

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL

**GEOFOAM BLOCK APPLICATION FOR BURIED
INFRASTRUCTURE**



ZAHRAA MOHAMMED JWAID ALMUSAWI

A THESIS OF MASTER OF SCIENCE
DEPARTMENT OF CIVIL ENGINEERING

ADVISOR: PROF. DR. ABDULLAH TOLGA ÖZER

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GEBZE TEKNİK ÜNİVERSİTESİ
LİSANSÜSTÜ EĞİTİM ENSTİTÜSÜ

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GEOFOAM BLOK UYGULAMALARI

ZAHRAA MOHAMMED JWAID ALMUSAWI

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ABSTRACT

Expanded polystyrene (EPS) geofoam (geofoam block), was used as a geotechnical material in infrastructure projects since the 1960s. The unit weight of geofoam blocks approximately less than 10% of the other lightweight fill alternatives and about 1% of the traditional compacted earth fill. Since geofoam is a lightweight fill, it lessens the loads when placed atop buried infrastructures. Geofoam block is meant to address specific technical challenges instead of being used as a general substitute for traditional earth fill. Therefore, it is considered as a type of geotechnology.

When it comes to below-ground applications, EPS offers two key benefits when compared to that of conventional earth fill materials. Its low mass density, which usually ranges from around 12 kg/m^3 to 46 kg/m^3 , is considered as its primary benefit. Therefore, geofoam blocks can be applied to lessen the horizontal and vertical loads for buried infrastructure. Its high strength to density ratio, which sets it apart from other lightweight fill materials, can be considered its second major benefit.

There are many geofoam block placement configurations that may be utilized to safeguard underground pipes and utility corridors that transverse beneath or through roads and railroads. The EPS embankment system, post and beam EPS system, slot-trench cover system with EPS block, and imperfect ditch are the possible block placement used in designs. The observed stress is significantly reduced by the geofoam block that is placed above the pipe, particularly at the crown and invert.

A historical review of the existing literature on the effectiveness of EPS to reduce the stress on buried pipes using the imperfect trench method was evaluated in this study. A comprehensive finite element modelling effort was conducted to evaluate the function of adding geofoam in reducing the soil pressure that is applied to the underground pipes. The performance of buried pipeline under service loads based on a well-known case study is calibrated. Then, comprehensive numerical analyses effort was conducted to investigate the impact of backfill height, geofoam thickness, and width on the effectiveness of reducing soil pressures. Therefore, outcomes of this study aimed to provide numerical modeling approach to utilize economical solutions for buried pipe designs. The calibration model mimicked the field condition, which in turn helped how to model geofoam blocks. The most significant information gleaned from the study is listed as: increasing thickness of geofoam blocks increases positive arching above the pipe and decreasing stress transferred to the pipe. The optimal ratio for geofoam block width to pipe diameter was found to be 1.5, as increasing the width decreases pressures in spring line and crown.

Keywords: Geofoam block, Buried pipes, Imperfect trench method, Pressure reduction.

ÖZET

1960'lı yıllardan itibaren altyapı projelerinde geoteknik malzeme olarak geliştirilmiş polistiren (EPS) geofam (geofam blok) kullanılmaktadır. Geofam blokların birim hacim ağırlığı, diğer hafif dolgu alternatiflerinin yaklaşık %10'undan az ve geleneksel sıkıştırılmış toprak dolgunun ise yaklaşık %1'dir. Geofam hafif bir dolgu olduğundan gömülü altyapıların üzerine yerleştirildiğinde yükleri azaltır. Geofam blok, geleneksel toprak dolgunun alternatifi olarak kullanılmak yerine, belirli teknik zorlukların üstesinden gelmeyi amaçlamaktadır. Bu nedenle bir tür geoteknoloji olarak kabul edilir.

Gömülü uygulamalar söz konusu olduğunda EPS, geleneksel toprak dolgu malzemeleriyle karşılaştırıldığında iki önemli avantaj sunar. Genellikle 12 kg/m^3 ila 46 kg/m^3 arasında değişen düşük birim hacim ağırlığı birincil faydasıdır. Bu nedenle gömülü altyapının yatay ve düşey yüklerini azaltmak için geofam bloklar uygulanabilir. Onu diğer hafif dolgu malzemelerinden ayıran yüksek mukavemet/yoğunluk oranı ise ikinci büyük avantajı olarak değerlendirilebilir.

Yolların, demiryollarının altından veya içerisinden geçen yer altı borularını ve hizmet koridorlarını korumak için kullanılabilecek birçok geofam blok yerleşim planı vardır. EPS dolgu sistemi, direk ve kirişli EPS sistemi, EPS bloklu yarık hendek kapatma sistemi ve kusurlu hendek tasarımlarda kullanılan olası blok yerleşimleridir. Boru hatlarına aktarılan toprak yükleri, borunun üzerine, özellikle tepe kısımlara yerleştirilen geofam blok sayesinde önemli ölçüde azaltılır.

Bu çalışmada, kusurlu hendek yöntemi kullanılarak gömülü borular üzerindeki gerilmeleri azaltmak için EPS'nin etkinliğine ilişkin mevcut literatürün tarihsel bir incelemesi değerlendirilmiştir. Yeraltı borularına uygulanan toprak basıncını düşürmede geofam işlevini değerlendirmek için kapsamlı bir sonlu eleman modelleme çalışması yapılmıştır. İyi bilinen bir vaka analizi çalışmasına dayalı olarak gömülü boru hattının servis yükleri altındaki performansı kalibre edilmiştir. Daha sonra dolgu yüksekliğinin, geofam kalınlığının ve genişliğinin toprak basınçlarını azaltma etkinliği üzerindeki etkisini araştırmak için kapsamlı sayısal analiz çalışmaları yürütülmüştür. Bu çalışmanın sonuçları, gömülü boru tasarımlarında ekonomik çözümlerden faydalanmak için sayısal modelleme yaklaşımı sağlamayı amaçlamıştır. Kalibrasyon modeli saha durumunu taklit ederek geofam blokların nümerik modellenmesi konusunda yardımcı olmuştur. Geofam blokların kalınlığının artması boru üzerindeki pozitif kemerlenmeyi artırarak ve boruya aktarılan gerilmeyi azaltmıştır, geofam blok genişliğinin boru çapına oranı için en uygun oran 1,5 olarak belirlenmiştir.

Anahtar Kelimeler: Geofam blok, Gömülü borular, Kusurlu hendek yöntemi, Basınç azaltımı.

