blocks is relatively high and decreases with increasing normal stress. Test results indicated that the interface friction angles of the 20kg/m³ EPS blocks are 52.1° for normal stress lower than 20 kPa. For normal stress of 20 to 40 kPa, the interface friction angle is approximately 33.7°, which is equivalent to a friction coefficient of 0.67.

The interface friction angle of the EPS and compacted Nepean sand is 30.1°, which is equivalent to a friction coefficient of 0.52.

The EPS material has been found to offer high shear resistance relative to its compressive strength. However, the interfacial joint between the blocks, especially the joint between EPS and soil, may form a plane of shear weakness, which could give rise to a problem of shear failure.

10.2.3. Creep Behaviour
In room temperature conditions the EPS material is generally long-term stable for a stress level less than 60kPa, where the creep rate continues to decrease with time. However, as the stress level increases above 60 kPa, a transitional though long-term stable condition exists where the creep rate eventually becomes constant with time. The threshold stress level at room temperature appears to be about 60 kPa.

The creep strain of EPS increases with increasing temperature. At a temperature of 40°C, and for stress levels less than 50 kPa, the EPS material is long-term stable. However, for a stress level of 50kPa at 40°C, the strain rate increases to a constant value at later elapsed times and into a transitional stage between long-term stable and unstable behaviour. This value is considered as threshold value for design purpose, in higher
temperature environments.

The modified four-element viscoelastic model and the power law model based on four month data of the creep tests have been found to be able to adequately predict the creep strain of EPS geofoam beyond the elapsed time as long as 14 month.

10.2.4. Resilient Behaviour of EPS

The studies from the cyclic tests on EPS at different static and dynamic stress levels show that the deformation of EPS is affected by applied stress level (or strain) and the number of loading and unloading cycles. The plastic (permanent, non-recoverable) strain increases with an increase in the static stress, but the static stress level has very little influence on the resilient strain. On the other hand, the resilient strain increases with increasing dynamic stress, but has little effect on the plastic strain under the stress ranges studied.

For total stress levels less than 70kPa, the resilient modulus of EPS only decreases slightly with increasing static stress, but remains constant as the dynamic stress is increased. Two distinct resilient behaviours have been shown: below 70 kPa the geofoam remains resilient but above 70 kPa, the material loses resilience very quickly.

The limiting stress and strain versus load repetition criteria for 20 kg/m³ EPS geofoam in pavement subgrade application developed in this thesis may be used predict the approximate load repetition to failure (at accumulated permanent strain of 0.5%). For given service life, these criteria may also be used to calculate the limiting applied stress or applied strain level in pavement design.

In terms of permanent deformation at subgrade level, EPS geofoam performs as well as, if not better than, compacted sand under the repeated traffic loading. However, for an EPS geofoam subgrade, the resilient deformation (resilient modulus) will have a significant influence on the rut depth of the pavement surface. The rut depth could be reduced by using an appropriately designed pavement structure (e.g. increase the pavement thickness or use a stiffer pavement).

Test results show that block size and lateral restraint did not significantly affect the performance of the EPS geofoam blocks under repeated loading. These findings should give design engineers added flexibility in design.
10.2.5. Modeling the Behaviour of EPS Fill Embankment

A semi-analytical one-dimensional EPS — soft soil model has been developed to analyse the typical EPS embankment resting on soft soil, which can easily be used to evaluate the long-term effectiveness of the EPS fill solution for overcoming settlement problems in soft soil. One of the important features of this simple model is that it is able to predict both the consolidation and creep deformation of the EPS embankment and soft sub-soil.

Finite element analysis have shown that the EPS fill embankment is able to largely reduce the excess pore pressure and the total surcharge load on underlying soil layers. Therefore, the problems of consolidation and creep in the foundation soil could be minimized. Furthermore, the EPS fill could be able to adjust its height and volume depending on the ground condition to meet the requirements of the project.

Another advantage of the EPS fill technique, probably the most important in geotechnical applications, is the time efficiency achieved in using this technique without the adverse consequences associated with long construction times.

10.3. FURTHER RESEARCH AND RECOMMENDATIONS

To date, most laboratory research work carried out on EPS geofoam has been for short term loading and limited to certain types of EPS. In this these studies, the triaxial tests have been undertaken for only three types of EPS with densities of 13, 20 and 27 kg/m³ respectively. Other tests such as cyclic loading tests, shear tests, creep tests and model tests have all been done on EPS of 20 kg/m³. It is recommended that further researches are needed to study the dependence of these mechanic properties, such as those related to resilient, shear and creep response, on the EPS densities. This will be helpful to fully understand the behaviour for the range of EPS geofoam that is manufactured.

Due to a lack of available data, no multiaxial creep model was developed. For the same reason, no development work was undertaken for a coupled temperature dependent creep model. It is suggested that a research program is needed to address these shortcomings if the long-term behaviour of this material is to be fully understood.

There is scope for developing more sophisticated constitutive models of EPS geofoam which are suitable for geotechnical applications.
More importantly, as the EPS geofoam is used widely in geotechnical engineering applications, it is necessary that the relevant test methods should be standardized for the determination of the properties of EPS for design and analysis. For carrying out a standard triaxial test on the EPS specimen, the following factors should be taken into consideration and standardized:

The size and geometry of the EPS specimen. It is recommended that the cylindrical EPS specimen with a diameter of 50 mm and a height of 50 mm be adopted as the standard for soil laboratory testing.

The loading rate. From the geotechnical viewpoint, a rate of 5 to 10% of the initial height of the EPS specimen per minute is considered compatible to similar tests for foundation soil and subgrade material.

The confining pressure. For the EPS specimen, it is recommended that the confining pressure be limited to not more than 20kPa, because of the large deformation expected at higher confining pressure and since higher confining pressures are not realistic in terms of the EPS geotechnical applications.

The method of evaluating the Poisson’s ratio of EPS should also be standardized. The method adopted in this thesis is suggested as one which could be considered as a standard test.
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Aaboe, R., 1987. 13 years of experience with expanded polystyrene as a lightweight fill material in road embankments, *Publication No. 61, Norwegian Road Research Laboratory*, Oslo, Norway. pp.21-27


REFERENCES


Chambers, R.E., 1984. Materials criteria for structural design, *Chapter 3 in Structural plastics design manual - Volume 1; ASCE manuals and reports on engineering


DIN 18164 Part 1, Schaumkunststoffe als Dämmstoffe für das Bauwesen (Cellular plastics for insulation in buildings).


REFERENCES


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APPENDIX A
PLEASE NOTE

The greatest amount of care has been taken while scanning the following pages. The best possible results have been obtained.
APPENDIX A

MANUFACTURE, CLASSIFICATION AND PROPERTIES OF EPS

A.1. EPS MANUFACTURE AND CLASSIFICATION

Today, EPS geofoam is produced in every continent by a host of different manufacturing companies. In Australia, the current biggest manufacturer of EPS products is the firm RMAX Rigid Cellular Plastics, the supplier of the EPS geofoam used in the present study. EPS is manufactured in Australia in accordance with the specifications in AS1366, Part 3, 1992.

A.1.1 Manufacturing Process
EPS geofoam is normally supplied as a rectangular block which has dimensions ranging from 0.50m × 1.00m × 2.00m to 0.60m × 1.20m × 6.00m (Horvath, 1996). The standard dimension of RMAX EPS foam is 0.30m × 1.20m × 5.00m, and can be cut as required. The raw material used to make EPS is polystyrene resin which is molded into a block in three stages as summarized below.

Pre-expansion
The polystyrene resin raw material is heated to about 80 °C to 110 °C in special foaming units, using steam. Under heat and pressure, the polymer softens and the gas bubbles expand to create cellular spheres about 50 times their original volume. Pre-expansion converts the compact beads into foam beads, each containing small, closed polyhedral cells. During this process the apparent density of the material drops from about 630 kg/m³ to about 10 kg/m³, depending on the temperature and residence time.

Intermediate aging
Several hours are allowed for cooling and stabilization to occur. As the freshly foamed particles cool down, a blowing agent and steam are passed into, and condense inside, the cells producing a vacuum, which has to be equalized by air diffusing into the cells. This imparts
greater mechanical stability to the beads, as well as extra foaming power, which is advantageous for further processing. This process takes place during intermediate aging in ventilated silos. At the same time, it gives the beads an opportunity to dry.

**Moulding**

The cavity of the foaming mould, which is usually in two parts, is now filled pneumatically with the pre-foamed material. The mould walls are equipped with holes or slits to connect the mould cavity with the steam chamber. Foam molding is accomplished by the use of steam to supply the necessary energy.

A surge of steam causes the beads to soften again and to expand. The expansion pressure compresses the beads and, at the same time, forces them against the rigid mould walls so that they fuse together.

The resultant part is then cooled by spraying water onto the mould and by applying a vacuum. When it has cooled down sufficiently, the molded part can be taken from the mould. Cooling is allowed over several days during which the block undergoes some dimensional change, usually shrinkage although swelling sometimes occurs. The change, however, decreases at a rate rapidly with time. The blocks are trimmed or cut to size by a cutting machine using heated wires according to specifications.

The special manufacturing process makes it possible to produce EPS foam of a wide range of bulk density. Molded EPS geofoam is normally produced within a range of densities between 10 kg/m³ and 40 kg/m³.

**A.1.2 Classification**

In Australia, EPS geofoam is available in six classes (AS 1366, part 3, 1992) with densities ranging from 11 to 28 kg/m³ as shown in Table A.1. The most important characteristic of EPS is its extremely low density. The density of EPS has a direct influence on the other physical properties, such as compressive strength and elastic modulus. In the manufacture of EPS blocks, however, densities are controlled by duration of expansion time rather than applied pressure. Therefore, each block has some variability in its density as a consequence of the molding process. Eriksson and Trank (1991) reported that the bulk density of an EPS block varies both within the layer and between layers, ±4% of the mean for the layer and ±8% of the whole block between layers. Variation of 25% has been measured for individual specimen
APPENDIX A: MANUFACTURE, CLASSIFICATION AND PROPERTIES OF EPS

from the same block.

Because the properties of the EPS material depend largely on its density, foams can be made with a range of application-specific properties. Elsewhere, EPS geofoam of density 20 kg/m³ is typically used as fill material. As a guide, Class M geofoam will most likely be the EPS geofoam of choice for geotechnical applications in Australia.

In the USA, EPS production is oriented towards traditional (non-geotechnical) construction markets where EPS is used as thermal insulation in above-ground applications (Horvath 1995). The ASTM Standard C 578-92 defines five “types” (densities) that are applicable to EPS, there are summarized in Table A.2.

In UK, in accordance with BS3837:Part 1:1986, polystyrene is classified into grades according to compressive stress/strength as determined by BS 4370: Part 1 : 1988 Method 3. The EPS is produced in SD, HD, EHD and occasionally UHD grades for using as fill. Details are given in Table A.3.

<table>
<thead>
<tr>
<th>Table A.1 Classes of EPS geofoam according to AS 1366, Part 3 (1992)</th>
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<tr>
<td>CLASSES</td>
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<tr>
<td>Nominal density(kg/m³)</td>
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<td>Compressive stress at 10% Deformation (kPa)</td>
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<th>Table A.2 Standard EPS densities from ASTM Standard C 578-92 (Horvath, 1995)</th>
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<td>Material type</td>
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<tr>
<td>XI</td>
</tr>
<tr>
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<tr>
<td>VIII</td>
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<tr>
<td>II</td>
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<td>IX</td>
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Table A.3 Classes of EPS geofoam according to BS3837:Part 1:1986

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<th>Grade</th>
<th>Dry density (Mg/m³)</th>
<th>Compressive strength*, kPa</th>
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<td>Standard Duty (SD)</td>
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<td>Ultra High Duty (UHD)</td>
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* where the maximum stress in the test occurs prior to 10% strain the result is reported as compressive strength.

A.2. PROPERTIES OF EPS

A.2.1 Chemical and Physical Structure

EPS is formed by polymerization of mono-styrene, with the addition of pentane. Both pentane, which is contained in petroleum, and styrene, which is a petroleum derivative, are pure hydrocarbons, i.e. they consist solely of carbon and hydrogen.

Styrene is supplied in the form of small spherical beads or cylindrical pellets. The expanded material is formed by fusion of the individual particles. The chemical structure is shown in Figure A.1. Blowing agents n-pentane and iso-pentane, the pneumatogen used in EPS, are members of the alkane family of hydrocarbons as are compounds like methane, ethane, propane and butane. The physical structure of cellular polystyrene consists of an assembly of myriads of tiny, air-filled cells.
A.2.2 Physical Properties of EPS

Most of the physical properties of cellular polystyrene depend on the density of the material. One important property of EPS foam is its mechanical strength when subjected to short-term and sustained loading. EPS foams are classed as rigid foam. When under load they exhibit the type of visco-elastic behaviour that is uncharacteristic of brittle-rigid materials. It is for this reason that the compressive stress at 10% compressive strain is measured instead of the compressive strength. Table A.4 lists the physical properties of EPS according to AS 1366, Part 3, 1992.

Mechanical Properties

As shown in Table A.4, all the mechanical properties of EPS are dependent on the density. The compressive strength, cross breaking (Flexural) strength, tensile strength and shear strength are all increase with increasing bulk density, as shown in Figures A.2 to A.5 (RMAX Technical Data, 1998).
Table A.4 Physical properties of EPS, according to AS1366, Part 3:1992

<table>
<thead>
<tr>
<th>Physical property</th>
<th>Unit</th>
<th>Class</th>
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<td>Compressive stress At 10% deformation</td>
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<tr>
<td>Cross-breaking strength</td>
<td>kPa</td>
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<td>95</td>
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<tr>
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<td>S</td>
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<td></td>
<td>M</td>
<td>200</td>
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<td></td>
<td></td>
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<tr>
<td>Rate of water vapor transmission measured parallel to rise at 23°C</td>
<td>μg/m²s</td>
<td>L</td>
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<tr>
<td></td>
<td></td>
<td>VH</td>
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<tr>
<td>Thermal resistance at 25°C (50mm sample)</td>
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<td>VH</td>
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Figure A.2 Compressive strength of EPS vs. density (RMAX, 1998)
Figure A.3 Cross-breaking strength vs. density (RMAX, 1998)

Figure A.4 Tensile strength vs. density (RMAX, 1998)
Figure A.5 Shear strength vs. density (at 23°C) (RMAX, 1998)

Figure A.6 Thermal conductivity vs. density (at 10°C) (RMAX, 1998)
**APPENDIX A: MANUFACTURE, CLASSIFICATION AND PROPERTIES OF EPS**

**Thermal Properties**
A further important physical property of rigid foam is its thermal insulation capacity. EPS foam is made up of polyhedral cells, 0.2 to 0.5 mm in diameter with a wall thickness of 0.001 mm. They are closed on all sides. The foam consists of about 98% of air and 2% of polystyrene. As is well known, the air entrapped within the cells is a very poor heat conductor and so plays a decisive role in providing the foam with its excellent heat insulation properties. Unlike foams containing other gases, the air stays in the cells so that the insulation effect remains constant.

The heat insulation capacity of a material is defined by its thermal conductivity. The thermal conductivity is a function of the bulk density (kg/m³) of the foam as shown in Figure A.6 (RMAX Technical Data, 1998). It is greater for foams of low bulk density, falls with rising bulk density, passes through a minimum value between about 30 – 50 kg/m³ and after that slowly rises again. The values measured in accordance with AS 1366 for EPS foam with a bulk density of 20 kg/m³ lie in the range of 0.033 – 0.036 W/(m·K) at 10 °C.

The specific heat capacity of EPS foam is not influenced by density. The material heat distortion temperature increases with increasing density and decreasing applied load. The freshly made foam attains its final heat distortion properties only after aging. Foams not under stress will withstand temperatures of up to about 100 °C for short periods, independent of their density.

**Electrical Properties**
EPS foam does not conduct electricity. The dielectric constant ε for foams with densities of between 20 and 40 kg/m³ is 1.02 – 1.04 between 100 Hz and 400 MHz. The dissipation factor tanδ up to 1 MHz is less than 0.0005 and up to 400 MHz is less than 0.00003. The specific dielectric strength reaches values of 2 kV/mm. The resistivity at 23 °C and 50% relative humidity is about 1014 Ω (DIN 53 482). Because of the high resistivity, the surface of foam moldings can become electrostatically charged, especially if the relative humidity is low. The surface resistance of molded parts can be reduced by treating them with antistatic agents.

**Chemical Properties**
EPS behaves exactly like polystyrene in the presence of chemical agents. Chemicals, which attack polystyrene, will destroy EPS foam more quickly than the solid material, because of the
thin-walled cells of which it is made. It follows that the rate of attack increases as the density of the cellular polystyrene decreases. EPS is unaffected by water, most acids and alkali solutions. Essential oils contained in the peel and juice of citrus fruit will attack EPS, but the material is resistant to animal and vegetable fats as well as to anti-corrosive agents containing paraffin, as long as they do not contain aggressive solvents.

Like many other construction materials, EPS foamed plastics are combustible. When assessing their fire behaviour, it must be taken into account that this depends to a substantial extent not only on material-related effects but also on application conditions. Of considerable importance, in particular, is where it is used in combination with other construction materials, the protective and covering layers are necessary.

**Biological Properties**

Rigid EPS foams made from EPS offer no breeding ground for microorganisms. It does not decay, rot or turn moldy. Bacteria in the soil do not attack the foam. Animals can damage it by gnawing or burrowing, but many years of road building experience have shown that they do not prefer it to other conventional insulating materials. EPS foams have no environmentally damaging effects and do not endanger water (crushed EPS waste is used in agriculture to break up and drain the soil).

As biological investigations have shown, if EPS rigid foam is involved in a fire, the toxicity of the gases from burning and carbonization is lower than that for the same amount of wood.

Foamed plastics from EPS have been in production and use for several decades. No harmful effects on health have been discovered in this time. The health-safety of the application of rigid foam boards of EPS is also evidenced by the fact that EPS is used in food packaging (BASF, 1998).
APPENDIX B

MATERIAL MODELING IN SOIL MECHANICS

B.1 ESSENTIALS OF MATERIALS BEHAVIOUR AND IDEALIZATION

For a long time, soil mechanics has been based on Hooke’s law of linear elasticity for stress and deformation analysis for a soil mass where no failure of the soil involved (elasticity problem). On the other hand, perfect plasticity is used to deal with the conditions of ultimate failure of a soil mass (stability problems), such as earth pressure on retaining walls, bearing capacity of foundations, and stability of slopes. Long-term settlement and consolidation problems, however, are considered essentially as viscoelastic problems (Chen and Mizuno, 1990).

As multi-phase materials, soils and rocks comprise mineral grains, air voids and water. these ‘microscopic’ or ‘discontinuous’ effects should be averaged and the soil could be idealized as a continuum. Thus, the mechanical behaviour of soil and rock can be based upon the principles of continuum mechanics, in which the basic sets of equations are:

1. Equilibrium (motion) equations;
2. Geometry conditions or compatibility of deformations; and
3. Material constitutive laws (stress-strain relations)

Once the material stress – strain relationship is known, equations of equilibrium and compatibility are used to determine the state of stress or strain when an idealized body is subjected to prescribed loading.

For common engineering materials like metals, plastics, ceramics and engineering soils,
the stress-strain curves are similar. However, for soils and other granular materials, the soil behaviour (stiffness and strength) is governed by the effective stress due to the pore pressures in the voids.

For materials essentially elastic and then plastic, the parameters required are Young’s modulus $E$, Poisson’s ratio $\nu$ and the yield and ultimate stresses which can be obtained from a uniaxial extension or compression test.

In soil mechanics, however, the shear and bulk moduli, $G$ and $K$, are preferable to Young’s modulus $E$ and Poisson’s ratio $\nu$ because it is important to consider shearing or deformation separately from compression or change size (Atkinson, 1993).

### B.2 ELASTICITY AND MODELING

Elastic material models are generally classified as linear elastic (generalized Hooke’s law) model and nonlinear elastic models as Cauchy elastic, hyperelastic, and hypoplastic models. Various publications are available on these theories. These models are briefly reviewed.

The linear elastic model has formed the basis of various nonlinear elastic stress-strain relations used in engineering practice which can be represented by the generalized Hooke’s law as:

$$\sigma_{ij} = B_{ij} + C_{ijkl} \varepsilon_{kl} \quad (B.1)$$

where $\sigma_{ij}$ and $\varepsilon_{kl}$ are stress and strain tensor respectively, $B_{ij}$ are components of initial stress tensor corresponding to the initial strain free state and $C_{ijkl}$ is a fourth-order tensor of elastic material constants.

In triaxial loading condition, a simple constitutive equation relating shearing and
volumetric stress-strain behaviour can be written as

$$
\begin{pmatrix}
\delta \varepsilon_s \\
\delta \varepsilon_v
\end{pmatrix} =
\begin{bmatrix}
C_{11} & C_{12} \\
C_{21} & C_{22}
\end{bmatrix}
\begin{pmatrix}
\delta q \\
\delta p
\end{pmatrix}
$$

(B.2)

where the $\varepsilon_s$ and $\varepsilon_v$ are shear and volumetric strain respectively, the $[C]$ is a compliance matrix, and $q$ and $p$ are the maximum shear stress and mean stress, respectively.