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TABLE OF CONTENTS

	<u>Page</u>
ABSTRACT	v
ÖZET	vi
ACKNOWLEDGEMENTS	vii
TABLE OF CONTENTS	viii
LIST OF SYMBOLS AND ABBREVIATIONS	x
LIST OF FIGURES	xi
LIST OF TABLES	xiii
1. INTRODUCTION	1
2. EPS GEOFOAM	4
2.1. Eps geofoam physical properties	6
2.1.1. Density	6
2.1.2. Compressive strength and stress strain curve	6
2.1.3. Poisson's ratio	7
2.1.4. Cycle loading	7
2.1.5. Creep	8
2.1.6. Shear strength	8
2.1.7. Tension	9
2.1.8. Thermal resistance	10
2.1.9. Absorption	11
2.1.10. Interface friction	12
2.2. Eps geofoam application	12
2.2.1. Lightweight fill	12
2.2.2. Slope stabilization	13
2.2.3. Retaining wall structures	14
2.2.4. Widening embankment	15
2.2.5. Railroad embankment	16
2.2.6. Bridge support	17
2.2.7. Culvert and buried pipelines	19
2.2.8. Theater and stadium seating	20
2.2.9. Airport runway and taxiway embankment	20
2.2.10. Levees	21
2.2.11. Rock fall/impact protection	22
2.2.12. Floating	23
2.2.13. Noise and vibration damping	24
2.2.14. Seismic application	24
3. USING EPS GEOFOAM AS A COMPRESSIBLE INCLUSION	25
3.1. Compressible inclusion function	25
3.2. Applications	26
3.3. Design and analysis for compressible inclusion	28
3.4. Pipeline protection using eps geofoam	28

3.5. Protection systems	30
3.5.1. Embankment system	30
3.5.2. Slot trench cover system	31
3.5.3. Geofoam post and beam cover system	32
3.5.4. Imperfect trench method	33
4. CASE STUDY	37
4.1. Prototype case	47
4.2. Experimental results for deformation	53
5. NUMERICAL ANALYSIS AND RESULTS	55
5.1. Field installation	55
5.2. Numerical modeling for calibration	56
5.2.1. Results of verification	63
5.3. Parametric numerical analysis	64
5.3.1. Results and discussion	66
5.3.1.1. Effect of the width	66
5.3.1.2. Effect of the Eps thickness and backfill height	69
5.3.1.3. Effect of unit weight	71
6. CONCLUSIONS	73
REFERENCES	75
BIOGRAPHY	81
PUBLICATIONS AND PRESENTATIONS FROM THE THESIS	82

LIST OF SYMBOLS AND ABBREVIATIONS

τ_e	: External shear resistance at the EPS interface (kPa)
σ_n	: Normal stress at the EPS interface (kPa)
δ	: EPS interface friction angle (degrees)
ε	: Modulus of elasticity
E_s	: Modulus of elasticity
EPS	: Permanent ground displacement.
PGD	: Kilometer
R	: Material's thermal resistance
ITI	: Imperfect trench installation.
Rms	: Reduction rate of maximum wall stress.

LIST OF FIGURES

	<u>Page</u>
Figure 2.1: Production procedure of EPS geofoam.	4
Figure 2.2: Using geofoam blocks in roadway widening application.	5
Figure 2.3: Construction of embankment around corrugated steel tunnel.	5
Figure 2.4: Stress-strain relationship for EPS geofoam.	7
Figure 2.5: Tensile strength of EPS geofoam (after BASF, CORP., 1997)	10
Figure 2.6: ASTM C 578 EPS R-Values.	11
Figure 2.7: A road embankment cross section including Geofoam blocks.	13
Figure 2.8: Slope Stabilization Utilizing EPS Geofoam.	14
Figure 2.9: Reducing Lateral Stress with EPS Geofoam.	15
Figure 2.10: Road widening application using Geofoam blocks.	16
Figure 2.11: Pavement insulation utilizing CPS geofoam.	17
Figure 2.12: Typical underfill bridge utilizing EPS geofoam for structural support.	18
Figure 2.13: Typical bridge abutment with geofoam backfill.	18
Figure 2.14: General techniques for protection pipelines and culverts.	19
Figure 2.15: Stair-step geofoam block placement for seating arrangements.	20
Figure 2.16: Conceptual geofoam block placement in an airport.	21
Figure 2.17: Levee repair using EPS geofoam.	22
Figure 2.18: Geofoam blocks applied for energy absorption on rock fall protection tunnel in Turkey.	23
Figure 2.19: Floating helipad supported by geofoam blocks.	23
Figure 2.20: Floating garden on one of the canals in Amsterdam.	24
Figure 3.1a: Categories of compressible inclusion applications.	27
Figure 3.1b: Categories of compressible inclusion applications.	27
Figure 3.2: EPS geofoam embankment constructed over culvert system.	30
Figure 3.3: Lightweight embankment built over a culvert or pipe.	31
Figure 3.4: EPS slot-trench lightweight cover system.	32
Figure 3.5: Head space vacuum in an EPS post and beam system.	33
Figure 3.6: An improper trench installation is a common installation.	34
Figure 3.7: Notation about soft zone geometry and faulty trench deployments.	35
Figure 3.9: Notes on faulty soft zone geometry and trench installations.	36
Figure 4.1: Optimization process of soft zone geometry soft zone geometry.	37
Figure 4.2: Effects of interface properties in ITI.	38
Figure 4.3: Imperfect trench installations versus embankment installation.	40
Figure 4.4: Schematic diagram of FE Models and Valsangkar's field Instrumentation.	40
Figure 4.5: AASHTO standard embankment installations.	42
Figure 4.6: Pronographic trench placement in a finite-element schematic model.	43
Figure 4.7: Experimental setup: (a) Schematic test configuration; (b) view of the test box.	44
Figure 4.8: View of the section of the numerical modelling of the geocomposite induced trench.	45
Figure 4.9: The experimental model Imperfect ditch method.	45
Figure 4.10: Numerical model in case of EPS block Imperfect ditch method.	46
Figure 4.11: 3D FE model of the pipe-fly ash system for 2D burial depth.	48

Figure 4.12: Particle size distribution and standard proctor curve For backfill material.	49
Figure 4.13: Geomtry of instalation of instromentation.	51
Figure 4.14: Measured vertical earth pressure above the pipe.	51
Figure 4.15: Messured vertical earth pressure on fill.	52
Figure 4.16: Messured horizontal earth pressure on the pipe.	52
Figure 4.17: Stress-strain curve for EPS based on full-scale test.	53
Figure 4.18: Deformation of the EPS.	54
Figure 5.1: Locations of pressure cells and EPS geofoam above the pipe.	55
Figure 5.2: A Cross-sectional view of PLAXIS geometry.	57
Figure 5.3: A Cross-sectional view of mesh generation.	58
Figure 5.4: Stress–strain behavior of an EPS.	60
Figure 5.5: PLAXIS 2D the model after calculations.	63
Figure 5.6: Calibration of the numerical analysis's field installation outcome.	64
Figure 5.7: Effect of increasing the width of the EPS at different densities.	66
Figure 5.8: Effect of increasing the width of the EPS at Crown for density 20 (kN/m ³).	67
Figure 5.9: Effect of increasing the width of the EPS at spring line For density 20 (kN/m ³).	67
Figure 5.10: Earth pressure reduction factor for deffrent W/D.	68
Figure 5.11: Reduction factor for different W/D ratios for T = 0.5 m at the crown.	68
Figure 5.12: Geofoam thickness effectiveness at the crown area.	69
Figure 5.13: Geofoam thickness effectiveness at the spring line area.	70
Figure 5.14: Reduction factor for 0.5m and 1m thikness for W/D = 1.5 crown area.	71
Figure 5.15: Reduction factor for 0.5m and 1m thikness for W/D = 1.5 spring line.	
Figure 5.16: Reduction factor of (16, 20 and 24) kg/m ³ at crown area.	72
Figure 5.17: Reduction factor of (16, 20 and 24) kg/m ³ at spring line.	72

LIST OF TABLES

	<u>Page</u>
Table 3.1: Physical Property Requirements.	6
Table 3.2: Volume of Absorption of Water, Percent	12
Table 5.1: Construction stages of numerical modeling.	59
Table 5.2: The model parameters and material models for EPS.	61
Table 5.3: Input parameters for calibration in PLAXIS 2D.	62
Table 5.4: Input parameters for Reinforced concrete pipe In PLAXIS 2D.	62
Table 5.5: Variables in the models.	65

1. INTRODUCTION

Expanded polystyrene (EPS) block (geofoam block) is a type of closed-cellular geosynthetic used in both above and belowground applications in geotechnical and civil engineering. It is manufactured in block format by heating expandable polystyrene beads with steam and then pressing them into molds [Horvath, 2010]. Geofoam blocks are primarily preferred to use in construction of highway embankments where settlement and bearing capacities are concern. In addition, it is also used in embankment construction for roadway widening, lightweight embankment for bridge approach embankments, and railway embankments. Geofoam blocks are also used in levees, airports, runways, taxiways, landscaping, green roof applications, slope stabilization, stadium and theater seating, pipeline, culvert, and subterranean infrastructure protection, compensating foundations, and other specific applications [Stark et al., 2012]. EPS geofoam weighs less than 10% of other lightweight fill choices and about 1% of soil. Geofoam lessens the loads placed on nearby and underlying soils and buildings since it is a lightweight fill. Geofoam is designed to address technical problems rather than serve as a generic substitute for soil fill. Therefore, it is considered a type of geotechnology.

EPS has two main advantages compared to that of traditional earthen materials when used in underground applications. The first advantage is its very low mass density, which typically ranges from about 12 kg/m^3 to 46 kg/m^3 . This advantage can be used to reduce the vertical and horizontal loads on buried structures. The second advantage is its relatively uniform compressibility, which can be used as a “compressible inclusion” in geosystems undergoing static, monotonic, and cyclic loadings [Horvath, 2005]. Due to its lower mass density compared to that of conventional backfill materials and very high compressibility, EPS geofoam has been utilized to build infrastructure that is resistant to earthquakes in seismically active areas. A 33 to 60 percent reduction was reported in the lateral soil-pipe forces when geofoam was used as a lightweight trench backfill cover for pipe undergoing horizontal displacement [Yoshika and Sakanoue 2003]. To prevent deformations or ruptures, various EPS geofoam cover systems were proposed [Bartlett et al., 2015]. The embankment Configuration, imperfect trench technique, a slot-trench cover system with an EPS

block put in the slot, and post and beam EPS system are listed as types of cover systems [Bartlett et al., 2015]. Steel pipes buried underground are protected against faulting and long-term ground deformation by lightweight cover systems made of EPS geofoam blocks [Marston, 1922]. Based on an experimental investigation, Kim et al. (2010) showed a 31%–36% decrease in vertical pressure. According to Kim et al. (2010), a single layer of EPS decreased pressure by 5%, whereas a double layer lowered pressure by 37%.

Hussein (2015) indicated that when geofoam block installed atop the pipe the observed stress was significantly decreases, particularly at the crown and invert. According to parametric tests, the maximum wall stress can be lowered by as much as 69% with the suggested imperfect trench placement [Kang et al., 2007].

Tarek (2004) indicated that the EPS30 imperfect ditch technique reduces deformations in both the vertical and horizontal directions by about 0 and 37%, respectively. EPS15 reduces deformations in both the vertical and horizontal directions by around 50% and 60%, respectively. Because EPS30 has a lower compressibility than EPS15, it is more effective in cases of surface displacement. The analysis conducted by [Kang et al., 2007] showed that the ideal soft zone form that has been suggested is very effective in reducing the earth pressure on corrugated steel pipes, both vertically and laterally. The results of the calculations showed that the modulus of elasticity of the lightweight material had an impact on the rates of reduction for deflections, maximum wall stresses, and arching factors. Another study by [Kang et al., 2007] found that backfill height, as well as bedding and sidefill installation processes, impact vertical arching parameters for buried concrete pipes. And moreover, the modulus of elasticity E_s of lightweight materials in the soft zone was discovered to be the single most important factor impacting the earth load reduction rate in the imperfect trench installation.

Furthermore, [Beju, 2017] indicated that the reduction of vertical pressure and strain in pipes is affected by various factors, such as location, density, shape, thickness, width, and the number of EPS layers. The development of the positive arching above the pipe was significantly influenced by the density of the geofoam, as demonstrated by the results within the investigated parameters. It is discovered that the ideal spacing between the EPS block and the pipe is 20% of the pipe's diameter or less. In order to Lessen the vertical pressure and strain on the pipe, the EPS block performed best when it was positioned exactly above the pipe or at 0.2D. According to the test results, the

thickness of the EPS affects how much the pipe's vertical pressure is reduced. For situations with two layers of EPS geofoam with 40 mm thickness, the ideal width was determined to be around 1.5D.

The goal of this study is to provide numerical modelling to investigate the effect of backfill height, geofoam density, thickness and width on the effectiveness of reducing soil pressures.



2. EPS BLOCK (GEOFOAM BLOCK)

Beads of polystyrene resin with a hydrocarbon blowing agent are first subjected to a steam treatment during manufacture. Pre-puff beads are produced when the blowing agent causes the beads to expand and the polymer softens. The beads are then placed into large rectangular block molds after they have already expanded. Once the molds are filled with steam, the beads expand even further and fuse together to produce the final product.

The schematic diagram of the EPS geof foam production process is shown in **Figure 2.1**.