The compliance parameters in Equations (B.2) will normally vary with strain, the current stresses and the history stresses. For isotropic and elastic materials, $C_{12} = C_{21} = 0$, $C_{11} = 1/3G$ and $C_{22} = 1/K$ (Chen and Mizuno, 1990).

### B.3 PLASTICITY AND MODELING

The difference between elastic and plastic models is the way loading and unloading are dealt with. In deformation theories of plasticity, the loading criterion is introduced to treat separately the behaviour of materials in loading and unloading. The flow theory of plasticity, by using the loading criterion, avoids the difficulty of neutral change in stress encountered in deformation theory (Chen, 1982).

The variable moduli models (Nelson and Baron, 1971; Nelson et al., 1971; Nelson and Baladi, 1977) are given in the incremental forms as:

$$
dp \equiv K \, d\varepsilon_{hk}
$$

(B.3)

and

$$
d\sigma_{ij} = 2G \, de_{ij}
$$

(B.4)

where the $dp$ and $d\varepsilon_{hk}$ are the mean hydrostatic stress and the volumetric strain increments and the $d\sigma_{ij}$ and $de_{ij}$ are the deviatoric stress and deviatoric strain increments, respectively. Different functions for shear and bulk modulus are applied in initial loading, unloading, and reloading.

In flow (incremental) theory of plasticity, the incremental plastic strain $dE_p$ is related
to the state of stress $\sigma_{ij}$ and stress incremental $d\sigma_{ij}$, and assume that a yield surface $f(\sigma_{ij})$ exists which depends only upon the state of stress. Plastic flow occurs when $f = 0$ and $df = 0$. Thus, $f = 0$ and $df < 0$ indicate unloading, and $f < 0$ indicates the elastic state.

The yield surface define the stress conditions under which plastic deformation will occur and initially given by:

$$f(\sigma_{ij}) = f_c$$  \hspace{1cm} (B.5)

where the $f_c$ is a constant value for a perfectly plastic material but a variable for strain- or work-hardening materials.

The development of modern theory of soil plasticity was strongly influenced by the well-established theory of metal plasticity. The Tresca criterion (maximum shear stress criterion), first proposed for metals, is given by,

$$\tau = k$$  \hspace{1cm} (B.6)

where $\tau$ is the maximum shear stress at failure and $k$ is the yield stress of the material determined from the experiment.

Later, the Von Mises criterion (maximum shear energy criterion), also for metals, has the simple form:

$$J_2 - k^2 = 0$$  \hspace{1cm} (B.7)

Where, $J_2$ is second invariant of the deviatoric stress tensor.

The Tresca's yield condition can be regarded as a special case of the condition of Coulomb's criterion on which the important concept of the limiting equilibrium of soil had been firmly established in soil mechanics (Terzaghi, 1943). The Mohr-Coulomb criterion states that failure occurs when the shear stress and normal stress acting on the material satisfy the linear equation,

$$\tau = c + \sigma_n \tan \phi$$  \hspace{1cm} (B.8)
where the $c$, $\sigma_n$ and $\phi$ are cohesive strength, normal stress and internal friction angle, respectively.

The form of Mohr-Coulomb yield surface is angular in the $\pi$-plane. Drucker and Prager (1952) extended the Coulomb criterion to three-dimensional soil mechanics problems, and approximated the Coulomb criterion by a simple smooth function as:

$$f = aI_1 + \sqrt{J_2} = k$$

(B.9)

where $I_1$ is the first stress invariant, and the constants $a$ and $k$ may be related to the Coulomb's material constants $c$ and $\phi$ as:

$$a = \frac{2 \sin \phi}{\sqrt{2 - \sin \phi}}, \quad k = \frac{6c \cos \phi}{\sqrt{3 - \sin \phi}}$$

(B.10)

The yield surface of Mohr-Coulomb, and Drucker-Prager approximations in principal stress space, are given in Figure B.1 (Zienkiewicz & Pande, 1977, Chen & Baladi, 1985).
B.4 HARDENING PLASTICITY AND MODELING

The elastic perfectly plastic material can readily be extended to a material with hardening. The incremental theory of plasticity for hardening materials is based on three fundamental assumptions (Chen and Mizuno, 1990):

(i) the shape of an initial yield surface;
(ii) the evolution of subsequent loading surface (hardening rule); and
(iii) the formulation of an appropriate flow rule.

The loading function in stress space changes with the plastic flow which is expressed in terms of the stress state $\sigma$, the plastic strain $\varepsilon^p$, and the hardening parameter $k$, and the general form of hardening plasticity models may be written as:

$$f(\sigma, \varepsilon^p, k) = 0$$  \hspace{1cm} (B.11)

The flow rule is of the associated type if the plastic potential function takes the same
form as that of the yield function as:

\[
d\varepsilon^p_{ij} = d\lambda \frac{\partial f}{\partial \sigma_{ij}}
\]  

(B.12)

Drucker et al. (1957) introduced the concept of work-hardening plasticity into soil mechanics and a spherical end-cap to the Drucker-Prager model, as shown in Figure B.2, in order to control the plastic volumetric change of soil, or dilatancy. They also used the current soil state variable (density, voids ratio, or plastic compaction) to determine the successive loading cap surfaces.

For a stable work-hardening material, Drucker (1951) postulated that:

1. During the application of the additional stresses, a positive work is taken place.
2. For a complete cycle of additional loading and unloading, additional stresses produce positive work if the plastic deformation takes place. The work will be equal to zero only if the deformation is totally elastic.

For the loading process:

\[
d\sigma_{ij} (d\varepsilon^e_{ij} + d\varepsilon^p_{ij}) > 0
\]  

(B.13)

For the cycle of loading and unloading:

\[
d\sigma_{ij} d\varepsilon^p_{ij} > 0
\]  

(B.14)

since the work on elastic strains is zero.

By extending the basic concept of Drucker et al., Roscoe et al. (1958, 1963) developed the famous Cambridge models to formulate the complete stress-strain model for normally consolidated or lightly overconsolidated clay. The Cam-clay combines the theories of critical state soil mechanics and the idea of a state boundary surface with the theories of plasticity, including yielding, hardening and plastic flow.
APPENDIX B: MATERIAL MODELING IN SOIL MECHANICS

Figure B.2 Drucker-Prager strain-hardening cap model in principal stress space

The complete constitutive equations for original Cam-clay (Schofield and Wroth, 1968) can be written as constitutive equation (B.2) where the components of the compliance matrix are

\[
\begin{align*}
C_{11} &= \frac{1}{\nu p} \left[ \frac{\lambda - \kappa}{M(M - \eta')} + \frac{\sigma_0}{3} \right] \\
C_{22} &= \frac{1}{\nu p} \left[ \frac{\lambda - \kappa}{M} (M - \eta') + \kappa \right] \\
C_{12} = C_{21} &= \frac{1}{\nu p} \left[ \frac{\lambda - \kappa}{M} \right]
\end{align*}
\]  

(B.15)

where \(\lambda, M, \nu, \kappa,\) and \(\sigma_0\) are soil parameters.

In Cam-clay, a set of non-linear constitutive equations was obtained in terms of these soil parameters together with parameters describing the current state and the loading history.

In Modified Cam-clay Model (Burland, 1965; Roscoe and Burland, 1968), an isotropic, nonlinear elastic strain-hardening plastic model, the elastic shearing strain is assumed to be identically zero and only the volumetric strain is assumed to be partially recoverable. The elastic volumetric strain is nonlinearly dependent on the hydrostatic stress and independent of
the deviatoric (shear) stresses. For certain stress histories, strain-softening may occur.

The volumetric strain increment expressed by:

$$d\varepsilon_{ik} = -\frac{\lambda}{(1+\varepsilon)p} \, dp$$  \hspace{1cm} (B.16)

And the recoverable (elastic) component is expressed by:

$$d\varepsilon^e_{\varepsilon} = -\frac{\kappa}{(1+\varepsilon)p} \, dp$$  \hspace{1cm} (B.17)

While the irrecoverable or plastic component is:

$$d\varepsilon^p_{\varepsilon} = d\varepsilon_{\varepsilon} - d\varepsilon^e_{\varepsilon} = -\left(\frac{\lambda - \kappa}{(1+\varepsilon)p}\right) \, dp$$  \hspace{1cm} (B.18)

and the tangential elastic bulk modulus, $K$, is given by:

$$K = \frac{(1+\varepsilon)}{\kappa}$$  \hspace{1cm} (B.19)

On the other hand, elastic shear strain is assumed to be identically zero. Therefore, elastic shear modulus $G$ can be made quite large, perhaps one hundred times the elastic bulk modulus, if it is desired to keep the computational model as close as possible to the Modified Cam-clay (Chen and Mizuno, 1990).

However, in program implementation of the Cam-clay models, the bulk and shear moduli ($K$, $G$) were used and considered as function of current stress state (Britto and Gunn, 1987), where the $K$ is given by equation (B.19) and the $G$ is obtained by experiment or calculated by:

$$G = \frac{3}{2} \left(1 - 2\nu\right) K$$  \hspace{1cm} (B.20)

**B.5 SUMMARY**

In numerical modeling in geomechanics, the constitutive models maybe classified by the degree to which the real soil behaviour is being matched. Brinkgreve and Vermeer (1992) classified
the perfect plastic Mohr-Coulomb model as a 1st-order approximation. In fact, it matches fully plastic failure behaviour quite well, but pre-failure regime is poorly modeled by using Hooke's law of elasticity. The Cam-clay model provides a 2nd-order approximation, at least for clays.

In developing material models in soil mechanics, either simple or advanced numerical models, the shear and bulk moduli, $G$ and $K$, are preferable because it may consider shearing or change of shape separately, from compression or change size. In advanced numerical models, it is assumed that the material moduli depend on the current stress states.

Although the finite-element method has had a profound effect on the rapid development of the analysis based on the nonlinear stress-strain behaviour of geotechnical materials there are still some analytical and theoretical difficulties associated with the present nonlinear stress analysis in geotechnical engineering problems such as

(i) selection of material models and material constants,
(ii) idealization of the ground conditions, and
(iii) assessment of analytical results

Nevertheless, any modeling result must be assessed by laboratory and in situ test data.

Then, the applicability of the material models used in a nonlinear stress analysis can be re-evaluated.
APPENDIX C
Computer Methods and Advances in Geomechanics

Edited by
Chandra S. Desai, Tribikram Kundu, Satya Harpalani, Dinshaw Contractor
& John Kemeny
The University of Arizona, Tucson, Arizona, USA

VOLUME 2
8 Laboratory and field testing, parameter determination; 9 Dynamics, earthquake analysis, liquefaction; 10 Soil/Rock-structure interaction; 11 Ground improvement and geosynthetics; 12 Petroleum geomechanics, hydraulic fracturing, geological simulation; 13 Pavement geomechanics; 14 Foundations, footings, piles; 15 Stopes, dams, walls, landslides 16 Tunnels, underground works, mining; 17 Flow, seepage, consolidation; 18 Education.
A one-dimensional model of EPS fill on soft soils

C.J. Leo & Y. Zou
Department of Civil Engineering, University of Western Sydney, N.S.W., Australia

ABSTRACT: Time-dependent long-term deformation of EPS fill overlaying soft soil is of practical interest to engineers and designers. A model of EPS fill resting on consolidating soft soil presented in this paper can enable designers to assess the long-term performance of the EPS fill solution used for overcoming settlement problems on soft soils. The model considers two distinct materials: the viscoelastic material which is the EPS fill and the poro-viscoelastic material which is the soft soil. The deformations from the two materials are superimposed to obtain the total settlement with time.

1 INTRODUCTION

Expanded Polystyrene (EPS) geofoam is essentially used in ground fill applications where the lightweight material is required to reduce the stresses on underlying soils. The long-term behaviour of the foam has always been of practical interest to engineers and designers needing to make a critical assessment of its performance for the duration of its design life.

Long term behaviour of a material is influenced by various aspects of the material response under stress and strain loading and includes the time-dependent phenomena of creep and stress relaxation. In fill applications, it is the creep response in the geofoam that will be a major design consideration. The authors in the present paper collected data from their laboratory creep experiments so as to develop a viscoelastic model of the EPS geofoam which include creep effects. This is then combined with the time-dependent consolidation of soft soil to produce a model that is capable of simulating the long term settlement behaviour of a typical EPS fill application.

It is worth noting from the onset that EPS fill and the underlying soft soils have to be regarded quite differently since the earlier does not deform in the same manner of consolidating soil where excess pore water are being expelled under loading. Thus the EPS geofoam is treated as a viscoelastic material whereas the consolidating soil is considered a poro-viscoelastic material. An illustrative example is given showing the relative merits of using the EPS fill where the foundation soil is very soft.

2 DEVELOPMENT OF A MODEL

The model in the present paper considers the deformation of two material types: EPS fill and the underlying consolidating soft soil. A schematic of this is illustrated in Figure 1.

Figure 1. EPS fill on soft soil.

2.1 EPS viscoelastic creep models

With few exceptions, the fundamental equation for creep models has the following qualitative form (Findley, 1976):

\[ \varepsilon(t) = \varepsilon_0 + \varepsilon_c(t) \]  

where: \( \varepsilon(t) \) = the total strain at time \( t \) after the stress application; \( \varepsilon_0 \) = the instantaneous strain upon a stress application and \( \varepsilon_c(t) \) = the time-dependent creep component of strain at some time \( t \) after the stress application. For the small stress ranges considered in the present paper and which occur under
representative conditions in pavement and embankments, the instantaneous strain gives,

\[ \varepsilon_0 = \frac{\sigma_0}{E_i} \]  

(2)

where \( \sigma_0 \) is the applied stress, \( E_i \) is the linear spring constant (or the Young's modulus) of the geofoam.

If the stress history is piecewise differentiable, then the creep component of strain under any given stress history may be expressed as a convolution integral,

\[ \varepsilon_c(t) = \int_0^t J(t-\tau) \frac{\partial \sigma_c(\tau)}{\partial \tau} d\tau \]  

(3)

where the kernel \( J(t) \) is the compliance for the creep component of strain whose form depends on the model adopted. Equation (3) forms the basis of the linear creep in that the material "remembers" the stress history it has been subjected to.

Laboratory creep tests on EPS results in typically parabolic curves presented in Figure 3. The present authors have made an evaluation of the viscoelastic mechanical models to obtain a suitable compliance relationship that is considered applicable to EPS geofoam. Mechanical models of various forms are already widely used to model a number of material and has been described as being suitable for polystyrene material. These models are made up of combinations of (Hooks) springs and (Newtonian) dashpots as exemplified by the well known Maxwell and Kelvin models (Figure 2a, 2b).

Initially, a four-element or Burgers model (Figure 2c, where a Maxwell model and a Kelvin model are connected in series), was thought to be suitable for capturing the creep characteristics of EPS geofoam since it could model both primary and secondary creep. In a Burger's model, the strain response when a constant stress \( \sigma_0 \) is applied at \( t = 0 \) is,

\[ \varepsilon_c(t) = \frac{\sigma_0}{E_i} + \frac{\sigma_e}{\eta_t} + \frac{\sigma_k}{E_k} \left( 1 - e^{-\frac{t}{\eta_k}} \right) \]  

(4)

so that the compliance equation gives,

\[ J(t) = \frac{1}{\eta_t} + \frac{1}{E_k} \left( 1 - e^{-\frac{t}{\eta_k}} \right) \]  

(5)

It is observed that the strain behavior described in Equation (4) allows for an instantaneous strain as well as the strain rate to approach asymptotically the value of \( \sigma_0/\eta_t \) as \( t \) tends to infinity. The fact that this model captures both primary and secondary creep characteristics is useful, but as experimental data yields typically parabolic creep curves a modification was introduced to enable the creep strain rate asymptote to a parabola as \( t \) tends to infinity. The strain equation for the modified four-element model for a constant stress \( \sigma_0 \) is applied at \( t = 0 \) is,

\[ \varepsilon_c(t) = \frac{\sigma_0}{E_i} + \frac{\sigma_e}{\eta_t} + \frac{\sigma_k}{E_k} \left( 1 - e^{-\frac{t}{\eta_k}} \right) \]  

(6)

where \( n \) is a creep parameter and the compliance equation for creep gives,

\[ J(t) = \frac{1}{\eta_t} + \frac{1}{E_k} \left( 1 - e^{-\frac{t}{\eta_k}} \right) \]  

(7)

In Figure 3a where the experimental data and the model results of both the 4-element model and modified 4-element model are plotted, it quite clearly shows a superior goodness of fit for the later model. Similar results were also presented by Taylor et al. (1997), in applying mechanical models to flexural creep deflection of structural insulated panels.

The present research is done for a geofoam material supplied by an Australian firm, with a small stress range (0-40 kPa) in mind. This stress range has practical interest since it typically occurs in representative conditions in road sub-base and embankments where EPS fills are being used. Creep data from the laboratory creep tests were collated and normalised with respect to the applied stress i.e. \( \varepsilon/\sigma_0 \), then the creep parameters were determined using a Levenberg-Marquardt nonlinear least squares regression procedure. The parameters of the modified 4-element model for EPS foam of specified 20 kg/m² density adopted in the current paper are tabulated in Table 1.
Table 1. Parameter values of modified 4-element model for EPS

<table>
<thead>
<tr>
<th>$n$</th>
<th>$E_1$ (kPA)</th>
<th>$\eta_1$ (kPA·yr)</th>
<th>$E_2$ (kPA)</th>
<th>$\eta_2$ (kPA·yr)</th>
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<td>0.165</td>
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</table>

Figure 3b,c,d shows good fit for the strains produced at various stress levels using the modified 4-element model.

2.2 Soft Soil

In field applications, the EPS blocks rest on a soft soil foundation, commonly separated by a thin layer of sand used both for leveling and as a drainage layer (Figure 1). The consolidation of the soil with time effects can be described by the constitutive equation:

$$\varepsilon_C(t) = \frac{\sigma_C(t)}{E_1} + \int_0^t \frac{\partial \sigma_C(t')}{\partial t} \frac{dt'}{t}$$

where $\sigma_C$ = effective vertical stress. If a 4-element (Burgers) model had been use for the consolidating soil as is the case in the present paper then the constitutive equation gives,

$$\varepsilon = \frac{\sigma}{E_1} + \int_0^t \frac{\partial \sigma(t')}{\partial t} \frac{dt'}{t} + \int_0^t \frac{\partial \sigma(t')}{\partial t} e^{-t/t_2} \frac{dt'}{t}$$

where: $E_1, E_2, \eta_1, \eta_2$ = model parameters. The constitutive equation has to be solved in combination with the continuity equation,

$$\frac{\partial \varepsilon_c}{\partial t} + u \frac{\partial \varepsilon_c}{\partial x} + \gamma \frac{\partial \epsilon}{\partial x} = \frac{k_x}{\eta} \frac{\partial^2 u}{\partial x^2}$$

in which: $\varepsilon_c$ = volumetric strain; $k_x$ = coefficient of vertical permeability; $\gamma$ = unit weight of fluid; $u$ = excess pore pressure. Interested readers may refer to the companion paper (Leo and Xie, 2001) for a discussion on the details of the consolidation of a layered system.

3 SOLUTION OF EPS GEOFOAM ON SOFT SOILS

The solution of the Equation (3) containing a convolution integral is significantly facilitated by introduction of a Laplace transform defined as,

$$\tilde{\varepsilon}_c = \int_0^t \varepsilon_c(t') dt'$$

which from Equations (1),(2) and (3) gives,
where \( \sigma(0) = 0 \). It is noted here that in the present model \( \sigma(t) = g(t) \), the surcharge load which is simulated by a ramp loading representing the construction sequences from beginning until completion when a steady load is imposed, although other loading functions may also be applied. The compliance Equation (7) yields,

\[
\frac{1}{\tilde{E}} = \frac{1}{\tilde{E}_1} + \frac{1}{\tilde{E}_2} + \frac{1}{\tilde{E}_3} \left( \frac{1}{\eta_1} \right) \left( \frac{1}{s^{\eta_2}} + \frac{1}{s(s\eta_2 + E_2)} \right)
\]

where \( \Gamma \) is the Gamma function. The strain in real time is found by numerically inverting Equation (12) using Talbot’s (1979) inversion algorithm. The settlement arising from the deformation of the EPS layer is obtained from the product of the strain and the depth of EPS fill layer. This result is superimposed on to the settlement from the consolidating soft soil (Leo and Xie, 2001) to obtain the total settlement of the system.

4 ILLUSTRATIVE EXAMPLE

The present model is being developed as an analytical tool to assess the long-term settlement of EPS fill resting on very soft consolidating soil. The same model can also be used to gauge these results against the ‘no EPS’ case where conventional fill material are used and considerably higher self-weight would be imposed on the consolidating soil.

The illustrative case considered in this paper has a 5 m high 20 kg/m³ density EPS embankment situated above 5 m of very soft clay, this in turn overlies fairly firm stratum. The values of the model parameters for the EPS fill are provided in Table 1 while those for the very soft soil are presented in Table 2 below.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_1 ) (kPa)</td>
<td>1000</td>
</tr>
<tr>
<td>( \eta_1 ) (kPa-yr)</td>
<td>0.5×10⁶</td>
</tr>
<tr>
<td>( E_2 ) (kPa)</td>
<td>1000</td>
</tr>
<tr>
<td>( \eta_2 ) (kPa-yr)</td>
<td>25</td>
</tr>
<tr>
<td>( C_2 ) (m²/yr)</td>
<td>5</td>
</tr>
</tbody>
</table>

A pavement was constructed directly above the EPS fill during a 6 months period, which imposes an ultimate surcharge of 20 kN/m². The present model is applied to evaluate the expected surface settlement over the next 100 years. In the ‘no EPS’ case where a conventional fill material has been used in the construction of the embankment, it is assumed that this would cause an ultimate surcharge of 100 kN/m² (instead of the 20 kN/m²) based on a unit weight of 16 kN/m³ for the fill material. The expected settlement results for both these cases are plotted in Figure 4. The results indicate that the EPS fill performed quite effectively for the duration of the 100 years in that settlement is restricted to a value of about 0.25 m. In the conventional case, a total settlement of about 1.1 m would have resulted assuming no other soil improvement was done.

5 CONCLUSION

The present paper is concerned with the development of an EPS-soft soil model used to evaluate the long-term effectiveness of EPS fill solution for overcoming settlement problems in soft soil. The developed model is simple to use and can be extended to simulate other aspects of construction and geotechnical design considerations currently not provided for.

Figure 4. Settlement plot for use and non-use of EPS foam in illustrative example.

REFERENCES


RESILIENT MODULUS OF EPS MATERIAL USED AS A PAVEMENT SUBGRADE

Y. Zou¹, C. J. Leo¹ & E. S. Bernard¹

ABSTRACT

A rigid closed-cell cellular plastic foam, Expanded Polystyrene (EPS), has been used successfully as a subgrade replacement material for pavement construction, especially for poor quality subgrade. In flexible pavement structures, the mechanical properties of the asphalt concrete, the base course and the soil subgrade have a considerable influence on the response of the whole pavement to dynamic traffic loading.

The design procedures for flexible pavements presented in the AUSTROAD (APRG Report No.8, 1993) or AASHTO Guide for Design of Pavement structures (AASHTO Guide 1993) utilize the mechanical properties of the asphalt concrete, base course and soil subgrade. The standard method of test for resilient response accounts for the repetitive nature of traffic loading. The property that describes this behavior of materials is called the resilient modulus of elasticity, which is defined as the deviator dynamic stress (due to the moving vehicular traffic) divided by the resilient (recoverable) strain. The resilient modulus of unbound pavement materials is normally determined in the laboratory from cyclic triaxial tests.

However, because the confining pressure has very little effect on the behavior of EPS material (Zou and Leo, 1998, Horvath et al. 1995, Preber et al. 1994, and Eriksson and Trank, 1991), a series of cyclic unconfined ($\sigma_0 = 0$) compression tests of EPS specimens were therefore carried out instead. The test consists of applying and holding an axial static load to the specimen, then applying the cycles of loading and unloading of the specified vertical stress (defined as dynamic stress). Therefore, the deviator stress in this study equals to the dynamic stress.

In the present study, 6 EPS specimens were selected and tested under dynamic loading. Each specimen is tested at a different stress level for a number of load repetitions. Cylindrical specimens with 50 mm diameter and 50 mm height of 20 kg/m² were used for the test. The tests on the different static and dynamic stress levels on the deformation behavior, in particular the resilient response, of EPS have been carried out and the following conclusions are obtained:

1. In unconfined compression test, the resilient modulus of EPS reaches a stable value as the preconditioning completed. Above the preconditioning limit, the resilient modulus does not change significantly with an increase in the number of loading cycles.
2. In the stress ranges considered, both the dynamic and static stress levels have only a small effect on the resilient modulus of EPS. The reason is due to the stress levels being in the linear elastic range of the EPS materials.
3. The resilient behavior of EPS material is different to both granular material and fine-grained soils. The resilient modulus of EPS decreases with the increase of static stress, but increases slightly as the dynamic stress is increased. This means that the resilient behavior of EPS material can be described as elastic.
4. The number of loading cycles has little effect on the permanent strain under the stress ranges studied.
5. The permanent strain increases with the increase of static stress. On the contrary, the static stress level has very little influence on the resilient strain.

¹ University of Western Sydney Nepean, PO Box 10, Kingswood, NSW 2747, Australia
Technical Paper by Y. Zou, J.C. Small and C.J. Leo

BEHAVIOR OF EPS GEOFOAM AS FLEXIBLE PAVEMENT SUBGRADE MATERIAL IN MODEL TESTS

ABSTRACT: The behavior of expanded polystyrene (EPS) geofoam used as subgrade and fill material under flexible pavement was investigated by carrying out a series of tests in a model pavement testing facility. In the experimental setup, pavement test sections that consisted of a wearing course, a gravel base layer, and a sand subbase were placed on EPS blocks inside a test tank. Traffic loading on the test pavement was simulated using a loaded wheel running on an oval-shaped test track. The investigation studied the effects of repeated traffic loading on the performance of the EPS geofoam, the influence of the EPS block size, and the presence or lack of lateral restraint. For comparison purposes, a pavement section using sand fill instead of EPS geofoam was also constructed and tested and is considered as the baseline case. The performance of the EPS geofoam was benchmarked against the baseline case to analyze its performance and to determine possible problems that may occur when used in full-scale pavement sections. Implications for pavement design are also discussed.

KEYWORDS: Expanded polystyrene, Geofoam, Pavement, Model testing, Subgrade.

AUTHORS: Y. Zou, Researcher, and C.J. Leo, Senior Lecturer, School of Civil Engineering and Environment, University of Western Sydney, Nepean, NSW 2747, Australia, Telephone: 61/247-360-132; Telefax: 61/247-360-132, E-mail: y.zou@eng.nepean.uws.edu.au and c.leo@uws.edu.au, respectively; and J.C. Small, Associate Professor, Department of Civil Engineering, University of Sydney, NSW 2006, Australia, Telephone: 61/2-93512128; Telefax: 61/2-93513343, E-mail: j.small@civil.su.oz.au.


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1 INTRODUCTION

The problems of excessive long-term settlement and low bearing capacity associated with highly compressible and weak soils in geotechnical design of pavements and embankments are well known and documented in the literature (e.g. Hui and Kong 1995, Brand and Renner 1981, and Charles 1993). Engineers use a variety of techniques to solve these problems; a commonly used method is to replace the heavier soils with lightweight fill materials so as to reduce the weight of the construction materials. A recently used ultra lightweight, synthetic fill material is expanded polystyrene (EPS) geofoam.

Several laboratory studies (e.g. Eriksson and Trank 1991, Horvath 1995, Preber et al. 1994, and Zou and Leo 1998) have shown that EPS geofoam can be used successfully in geotechnical applications including pavements and embankments. Typically, the basic material behavior is investigated by carrying out the appropriate standard laboratory tests under controlled conditions and then extrapolating the results to field applications. There is a limit, however, to the use of laboratory results for the interpretation of field performance because standard laboratory tests cannot fully replicate actual field conditions. It has been shown in similar investigations (Sparks 1970; Taylor 1966) that some of the limitations of standard laboratory tests can be overcome using physical models, thereby providing better insight into material performance under more realistic conditions.

In order to study the behavior of EPS geofoam as a replacement subgrade material under a flexible pavement, the authors of the current paper performed model tests simulating the application of repeated vehicular loading in realistic yet controlled conditions. The current study follows earlier successful investigations on the settlement problems of bridge approaches (Wong and Small 1994) and the effectiveness of geosynthetic-reinforced pavement (Moghaddas-Nejad and Small 1996) carried out using the same test facility. For the purpose of the current investigation, EPS geofoam fill was used below the model pavement instead of conventional soil. The test facility instrumentation is capable of accurately measuring the displacement of the pavement and the EPS blocks when subjected to repeated vehicular loading simulating live traffic.

In the series of model pavement tests, the influence of the EPS block size and the effects of lateral restraint on EPS blocks were studied. Block size and lateral restraint are issues directly affecting EPS geofoam design in pavements and embankments. In addition, the performance of EPS geofoam blocks and conventional sand (the baseline case) as subgrade materials are compared and assessed.

2 OVERVIEW OF TESTING FACILITY

The model test facility used in the current series of tests was first developed by Wong and Small (1994) at a one-quarter scale to study the effect of the orientation of approach slabs on pavement deformation. The facility is described in detail by Wong (1991), however, a brief overview is given herein.

There are four main structural components in the test facility, namely, the overhead track, the test tank, the running track, and the loading carriage. A loading wheel is driven around an oval-shaped track by an overhead guided rail system as shown in Figure 1. The oval-shaped track is chosen because it offers some advantages over both the lin-
ear and circular tracks - typically, a linear track requires an elaborate mechanism to control the loading wheel so that it can: (i) accelerate from the initial starting position to the desired speed; (ii) pass over the test pavement at a constant speed; (iii) decelerate and stop at the end of travel; and (iv) be lifted to the initial starting position to commence the next pass (Sparks 1970; Brown and Brodick 1981). The disadvantage of a circular track is that it requires a large area and that the wheel does not follow a straight path across the pavement. The oval-shaped track at Sydney University by contrast has a relatively simple mechanism, is compact, and allows the use of a straight pavement test section.

The model pavement test section was constructed inside the test tank and positioned below one of the straight sections of the overhead track. The test tank consists of two hollow steel boxes bolted together on the flanges and has internal dimensions of 1.4 m (length) × 0.5 m (width) × 0.8 m (depth). Several linear variable displacement transducers (LVDTs) were installed to measure subsurface pavement and subgrade displacement. A conductor rail system supplies power to the motor, which drives the test wheel. The wheel passes over the pavement test section once during each revolution around the running track and triggers a microswitch that starts the recording of data by a computer. Wheel load data from the loading carriage can also be sent back to the computer.

The loading carriage consists of two bogies, a spring loading system, a driven pneumatic tire, and a driving unit utilizing timing belts and grooved pulleys (Figure 2). The tire is supported by the rotating arm, which is pivoted at the top by means of roller bearings. The loading unit comprises four identical springs containing concentrically placed guide rods. The spring loading system is connected to the bottom bogie plate by means of a hinged connection. The compression in the springs can be changed by adjusting nuts above and below the rotating arm. A LVDT fitted between two reaction
plates is used to monitor the variation of spring compression. The top of the loading system is connected to the bottom bogie plate of the trolley; therefore, as the trolley runs along the test track, the wheel load is influenced by how level the track is.

The test facility is situated in a temperature-controlled room, where the temperature was maintained at approximately 22°C to minimize thermal effects that may have influenced the properties of the pavement and subgrade materials.

The maximum wheel load was limited to 1.4 kN and the maximum speed to 7.2 km/h for safety reasons.

Figure 2. Profile view of the loading carriage.
3 EXPERIMENTAL PROCEDURE

3.1 Pavement and Subgrade Material

The model pavement section consisted of a wearing course, a road base, and a sand subbase, all of which overlaid the subgrade (which in this series of tests was either compacted sand or EPS geofoam blocks) and an uncompact sand layer used for leveling at the bottom of the test tank. A typical model pavement test section is presented in Figure 3.

Wearing Course. The wearing course used in road pavement varies widely, but typically the maximum stone size is approximately 20 to 25 mm. Thus, a proprietary bituminous mix (known commercially as Pavefix) with a bitumen content of approximately 5% and a maximum aggregate size of 5 mm was chosen for the wearing course, based on the one-quarter scale of the models. Although the mix could have been compacted at ambient temperature, as claimed by the manufacturer, it was heated for approximately four hours at 150°C before compaction because this practice resulted in superior performance as shown in the test results.

Road Base. The base layer comprised crushed aggregate (basalt) with a nominal size of 5 mm and mainly subangular particles (coefficient of uniformity, Cu = 2; diameters corresponding to 10 and 60% by weight of finer particles, D10 = 2 mm and D60 = 4 mm, respectively). It is classified as a uniform fine gravel with less than 1% of particles finer than 75 μm. The particle size was chosen so that it was roughly scaled in accordance to the model dimensions. The minimum and maximum unit weight values of the base material were 13.5 and 14.6 kN/m³, respectively.

Subbase and Subgrade. Two types of subgrade were used in this series of testing. For the base case, Sydney sand, which is a silica sand, was used as the conventional subgrade (Cₜ = 1.5, D₁₀ ≈ 0.25 mm and D₆₀ ≈ 0.37 mm). Sydney sand is classified as a well-graded sand with less than 1% fines. The minimum unit weight was 14.1 kN/m³, while the average maximum unit weight was 16.6 kN/m³. The same sand was also used for the compacted sand subbase.

The second subgrade material used was EPS geofoam having a density 21 kg/m³; the density most commonly used in geotechnical filling applications. The compressive strength (at 10% compressive strain) of this grade of EPS foam was approximately 90 kPa (cylindrical specimens with a diameter of 50 mm and height of 50 mm were compressed at a constant strain rate of 1%) and the yield stress was 80 kPa (Zou and Leo 1998).

Compaction Methods. Due to the strong influence of the degree of compaction on the granular material deformation characteristics, the compaction procedure must be chosen so as to give uniform layers with similar density and particle structure. To obtain a reproducible test result, close control of compaction processes should be exercised.

A manual compaction method was selected for this series of tests. In order to ensure uniformity and repeatability, filling was completed in seven sections in any single layer as described by Wong (1991). The number of blows for all sections was specified to be 20 and the fall height for the compacting tool was 50 mm at each section of a layer. After
compaction, the average measured unit weights of the wearing course, base layer, and sand subgrade were 17.1, 14.7, and 16.2 kN/m³, respectively.

Strength and Deformation Parameters of Pavement and Subgrade Material. The strengths of pavement materials are varied and influenced by many factors such as thickness, compaction efforts, moisture content, load level, temperature, and methods
of sample preparation. It is worthwhile measuring the strength parameters of materials under the same conditions as they are tested. For this reason, standard California Bearing Ratio (CBR) tests were carried out on the crushed aggregate and compacted sand, while cone penetration tests were conducted on the wearing course, crushed aggregate, and compacted sand, as well as the EPS blocks, using a hand-held penetrometer. For the EPS geofoam, with a density of 21 kg/m³, unconfined cyclic compression tests were also carried out on cylindrical specimens (diameter = 50 mm, height = 50 mm) to measure the resilient modulus, \( M_r \). The strength parameters of the pavement and subgrade materials are shown in Table 1.

### 3.2 Wheel Load

The pneumatic tire selected for the test facility has a diameter of 230 mm, which is approximately one quarter the size of a full-scale truck tire. For the current series of tests, the applied wheel load ranged from 490 to 690 N. The tire pressure was maintained at 210 kPa. As described in Section 2, the wheel load is provided by four springs in compression; the magnitude of the load depends on the spring compression and stiffness. Based on the calibration data reported by Wong (1991), the relationship between wheel load and spring compression, and wheel load and contact area are, respectively:

\[
F = 26.9E + 4,192 \quad (1)
\]

\[
A = 0.027F + 14.433 \quad (2)
\]

where: \( F \) = wheel load (N); \( E \) = compression of the springs (mm); and \( A \) = contact area of the tire at a pressure of 210 kPa (cm²). Equation 1 is used to calculate the wheel load, which is then divided by the contact area in Equation 2 to give the contact stress.

The loading unit cannot maintain a constant wheel load during a test. As the deformation of the pavement is increased, it will release some of the spring compression in the loading system, thereby causing the wheel load to decrease. A 1 mm displacement in the pavement surface will result in a wheel load decrease of approximately 13.5 N, which translates to a contact stress of approximately 2.0 kPa for the applied loading in the current experiment. During testing, the wheel load was regularly checked and, if required, the spring compression was adjusted so as to maintain the desired wheel load.

### Table 1. Strength and deformation parameters of the pavement materials used in the model tests.

<table>
<thead>
<tr>
<th>Material</th>
<th>Resilient modulus, ( M_r ) (Pa)</th>
<th>Cone penetrometer resistance (MN/m²)</th>
<th>CBR (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wearing course</td>
<td>N/A</td>
<td>7.6</td>
<td></td>
</tr>
<tr>
<td>Crushed aggregate</td>
<td>( 140 \times 10^6 ) *</td>
<td>3.8</td>
<td>14</td>
</tr>
<tr>
<td>Compacted sand</td>
<td>( 50 \times 10^6 ) *</td>
<td>1.0</td>
<td>5</td>
</tr>
<tr>
<td>EPS</td>
<td>( 5 - 6 \times 10^6 )</td>
<td>0.5</td>
<td>1.5 to 2.0</td>
</tr>
</tbody>
</table>

Notes: *The resilient modulus was estimated using the following relationship: \( M_r (\times 10^6 \text{ Pa}) = 10 \times \text{ CBR value} \) (Austroads 1992). N/A = not available. \( \rho_{EPS} = 21 \text{ kg/m}^3 \).
The speed of the wheel circling the test track was set to 2.7 km/h or approximately 11 km/h at prototype scale. This resulted in a loading frequency of approximately 241 passes per hour for the current series of tests.

3.3 Test Program

The current test series consisted of five tests using an EPS geofoam subgrade and one test using a sand subgrade (baseline case). Table 2 summarizes the parameters used for this test series. Tests 1, 2, and 3 used “large” blocks (the definition of “large” block is given in Section 3.4); all had an identical experimental setup except for the magnitude of the wheel load. All of the tests were run to gather information on pavement and EPS geofoam response when subjected to increased wheel loads. In Test 4, no lateral restraint was provided for the EPS blocks. As a result, an approximate 10 mm-wide gap was left between the EPS blocks and the side walls of the test tank. Test 5 examined the effects of block size by using “small” blocks (the definition of “small” is in Section 3.4) instead of the standard large blocks. Finally, for the purposes of a comparative study, Test 6 (the baseline case) used conventional compacted sand instead of an EPS geofoam subgrade.

3.4 Pavement Thickness and EPS Blocks

Pavement Thickness. The pavement layers were made thick enough to diffuse the imposed loading so that the combination of pavement live and dead loads did not stress the EPS geofoam beyond a given stress-strain level (typically below its elastic limit as shown in Figure 4). This ensured that unacceptable deflections and permanent deformations of the pavement would be avoided. At full scale, typical thicknesses of a pavement wearing course, base layer, and sand subbase are 80, 200, and 200 mm, respectively. Thus, the corresponding model thicknesses of these layers at the one-quarter scale, i.e. the model scale used in the current study, were 20, 50, and 50 mm, respectively.

Table 2. Summary of the test parameters.

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Subgrade</th>
<th>Laterally restrained</th>
<th>Maximum wheel load (N)</th>
<th>Contact stress* (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EPS (large blocks)</td>
<td>Yes</td>
<td>690</td>
<td>$209 \times 10^3$</td>
</tr>
<tr>
<td>2</td>
<td>EPS (large blocks)</td>
<td>Yes</td>
<td>590</td>
<td>$196 \times 10^3$</td>
</tr>
<tr>
<td>3</td>
<td>EPS (large blocks)</td>
<td>Yes</td>
<td>490</td>
<td>$179 \times 10^3$</td>
</tr>
<tr>
<td>4</td>
<td>EPS (large blocks)</td>
<td>No</td>
<td>590</td>
<td>$196 \times 10^3$</td>
</tr>
<tr>
<td>5</td>
<td>EPS (small blocks)</td>
<td>Yes</td>
<td>590</td>
<td>$196 \times 10^3$</td>
</tr>
<tr>
<td>6</td>
<td>Sand (baseline case)</td>
<td>Yes</td>
<td>540</td>
<td>$188 \times 10^3$</td>
</tr>
</tbody>
</table>

Note: *Tire pressure = 210 $\times 10^3$ Pa for all tests.
Figure 4. Typical EPS geofoam stress-strain curve obtained from a compression test ($\rho_{EPS} = 21$ kg/m$^3$)

**EPS Geofoam Blocks.** The block sizes in actual applications vary over a range of dimensions. Based on the size of the test tank and the possible block sizes in the field, the size of the EPS geofoam chosen for the model was 500 mm (length) × 160 mm (width) × 100 mm (depth) (this is referred to as the "large" block in the current test series). Each is a rectangular parallelepiped block that was carefully cut using a hot wire. Since a one-quarter scale was used, a large block represents a 2,000 mm × 480 mm × 400 mm prototype EPS geofoam block.

The small blocks, having half the length of the large blocks, i.e. measuring 250 mm × 160 mm × 100 mm, were used in Test 5 to study the potential effects of block size. It should be noted that a small block arrangement results in more joints and block-to-block interfaces than that for the large blocks.

In all of the experiments involving EPS geofoam, four layers of geofoam, each 100 mm-thick, were arranged inside the test tank. Each layer was placed horizontally and in a pattern that minimized the vertical continuity of joints between blocks. The longest dimension of the blocks in one layer was aligned orthogonal to the longest dimension of the blocks in the adjacent layers both above and below. This arrangement is illustrated in Figure 5.