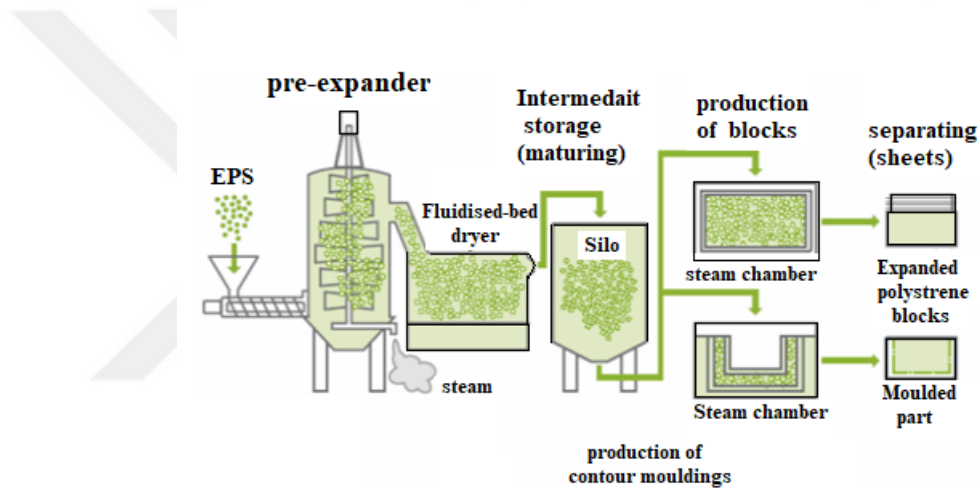


Figure 2.1: Production procedure of EPS geof foam [Elragi, 2000].

In order to satisfy the demands for particular projects, geof foam is created in blocks that may be sliced into a variety of sizes, shapes, and compressive resistances. It can be manufactured as a designed product to have the necessary compressive resistance.

As part of an experiment to ascertain geof foam's feasibility as a pavement insulator, the material was initially employed in the construction of a pavement in Germany in the 1950s. Despite its effectiveness in this use, EPS geof foam was only used as pavement insulation in highlands and hilly regions where harsh winters would necessitate the employment of frost protection measures [Beinbrech and Hillmann 1997]. Globally, EPS-geof foam has been widely used. Norway, the Netherlands, the US, Japan and Germany are a few of the nations that have employed EPS-geof foam. In Europe, EPS has been utilized in highway constructions since the early 1970s. In Norway, the first Road insulating project utilized EPS-geof foam in 1965, and the first project to build a

road embankment used it in 1972. Some of the typical geofoam embankment applications were provided in **Figures 2.2 and 2.3.**



Figure 2.2: Using geofoam blocks in roadway widening application, Japan [Kawashima et al. 2001].



Figure 2.3: Construction of embankment around corrugated steel tunnel [Aaboe et al., 2019].

2.1. EPS Geofoam Physical and Mechanical Properties

2.1.1. Density

EPS has a very low density. Because its density is just 1/100th that of conventional fill materials, it is an excellent lightweight fill material [Arellano, 2005]. It's interesting to note that expanded polystyrene's engineering characteristics are inversely proportional to density.

Young's modulus, compressive strength, and creep behavior are covered in this section. The cost of producing an EPS geofoam block is usually directly proportional to its density. Non-mechanical characteristics like insulating coefficients are also impacted by density. EPS blocks are produced with densities ranging from 12 to 46 kg/m³ [ASTM D4486].

2.1.2. Compressive Strength and Stress Strain Curve

Compressive strength is an essential property of EPS as it is the main means of loading in geotechnical applications [Stark 2004]. Remarkably, the ASTM standards require that the compressive stress at a 10% strain be used to compute the compressive strength of EPS (Table 2.1 displays typical results).

Table 2.1: [ASTM D6817] Minimum physical and mechanical property requirements of Geofoam.

Property	EPS12	EPS15	EPS19	EPS22	EPS29	EPS39	EPS46
Density (kg/m ³)	11.2	14.4	18.4	21.6	28.8	38.4	45.7
Compressive Resistance at 1% strain (kPa)	15	25	40	50	75	103	128
Compressive Resistance at 5% strain (kPa)	35	55	90	115	170	241	300
Compressive Resistance at 10% strain (kPa)	40	70	110	135	200	276	345

The stress-strain characteristics of geofoam block, which is linear elastic up to a strain of around 1%, is shown in **Figure 2.4**. The stress at 1% compressive strain is known as the elastic limit stress (σ_e), and it is measured in a standard rapid-loading compression test. Since they exceed the EPS's yield strength and may cause permanent strains, higher compressive stresses, corresponding to 5% or 10% strains, are rarely used to determine the geofoam strength unless in certain compressible applications.

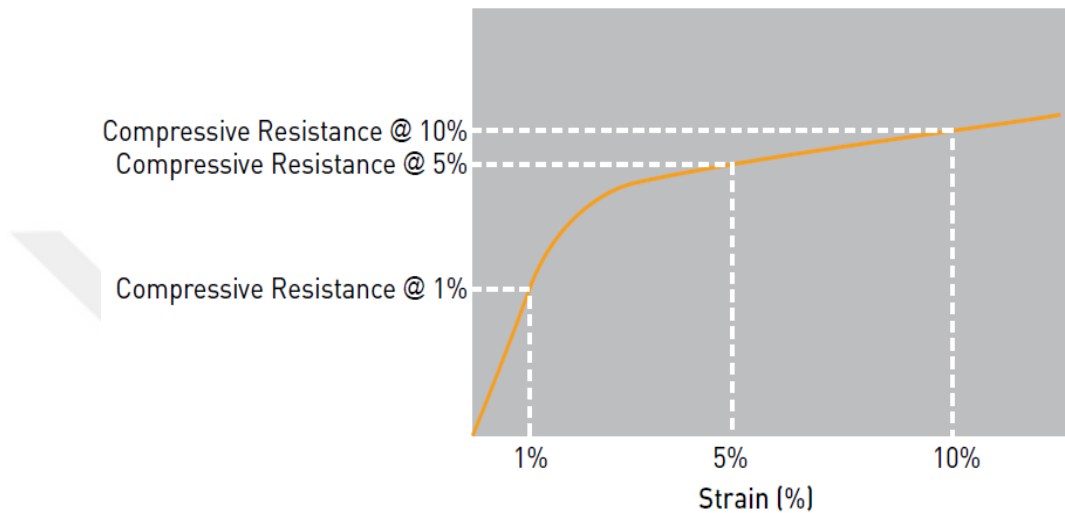


Figure 2.4: Stress-strain relationship for EPS geofoam [Stark et al 2012].

2.1.3. Poisson's Ratio

Poisson's ratio, for a specific applied vertical load on the EPS geofoam mass, indicates the lateral pressure of the geofoam in contact with adjacent structural elements. It is a density-dependent function that, assuming linear stress-strain behavior, rises linearly with block density but drops quickly for greater strains [Santiago, 2014]. For EPS geofoam, Poisson's ratio is roughly equal to 0.12 when the material behaves in the elastic region [Stark, 2012].

2.1.4. Cycle Loading

In a variety of cases, EPS geofoam may be subjected to cyclic loading. This may involve both static and dynamic loading. The bulk of laboratory research and field Studies indicate that the cyclic load behavior of block-molded EPS geofoam is linearly elastic when stresses are less than about 1% [Eriksson and Trank, 1991]. Laboratory

studies were performed on varied-density EPS samples to determine behavior under constant and cyclic loading circumstances. The initial and modulus of elasticity both rise as the unit weight of the EPS material increases. When exposed to cyclic loads, permanent deformation of EPS foam reduced as EPS geofoam density increased. When the stress level exceeded the yield stress, the stress-strain mechanism of EPS hardened without failure [Malai, 2017].

2.1.5. Creep

Creep deformation is a problem in geofoam design. Geofoam is subjected to a constant dead load after construction, causing gaps between blocks to close and creep deformations to occur. The live load caused by traffic and its related deformations is often temporary. The dead loading margin largely determines the post-construction deformations of geofoam in creep. The creep behavior of EPS geofoam is affected by a variety of characteristics, one of which is density.

With increased density, creep deformations decrease [Sun, 1997]. Additionally, it has been noted that creep deformation behavior will be caused after the long-term continuous loading exceeds the 2% compressive strain limit. The compressive stress-compressive strain curve's linear component should be followed while designing in order to limit the creep strains [Horvath, 2010].

2.1.6. Shear strength

Expanded polystyrene shear strength is divided into two categories: internal shear strength and interface shear strength. The former focuses on the sliding resistance of an EPS block, while the latter, often referred to as external shear strength, focuses on the sliding resistance at the interfaces between EPS blocks and other materials [Santiago, 2014].

[Stark et al., 2004] conducted direct shear experiments on geofoam samples of different densities, and the results showed a proportional relationship between the cohesive strength and material density. In the usual stress range of 10–40 kPa, [Özer And Akay, 2016] performed direct shear tests on EPS samples and found that cohesion is the primary factor influencing the shear strength of the tested geofoam blocks, while

adhesion and friction coefficient are the main factors influencing the shear strength at the interface.

Shear strength in an EPS block with a higher density is greater. An EPS geofoam specimen's internal shear strength can also be evaluated by rapid loading. [Stark, 2004]. The following modified Mohr-Coulomb equation, as given in Eq1, may be used to compute the shear strength at the EPS/EPS contact, the first subcategory of external shear strength.

$$\tau_e = \sigma_n \tan (\delta) \quad (2.1)$$

Where

τ_e = external shear strength at the EPS/EPS interface (kPa)

σ_n = normal stress (kPa)

δ = interface friction angle of EPS (degrees)

It is also important to emphasize that, in cases where substantial horizontal loading is anticipated, mechanical connections might need to be made in order to prevent the blocks from sliding laterally [A .Corporation 2011].

2.1.7. Tension

The tensile strength of EPS geofoam is significantly influenced by the level of fusion between the expanded polystyrene beads. Tensile and flexural tests are valuable indicators, as they reflect the effectiveness of bead fusion during EPS manufacturing. Among the various strength measures, tensile strength is particularly sensitive to processing conditions, more so than flexural and shear strength [BASF 1998]. As may be seen in **Figure 2.5**, tension strength increases with density.

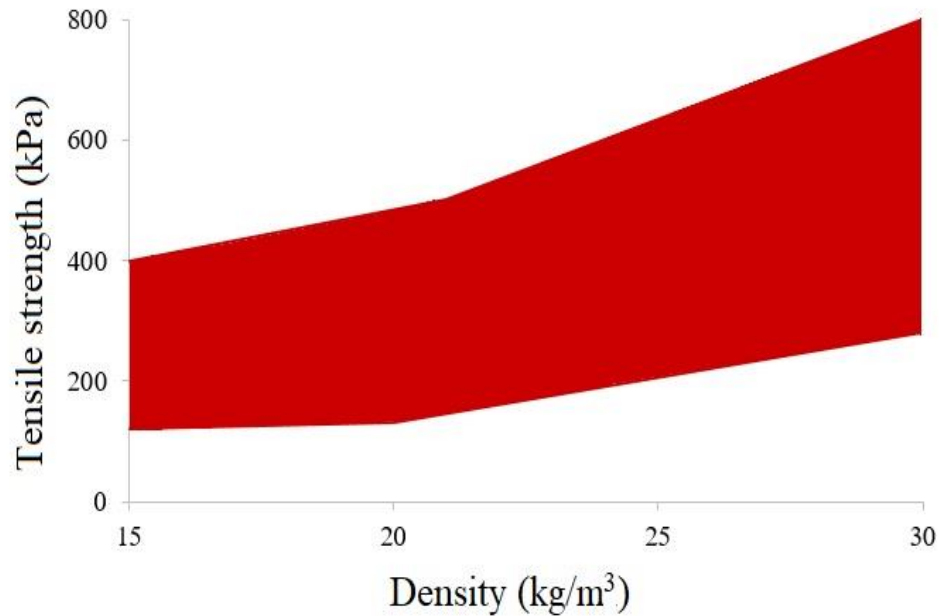


Figure 2.5: EPS geofoam tensile strength [BASF, CORP., 1997].

2.1.8. Thermal Resistance

Approximately 98% of the contents of EPS geofoam is air [BASF Corp., 1997]. Because the huge volume of air is partitioned into smaller quantities confined within cells, convection currents are minimized. Because of the little amount of solid material and the air within smaller cells, EPS geofoam has low thermal conductivity (R). The R-value indicates a material's thermal resistance.

R-values are often substantially less than $0.1 \text{ m}^3 \text{ }^\circ\text{C/W}$ for both normal soil and concrete. **Figure 2.6** illustrates the R-values for EPS geofoam as specified in ASTM C 578; these values are significantly higher than those for normal soil and concrete, ranging from 0.5 to $0.8 \text{ m}^3 \text{ }^\circ\text{C/W}$ for EPS geofoam.

The density of geofoam raises the R-value of the material. As density rises, the curves tend to horizontally level out. A study conducted by [van Drop 1988], the maximum R-value is found in foam with a density of 35 kg/m^3 . The R-value disappears or reduced with moisture absorption, although aging has no discernible influence on the R-value [Huntsman, 1999]. This is due to the fact that EPS's closed cell structure includes only air.