### 3.5 Measurement of Surface and Subsurface Deformation

The displacement measurements were taken at two sections in the test tank using LVDTs. The plan and section views showing the positions of the LVDTs are given in Figure 3. Three LVDTs were placed in each section (LVDTs 1, 2, and 3 in Section 1; LVDTs 4, 5, and 6 in Section 2) along the longitudinal centerline of the pavement at three elevations, namely the wearing course-base interface, subbase-EPS block interface, and in
Figure 5. The arrangement of EPS geofoam blocks in a typical model pavement test section.

the middle of the EPS blocks. Also, two additional LVDTs were installed in each section (LVDTs 7 and 8 in Section 1; LVDTs 9 and 10 in Section 2) at the subbase-EPS block interface, beside the center transducers (LVDTs 2 and 5). In Test 6, LVDTs were placed at the same positions and elevations as the EPS geofoam. This LVDT setup was capable of measuring displacements to an accuracy of ±0.01 mm.

Several drawing pins were pushed into the surface of the pavement directly above the LVDTs, along a line perpendicular to the travel direction of the wheel, to act as settlement points. Displacement readings taken at the settlement points allowed construction of the surface displacement profile at the same cross section as the LVDTs. The surface measurements were made using a micrometer for measuring deflections to an accuracy of ±0.01 mm and with the help of a straight edge placed across the sides of the test tank as reference.

As the driving wheel passed over the test tank, the subsurface displacements were recorded every second pass for the first 20 passes, then every fifth pass up to 100 passes, every tenth pass up to 200 passes, every fiftieth pass between 200 to 1,000, and every hundredth pass after 1,000 passes. The surface displacement was measured at 5, 10, 20, 50, 100, 200, 500, and 1,000 passes, and after that, at every thousandth pass. A typical displacement curve for the first three passes is shown in Figure 6 for LVDT 2 at the top of subgrade. The maximum displacement was achieved as the driving wheel passed directly above the LVDT and then decreased as the wheel load was released. For the pavement system as a whole, some permanent displacement was clearly recorded with each pass of repeated loading. There is a small displacement gap between the end of the first pass and the beginning of the second pass (Figure 6), indicating a slow recovery between the two passes. In the current paper, the maximum displacement of each pass is
Figure 6. Typical amount of displacement at the top of the subgrade after the first three passes.

Note: Measurements obtained from LVDT 2 at the top of the subgrade under the centerline (Section 1).

defined as the maximum instantaneous displacement, the initial displacement of the $n$th pass is taken as the cumulative permanent (plastic) displacement of the $(n-1)$th pass, and the resilient displacement is the recoverable amount of the total deformation (Figure 6). The typical maximum and cumulative permanent displacement curves are shown in Figure 7. The largest incremental plastic deformation occurs in the first few hundred passes of the driving wheel, after which the rate of increase is negligible.

4 TEST RESULTS

4.1 Deformation of Subgrade

The results for subgrade deformation have been categorized into permanent (plastic) and resilient (recoverable) displacements as measured by LVDTs 1 and 4, which were installed in the middle of the subgrade at a depth of 320 mm.

Figure 8 shows typical cumulative permanent displacements at the EPS geofoam subgrade surface under the center of the wheel load for Tests 1, 2, and 3 at maximum wheel loads of 690, 590, and 490 N, respectively. For the same wheel load, cumulative permanent deformation increases rapidly in the first approximately 500 passes and
Figure 7. Typical maximum and cumulative permanent displacement curves for Test 3.

Figure 8. Cumulative permanent displacement at the subgrade surface under the center of the wheel load.
more gradually after that. Differences in deformation readings were recorded at Sections 1 and 2 even though these were two identical sections in the test tank. It was therefore decided that, for the sake of consistency, comparisons of test results should be made only for the same test section.

Figure 9a shows that increasing the wheel load from 590 to 690 N results in significantly higher cumulative permanent deformation than for the same wheel load increment from 490 to 590 N. Therefore, permanent (plastic) displacement appears to

![Graph (a)](image)

**Figure 9.** Cumulative deformation at the top surface of the EPS geofoam subgrade after 5,000 wheel passes: (a) permanent; (b) resilient.
increase (in a highly nonlinear fashion) as the magnitude of the wheel load increases even though the stress levels in the EPS geofoam are fairly small, falling within the linear range of the stress-strain relationships. The previous remark is based on finite element analyses, which revealed that the maximum stress at the top of the blocks for the imposed loading in this series of tests (estimated to range between 10 to 15 kPa) is well below the elastic limit.

In Figure 10, the cumulative permanent displacements at the top surface of the EPS geofoam layer after 5,000 passes for Tests 2, 4, and 5 are shown (maximum wheel load = 590 N). It may be recalled that large blocks were used in Test 2, large blocks with no lateral restraint were used in Test 4, and small blocks were used in Test 5. After the first 1,000 passes, the deformation curve flattens becoming fairly uniform and displaying a small rate of increase per pass.

It is worth noting that the length of a large block (500 mm) is twice the length of a small block (250 mm), which results in a block-block interface area for Test 5 that is approximately 1.4 times that of Test 2. The test data provide no evidence to suggest that there was any significant change in deformation behavior of the EPS blocks when the block length was halved.

Data from Tests 2 and 5 also suggest that the presence or lack of lateral confinement of the EPS blocks did not result in any significantly different behavior. On the contrary, a higher deformation was measured in Test 2 (lateral restraint on the blocks) when compared to Test 4 (no lateral restraint). This behavior is vastly different in comparison to granular material, which derives much of its shear strength and stiffness from the presence of lateral confinement.

Figure 10. Cumulative permanent displacement at the top surface of the EPS geofoam subgrade.
Also shown in Figure 8 are the results for Test 6 representing the baseline case (maximum wheel load = 540 N) where the subgrade comprised compacted sand instead of EPS geofoam. For sand, the increment of the cumulative permanent deformation is more gradual over the first 3,000 passes, but also flattens out to a more gradual rate of increase. After 5,000 passes, the deformation reached approximately the same magnitude, if not marginally higher than the EPS geofoam cases. Tests 2 and 3, which had wheel loads of 590 and 490 N, respectively, provided the lower and upper loading bounds for comparisons with the baseline case.

Unlike the permanent deformation, when resilient deformation after 5,000 passes is plotted against the wheel load in Figure 9b (for Tests 1, 2, and 3), the resulting curve is approximately linear because the stress levels are small and fall within the initial linear section of the stress-strain curve.

The data for the resilient deformation per pass are shown in Figure 11. For Tests 2, 3, 4, and 5, the curve flattens after approximately 1,000 passes, while, for Test 6, this only occurs after approximately 3,000 passes. While the magnitude of the cumulative permanent deformation, particularly after 5,000 passes, did not differ significantly between EPS blocks and compacted sand for the same approximate wheel load, the same did not apply for resilient deformation. The results show that after 5,000 passes, the magnitude of the resilient deformation for the baseline case is approximately 25 μm, while for the EPS cases it is between 65 and 110 μm or approximately 2.5 to 4.5 times greater.

**Figure 11.** Resilient deformation in the subgrade at a depth of 120 mm.
The breakdown of the total deformation (i.e. the previously defined maximum instantaneous deformation) during the load cycle into its component permanent and resilient parts after 5,000 passes is presented in Table 3. The experimental evidence suggests that, at the subgrade level, the EPS geofoam performs just as well as compacted sand in terms of the permanent deformation, but fared worse for resilient deformation during a load cycle. It is quite clear that resilient deformation dominates over the permanent deformation during a load cycle, for readings up to 5,000 passes, although the cumulative permanent deformation is much higher than the resilient deformation.

The effects of resilient deformation on surface deformation are discussed further in Section 5.2.

4.2 Deformation of Pavement Surface

Figure 12 shows cross-section views of the cumulative permanent displacements of the pavement surface after 5,000 passes for the current series of tests. A rut developed at the center of the pavement where the wheel load passed, whereas heaving occurred on both sides of the wheel path. The rutting depth is the smallest for Test 6 (baseline case) and the largest for Test 1 (the greatest wheel load, 690 N, of all of the tests was used). A permanent deformation profile similar to the one at the pavement surface was observed for the top surface of the EPS blocks (i.e. at the subbase-EPS interface), but at significantly reduced amplitude. These results may seem paradoxical when it is observed that, at the subgrade level (see Figure 10), there is little difference between the recorded cumulative permanent deformation at 5,000 passes for sand (Test 6) and the EPS geofoam at an approximately similar wheel load (Tests 2, 3, 4, and 5).

Figure 13 presents the curves for cumulative permanent deformation at the pavement surface and at the top of the subgrade for both the compacted sand (Test 6) and EPS geofoam (Tests 2, 4, and 5). After the initial rapid deformation increment, the curve flattens to a gradual rate of deformation increase for both the pavement surface as well as the subgrade. It clearly shows that most of the deformation is taking place in the pavement layers. Figure 13 again suggests that, while the permanent deformation of EPS and compacted sand are not significantly different at the subgrade level, the difference is however significantly accentuated on the pavement surface. Compacted sand clearly performs better than EPS geofoam in relation to pavement surface deformation. This is discussed further in Section 5.

Table 3. Cumulative permanent and resilient displacement after 5,000 passes.

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Subgrade</th>
<th>Cumulative permanent displacement (mm)</th>
<th>Resilient displacement (mm)</th>
<th>Maximum wheel load (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EPS (large blocks)</td>
<td>0.66</td>
<td>0.16</td>
<td>690</td>
</tr>
<tr>
<td>2</td>
<td>EPS (large blocks)</td>
<td>0.31</td>
<td>0.11</td>
<td>590</td>
</tr>
<tr>
<td>3</td>
<td>EPS (large blocks)</td>
<td>0.27</td>
<td>0.07</td>
<td>490</td>
</tr>
<tr>
<td>4</td>
<td>EPS (large blocks)</td>
<td>0.30</td>
<td>0.10</td>
<td>590</td>
</tr>
<tr>
<td>5</td>
<td>EPS (small blocks)</td>
<td>0.29</td>
<td>0.12</td>
<td>590</td>
</tr>
<tr>
<td>6</td>
<td>Sand (baseline case)</td>
<td>0.31</td>
<td>0.03</td>
<td>540</td>
</tr>
</tbody>
</table>
Figure 12. Deformation of the pavement surface after 5,000 wheel passes.

Figure 13. Cumulative permanent displacement at the pavement surface and at the compacted sand and EPS geofoam subgrade top surfaces.
5 DISCUSSION

5.1 Extrapolation of Results (Permanent Deformation)

It is observed (Figures 8 and 10) that the cumulative permanent deformation versus pass number curve after the first 1,000 passes for EPS geofoam is not only very gradual but also approximately linear. Based on this observation, permanent deformation data from Tests 2, 4, and 5 were plotted on a common plot after excluding the first 1,000 passes. The data are shown in Figure 14 where, using linear regression analysis, the relationship between the cumulative permanent deformation and cumulative number of load cycles (passes), $N$, was established as:

$$d_{EPS} = 6.05 \ (N - 1,000) \quad N \geq 1,000$$  \hspace{1cm} (3)

where: $d_{EPS}$ = cumulative permanent deformation after 1,000 load cycles, or passes ($\mu$m); and $N$ = cumulative number of load cycles (passes) in kilocycles. The correlation coefficient of the linear regression is 0.97 and the standard error is 0.002. Equation 3 yields a permanent deformation rate of approximately 6.05 $\mu$m per kilicycle. The relationship between the cumulative permanent deformation and cumulative number of load passes after 3,000 passes (Figure 15) for compacted sand was established as:

$$d_{sand} = 7.36 \ (N - 3,000) \quad N \geq 3,000$$  \hspace{1cm} (4)

![Graph showing cumulative permanent deformation at the top of the EPS blocks at a depth of 120 mm.](image)

**Figure 14.** Cumulative permanent deformation at the top of the EPS blocks at a depth of 120 mm.
where: $d_{\text{pass}}$ = cumulative permanent deformation after 3,000 load cycles, or passes (µm); and $N$ = cumulative number of load cycles (passes) in kilocycles. The cumulative permanent deformation rate is marginally higher than that for EPS geofoam. This is another clear indication that, at the subgrade level, EPS geofoam did not incur more permanent deformation than the compacted sand. Extrapolating to a hypothetical $1 \times 10^6$ passes yields a deformation of 6.05 mm for EPS geofoam and 7.36 mm for compacted sand. At full scale, these represent cumulative permanent deflections of approximately 24 and 29 mm, respectively, which normally would be considered as acceptable. As discussed in Section 5.2, within each load cycle or pass, it is the resilient rather than the permanent component of the deformation of the subgrade that appears to govern the rutting deformation at the pavement surface.

5.2 Resilient Deformation and Pavement Rutting

It is generally thought that pavement rutting is related to the relative stiffness of the pavement material and its subgrade. It should be noted that the pavement structures for all of the tests were constructed identically and, therefore, had identical stiffness. Based on the experimental data, any difference in pavement performance and behavior among the tests could only have been a result of the wheel load and/or subgrade (Table 2) after ruling out the effects of lateral restraint. As well, any permanent deformation at the subgrade would have propagated upward and been manifested as a part of the surface rut. However, the experimental data show that the cumulative permanent deformation in
either the EPS geofoam or compacted sand at the subbase-subgrade interface is significantly small (approximately 0.3 mm after 5,000 passes) compared to the surface rut depth (15 to 20 and 8 mm, respectively). Therefore, while permanent subgrade deformation can account for a small amount of the rut depth, it cannot clearly account for all of the rutting that occurred. This raises the question: "If the permanent deformations of the compacted sand and the EPS geofoam are approximately the same at the subgrade level, why then is the difference magnified at the pavement surface (EPS geofoam to compacted sand rut depth ratio ranges from 2 to 3)?"

The answer could possibly be found by observing the deformation within each cycle or pass. After the initial passes, the permanent deformation per pass becomes constant at approximately $6.05 \times 10^{-6}$ mm for EPS, and $7.36 \times 10^{-6}$ mm for compacted sand (from Equations 3 and 4, respectively), whereas the resilient deformations are 65 to 110 $\times 10^{-3}$ and 25 $\times 10^{-3}$ mm, respectively. Apart from the fact that resilient deformation dominates over permanent deformation for each pass, resilient deformation is also approximately 2.5 to 4.5 times higher for the EPS geofoam than for the compacted sand as mentioned in Section 4.1. It is suggested that, for the current series of tests, the large difference in resilient deformation at the subgrade is ultimately manifested as a large difference in deformation at the pavement surface. EPS geofoam is a soft material (as the strength and estimated modulus values in Table 1 demonstrate), particularly when compared to compacted sand, indicating that the results could have been predicted.

5.3 Implications for Pavement Design

The EPS geofoam block size and lateral restraint did not appear to significantly influence pavement performance; the latter is in general agreement with previous laboratory studies indicating that there is little dependency between lateral confining pressure and the EPS geofoam strength and stiffness (e.g. Preber et al. 1994, and Zou and Leo 1998). Therefore, the results of the current study present engineers with added flexibility in design.

The model tests demonstrate that the resilient modulus of EPS geofoam is the most important factor influencing pavement deformation at the surface. Correlation to the resilient modulus of EPS geofoam or its equivalent would be useful for pavement design.

It is important not to lose sight of the fact that EPS geofoam is typically used to address the problems of soft soil because of its low compressibility and bearing capacity. Also, the low resilient modulus of EPS geofoam can easily be overcome by designing a suitable pavement structure, i.e. by increasing the pavement thickness or using stiffer EPS geofoam.

6 CONCLUSIONS

The behavior of an EPS geofoam subgrade under pavement subjected to repeated wheel loads in a model testing facility was investigated and resulted in the following conclusions:

• In terms of permanent (plastic) deformation at the subgrade level, EPS geofoam performs as well as, if not better than, compacted sand under repeated traffic loading.
• Plastic deformation occurs even at small stress levels. The magnitude of cumulative permanent deformation increases nonlinearly with loading.

• For an EPS geofoam subgrade (and a compacted sand subgrade), the resilient deformation (or, by extension, the resilient modulus) will have a significant influence on the rut depth of the pavement surface.

• Resilient deformation of EPS geofoam at the subgrade level is much higher than that for compacted sand. The resilient deformation manifests as a deeper rut on the pavement surface of the EPS geofoam subgrade test section than on the compacted sand subgrade test section, even when both have the same pavement structure. The rut depth could be reduced, however, by using an appropriately designed pavement structure (e.g. increase the pavement thickness or use stiffer pavement).

• Test results showed that block size and lateral restraint did not significantly affect the performance of the EPS geofoam blocks. These findings will give design engineers added flexibility in design.

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NOTATIONS

Basic SI units are given in parentheses.

\[ A = \text{contact area of the tire (m}^2\text{)} \]
\[ D_{10}, D_{60} = \text{particle diameter corresponding to 10 and 60\% by weight of finer particles, respectively (m)} \]
\[ d_{\text{EPS}} = \text{cumulative permanent deformation of EPS after 1,000 passes (m)} \]
\[ d_{\text{sand}} = \text{cumulative permanent deformation of sand after 3,000 passes (m)} \]
\[ E = \text{compression of springs (m)} \]
\[ F = \text{wheel load (N)} \]
\[ M_r = \text{resilient modulus (Pa)} \]
\[ N = \text{cumulative number of load cycles (passes)} \times 10^3 \text{cycles (dimensionless)} \]
\[ \rho_{\text{EPS}} = \text{density of EPS geofoam (kg/m}^3\text{)} \]
Mechanics of Structures and Materials

Edited by

Mark A. Bradford
School of Civil and Environmental Engineering, The University of New South Wales, Sydney, N.S.W., Australia

Russell Q. Bridge
School of Civic Engineering and Environment, University of Western Sydney, Nepean, N.S.W., Australia

Stephen J. Foster
School of Civil and Environmental Engineering, The University of New South Wales, Sydney, N.S.W., Australia
The behavior of expanded polystyrene (EPS) under repeated loading

Y. Zou, C. I. Leo & E. S. Bernard
School of Civil Engineering and Environment, University of Western Sydney, Nepean, N.S.W.,
Australia

ABSTRACT: EPS geofoam is an ultra-lightweight filling material frequently used in building road embankments and as a load-bearing subgrade layer on very soft soils. In these and other geotechnical applications, the EPS geofoam is subjected to repeated loading arising from the motion of vehicles. The response of EPS geofoam to cyclic loading has been studied in a series of unconfined cyclic loading tests at various stress levels. Useful relationships describing the number of loading cycles to cause failure for a given peak stress, and the permanent strain accumulation with number of load repetitions, have been established.

1 INTRODUCTION

EPS (Expanded Polystyrene) is a rigid closed-cell cellular plastic commonly made from expandable polystyrene bead. It is very lightweight and compressible, and exhibits good insulation properties. As a ultra-lightweight material, EPS geofoam is an excellent fill for road embankments and retaining structures located on soft soils. The use of EPS geofoam as a replacement material for poor quality subgrade in road construction is gaining acceptance. Current engineering design methods for geotechnical application of EPS geofoam remains based on empiricism and experience, even though the material has been used successfully for nearly 30 years in Europe, North America and Japan. EPS materials exhibit a visco-plastic behavior and therefore suffer from permanent deformation under repeated loading, even at small stress levels. The long-term performance and durability of EPS remain as technical barriers to its wider acceptance and use in geotechnical applications. In this study, a series of unconfined cyclic loading tests at various peak stress levels have been carried out and the response of EPS geofoam to cyclic loading is studied. The effects of stress level and number of load repetitions on EPS material are investigated. Two relationships have been established: (1) permanent strain accumulation with the number of load repetitions; and (2) the number of cycles to cause failure, N, for various cyclic stress level, S (the classical S-N relationship of Wohler (in Timoshenko, 1953)).

2 LABORATORY TESTS ON THE BEHAVIOUR OF EPS

A number of studies have recently investigated the mechanical behavior of EPS, including triaxial compression tests, unconfined/confined compression tests, hydrostatic compression testing, shear tests, creep tests and cyclic unconfined compression tests (Hamada and Yamanouchi, 1987, Eriksson and Tranl, 1991, Preber et al., 1994, Horvath, 1995, Zou and Leo, 1998). Field monitoring of existing EPS sites (Refsdal, 1985, Aaboe, 1987) have generally indicated that
there are no mechanical problems with EPS fill after several years of traffic loading. However, these results are confined to materials subject to low magnitude stresses and strains. At larger stresses and strains, the long-term effects of repeated loading appear less certain.

When EPS material was subjected to the repeated loading condition, both the maximum stress level and number of repetitions affect the plastic deformation of the material. The effects of number of load repetitions can be investigated in cyclic loading tests where the maximum cyclic stress was kept constant. The effects of stress levels can be studied by conducting of cyclic loading tests at different peak stresses.

2.1 Definitions of key parameters

In unconfined cyclic loading tests, the key parameters of interest for the stress-strain curve are defined as follows, and are illustrated in Figure 1.

- peak/maximum cycle stress, \( \sigma_m \): the maximum stress imposed during loading cycle number
- compressive strength, \( \sigma_c \): compressive stress at 10% of axial strain on stress-strain curve of monotonic compression test
- peak strain, \( \varepsilon_p \): the axial strain exhibited in response to the maximum cycle stress
- residual strain, \( \varepsilon_r \): the plastic (permanent) strain after unloading of each cycle
- initial tangent Young's modulus, \( E_0 \): the slope of the initial linear-elastic portion of the stress-strain curve
- recoverable strain, \( \varepsilon_0 \): the peak strain minus plastic strain, \( \varepsilon_0 = \varepsilon_p - \varepsilon_r \)
- resilient modulus, \( E_r \): is defined as the ratio of the maximum cycle stress, \( \sigma_m \), to recoverable (resilient) strain, \( \varepsilon_0 \): \( E_r = \frac{\sigma_m}{\varepsilon_0} \)
- stress ratio, \( S = \frac{\sigma_n}{\sigma_c} \)

![Figure 1 Parameters describing stress-strain relation in cyclic test](image)

2.2 Test conditions

Unconfined cyclic compression tests were performed on cylindrical specimens of dimensions Ø50 x 50 mm with a density of 21kg/m³. Test conditions were those of a laboratory environment (approximately 20-25°C and 50% relative humidity). The tests were displacement controlled to produce a rate of loading of 10% of the thickness of the specimen per minute (5 mm/min) in accordance with Australian Standard AS2498.3 (1993) and ASTM Standard D 1621-73/79.

The cyclic tests were carried out using an INSTRON 6027-R5000 Electro-mechanical Universal Testing Machine equipped with high speed digital control and data acquisition. Axial loading and unloading was effected by moving the crosshead on which the base of specimen was seate
Figure 2 Typical loading waveform of cyclic test

Table 1 Loading Condition and Results for Constant Stress Cyclic Loading Tests

<table>
<thead>
<tr>
<th>Max. load, N</th>
<th>Max. cyclic stress, kPa</th>
<th>Stress ratio, %</th>
<th>No. of cycles to failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>240</td>
<td>123.3</td>
<td>112.1</td>
<td>10.1</td>
</tr>
<tr>
<td>220</td>
<td>112.6</td>
<td>102.4</td>
<td>3.09</td>
</tr>
<tr>
<td>200</td>
<td>102.7</td>
<td>93.4</td>
<td>1.247</td>
</tr>
<tr>
<td>180</td>
<td>92.4</td>
<td>84.0</td>
<td>0.225</td>
</tr>
<tr>
<td>160</td>
<td>82.8</td>
<td>74.7</td>
<td>0.135</td>
</tr>
<tr>
<td>100</td>
<td>51.1</td>
<td>46.7</td>
<td>0.02</td>
</tr>
</tbody>
</table>

* Stress ratio = Max. cyclic stress/compressive strength, where the compressive strength of 110 kPa (at 10% axial strain) was obtained from the monotonic compression test.

up and down. The magnitude of loading was measured by a calibrated load cell and the test system was monitored using a proprietary program called Merlin. The saw-tooth loading waveform used for this study is shown in Figure 2.

2.3 Test results

A total of six samples were used in constant stress cyclic loading tests where the maximum cyclic stress was keep constant. Failure was achieved in all tests except for the case of when the maximum cyclic stress was 51.1 kPa. In this test, the number of cycles to failure was based on a projection of existing data as the test would otherwise have required a very long time to reach failure.

The details of the test conditions and test results are listed in Table 1. Figure 3 shows some typical stress-strain curves developed in the constant stress cyclic loading tests. As expected, it was found that the higher the stress level, the higher the initial strain and residual strain rate in term of the number of cycles. The strains induced at all levels of loading of EPS are not fully recoverable upon unloading. This is true even for small levels of peak stress, σ<sub>p</sub>. The resilient moduli, developed from the data shown in Figure 3, are shown in Figure 4. This indicates that in the post-yield range the resilient modulus, E<sub>r</sub>, does not change much with the number of loading cycles. It suggests that the recoverable behavior of EPS will not be affected by repeated loading.

The residual strain accumulation was measured as a function of the number of load cycles for 5 specimens. The results are shown in Figure 5. This curve shows an approximately linear relationship between log(ε<sub>r</sub>) and log(N). The nature of this plot suggests that the relationship between ε<sub>r</sub> and N can be described by:

\[ \log(\varepsilon_r) = m \times \log(N) + \log(\varepsilon_i) \quad \text{or} \quad \varepsilon_r/\varepsilon_i = N^m \]  

(1)
Figure 3 Typical stress-strain curve in constant stress cyclic test

Figure 4 Resilient modulus in constant stress cyclic test

Figure 5 Residual strain in log diagram for various stress ratio ($S=\sigma_i/\sigma_i$)

where $\varepsilon_i$ is residual strain in the first cycle ($N=1$), and $m$ is an experimentally derived parameter. The normalized residual strain, $\log(\varepsilon/\varepsilon_i)$, is plotted against the logarithm of the number of cycles, $\log N$, on a log-log curve in Figure 6. The parameter $m$ is the slope of the linear regres-
Figure 6 Normalized residual strain in log diagram

Figure 7 Cyclic stress ratio versus number of cycles to cause failure

sion for all the test data. Based on the result of the current cyclic tests, an m value of 0.318 was obtained for all stress levels.

The cyclic test results show that a decrease in the maximum stress level requires a significant increase in the number of cycles to cause failure. At lower stress levels, the post-yielding stage may be absent. For stress levels lower than 50%, it may take several years to complete a single cyclic test to failure. Equation (1) may be used to extrapolate the approximate number of load cycles to yield a strain at failure of 10%. Thus, the predicted number of cycles to failure for a stress level of 51.1kPa (stress ratio, S=0.467) in accordance with Equation (1) is approximately $3.1 \times 10^6$.

Figure 7 shows the relation between the number of cycles to failure, N, and the stress ratio, S, for data obtained from the five unconfined constant stress cyclic tests conducted in this investigation. Failure means that either the prescribed maximum stress under cyclic loading cannot be achieved or the accumulated residual strain exceeds a threshold value (e.g. 10%). An approximately linear relationship between log(N) and log(S) was obtained as:

$$\log(N) = -21.5 \log(S) + 1.66$$

or

$$N = \frac{1.195}{S^{21.5}} \quad (2)$$

Equation (2) can be used to predict the cycles of loading to cause failure for EPS material under a given stress level.
From unconfined cyclic compression test results, the influence of different stress levels and number of loading cycles on EPS behavior have been investigated. The conclusions were:

1. The permanent deformation of EPS is affected by both the applied stress level and the number of loading and unloading cycles. The cumulative plastic (residual) strain $e_r$ is increased as the number of cycles increases. The rate of increase of residual strain is also proportional to the peak stress level.

2. The permanent deformation of EPS accumulates with the number of loading and unloading cycles, even in the so-called elastic range of the stress-strain curve obtained from a monotonic compression test.

3. An approximate relationship between normalized strain and number of load cycles can be developed to predict the approximate residual strain for a given number of load repetitions.

4. An approximate relationship between stress level and number of cycles to failure can be developed to predict the approximate numbers of cycles to failure (10% of residual strain).

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Edited by

C.M. Wang
Department of Civil Engineering, The National University of Singapore, Singapore

K.H. Lee
Centre for Advanced Computations in Engineering Science, Department of Mechanical and Production Engineering, The National University of Singapore, Singapore

K.K. Ang
Department of Civil Engineering, The National University of Singapore, Singapore

1999
ELSEVIER
AN INCREMENTAL DEFORMATION MODEL FOR EPS GEOFOAM

C.J. Leo and Y. Zou

School of Civic Engineering and Environment, University of Western Sydney, Nepean, NSW 2747, Australia

ABSTRACT

A simple nonlinear incremental deformation model is presented in this paper for analyzing the stress-dependent deformation of EPS geofoam. The model was developed on the basis of data collected from a series of laboratory tests. Laboratory testing is briefly discussed and explained in relation to model development and estimation of model parameters. Results of model verification studies indicate that it generally accords well with experimental data.

KEYWORDS

EPS, geofoam, nonlinear model, lightweight fill, soft soils

1. INTRODUCTION

EPS (Expanded Polystyrene) geofoam is a very lightweight material used in the construction of embankment over very soft soils. The material is manufactured from expanded polystyrene beads fused together under high temperature and pressure and so in some ways, it can be regarded as a particulate material with particularly strong inter-particle bonding or cohesion. As with other material, the stress-strain behavior of EPS displays a series of characteristics which are specific to the material. Like soil, the strains induced on all loading of EPS are not fully recoverable upon unloading. This is true for every loading even at small stresses, not just for stress levels in the “plastic” regime. The mechanical behavior is clearly dependent on the stress history and on the initial density (Zou and Leo, 1998; Hovarth, 1994; Eriksson and Trank, 1991). However, in this case the initial density depends on the particular product from the manufacturer (usually EPS density of about 20 kg/m³ is used in geotechnical applications). Laboratory tests (Zou and Leo, 1998) have also shown that its stress-strain behaviour has low or no dependency on the confining pressure before yield.
The aim of this paper is to present a simple nonlinear (variable moduli) model of EPS geofoam for the prediction of deformation in geotechnical applications. From a practical engineering and a computational standpoint, providing that the essential characteristics of EPS response are properly captured, a simple model is preferred to more complex stress-strain relationships. The values of the material parameters for the model are easily obtained from standard triaxial laboratory tests and these procedures are briefly discussed herein. Results of verification studies are briefly outlined.

2. FORMULATION OF PROPOSED MODEL

The proposed model may be described as a variable moduli model in which the mechanical behaviour is further restricted to be incrementally isotropic. The tensor relating stress and strain increments in this model depends on the invariants, but not on the stress (or strain) tensor itself. The response of a variable moduli model is irreversible even in incremental loading and does not contain explicit yield condition used in elasto-plastic approaches. Hence yield stress and the presence of strain hardening (or lack thereof) are not formally incorporated within the model.

2.1. Generalized Hooke’s Law

From a general standpoint, the stress-strain relationship of the proposed model can be expressed incrementally by the isotropic generalized Hooke’s law as:

\[ \sigma = K \varepsilon_u \delta_0 + 2G \varepsilon_y \]  

(1)

where \( \sigma, \varepsilon, \delta \) are the incremental stress, strains and deviator strains respectively; \( K, G \) are the tangential bulk modulus and the tangential shear modulus respectively. Decomposition of Equation (1) yields the stress-strain relations (e.g. Nelson and Baron, 1971, Chen and Saleeb, 1994):

\[ \dot{\varepsilon}_y = 2G \dot{\varepsilon}_y \]
\[ \dot{\rho} = K \dot{\varepsilon}_u \]

(2)

(3)

where \( \varepsilon, \rho \) are the incremental deviator and mean stresses respectively. The separation of the constitutive relations into deviatoric and volumetric components as represented by equations (2) and (3) is convenient for determination of the model tangential moduli \( K \) and \( G \). It may be noted that the tangential moduli relationships in this model cannot be uniquely constructed from one or other loading-unloading path but instead must be found in accordance with each of the following loading-unloading paths: initial loading, unloading and reloading. \( K \) and \( G \) in this model are prescribed as functions of the mean stress \( p \) and the second deviatoric stress invariant \( J_2 \) as described below.
2.2. **Tangential moduli in initial loading**

The tangential moduli are functions of the stresses. In initial loading, the moduli are given by the following nonlinear relationships:

\[
K_r = \begin{cases} 
\frac{1}{2} \left( K_a + K_p \right) + \frac{1}{2} \left( K_a - K_p \right) \left( 1 - \exp \left( -a \frac{G_a}{G_{dp}} \right) \right) & \text{when } \frac{p}{p_f} \leq 1 \\
\frac{1}{2} \left( K_a + K_p \right) - \frac{1}{2} \left( K_a - K_p \right) \left( 1 - \exp \left( -a \frac{G_a}{G_{dp}} \right) \right) & \text{when } \frac{p}{p_f} > 1 
\end{cases} 
\]

\[
G_r = \begin{cases} 
\frac{1}{2} \left( G_a + G_p \right) + \frac{1}{2} \left( G_a - G_p \right) \left( 1 - \exp \left( -b \frac{\sqrt{J_2}}{\sqrt{J_2}} \right) \right) & \text{when } \frac{\sqrt{J_2}}{\sqrt{J_2}} \leq 1 \\
\frac{1}{2} \left( G_a + G_p \right) - \frac{1}{2} \left( G_a - G_p \right) \left( 1 - \exp \left( -b \frac{\sqrt{J_2}}{\sqrt{J_2}} \right) \right) & \text{when } \frac{\sqrt{J_2}}{\sqrt{J_2}} > 1 
\end{cases} 
\]

where \( K_{ri}, G_{ri} \) are the initial tangential moduli and \( K_{ri}, G_{ri} \) are the tangential moduli beyond the reference stresses \( p_f \) and \( \sqrt{J_2} \), respectively (see Figure 1). \( K_{ri}, G_{ri}, K_{ri}, G_{ri}, p_f, \sqrt{J_2} \), \( a \) and \( b \) are parameters which can be obtained by conventional triaxial test as described in a later section.

2.3. **Tangential moduli in unloading and reloading**

The tangential moduli in unloading and reloading are approximated by:

\[
K_r = K_{ri} 
\]

\[
G_r = G_{ri} 
\]

The unloading and reloading moduli may also be obtained from laboratory tests.

3. **LOADING, UNLOADING AND RELOADING CRITERIA**

Following soil mechanics sign convention where compressive stress is taken as positive, the loading, unloading and reloading may be written as (Chen and Saleeb, 1994):

- **Bulk modulus** \( K_t \)
  - Loading: when \( p = p_{\max} \) and \( \dot{p} > 0 \)
  - Unloading: when \( p \leq p_{\max} \) and \( \dot{p} < 0 \)
  - Reloading: when \( p < p_{\max} \) and \( \dot{p} > 0 \)

- **Shear modulus** \( G_t \)
  - Loading: when \( J_t = J_{t_{\max}} \) and \( J_t > 0 \)
  - Unloading: when \( J_t \leq J_{t_{\max}} \) and \( J_t < 0 \)
  - Reloading: when \( J_t < J_{t_{\max}} \) and \( J_t > 0 \)

\( p_{\max} \) is the larger of \( p_f \) and the historical maximum value of the mean stress while \( \sqrt{J_{t_{\max}}} \) is the larger of \( \sqrt{J_2} \) and the historical maximum value of the second stress invariant. It may be
noted that the prescribed loading criteria allows for the possibility of the material loading in shear while unloading in mean stress simultaneously and vice versa.

4. LABORATORY TESTS AND CALIBRATION OF MODEL PARAMETERS

The proposed model in the previous section was developed on the basis of experimental data obtained from laboratory testing conducted at the University of Western Sydney (Nepean) using cylindrical EPS specimens of density 21 kg/m$^3$. The specimens used in the testing were supplied by RMAX Pty Ltd, an Australian manufacturer of EPS products. Test specimens were 50 mm in diameter and 50 mm in height and test conditions were typical of a laboratory environment (approximately 23°C and 50% relative humidity).

For the purpose of determining the material parameters, the EPS samples were subjected to standard triaxial undrained compression tests where the confining pressure ranged from 0 to 20 kPa. This is the range of confining pressure expected in filling applications.

In each of the experiments, an EPS sample is situated on a pedestal completely isolated from the cell fluid by a layer of geomembrane. A pressure/volume controller accurate to 1 mm$^3$ was used to control the cell pressure and to measure any volume change in the cell fluid during the test. On this basis, the volume change in the specimen can be determined from the volume change recorded by the controller (after taking into account the necessary corrections for the displaced plunger). The bulk modulus was by plotting the mean stress $p$ against the volumetric strain $\varepsilon_v$ (Figure 1). If the axial strain is $\varepsilon_a$ and the radial strain is $\varepsilon_r$ then the volumetric strain is given by $\varepsilon_v = \varepsilon_a + 2\varepsilon_r$ so that $\varepsilon_v = \frac{1}{2}(\varepsilon_a - \varepsilon_r)$. A plot of the shear stress $\sqrt{3J_2/2}$ against the shear strain $(\varepsilon_a - \varepsilon_r)$ enables the shear modulus to be determined (Figure 2).

![Fig 1: Typical $p$-$\varepsilon_v$ curve in triaxial test](image1)

![Fig 2: Typical $\sqrt{3J_2/2}$-$\gamma_s$ curve in triaxial test](image2)

The tangential bulk and shear moduli are often expressed as functions of the stress invariants such as given by $\kappa_1, (I_1), G, (I_1, \sqrt{J_2})$ where $I_1, \sqrt{J_2}$ are the first and second stress invariants.
respectively. While many possible and theoretically sound relationships could be established in the case of EPS, it was found that both $K_y, G_y$ could be reasonably approximated as functions of $I_1$ (or mean stress $p$) and $\sqrt{J_2}$ respectively based on experimental evidence. This has a major advantage of permitting a simple and neat characterization of the material. The model parameters $K_{uu}, K_{up}, G_{uu}, G_{up}$ are estimated by fitting the appropriate linear relationships in the respective plots as shown in Figures 1 and 2 respectively. $p_y, \sqrt{J_2_y}$ are determined by plotting the logarithmic relationships in equation (6) which is derived from equation (4) and finding the intersection points of the two piecewise linear parts of the relationships. The parameters $a$ and $b$ are found by using linear regression analysis on the logarithmic relationships:

\[
\begin{align*}
\ln \left( 1 - \frac{K_y}{\frac{1}{2}(K_u - K_y)} \right) &= a \left( \frac{p}{p_y} - 1 \right) \quad \text{when } \frac{p}{p_y} \leq 1 \\
\ln \left( 1 - \frac{G_y}{\frac{1}{2}(G_u - G_y)} \right) &= b \left( \frac{\sqrt{J_2}}{\sqrt{J_2_y}} - 1 \right) \quad \text{when } \frac{\sqrt{J_2}}{\sqrt{J_2_y}} \leq 1
\end{align*}
\]  

(6a) (6b)

The results of the triaxial testing yielded the following values for the model parameters (Table 1):

**Table 1: Model parameters from triaxial tests ($\rho = 21$ kg/m$^3$)**

<table>
<thead>
<tr>
<th>$K_{uu}$, MPa</th>
<th>$K_{up}$, kPa</th>
<th>$p_y$, kPa</th>
<th>$a$</th>
<th>$G_{uu}$, MPa</th>
<th>$G_{up}$, kPa</th>
<th>$\sqrt{J_2_y}$, kPa</th>
<th>$b$</th>
<th>$K_{to}$, MPa</th>
<th>$G_{to}$, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>35</td>
<td>37</td>
<td>11</td>
<td>2.4</td>
<td>50</td>
<td>45</td>
<td>15</td>
<td>2.8</td>
<td>4.0</td>
</tr>
</tbody>
</table>

4.1. **Model verification**

A number of laboratories studies (Zou and Leo, 1998; Hovarth, 1994; Preber et al., 1994; Eriksson and Trank, 1991 among others) have shown that EPS exhibit specific characteristics which are:

1. The stress in initial loading increases almost linearly with strain but at some stage the rate of increase diminishes very quickly and asymptotically to a significantly lower rate.
2. The deformation is irreversible even at small stress levels and the “elastic regime” does not really exist.
3. For each cycle of loading-unloading cycle, a hysteresis loop is observed.
4. Strain-hardening occurs when the material is loaded beyond the yield stress.
5. Very small, sometimes almost zero values of Poisson ratios.
Independent tests have been carried out to verify that the proposed model for EPS could capture the essential characteristics of the material in response to loading and unloading stress paths. Generally the model has been able to reproduce the specific characteristics of the material listed above as well as the stress-strain response quite satisfactorily (Leo and Zou, 1999). In its current form, however, it cannot capture hysteresis effects during a loading-unloading cycle.

5. CONCLUSIONS

In this paper, a simple nonlinear variable model for predicting deformation of EPS geofoam is developed for use in the wider geotechnical community. Verification studies have indicated that the model can generally capture the specific characteristics of the material behavior. One exception however is the hysteresis effects which cannot be adequately captured by the model its current form.

ACKNOWLEDGEMENTS

The authors wish to thank RMAX Pte Ltd for generously providing the EPS samples used in the testing.

REFERENCES

MODELLING THE BEHAVIOUR OF EXPANDED POLYSTYRENE (EPS) FILL

C. J. Leo & Y. Zou
School of Civil Engineering and Environment, University of Western Sydney
New South Wales, Australia

ABSTRACT

EPS geo-foam is increasingly used as an ultra lightweight fill of choice to alleviate settlement and bearing capacity problems on soft soils. This paper describes a variable moduli model used in predicting deformation of EPS geo-foam subjected to loading and unloading stress paths. Results from verification studies are presented showing that for the most part the model is sufficiently capable of capturing the response of the EPS material. The model is then used in a finite element simulation example to predict the deformation of a typical EPS embankment.

INTRODUCTION

Lightweight filling materials have long been used to reduce the impact of the fill’s self weight acting on sites underlain by poor quality soft soils of low bearing capacity and high volume compressibility. However, use of conventional lightweight materials such as pumice soil, fly ash and burnt tailing refuse taken from coal mine have had only mixed success. At one-third to one-half as heavy as normal soil, the self weight reduction in employing these materials would appear not significant enough to alleviate settlement and bearing capacity problems at a number of poor quality sites. Under these circumstances, an ultra lightweight material like EPS (Expanded Polystyrene) geo-foam with only about one-hundredth the weight of normal soil can be an effective solution as well as a better engineering option.