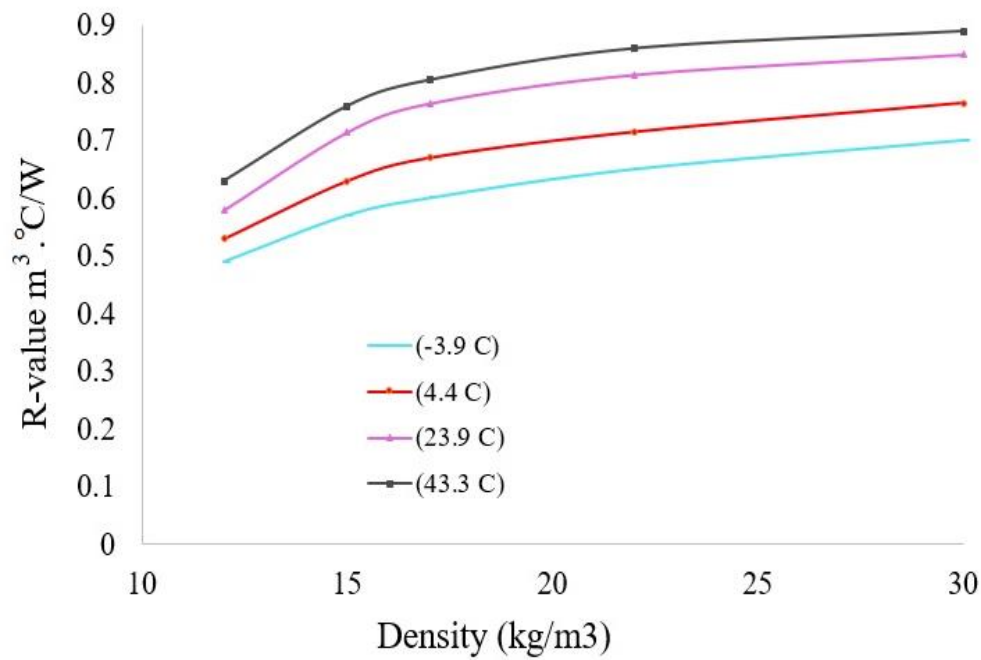


Figure 2.6: ASTM C 578 EPS R-Values [Elragi, 2000].

2.1.9. Absorption

Expanded polystyrene has a low water absorption rate. Though density increases cause a reduction in water absorption, as seen in **Table 2.2** absorption rates is a function of the density, length of moisture exposure, and degree of bead fusion during manufacture. Less moisture is absorbed when there is a higher density, excellent fusion, and a smaller exposure, and the opposite is true. In general, density and thermal conductivity are likely increasing as a result of moisture absorption. Water absorption is decreased in good fusion. In EPS fills below the ground's water table, equilibrium values of 8–9% volume have been discovered after 9–12 years of service [van Dorp, 1988]. However, the mechanical qualities of EPS geofoam are little impacted by the water that it absorbs [BASF, 1998].

Table 2.2: % Volume of water absorption [van Dorp, 1988].

Density kg/m ³	After 7 Days	After 1 year
15	3.0	5.0
20	2.3	4.0
25	2.2	3.8
30	2.0	3.5
35	1.9	3.3

2.1.10. Interface Friction

According to [Negusse et al. 2001; NRRL, 1992], the interface friction between EPS geofoam blocks can be substantial and is similar to the internal friction angle of sand. But for the EPS geofoam to EPS geofoam interface, values of 0.65 have been measured [Nomaguchi, 1996] based on findings from both static and dynamic testing, therefore 0.5 may be considered a conservative value for coefficient of friction.

2.2. Geofoam Block Applications

2.2.1. Lightweight Fill

In Norway, EPS geofoam was initially used as a lightweight fill material for restoring the approach fill of the Flom Bridge, which is located close to Oslo [Frydenlund and Aabe, 1996]. Airfields, railroads, and other construction projects have all made use of geofoam blocks, which are frequently utilized as a lightweight fill material for road construction [Kongkitkul, 2011; Beinbrech, 1997]. The blocks might be used to decrease the stress increase on soft subsoils for stability and settlement issues. A typical geofoam block road cross section is provided in **Figure 2.7**.

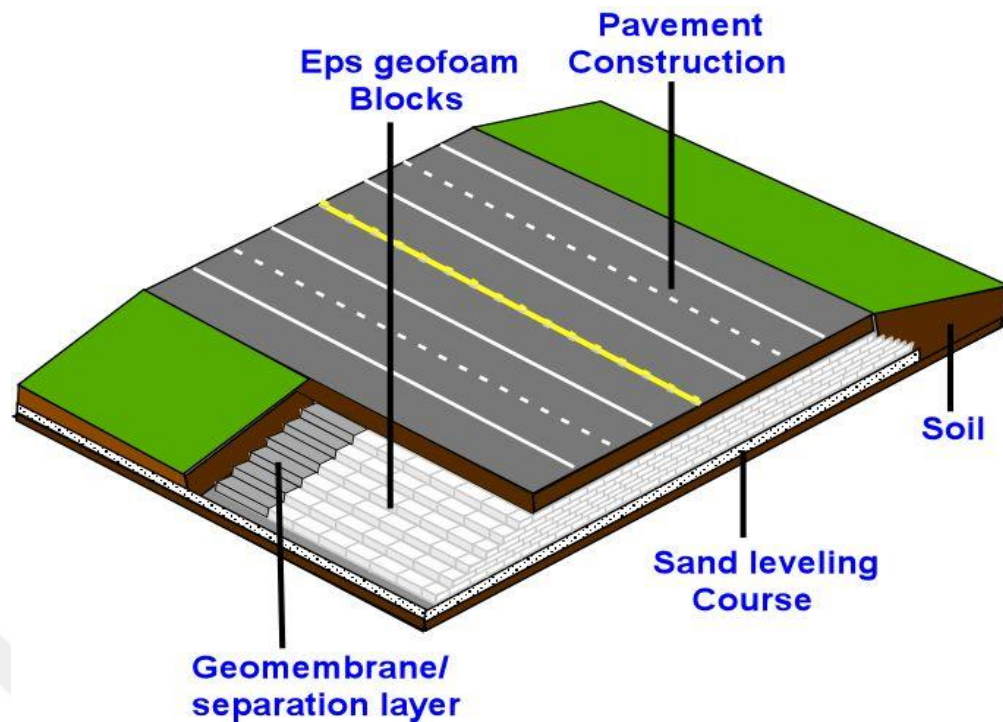


Figure 2.7: Cross section of road embankment with Geofoam blocks [Stark et al 2012].

Geofoam blocks can be used as a lightweight fill behind retaining walls and bridge abutments, serving two primary purposes. Firstly, its use reduces vertical strains, which helps prevent backfill settlements and avoids elevation differences between the approach road and the bridge deck. Secondly, due to its higher compressibility, greater interface friction, and lower density, EPS geofoam reduces lateral pressures on the wall. A study by [Ishihara et al., 1996] demonstrated that using a 9m EPS geofoam fill significantly reduced settlement in a bridge abutment.

2.2.2. Slope Stabilization

The EPS geofoam's lightweight property was also used to improve the slopes' safety factor [Elragi 2000; Sheeley 2000] as shown in **Figure 2.8**.

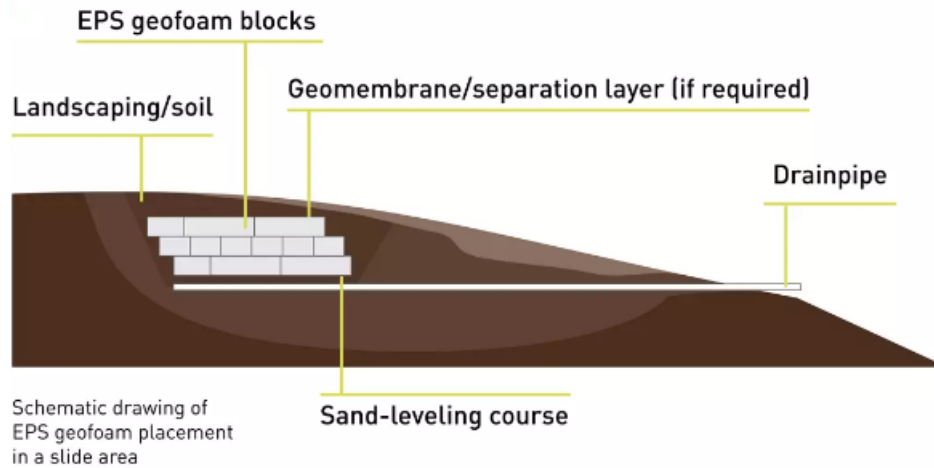


Figure 2.8: Slope stabilization utilizing EPS geofoam [Stark et al., 2012].

In Japan, 1834 cubic meters of EPS geofoam were used to build a road embankment on a precipitous mountainside. The EPS was used in a 104 m-long part of the road [Suzuki et al., 1996]. By using EPS, the overall cost of stabilization activities was decreased. This project's building duration was also decreased. Expanded polystyrene geofoam blocks were used in New York to stabilize a clayey soil roadway embankment slope [Jutkofsky et al., 2000].

The choice of the geofoam solution was made based on its constructability and low influence on the surrounding homeowners and the environment. The use of a thicker subbase layer in the geofoam treated region helped to reduce any potential traffic safety issues brought on by differential icing of roads caused by the presence of geofoam blocks beneath pavements. The use of geofoam to lessen the driving forces. An instrumentation program made up of an inclinometer, extensometers, and thermistors. Since the remedial solution was finished in 1996, there has been no slope movement [Jutkofsky et al., 2000].

2.2.3. Retaining wall Structures

For the purpose of lowering lateral pressure, EPS Geofoam can be used in between the soil and retaining structure. A comparison of the two main geofoam layouts is shown in **Figure 2.9**. During and after backfill construction, a 14-meter-high abutment was subjected to static earth pressure. Additionally, a 0.5-meter-long strip of EPS geofoam was used as a cushion between the abutment and the backfill to reduce dynamic earth

pressure caused by traffic loads and earthquakes [Matsuda et al., 1996]. Rolling compaction reduced the total bending moment by 85% when 12 kg/m³ of EPS geofoam was used, according to a finite element analysis. In comparison to the scenario of no geofoam, the 20kg/m³ geofoam exhibited a 70% reduction. The geofoam block design used in the Syracuse Mall's basement wall [Sun, 1997].

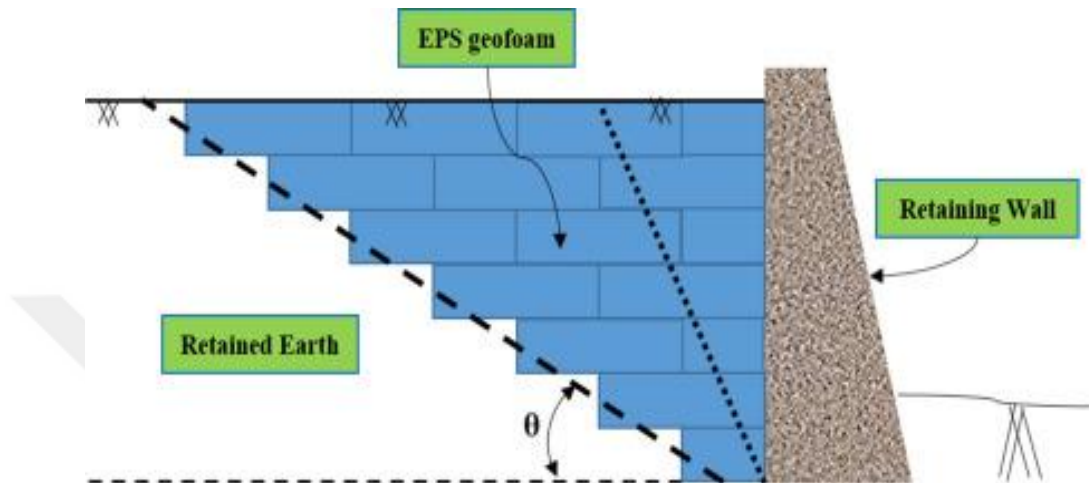


Figure 2.9: Reducing lateral stress with EPS geofoam [Stark et al., 2012].

2.2.4. Embankment Widening

When enlarging existing roadways (Figure 2.10), using geofoam help to enhance stability and prevent differential settlements between the old and new sections of the roadways[Özer, 2016].When expanding or constructing roads on steep side slopes, geofoam makes it simple to broaden embankments.. The geofoam face will need to be protected, nevertheless, by a fascia wall face.