The Norwegian Road Research Laboratory first experimented with EPS geo-foam in 1972 in an embankment built on a site that repeatedly suffered settlement problems for a number of years (Refsdal, 1985; Frydenlund, 1986, 1991). The prevailing soil at the site consists of 3 m thick of peat and 10 m of soft sensitive clay. Earlier methods of arresting settlement had failed quite badly until the innovative use of EPS geo-foam was implemented. The success of the Norwegian experiment has resulted in today the EPS geo-foam rapidly gaining acceptance as a fill of choice. A major advantage has also been that time taken for constructing an EPS embankment is significantly shorter when pre-loading or soil treatment work are dispensed with.

Laboratory and scale model studies (e.g. Zou and Leo, 1998; Preber et al., 1994; Eriksson and Trank, 1991; Hamada and Yamanouchi, 1987) provided useful insights of the behaviour of the EPS material through testing under controlled conditions. As EPS is manufactured from expanded polystyrene beads fused together under high temperature and pressure, in some ways it can be regarded as a particulate material with strong inter-particle bonding or cohesion. Tests have shown that the strains induced on primary loading of EPS are generally not fully recoverable upon unloading (Zou and Leo, 1998). Its compressive strength is dependent on the stress history (i.e. there is some strain hardening) but it has only a small dependency on the confining pressure. As the confining pressure in most practical situation is small in any case, this dependency can be ignored.
Horvath (1996) has quite appropriately pointed out that to make the best use of the material, there needs to be a consistent program for developing constitutive models of EPS geofoam. In the light of this, this paper presents a simple deformation model developed by the authors based on triaxial testing of the EPS material. This model is best described as a K-G (bulk/shear modulus) variable moduli model which is the product of a series of triaxial and other laboratory studies on EPS. It falls into the same category as the well-known hyperbolic (hypoelastic) model of Duncan and Chang (1970) which provides a simple procedure for representing nonlinear, stress-dependent, inelastic stress-strain behavior for soils. Despite certain shortcomings as not being able to compute collapse load in the fully plastic range, the Duncan-Chang model is still much favoured by the geotechnical community for its ease of use. It may also be argued that in many cases of EPS applications, the deformational rather than strength behavior is the main subject of interest to geotechnical engineers. The proposed EPS model fulfils the first requirement quite adequately. Verification results are presented showing that the particular characteristics of EPS behaviour have generally been captured by the model.

This paper also gives a finite element simulation example using the K-G model of a typical road embankment on soft soils. The modelling results are briefly discussed within.

THE VARIABLE MODULI K-G MODEL FOR EPS GEOFOAM

Generalized Hooke’s Law

The stress-strain relationship of the proposed model can be expressed incrementally by the generalized Hooke’s law as:

$$\Delta \sigma = D_t \Delta \varepsilon$$  \hspace{1cm} (1)

where,

$$\varepsilon = \begin{bmatrix} \varepsilon_x, \varepsilon_y, \varepsilon_z, \gamma_{xy}, \gamma_{yz}, \gamma_{zx} \end{bmatrix}^T$$ is the vector of the strain

$$\sigma = \begin{bmatrix} \sigma_x, \sigma_y, \sigma_z, \sigma_{xy}, \sigma_{yz}, \sigma_{zx} \end{bmatrix}^T$$ is the vector of the stress

$$D_t = \text{the tangential material stiffness matrix}$$

It is assumed that the EPS material is isotropic, therefore the tangential elasticity matrix is given by:

$$D_t = \begin{bmatrix} K_t + \frac{4}{3} G_t & K_t - \frac{2}{3} G_t & K_t - \frac{2}{3} G_t & 0 & 0 & 0 \\ K_t - \frac{2}{3} G_t & K_t + \frac{4}{3} G_t & K_t - \frac{2}{3} G_t & 0 & 0 & 0 \\ K_t - \frac{2}{3} G_t & K_t - \frac{2}{3} G_t & K_t + \frac{4}{3} G_t & 0 & 0 & 0 \\ 0 & 0 & 0 & G_t & 0 & 0 \\ 0 & 0 & 0 & 0 & G_t & 0 \\ 0 & 0 & 0 & 0 & 0 & G_t \end{bmatrix}$$  \hspace{1cm} (2)
where $K_t$, $G_t$ are the tangential bulk modulus and the tangential shear modulus respectively. Relationships for the tangential moduli are given below.

**Tangential Moduli in Initial Loading**

The tangential moduli are functions of the stresses. In initial loading, the moduli are given by the following nonlinear relationships:

\[
K_t = \begin{cases} 
\frac{1}{2} \left( K_n + K_p \right) + \frac{1}{2} \left( K_n - K_p \right) \left( 1 - \exp^{-a \left( \frac{p}{p_y} \right)} \right) & \text{when } \frac{p}{p_y} \leq 1 \\
\frac{1}{2} \left( K_n + K_p \right) - \frac{1}{2} \left( K_n - K_p \right) \left( 1 - \exp^{-a \left( \frac{p}{p_y} - 1 \right)} \right) & \text{when } \frac{p}{p_y} > 1 
\end{cases}
\]

(3a)

\[
G_t = \begin{cases} 
\frac{1}{2} \left( G_n + G_p \right) + \frac{1}{2} \left( G_n - G_p \right) \left( 1 - \exp^{-a \left( \frac{\sqrt{J_2}}{\sqrt{J_{2y}}} \right)} \right) & \text{when } \frac{\sqrt{J_2}}{\sqrt{J_{2y}}} \leq 1 \\
\frac{1}{2} \left( G_n + G_p \right) - \frac{1}{2} \left( G_n - G_p \right) \left( 1 - \exp^{-a \left( \frac{\sqrt{J_2}}{\sqrt{J_{2y}}} \right)} \right) & \text{when } \frac{\sqrt{J_2}}{\sqrt{J_{2y}}} > 1 
\end{cases}
\]

(3b)

where $K_{ti}$, $G_{ti}$ are the initial tangential moduli and $K_{tp}$, $G_{tp}$ are the tangential moduli beyond the reference stresses $p_y$ and $\sqrt{J_{2y}}$ respectively. $K_{ti}$, $G_{ti}$, $K_{tp}$, $G_{tp}$, $p_y$, $\sqrt{J_{2y}}$, $a$ and $b$ are parameters which can be obtained by conventional triaxial test as described in Leo and Zou (1999).

**Tangential Moduli in Unloading and Reloading**

The tangential moduli in unloading/reloading is given by:

\[
K_t = K_n \\
G_t = G_n
\]

(4a)

(4b)

**Loading, Unloading and Reloading Criteria**

The tangential moduli corresponding to loading, unloading and reloading may be written as (Chen and Saleeb, 1994):

**Bulk modulus $K$**

- **Loading:** when $p = p_{\text{max}}$ and $\dot{p} > 0$
- **Unloading:** when $p \leq p_{\text{max}}$ and $\dot{p} < 0$
- **Reloading:** when $p < p_{\text{max}}$ and $\dot{p} > 0$
Shear modulus $G$

Loading: when $J_2 = J_{2,\text{max}}$ and $J_2 > 0$

Unloading: when $J_2 \leq J_{2,\text{max}}$ and $J_2 < 0$

Reloading: when $J_2 < J_{2,\text{max}}$ and $J_2 > 0$

It may be noted that the prescribed loading criteria allows for the possibility of the material loading in shear while unloading in mean stress simultaneously and vice versa.

The calibration of model parameters (for 21 kg/m$^3$ EPS geofoam supplied by the RMAX firm in Australia) is described in Leo and Zou (1999) and the values given in Table 1. A finite element program incorporating the K-G constitutive relationships given above has been written to make predictions of EPS deformations in geotechnical applications.

<table>
<thead>
<tr>
<th>$K_{\text{tt}}$, MPa</th>
<th>$K_{\text{tp}}$, kPa</th>
<th>$p_{\gamma}$, kPa</th>
<th>a</th>
<th>$G_{\text{tt}}$, MPa</th>
<th>$G_{\text{tp}}$, kPa</th>
<th>$\sqrt{J_{2,\gamma}}$, kPa</th>
<th>b</th>
<th>$K_{\text{tu}}$, MPa</th>
<th>$G_{\text{tu}}$, MPa</th>
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</table>

MODEL VERIFICATION

A number of laboratories studies (Zou and Leo, 1998; Hovarth, 1994; Zou and Leo, 1998; Eriksson and Trank, 1991 among others) have shown that EPS exhibit particular characteristics which are:

1. The stress in initial loading increases almost linearly with strain but at some stage the rate of increase diminishes very quickly and asymptotically to a significantly lower rate.
2. The deformation is irreversible even at small stress levels, therefore the “elastic regime” does not strictly exist.
3. For each cycle of loading-unloading cycle, a hysteresis loop is observed.
4. Strain-hardening occurs when the material is loaded beyond the yield stress.
5. Very small, sometimes almost zero values of Poisson ratios have been measured.

Independent tests have also been carried out to verify that the proposed model for EPS could capture the essential characteristics of the material response to loading and unloading stress paths.

Figure 1 shows the experimental and predicted results of a constrained loading-unloading-reloading uniaxial compression test on an EPS specimen. The predicted results are obtained by numerically integrating Equation (1) while maintaining the lateral strains at zero. In the initial loading stage, the characteristic mechanical response described above is reproduced fairly accurately by the model. The presence of the hysteresis loop is evident in the loading-unloading cycle but in the model this is represented by an “average” linear response. At low stress levels however, this deviation from experimental data is considered as not significant. Importantly, the residual strain arising from the irreversibility of the deformation at the stress level before yielding is predicted quite accurately.
Figure 1: Stress-strain curve in constrained uniaxial compression test

Figure 2: Stress-strain curve in triaxial test ($\rho = 21\text{kg/m}^3$)

Figure 2 shows the vertical stress-strain results for a triaxial test with a confining pressure of 10 kPa and the predictions obtained by numerical integration of the model. In the loading sequence, the shape of the response curve is once again clearly reproduced by the model. The first unloading is in the linear part of the curve but it produces a residual strain which is again quite accurately predicted by the model. The reloading curve rejoins the linear part of the virgin curve where loading continues until the second unloading takes place at a higher stress level beyond the yield value. The second reloading curve eventually rejoins the flat part of the virgin loading curve once the historical maximum stress level is exceeded indicating the presence of strain hardening. The model could reproduce all these responses quite satisfactorily (including the effects of strain hardening) except for the response in the second unloading/reloading cycle. As hysteresis effects increase with stress levels, the limitation of the model in its current form to reproduce these effects is exposed.
Figure 3: lateral strain in triaxial test ($\rho = 21$kg/m$^3$)

The plot of the radial strain against deviator stress from the triaxial test of various confining stresses (0-20 kPa) is shown in Figure 3. As described above, one of the characteristics of the EPS material is the very low or sometimes almost zero-valued Poisson’s ratio which is manifested in the very low lateral strain produced in axial loading. The model results quite accurately predicted the uniquely small lateral strain for EPS geofoam under axial loading until yielding when the lateral strain actually become compressive.

Shown in Figures 4a and 4b are the plots of the volumetric strain against deviator stress and the volumetric strain against the axial strain respectively for various confining stresses (0-20 kPa). Shear loading is imposed as the deviator stress/axial strain is increased. Clearly as the material is loaded axially the volumetric strain is decreased, this characteristic is also quite correctly predicted by the model.

Generally the model has been able to reproduce the specific characteristics of the material listed above as well as the stress-strain response quite satisfactorily. In its current form, however, it cannot capture hysteresis effects during a loading-unloading cycle.
FINITE ELEMENT MODELLING OF EPS ROAD EMBANKMENT

A finite element simulation of an EPS road embankment build on overconsolidated clay has been undertaken (similar to an example in Hamada and Yamanouchi, 1987). The dimensions of the embankment are given in Figure 5. The embankment is subjected to concentrated wheel loads (4 x 19.6 kN) and its self weight in a typical loading case. The properties used for the non-EPS component parts of the embankment are tabulated in Table 2, where it is noted that the clay and the covering reinforced soil had been modelled as “Duncan-Chang” material (Hyperbolic model). The properties of the EPS fill are as given in Table 1 and the finite element mesh (and the deformed outline under the applied loading and self weight) is presented in Figure 6.

Table 2: Material properties of non-EPS material

<table>
<thead>
<tr>
<th>Material</th>
<th>Cohesion (kPa)</th>
<th>Friction angle ($\phi'$)</th>
<th>$R_f$</th>
<th>n</th>
<th>Initial $E'$ (kPa)</th>
<th>Poisson ratio ($\nu'$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OC clay</td>
<td>49</td>
<td>0</td>
<td>0.9</td>
<td>0</td>
<td>9.8</td>
<td>0.3</td>
</tr>
<tr>
<td>Reinforced soil</td>
<td>98</td>
<td>35°</td>
<td>0.9</td>
<td>0.55</td>
<td>49</td>
<td>0.25</td>
</tr>
<tr>
<td>Pavement</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>200</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The results show that large parts of the embankment itself are under some tension. For the most part, tensile (principal) stresses in the EPS fill are typically low (less than 10 kPa), except in the region next to the EPS-pavement interface as shown in Figure 7. Stresses are within the tensile strength of the EPS material (where tensile stress is considered as positive). The reinforced covering soil would have assisted to take some tensile stresses.

The highest compressive (principal) stress in the EPS exist in a small region at the centre of the fill just below the pavement, but even these stresses (about 30 kPa) are also well within the compressive strength of the material (Figure 8). The clay remains overconsolidated after loading. Had the EPS fill not been used, a conventionally build embankment would have
stressed the clay beyond the preconsolidation stress \( \sigma_{oc}' = 50 \text{kPa} \) into the range of normal consolidation.

The maximum settlement which occurs at the centre and top of the embankment is about 37 mm and maximum differential settlement is about 13 mm, within acceptable tolerance which in normal circumstances using normal fill would not have been possible to achieve.

Figure 5: EPS embankment on overconsolidated clay

Figure 6: Finite element mesh (and deformed outline) of EPS embankment on overconsolidated clay
CONCLUSION

This paper describes a variable moduli model for predicting the deformation of EPS geofoam fill in geotechnical applications. Verification studies have indicated that the model is able to capture for the most part the essential characteristics of the material behavior.

ACKNOWLEDGEMENT

The authors wish to thank RMAX Pte Ltd for generously providing the EPS samples used in the testing.
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MODEL TESTING OF EPS FILL UNDER VEHICULAR LOADING

Yong Zou1, C.J. Leo1 and J.C. Small2

1Department of Civil Engineering, University of Western Sydney Nepean, NSW 2747, Australia
2Department of Civil Engineering, University of Sydney, NSW 2006, Australia

Abstract: The behaviour of EPS geofoam used as a replacement subgrade material under flexible pavement was investigated by carrying out a series of tests on a model pavement testing facility. The test section of the pavement and the selected subgrade were constructed inside a test tank where vehicular loading was simulated in the form of a loaded wheel running on an oval-shaped test track which passed over the test pavement. The investigation studied the effects of repeated traffic loading on the performance of the EPS geofoam including the influence of the size of the EPS blocks and the presence or lack of lateral restraint. A comparative study with conventional fill, considered the base case, was also carried out. The test results shown that the performance of EPS geofoam is very well as a replacement subgrade material under the repeated traffic loading.

Key Words: EPS, geofoam, model testing, fill material

1. Introduction

In geotechnical applications, the mechanical behavior of an engineering material is complicated and influenced by many factors present in the field. As a first approach, the determination of the basic material properties may be achieved by carrying out the appropriate standard laboratory tests. However, there is a limit to which the results can be used to interpret field performance since standard laboratory tests are usually not designed to fully capture the field conditions. It has been shown from past experience that if carefully designed, testing on a physical model can provide a way to overcome some of the limitations of the standard laboratory test thereby giving better insights into the material performance under more realistic conditions.

In order to study the behavior of Expanded Polystyrene (EPS) geofoam as a replacement subgrade material under a flexible pavement, the authors undertook some testing on a road pavement testing facility which was first developed by Wong and Small (1994). EPS geofoam is an extremely lightweight polymeric material used for filling over very soft grounds. Several laboratory studies (e.g. Zou and Leo, 1998, Horvath, 1995, Preber et al. 1994, and Eriksson and Trank, 1991) and field observations (Aabo, 1987) have indicated that EPS geofoam can be used to overcome many of the geotechnical problems in areas beset with soft soils. The test facility developed by Wong and Small (1994) provided an opportunity to closely study the behaviour of EPS blocks under vehicular loading in realistic yet controlled conditions. Some earlier studies on the settlement problems of bridge approaches (Wong and Small, 1994) and the effectiveness of geosynthetic reinforced pavement (Moghaddas-Nejad and Small, 1996) had already been successfully carried out using the test facility. For this investigation, the physical pavement model in the facility was set up to incorporate the EPS fill, then a series of tests were conducted which accurately monitored the deformation of the EPS blocks under conditions as closely as possible replicating those found in the field.

The objects of the model pavement tests are: to investigate the behaviour of the EPS geofoam as filling material for construction of road subgrade; to investigate the influence of the EPS block size on the deformation of the subgrade; to study the effects of lateral restraint on EPS blocks; to compare the deformation of EPS blocks with that of a conventional filling material sand. The results obtained suggest that EPS geofoam performed very well under repeated vehicular loading, halving the size of the EPS blocks and the lack of lateral restraint did not adversely affect the performance of the geofoam. However, the results also indicate that there is a possibility that the use of EPS geofoam may accentuate pavement rutting if the stiffness or the thickness of the pavement layers were not adequate.

2. Testing facility

The testing facility consists of four main structural components, namely, the test tank, the overhead track, running track and the loading carriage. The support and guidance for the moving loading carriage is provided by the overhead track, which has an oval shape with straight sections on the two long sides. An overview of the test facility is shown in Figure 1.

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The test section of model pavement was constructed inside the test tank and positioned below one of the straight sections of the overhead track. The test tank used in the tests consisted of two hollow steel boxes, 1.4m by 0.5m by 0.4m deep internally and bolted together on the flanges. Several LVDT transducers were installed to measure subsurface displacement in the test pavement and the subgrade. A driven wheel was directed around the oval shaped track by a guide rail system. A conductor rail system supplied power to the motor, which drove the test wheel. The wheel passed over the test section of pavement once during each revolution around the track and triggered a microswitch that started a microcomputer recording data. Also wheel load data from the moving carriage was sent back to the microcomputer.

3. Description of Experiment

3.1 Model Scale

To capture the essential behavior of an engineering material, the appropriate stress levels existing in field conditions should necessarily be reproduced in the test model. As the primary aim of the study was to investigate the behavior of the EPS, consideration was given to ensure that the stress levels applied to the EPS blocks were approximately of the same magnitude as in the field. From finite element analysis (author's unpublished data), it was determined that the maximum stress level in the EPS blocks in a typical flexible pavement embankment application corresponds to about 20 - 30 kPa. The test facility, which has a model scale of ¼, allowed the stress levels in the EPS of the model to be achieved easily without need for any untoward compromises.

The largest truck tyre is about 910 mm in diameter, therefore, the pneumatic tyre of the model loading wheel of 230 mm diameter, the smallest available, was chosen on the basis of the given scale. On the same scale, the required loading on the model wheel ranged from 490-680 N at a tyre pressure of about 210 kPa and corresponding to the actual tyre loading.

3.2 Pavement and Subgrade Material

The model pavement section, consisting of a wearing course, a road base and a sand sub-base, overlaid the subgrade (which in this series of testing was either compacted sand or EPS blocks) and an uncompacted sand layer used for leveling at the bottom of the test tank. A typical section of a model pavement used for the tests is presented in Fig.2.

3.2.1 Wearing course

The wearing course used in road pavement varies widely, but typically the maximum stone size of the wearing course is about 20-25mm. Thus a commercial bituminous mix with a bitumen content of about 5% and a maximum aggregate size of 5 mm was chosen for the wearing course based on the scale ratio of 1:4. Although advertised and sold as a mix not requiring any special heating before laying, nevertheless, the mix was heated for about 4 to 5 hours at 150°C before compaction as tests showed that this practice gave a superior performance.
3.2.2 Base material

The base layer was made up of crushed aggregate of basaltic origin and of 5mm nominal size with particles that are mainly subangular. It is classified as a uniform fine gravel with less than 1% of particles finer than 75μm. The particle size was chosen so that it is roughly scaled in accordance to the model dimensions. The minimum and maximum densities of the base material were 1.38 t/m³ and 1.49 t/m³, respectively.

3.2.3 Sub-base and Subgrade

As discussed previously, two types of subgrade were used in this series of testing. For the base case, a silica sand called Sydney sand was used as the conventional subgrade. Sydney sand is classified as a well graded sand with less than 1% of fine. The minimum density was 1.44 t/m³, while the average maximum density was 1.69 t/m³. The same sand was also used for the compacted sand sub-base.

The second subgrade material used was EPS geofoam of density 21kg/m³, which is the most popular type used as filling material in geotechnical applications. The compressive strength of this grade of EPS foam ranges from 90 to 120 kPa. Figure 3 shows a typical stress - strain curve from a strain-controlled triaxial test on a cylindrical EPS specimen at strain rate of 1% deformation per minute (Zou and Leo, 1998).

The geofoam was cut into rectangular parallelepiped blocks of dimensions measuring 500×160×100 mm³ for the "big blocks" in this paper, and 250×160×100 mm³, for the "small blocks". The small blocks have more joints and block to block interfaces. The effect of halving the block size on the mechanical behavior was studied. A total of 5 tests were conducted on big blocks, small blocks and compacted sand. The details of these tests are listed in Table 1.

<table>
<thead>
<tr>
<th>test No.</th>
<th>filling material</th>
<th>laterally restrained</th>
<th>Max wheel load, N</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EPS (big)</td>
<td>Yes</td>
<td>680</td>
</tr>
<tr>
<td>2</td>
<td>EPS (big)</td>
<td>No</td>
<td>590</td>
</tr>
<tr>
<td>3</td>
<td>EPS (big)</td>
<td>Yes</td>
<td>490</td>
</tr>
<tr>
<td>4</td>
<td>Compacted sand</td>
<td>Yes</td>
<td>540</td>
</tr>
<tr>
<td>5</td>
<td>EPS (small)</td>
<td>Yes</td>
<td>590</td>
</tr>
</tbody>
</table>

3.2.4 Pavement thickness

Generally, the thickness of the pavement construction should be sufficient to spread the loads in such a way that the combination of live load and the dead load of the pavement plus any additional loading do not cause the EPS to compress beyond its elastic limit. This ensures that unacceptable deflections and permanent deformations of highway surface are avoided. The typical pavement thickness at full scale for the wearing course, base layer and sand sub-base were chosen to be: 80 mm, 200 mm and 200 mm respectively. Thus the corresponding pavement thicknesses chosen in the model are 20 mm, 50 mm and 50 mm.
3.2.5 Compaction Methods

Due to the strong influence of the degree of compaction on the granular material deformation characteristics, the compaction procedure must be chosen so as to give a uniform layer with similar densities and particle structures. To obtain a reproducible test result, close control of compaction processes should be exercised.

A manual compaction method was selected for this series of tests. In order to ensure uniformity, filling was completed in seven sections in any single layer as described by Wong (1994). The number of blows for all sections was specified to be 20 and the height of fall of the compacting tool was 50 mm at each section of a layer. After compacting, the average densities of wearing course, base layer and sand subgrade were found to be 1.75 t/m³, 1.5 t/m³ and 1.65 t/m³ respectively.

![Figure 3 Typical stress – strain curve (ρ=21kg/m³)](image)

![Figure 4 Typical displacement curve of model test](image)

3.3 Measurement of Surface and Subsurface Deformation

In each test, the displacement measurement was taken at two sections in the test tank. For each section, three transducers were placed along the centerline of the pavement at three levels i.e. wearing course-base interface, subgrade-EPS block interface and in the middle of the EPS blocks. Two additional transducers were also setup at the level of the subgrade-EPS block interface in a line perpendicular to the centerline of the pavement to measure the deformation of the top surface of the EPS blocks. The position of the transducers is indicated in Figure 2. Several drawing pins were pushed into the surface of the pavement directly above the transducers, which is perpendicular to the centerline, to measure the surface deformation at the same cross-section as the transducers.

As the driving wheel passed the test tank, the displacements of the pavement and substructure were recorded. A typical displacement curve for the first three passes is shown in Figure 4. The displacement reaches the maximum as the driving wheel directly passes above the location of transducer and then goes back as the wheel load released. In this paper, the maximum displacement of each pass was defined as "maximum instant displacement", and the initial displacement of each pass was taken as the "permanent displacement" of the previous pass. The typical maximum and permanent displacement curves are shown in Figures 5a and 5b. The largest incremental plastic deformation occurs in the first few hundred passes of the driving wheel, after which the increment if any is considerably more gradual.

3.4 Other Notable Aspects of Testing

The other notable aspects of the testing are summarized below:

1. The speed of the wheel going around the test track was set to 2.7 km/h or about 11 km/h on the actual scale. This is a major limitation of the testing facility as it can only model slow moving vehicles.

2. The compacting of all layers was conducted using a standardized procedure. This ensured a uniform pavement with repeatable densities.
3. The room temperature was kept at 22°C. With the temperature held constant, any potential thermal effects, which may change the properties of the bituminous layer, were eliminated.

4. In all tests involving EPS geofoam, four layers of EPS blocks with a total thickness of 400mm were used. Each layer was placed on a plane parallel to the centerline of the pavement surface. The EPS blocks were arranged in a pattern that minimized the vertical continuity of joints between blocks.

5. For testing of the “no lateral restraint” case, a gap of approximately 1 cm wide was provided between the EPS blocks and the side walls of the test tank.

4. Test results and discussion

Figure 6 shows the typical permanent displacement curve of the EPS layer under the center of the wheel load. The results show that there were two distinctly different sets of deformations from what were clearly two similar sections of the test tank. While there are a number of possible reasons why this had occurred, it was decided that in the interest of consistency comparison of test results should only be made for the same section.
Figure 7a and 7b show the permanent displacement at the top surface of the EPS layer and at the same depth in compacted sand (for the base case) at sections 1 and 2 respectively. These results will be of most interest in the interpretation of the effects of block size, lateral restraint and in comparison with the base case. An indication of the effects of the block size is seen by comparing the deformation results from Test 3 ("big blocks") and Test 5 ("small blocks"). It may be noted that the length of a big block is twice that of the small block, therefore, Test 5 had a block-block interface area which is about 1.4 times more than the interface area in Test 3. On the basis of the data obtained, it is observed that there is no perceptible change in deformation behaviour of the EPS blocks when the size was halved. In fact, the deformation in Test 5 was slightly smaller than in Test 3 after 5000 passes even though it was carrying a slightly higher wheel loading.

The effects of lateral restraint, if any, may be observed by comparing Test 2 and Test 3. There appears to be little or no benefit gained by laterally confining the blocks as the results from the model testing suggest. Somewhat against expectation, there was actually a higher deformation occurring in Test 2 where there was no lateral restraint on the blocks and when the wheel load was a little higher than in Test 3.

Test 4 represents the base case where the subgrade is compacted sand. The first observation which may be made is that the incremental deformation for compacted sand is more gradual in the first few thousand cycles, but eventually the deformation after 5000 cycles is about the same magnitude, if not marginally higher, than the other cases for approximately similar wheel loading.

After 5000 passes of the wheel, the permanent deformation of the EPS material is about 0.5% of the thickness of the EPS layer which is of a similar magnitude in the permanent strain obtained for the base case. This suggests that the EPS geofoam should perform just as well as compacted sand for the range of stresses investigated. The projected permanent deformation after 1 million passes, shown in Figure 8, also indicated that when used as a subgrade material for road pavement, EPS geofoam does not suffer any more excessive deformation than conventional subgrade material.

![Figure 9 Permanent displacement of top surface of pavement](image-url)

Figure 9 shows the permanent displacement of the top surface of the pavement for the series of tests. From consideration of the rutted road surface only, it is interesting to note that the compacted sand subgrade was a better material even though the deformation of compacted sand was marginally higher than the EPS material at subgrade level. This implies that for the EPS blocks, a higher permanent deformation must have occurred in the pavement layer above the subgrade than in the base case. A good perspective of the magnitude of displacement at the road surface and at the level of the EPS is provided in Figure 10 demonstrating quite clearly that the deformation on the road surface developed mostly within the pavement layer. An explanation why pavement deformation is much greater in the case of an EPS subgrade is that a larger deflection amplitude was being experienced in the deformation cycle (i.e. the difference between maximum and minimum deflection) for the EPS blocks. Generally, rutting in the pavement layer is related to the relative stiffness of the pavement material and its subgrade, the stiffness for EPS being only about 10% of compacted sand (see Table 2 below).

<table>
<thead>
<tr>
<th>Subgrade</th>
<th>Secant Young's Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPS (21 kg/m³)</td>
<td>4-5 (Zou and Leo, 1998)</td>
</tr>
<tr>
<td>Compacted sand</td>
<td>50 (Moghaddas-Nejad, 1996)</td>
</tr>
</tbody>
</table>

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Figure 11 shows the cross section of the pavement after 5000 passes and an indication of the rutting, which had occurred. The base case outperformed the EPS material when the results are assessed based on pavement rutting.

Figure 10 Deformation of the Pavement after 5000 passes

Figure 11 Deformation of the pavement surface after 5000 passes

5. Conclusions

The investigation has shown that:

1. For the expected stress levels found in the field, EPS has the potential to perform very well under the repeated traffic loading.

2. Even though the loading is within the elastic limit of the EPS blocks, plastic deformation occurs as the wheel loading passes on the top of EPS block. The magnitude of permanent deformation depends on the load level.

3. The deformation of the EPS is relatively large in the initial passes. However, after 5000 passes, the maximum permanent deformations were within 0.5% of the thickness of blocks. This indicates that the projected deformation in EPS fill for 1 million cycles would also be quite small.

4. The difference between EPS and sand fill is clearly seen in Figure 7. The deformation of EPS increases sharply in the initial passes then tapers off, but for sand the deformation increases more gradually. Ultimately, there was little difference in the deformation between the sand and EPS subgrade for the stress levels investigated.

5. Limited test results showed that halving the block size and not having any lateral restraint did not reduce the performance of the EPS blocks. This will have some implications on the design of EPS embankment for road pavement.

6. The test results also suggest that in areas where rutting may be a problem, it would be prudent to increase the stiffness of the pavement such as by using a rigid pavement or increasing the thickness of the pavement itself when EPS subgrade is being used. As it is not the one of the aims of this investigation to study the problem of rutting, further research is needed to resolve potential aggravation of rutting problems caused by using EPS blocks.

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References

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Laboratory Studies on the Engineering Properties of Expanded Polystyrene (EPS) Material for Geotechnical Applications

Y Zou, University of Western Sydney, Australia
C J Leo, University of Western Sydney, Australia

ABSTRACT: EPS is a very lightweight filling material increasingly seen as providing an effective engineering solution for overcoming bearing capacity and settlement problems of embankments built on very soft soils. As wider use of EPS is found in countries around the world, there is a need to better understand the behavior of this material especially in the context of geotechnical applications as well as to develop appropriate constitutive models used for engineering analysis and design. In this paper, the results of some testing carried out in the geotechnical laboratory are presented to provide information on the material’s deformational behavior and as a first step leading to development of methods of analysis which can sufficiently model the complex behavior of EPS in problems of practical interest.

1. INTRODUCTION

Expanded Polystyrene (EPS) is a lightweight polymeric material made by expansion of raw plastic beads. When expanded, the beads become spherical shaped particles, each containing closed hollow cells in which air is trapped. This enables the material to be very lightweight, compressible, as well as possessing good insulation properties. EPS has been used for thermal insulation in housing and road constructions. However, as it is a very lightweight material, geofoams made from EPS are also an excellent material for building road embankments and for filling behind retaining structures resting on very soft soils. In South-East-Asian countries where there is a predominance of soft soils, EPS offers an attractive engineering solution for overcoming many of the geotechnical problems facing conventionally built embankments and retaining structures.

EPS has potential for wider geotechnical applications than it is the case today. From an engineering perspective, it will be both necessary and useful that the engineering properties of EPS relevant to geotechnical applications should be investigated and discussed. This paper therefore presents the results of some EPS material testing undertaken by the authors in the geotechnical laboratory. The testing is also a first step of an effort to develop useful constitutive relationships for characterizing the behavior of EPS and implementation in computer models.

A series of conventional laboratory testing were carried out which include “undrained” triaxial compression test, hydrostatic compression test, shear box test and creep test. These were used to determine the dependency of EPS mechanical properties on the material density and on the confining pressure. Relationships are established to describe the compressive strength, yield stress and Young’s modulus in terms of these two variables. It is noted that these parameters are generally only mildly dependent on the confining pressure, but is strongly related to the material density. Results for Poisson’s ratio, bulk modulus and shear modulus of EPS are discussed. A plastic yield criterion based on test results is proposed for analysis using elastic-perfectly plastic approaches. Finally, some results from creep testing are presented to illustrate the long term deformational behavior of EPS under constant loading.
2. LABORATORY TEST ON THE BEHAVIOUR OF EPS

2.1 TRIAXIAL TEST APPARATUS

The main testing were carried out in a triaxial apparatus using cylindrical EPS specimens with density ranging from 12 kg/m³ to 27 kg/m³. A total 69 of specimens were used in "undrained" triaxial and hydrostatic testing. Test specimens were 50 mm in diameter and 50 mm in height. The specimens were prepared to a height to diameter aspect ratio of 1:1 rather than the conventional 2:1 ratio after it was observed that the later produces a number of the specimens that buckle easily under axial loading.

In triaxial testing, the EPS specimen sits on a pedestal completely isolated from the cell fluid by a layer of geomembrane. The axial loading is provided by a triaxial compression machine and the magnitude of loading is measured by a calibrated proving ring.

A pressure/volume controller accurate to volume measurement of 1mm³ was used to control the cell pressure constant at a pre-set value and to measure any volume change in the cell fluid during the test. It is observed that except for the drainage line connecting the controller and the cell, no other drainage paths are provided in the set up. Therefore, any recorded volume change is the sum total of the displaced fluid due to the loading plunger and the volume change in the specimen itself as it is loaded axially. On this basis, the volume change in the specimen can be determined from the volume change reading recorded by the controller after making the necessary corrections for the displaced plunger.

Test conditions are typically those of a laboratory environment (approximately 23°C and 50% relative humidity). A strain-controlled loading rate of 0.5 mm/min was applied to the specimen in accordance with standard ASTM practice of loading at strain rate of 1% /min. In any normal undrained test, corrections should be made for the average cross-sectional area of the specimen for a given strain in the specimen. However, as this is not strictly a classical undrained test and given that the Poisson ratio value is very low, no correction was made for the cross-sectional area of the specimen. After failure, the dimensions of each specimen were carefully measured, the results clearly showing that there was negligible change in the specimen's diameter. This backs up our decision not to make any correction for the cross-sectional area.

2.2 TEST RESULTS

2.2.1 Definitions and general observations

Figure 1 shows a typical approximating bilinear type stress-strain relationship obtained from the undrained triaxial test. For the sake of clarity, the key strength parameters and Young's moduli are defined below and further illustrated in Figure 2. These definitions are consistent with those used by Horvath (1995)⁴⁷.
Compressive Strength, \( \sigma_c \), is defined as the compressive stress measures at a strain level, usually 10\%, which corresponds approximately to the end of the yielding range.

Initial Tangent Young’s modulus, \( E_h \), is defined as the slope of the initial linear-elastic portion of the stress-strain curve.

Plastic Tangent Young’s modulus, \( E_p \). The slope of the post-yielding linear portion of the stress-strain curve is defined as the plastic tangent Young’s modulus, \( E_p \).

Yielding stress, \( \sigma_y \). The yielding stress is determined graphically by forward extrapolation of the initial linear portion and backward extrapolation of the post-yield linear portion of the stress-strain curve. The stress at which the two extrapolated lines cross is defined as the yielding stress, \( \sigma_y \).

In general, the key aspects of the stress-strain curve are listed below:

1. the initial part of the curve between 1\% and 2\% strain is essentially linear-elastic. The elastic limit increases with increasing EPS density.
2. yielding occurs over a range, which extends to strain between approximately 3\% and 5\% (also depend on EPS density).
3. the stress-strain behavior immediately after yielding is linear and work hardening in nature.

2.2.2 Average values of compressive strength, yield stress and Young’s moduli

The average values of compressive strength, yield stress, initial tangent and plastic tangent Young’s modulus for each density are shown in Table 1 below.

<table>
<thead>
<tr>
<th>Density ( \rho ) (kg/m(^3))</th>
<th>( \sigma_c ) (kPa)</th>
<th>( \sigma_y ) (kPa)</th>
<th>( E_h ) (MPa)</th>
<th>( E_p ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>44</td>
<td>35</td>
<td>2.13</td>
<td>110.6</td>
</tr>
<tr>
<td>21</td>
<td>91</td>
<td>81</td>
<td>4.62</td>
<td>112.9</td>
</tr>
<tr>
<td>27</td>
<td>135</td>
<td>127</td>
<td>6.46</td>
<td>107.4</td>
</tr>
</tbody>
</table>

It is interesting to note that when these values are compared with typical strength and modulus values obtained for very soft clay on which the EPS geofoams sit (Table 2), it is evident that EPS is neither significantly stronger nor stiffer. However, the main advantage of using EPS fill lies in the fact that its weight is one to two order less so that the loading of EPS embankment on the soft clay is reduced significantly. Part of the engineering solution lies in dispersing the vehicular loading so as not to stress the EPS geofoam in excess of its material strength. Past experience has shown that this can be achieved easily \[2,3\].

<table>
<thead>
<tr>
<th>Unit Weight (kN/m(^3))/Density (kg/m(^3))</th>
<th>Undrained cohesion ( c_u ) (kPa)</th>
<th>Young’s Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.5/1888</td>
<td>25-40(^{[4]})</td>
<td>2-15(^{[5]})</td>
</tr>
</tbody>
</table>

2.2.3 Dependency of strength and Young’s moduli on material density and confining pressure.

The engineering properties of EPS relate closely with the density of the material as is evident from several previous studies \[6,7\]. Test results of compressive strength (\( \sigma_c \)) and yielding stress (\( \sigma_y \)), initial Young’s modulus (\( E_h \)) and plastic tangent modulus (\( E_p \)) are shown in Figures 3 to 5 respectively. The relationships between the aforementioned properties and the measured density is summarised in Table 3 below:

<table>
<thead>
<tr>
<th>Material property</th>
<th>Relationship with density ( \rho ) (kg/m(^3))</th>
<th>Maximum variation from average values</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_c ) (kPa)</td>
<td>( 6.23\rho-36 )</td>
<td>± 3%</td>
</tr>
<tr>
<td>( \sigma_y ) (kPa)</td>
<td>( 6.28\rho-46 )</td>
<td>± 5%</td>
</tr>
<tr>
<td>( E_h ) (MPa)</td>
<td>( 0.3\rho-1.65 )</td>
<td>± 1%</td>
</tr>
<tr>
<td>( E_p ) (kPa)</td>
<td>( 0.13\rho+107.5 )</td>
<td>± 3%</td>
</tr>
</tbody>
</table>
For a given density, the maximum variation of \( \sigma_c \) and \( \sigma_p \), \( E_d \) and \( E_p \) were computed based on the difference between the values calculated from the relationships in Table 3, and the average values, shown in Table 1. As shown in Table 3, the variations were relatively small (±5% and less) indicating that the quality of the material is fairly consistent. The difference between compressive strength and yielding stress is about 10 kPa. Test results clearly show that, as the density of the EPS material increase, the strength parameters and the initial tangential modulus increase as well. Interestingly, it is also observed that density has negligible effect on the value of the tangent modulus, \( E_p \), which can be ignored for practical purposes.

![Figure 3. Variations of compressive strength with density](image1)

![Figure 4. Initial Young's modulus with density](image2)

The strength and deformation of particulate material, such as sand, depend on the confining pressure. It was initially thought that since EPS geofoam is made from beads, its engineering properties may be strongly affected by the confining pressure. The study shows that confining pressure has only a small effect on the compressive stress, yield stress and initial tangential modulus. However, the effect of confining stress on the tangential plastic modulus is stronger (Figure 5). The influence of the confining pressure on the engineering properties is summarized in Table 4.

### Table 4: Dependency of material properties on confining pressure, \( \sigma_3 \) (kPa)

<table>
<thead>
<tr>
<th>( \rho ) (kg/m(^3))</th>
<th>( \sigma_c ) (kPa)</th>
<th>( \sigma_s ) (kPa)</th>
<th>( E_d ) (MPa)</th>
<th>( E_p ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>0.12( \sigma_3 )+42</td>
<td>0.02( \sigma_3 )+34</td>
<td>0.006( \sigma_3 )+2.03</td>
<td>2.28( \sigma_3 )+81.7</td>
</tr>
<tr>
<td>21</td>
<td>0.17( \sigma_3 )+88</td>
<td>-0.2( \sigma_3 )+84</td>
<td>0.007( \sigma_3 )+4.42</td>
<td>4.01( \sigma_3 )+54.6</td>
</tr>
<tr>
<td>27</td>
<td>-0.03( \sigma_3 )+135</td>
<td>-0.3( \sigma_3 )+130</td>
<td>0.018( \sigma_3 )+6.23</td>
<td>3.61( \sigma_3 )+62.9</td>
</tr>
</tbody>
</table>

![Figure 5. Plastic modulus with confining pressure](image3)

![Figure 6. Volume strain vs. axial strain](image4)

#### 2.2.4 Poisson’s ratio, \( \nu \)

Poisson’s ratio of the EPS material was evaluated by measuring the volume change of EPS samples during the “undrained” triaxial tests. The volume change of the EPS sample was determined by measuring the amount of water flowing into or out of the triaxial chamber during the test. Figure 6 shows that the volume strain, \( \varepsilon_v \), is linearly decreasing with increasing axial strain, \( \varepsilon_a \). The sign convention used is: the volumetric strain is positive if the volume is increased. For triaxial test, the volumetric strain defined as:

\[ \varepsilon_v = \frac{V_f - V_0}{V_0} \]
volumetric strain, $\varepsilon_v = \varepsilon_a + 2\varepsilon_r$  \hfill (1)

where $\varepsilon_r$ is lateral strain. Thus,

lateral strain, $\varepsilon_r = \frac{1}{2}(\varepsilon_v - \varepsilon_a)$  \hfill (2)

which leads to,

Poisson's ratio, $\nu = \frac{\varepsilon_r}{\varepsilon_a} = \frac{(\varepsilon_v - \varepsilon_a)}{2\varepsilon_a}$  \hfill (3)

Figure 7 shows the variation of typical Poisson's ratio during triaxial test. It indicates that the Poisson's ratio of EPS material is much less than that of soil and decreases as the yield point is reached. Table 5 shows Poisson's ratios for density of 21 kg/m$^3$ EPS calculated at 1%, 5% and 10% axial strain. The Poisson’s ratio at failure (10% axial strain) of EPS material varies with confined stress as shown on Figure 8. It appears that there is a possibility of the Poisson’s ratio becoming negative. Negative values of Poisson’s ratio were also reported by Preber et al. [7]

<table>
<thead>
<tr>
<th>Confining Pressure (kPa)</th>
<th>$\nu$, at 1% $\varepsilon_a$</th>
<th>$\nu$, at 5% $\varepsilon_a$</th>
<th>$\nu$, at 10% $\varepsilon_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.09</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>20</td>
<td>0.06</td>
<td>0.00</td>
<td>0.01</td>
</tr>
<tr>
<td>30</td>
<td>-0.11</td>
<td>-0.14</td>
<td>-0.07</td>
</tr>
</tbody>
</table>

Figure 7. Typical Poisson's ratio in triaxial test

Figure 8. Poisson's ratio with confining pressure

2.2.5 Bulk and Shear Moduli

Stress-strain relationships may alternatively be described in terms of bulk and shear moduli. Typical curves obtained from "undrained" and hydrostatic tests are shown in Figures 9 and 10. In the same manner as in the case of Young's modulus, the corresponding initial and plastic tangential bulk and shear moduli ($K_n$, $K_p$ and $G_n$, $G_p$) may be used to help describe the relationships. Average values for the given EPS densities are tabulated below:

<table>
<thead>
<tr>
<th>Density $\rho$ (kg/m$^3$)</th>
<th>$K_n$ (MPa)</th>
<th>$K_p$ (kPa)</th>
<th>$G_n$ (MPa)</th>
<th>$G_p$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>0.61</td>
<td>185</td>
<td>1.22</td>
<td>75</td>
</tr>
<tr>
<td>21</td>
<td>1.13</td>
<td>182</td>
<td>2.12</td>
<td>77</td>
</tr>
<tr>
<td>27</td>
<td>1.62</td>
<td>176</td>
<td>2.92</td>
<td>74</td>
</tr>
</tbody>
</table>

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2.2.6 Yield Surface

The results presented earlier should provide information to proceed with linear elastic or even nonlinear analysis of EPS deformation and behavior in field applications. In most geotechnical applications, the stress range is compressive and should probably be within the elastic range (below yield stress) and so the data presented previously would be useful. For more extensive analysis, however, particularly where strength calculations are involved, more complex elasto-plastic relationships or similar types of models are required. Generally, it would be prudent to adopt the simpler approach in the first attempt before proceeding to the more complex. While there is evidence of a small amount of post yield work hardening, the simpler perfectly-plastic representation of EPS yielding would probably suffice in most analysis.

Based on the results of triaxial testing, a Drucker-Prager perfectly plastic failure envelope is presented. A Drucker-Prager material yields when:

\[ f = \sqrt{J_2} - \alpha I_1 - \kappa = 0 \]  

(4)

where,

\[ \sqrt{J_2} = \frac{(\sigma_1 - \sigma_3)}{\sqrt{3}} \]

\[ I_1 = (\sigma_1 + 2\sigma_3) \]

\[ \alpha = \frac{2 \sin \phi}{(3 - \sin \phi) \sqrt{3}} \]

\[ \kappa = \frac{6 c \cos \phi}{(3 - \sin \phi) \sqrt{3}} \]

\( \alpha \) and \( \kappa \) are material constants, related to the frictional strength \( \phi \) and cohesive strengths \( c \) of the material, respectively. Figure 11 shows the yield surface in the \( \sqrt{J_2} - I_1 / 3 \) space. The shear strength parameters are shown in Table 7. Given the small values of \( \phi \), EPS is found to be only a slightly frictional material.
Table 7: Shear strength parameters from triaxial test

<table>
<thead>
<tr>
<th>ρ  (kg/m³)</th>
<th>σ₀</th>
<th>k</th>
<th>φ</th>
<th>c (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>0.07</td>
<td>23.0</td>
<td>3.6</td>
<td>19.5</td>
</tr>
<tr>
<td>21</td>
<td>0.11</td>
<td>47.4</td>
<td>5.4</td>
<td>39.9</td>
</tr>
<tr>
<td>27</td>
<td>0.06</td>
<td>74.4</td>
<td>3.1</td>
<td>63.4</td>
</tr>
</tbody>
</table>

The shear box test was also carried out to determine the shear strength of EPS material. The shear behavior of EPS material under different normal stress condition was studied. The non-linear shear stress-shear strain curves for 21 kg/m³ EPS are shown in Figure 12. Figure 13 shows the relationship between shear strength and normal stress. The results from the shear box test suggest that the failure surface can be represented by a Mohr-Coulomb type of failure criterion as well. Hence the simpler Drucker-Prager model, which neglects the influence of J₂ can be considered as a first step approximation of the well-known Mohr-Coulomb failure surface.

![Shear stress vs. Normal Stress](image1)

**Fig. 13 Shear stress vs. Normal Stress**

![Creep Test on EPS material](image2)

**Figure 14. Creep Test on EPS material**

2.2.7 Creep Test

Figure 14 shows the results of a creep test on a cylindrical specimen of 21 kg/m³ EPS, 50 mm in diameter and 50 mm in height, under an unconfined axial stress of 51 kPa, which is about the elastic stress limit. The initial strain, ε₀ is 0.0089. From data obtained, the total strain (based on the popular power law) is given by:

\[
total\ strain, \ \varepsilon(\%) = \varepsilon₀ + \epsilon_c = 0.89 + 0.119 t^{0.22}
\]

(5)

where the \( \epsilon_c \) is creep strain in %, \( t \) is the time in hours. At the projected rate, a creep strain of less than 0.9% is obtained a year. It would appear that creep effects will not be significant provided the stress levels are not excessive (greater than yield stress).

3. CONCLUSION AND DISCUSSION

The compressive strength, yield stress and Young’s modulus of EPS material are relevant material parameters for elastic analysis and design. These values can be estimated from data given in the tables. In general, the properties (except for plastic modulus) depend on the material density but only weakly dependent on the confining stress. This dependency can probably be ignored for most practical purposes. The plastic modulus, however, has low dependency on the material density but it is influenced by the confining pressure.

The Poisson’s ratio of EPS, within its elastic range, is very small and is sometimes taken to be zero for practical design. The possibility of occurrence of negative Poisson’s ratio suggest that the lateral strain of EPS blocks at the interface with retaining structures may be reduced as vertical strain or loading is increased. To this extent, if the Poisson’s ratio is reduced to negative, it is envisaged that the EPS blocks and retaining wall may eventually lose contact and separate in field applications. Although this has not been confirmed in practice, Refsdal [6] reported that in one case where a small gap was left between the blocks and retaining wall, the original gap was essentially maintained during operation.
Alternative representations of deformational stiffness in terms of bulk and shear moduli are given. The shape of the mean stress vs. volumetric strain and shear stress versus shear strain curves are fundamentally the same as the axial deviator stress vs. axial strain curves, as should be the case. These curves can be defined in terms of the corresponding initial tangent and plastic moduli.

The results from triaxial compression tests have shown that EPS is only a slightly frictional material. Its friction angle, $\phi$, is very small and cohesive strength $c$ is about $\frac{1}{2}$ of the yield stress, $\sigma_y$. If it is assumed that $\phi = 0$, then the yield criterion of EPS material given by Equation (4) simplifies to the von Mises criterion as:

$$f = \sqrt{J_2 - \kappa} = 0$$

The results given herein are largely for time independent behavior, except for the results from creep testing. More testing are necessary before long term deformations of EPS material under different stress path loading can be truly assessed.

4. REFERENCES

BEHAVIOR OF THE EXPANDED POLYSTYRENE (EPS) GEOFOAM ON SOFT SOIL

Yong Zou
B.S., M.S.

A thesis submitted for the degree of Doctor of Philosophy
School of Civic Engineering and Environment
The University of Western Sydney Nepean
January, 2001
PLEASE NOTE

The greatest amount of care has been taken while scanning this thesis,

and the best possible result has been obtained.
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SYNOPSIS

Excessive settlement and foundation instability are some of the main problems commonly encountered in fills and embankment structures built on a soft soil of low bearing capacity. The Expanded Polystyrene (EPS) replacement method, by partially or fully replacing the conventional fill material with extremely lightweight EPS geofoam, may solve these problems. In essence, the method produces a negligibly small increase in stress level and so enables the foundation soil to remain over-consolidated relative to the loading from above.

In this thesis, experimental and theoretical work have been carried out to investigate the behaviour of EPS under various loading conditions in geotechnical applications and the effectiveness of EPS replacement technique has been studied using numerical models. The thesis work is broadly divided into the following sections:

1. A series of laboratory tests were performed on specimens cut from EPS blocks to characterize the material behaviour under various loading conditions. The stress-strain behaviour, strength, stiffness and yielding characteristics were obtained from uniaxial and triaxial compressive tests on EPS specimens. The effects of the confining stress, geometry, strain rate and temperature on the stress-strain behaviour of EPS specimen were evaluated. The test results have formed a comprehensive database useful for the development of constitutive models of the EPS geofoam.

2. The internal shear resistance of EPS geofoam and the interfacial friction between EPS–EPS and EPS–soil interfaces were studied by performing shear box and triaxial tests. The test results indicated that, in geotechnical applications, the shear strength of the EPS material is high relative to its compressive strength, but the joints of the EPS blocks, especially the interface of EPS and soil, may form the plane of weakness for shear failure.

3. The creep behaviours of EPS samples were studied during the creep tests. Creep models were developed for the EPS geofoam under sustained loading in the typical stress range of geotechnical application. The effects of temperature on the creep
characteristics were also investigated.

4. A series of cyclic loading tests were carried out covering a wide range of static and dynamic stress levels. The effect of the stress level on the cyclic behaviour of EPS and resilient stiffness were evaluated. The relationship of load repetition and material fatigue was established, and the limit strain and stress criteria were given in empirical relationships.

5. A scale model test of an EPS pavement embankment with an elaborate instrumentation monitoring system was carried out to study the behaviour of EPS geofoam as a replacement subgrade material under flexible pavement subjected to repeated wheel loading. Test results showed that block size and lateral restraint did not significantly affect the performance of the EPS blocks. However, the study also found that certain care with the pavement design above the EPS backfill is needed to ensure that rutting does not develop into a problem.

6. Based on the experimental test results, a nonlinear variable moduli constitutive model of EPS geofoam has been developed. This model has been incorporated into a finite element program providing an efficient method of analyzing the stress state and the deformation behaviour of the EPS geofoam in geotechnical applications.
PREFACE

The works described in this thesis were carried out by the candidate in the School of Civic Engineering and Environment at the University of Western Sydney during the period of 1997–2001. The candidate was supervised by Dr. Chin Jian Leo (Chair Supervisor), Associate Professor Steven James Riley and Professor Russell Quinlin Bridge (co-supervisors).

The Doctor of Philosophy (PhD) Rule, Policies and Procedures Directory of the University of Western Sydney, requires a candidate's thesis must be the candidate's own account of the work undertaken. The candidate shall also indicate in the thesis the sources of information and the extent to which the candidate has used the work of others. In accordance with this Rule, any information or ideas derived from other sources have been acknowledged in the text. The text is original. The candidate claims the following experimental and analytical portions of this thesis as original:

1. In Chapter 3:
   (i) The planning and conduct of the monotonic and triaxial compression tests on the $\varnothing 50\text{mm} \times 50\text{mm}$ EPS specimens under various conditions including:
       • Specimen densities of 13, 20 and 27 kg/m$^3$,
       • Confining pressure of 0, 5, 10, 15 and 20 kPa,
       • Temperature of 23, 30, 35, 40, 45 and 50 $^\circ$C,
       • Strain rates of 0.5, 1.0, 2.5, 5.0 and 10.0 mm/min, and
       • Various geometry and size.
   (ii) The analysis of the stress–strain characteristics of EPS under various conditions and the influence of these factors on the behaviour of EPS specimen under compressive load.
   (iii) The establishment of the relationships of compressive strength, yield strength and initial elastic modulus of EPS depending on the material density and confining stress based on the experimental results.
   (iv) The development of the method to evaluate the Poisson’s ratio of EPS.
   (v) The introduction of an approximate method to estimate the bulk and shear moduli of EPS based on the experimental results from triaxial test.
2. In Chapter 4:
   (i) The planning and conduct of shear box test on the internal shear resistance of 
       EPS block, and the shear-friction of EPS–EPS and EPS–compacted sand 
       interfaces.
   (ii) The analysis and discussion of the shear characteristics of EPS material and 
       interfaces based on the experimental results.
   (iii) The comparison of the shear strength parameter results from shear box test 
       with those from triaxial test.
3. In Chapter 5:
   (i) The planning and conduct of creep tests under different sustained stress levels
   (ii) The setup of the temperature controlled creep test system and the conduct of 
       the creep tests at elevated temperature.
   (iii) The testing of the different creep models and the selection of the best creep 
       model for describing the creep behaviour of EPS.
   (iv) The analysis of the influence of stress level and temperature on the creep 
       behaviour of EPS.
   (v) The discussion of the stability of EPS under sustained load at different 
       temperatures
4. In Chapter 6:
   (i) The planning and conduct of cyclic loading test on EPS specimen.
   (ii) The investigation of the influence of the static stress and dynamic stress on the 
       permanent and resilient deformation of EPS.
   (iii) The investigation of the influence of the static stress and dynamic stress on the 
       stiffness (resilient modulus) of EPS.
   (iv) Based on the experimental data, the development of the strain, and stress limit 
       criteria for EPS under cyclic loading condition for geotechnical applications.
   (v) The validation and discussion of the creep model of EPS under repeated type 
       loading.
5. In Chapter 7:
   (i) The planning and conduct of scale model pavement test.
   (ii) The investigation of the behaviour of EPS subgrade under different dynamic 
       (driven) loading.
(iii) The analysis of the experimental results and the influence of block size and lateral constrain.

(iv) The comparison of the experimental results of EPS subgrade and compacted sand subgrade.

(v) The discussion of the rutting mechanisms of pavement with EPS subgrade.

6. In Chapter 8:

(i) The development of a semi-analytical EPS-soil model for time-dependent behaviour (consolidation and creep) of EPS embankment on soft soil.

(ii) The analysis of an illustrative example case to show the merits of the semi-analytical EPS-soil model in geotechnical applications.

7. In Chapter 9:

(i) The development of the constitutive model for EPS in geotechnical application based on the experimental results.

(ii) The performance of finite element case analyses for EPS embankment construction.

(iii) The investigation of the stability, deformation (settlement and lateral displacement) and stress state in the embankment and foundation soil and the evaluation of the effectiveness of EPS construction technique.

(iv) The comparison of the EPS geofoam method with other ground improvement techniques and the investigation of the effectiveness of the EPS fill and vertical drains.

Eight published research papers were jointly prepared by the author and Dr. C.J. Leo, Professor J.C. Small and Dr. E.S. Bernard during the period of the candidature based on the experimental and theoretical results. These are submitted in support of this thesis. They are:


ACKNOWLEDGMENT

The work described in this thesis was made possible by the award of a Australian Research Council research scholarship administered by University of Western Sydney Nepean, which is gratefully acknowledged.

I wish to express my sincere gratitude and thanks to my supervisor, Dr. Chin Jian Leo, for his unforgettable enthusiastic supervision, encouragement, patience, guidance and support throughout the course of this research. I am greatly indebted to my co-supervisors Associate Professor Steven James Riley and Professor Russell Quinlin Bridge, for their helpful advice, suggestion, support and guidance for the research work in this thesis.

I appreciate the help which I have received from the academic staff of the Department of Civic Engineering and Environment at the University of Western Sydney, especially Dr. E.S. Bernard for many useful discussions and helpful advice for the experimental work presented in this thesis. I gratefully acknowledge the assistance of the technical staff and administration staff of the Department of Civic Engineering and Environment at the University of Western Sydney, especially Mr. Andrew Townsend, Ms. Tanya Hobson, Mr. Bert Finnigan, Mr. Robert Caley and Mr. Mick Greentree.

The model pavement embankment test of this research was made possible with the help of the academic and technical staff of the Center for Geotechnical Research at the University of Sydney, especially Professor John C. Small, Dr. Tim Hull and Mr. Ross Barker, who are gratefully acknowledged.

Finally, I would like to thank my parents, family and especially my wife Tong Chen and our friends for their support and understanding during the course of my candidature.
NOTATIONS

All notations and symbols have been defined where they first appear in the text. For convenience, the most frequently used symbols and their meanings are listed below.

A  area

a, b  experimental parameters of variable moduli model

a'  intercept of t's' failure envelope

CBR  California Bearing Ratio

c  cohesion, Mohr-Coulomb shear strength parameter

Cv  compressive coefficient

Di  tangential material stiffness matrix

d_{EPS}  cumulative permanent deformation of EPS subgrade after 1000 cycles (μm)

d_{sand}  cumulative permanent deformation of sand subgrade after 3000 cycles (μm)

E  Young's modulus

E_{1}  spring constant 1 of viscoelastic mechanical models

E_{2}  spring constant 2 of viscoelastic mechanical models

E_{i}  initial tangent Young's modulus

E_{p}  plastic tangent Young's modulus

E_{r}  resilient modulus

F  wheel load in Newton

G  shear modulus

G_{i}, (G_{ii})  initial shear modulus
\( G_p (G_{tp}) \) plastic shear modulus

\( G_t \) tangential shear modulus

\( G_{tu} \) reloading tangential shear modulus

\( I_1 = \sigma_{ii} \), first stress invariant

\( J_2 = \frac{1}{2} (I_1^2 - \sigma_{ij}\sigma_{ij}), \) second stress invariant

\( J_{2y} \) reference (yield) point of \( J_2 \)

\( J(t) \) time-dependent creep compliance

\( K \) bulk modulus

\( K_i (K_{ii}) \) initial bulk modulus

\( K_p (K_{ip}) \) plastic bulk modulus

\( K_t \) tangential bulk modulus

\( K_{tu} \) reloading tangential bulk modulus

\( K_v, K_y \) coefficient of vertical permeability

\( K_s \) coefficient of horizontal permeability

\( L \) compression of springs (mm)

\( m \) dimensionless experimental creep parameter

\( m_F \) Findley's creep coefficient

\( N \) number of cycles in repeated loading test

\( n \) dimensionless experimental creep parameter

\( n_F \) Findley's creep power parameter

\( p \) mean stress = \( \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3) \)

\( p_y \) reference meanstress

\( q \) deviator stress = \( \frac{1}{2} (\sigma_1 - \sigma_3) \)

\( r^2 \) coefficient of determination of regression
\( S \) stress ratio

\( s' \) mean stress in plane strain condition = \( \frac{1}{2} (\sigma_1 + \sigma_3) \)

\( t \) time in creep test,

\( t' \) maximum shear stress in plane strain condition = \( \frac{1}{2} (\sigma_1 - \sigma_3) \)

\( T \) temperature, °C

\( u \) excess pore pressure

\( \alpha \) slope of \( t' : s' \) failure envelope (\( \tan \alpha = \sin \phi \))

\( \varepsilon \) strain vector

\( \varepsilon (\varepsilon_0) \) total strain

\( \varepsilon_0 \) initial strain

\( \varepsilon_a \) axial strain

\( \varepsilon_c \) creep strain

\( \varepsilon_l \) lateral strain

\( \varepsilon_{n,n} \) resilient strain of \( n \)th cycle under repeated load

\( \varepsilon_{p,n}, \varepsilon_{p,n} \) plastic strain of \( n \)th cycle under repeated load

\( \varepsilon_v \) volumetric strain

\( \phi \) angle of friction, Mohr-Coulomb shear strength parameter

\( \gamma_{bulk} \) bulk unit weight (kN/m³)

\( \gamma_w \) unit weight of fluid (kN/m³)

\( \eta_1 \) viscous constant 1 of viscoelastic mechanical models

\( \eta_2 \) viscous constant 2 of viscoelastic mechanical models

\( \nu \) Poisson's ratio

\( \rho \) density, kg/m³
\( \sigma \)  stress vector

\( \sigma_0 \)  constant stress

\( \sigma_i \)  maximum principal stress

\( \sigma_3 \)  minimum principal stress, confining stress

\( \sigma' \)  effective stress

\( \sigma_c \)  compressive strength

\( \sigma_{d(n)} \)  dynamic stress

\( \sigma_{ij} \)  stress tensor

\( \sigma_n \)  normal stress, maximum cyclic stress

\( \sigma_s \)  static stress

\( \sigma_y \)  yield stress

\( \tau \)  shear stress