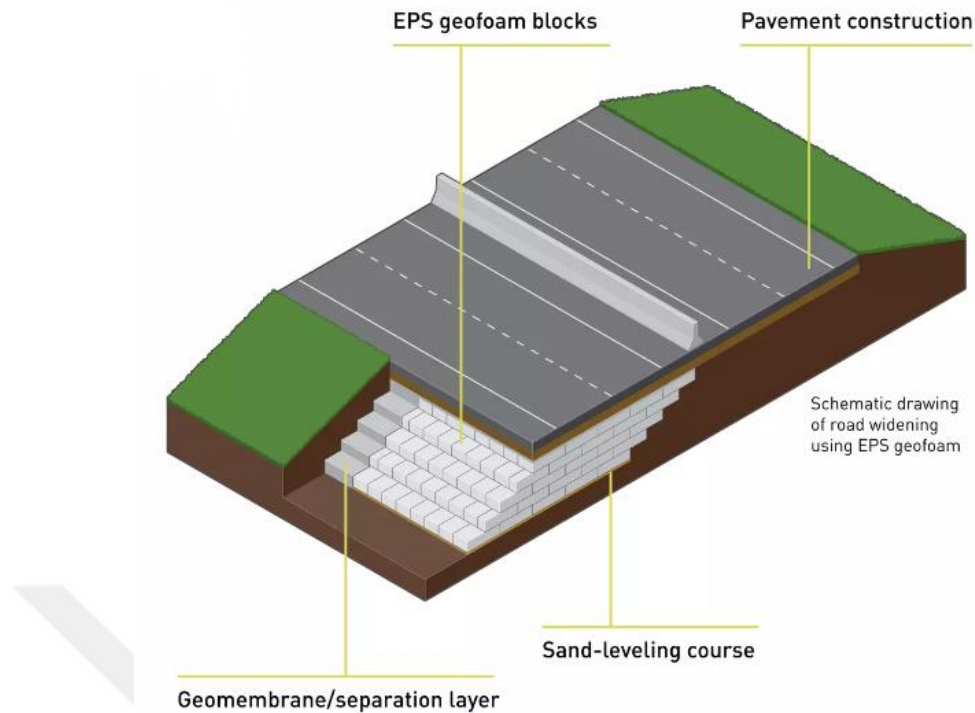


Figure. 2.10: Road widening application using Geofoam blocks [Stark et al., 2012].

2.2.5. Railroad Embankment

Transportation infrastructure, such as roads and railroads, may be impacted by the soil's seasonal cycle of winter freezing and spring thawing. During freezing, the earth surface heaves and then settles. As a result, the lifespan of the pavement segment is decreased.

The subgrade is weakened, which could provide a safety risk for roads, railroads, or airports. **Figure 2.11** cross section depicts the positioning of EPS geofoam layers beneath a segment of pavement to prevent frost related issues [Elragi, 2000].

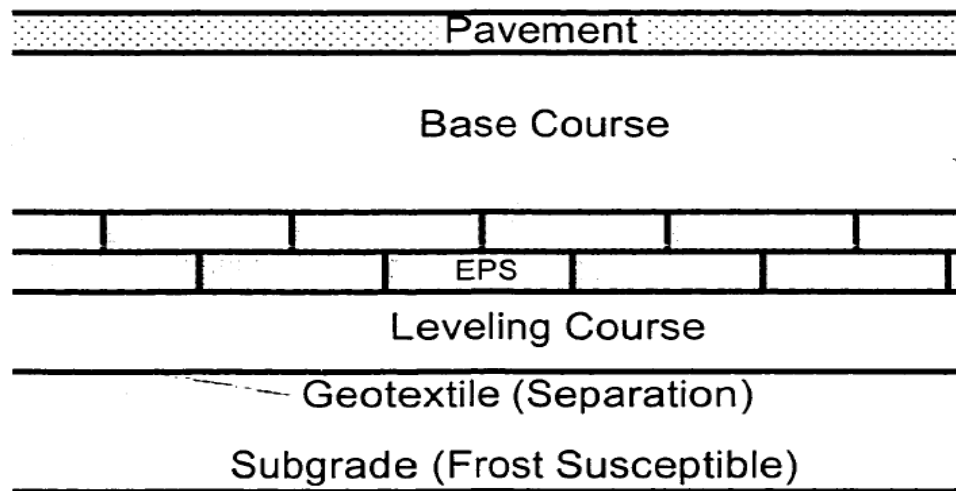


Figure 2.11: Pavement insulation utilizing CPS geofoam [Elragi, 2000].

2.2.6. Bridge Approach Embankments

When properly designed, EPS geofoam can effectively support bridges. Its light weight adds minimal additional load to the underlying ground. In situations where an existing bridge can no longer support the required traffic loads, EPS geofoam infill can help by supporting the span and safely transferring the traffic load to the foundation or underlying soil **Figure 2.12** showing Underfill Bridge Utilizing EPS Geofoam for Structural Support. Although the EPS embankment technology is well-suited for these purposes, it has not been used for the direct support of bridge structures that is, without the need for the installation of deep foundations like piles, shafts, caissons, or piers.

Using EPS geofoam to construct approach fills for bridge abutments offers several advantages. Thanks to its high compressive resistance, EPS geofoam can support highway loading without over-stressing the underlying soils. By calculating the loads induced by the new roadway and replacing an equivalent amount of existing soil with EPS geofoam fill, zero net loading on the existing building foundations is achieved. **Figure 2.13** illustrates a typical bridge abutment with a geofoam backfill.



Figure 2.12: Typical underfill bridge utilizing EPS geofoam for structural support [Stark et al., 2012].

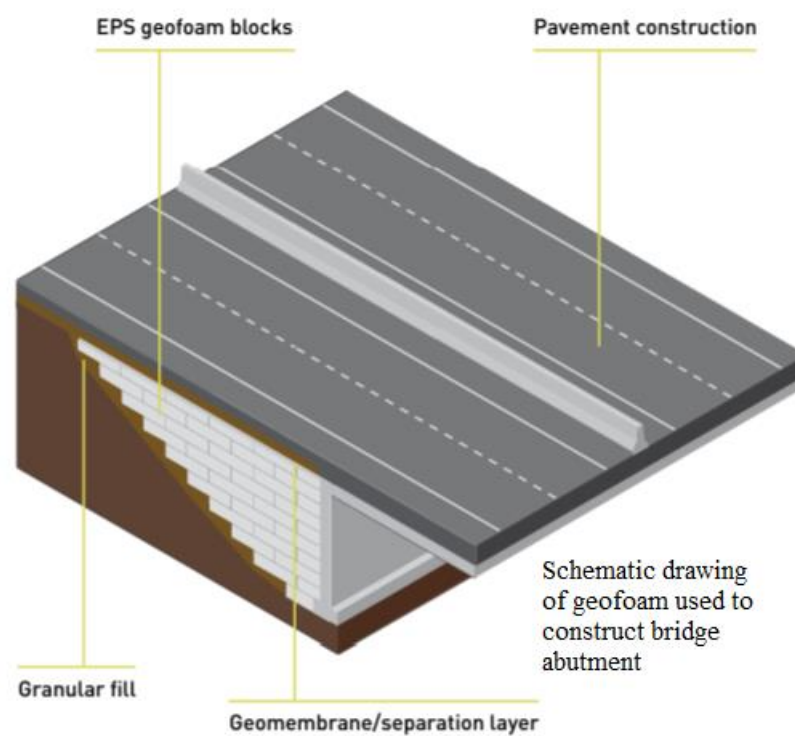


Figure 2.13: Typical bridge abutment with geofoam backfill [Stark et al., 2012].

2.2.7. Culvert and Buried Pipelines

Geofoam has two crucial material qualities that make it useful for protection underground pipes and culverts (**Figure 2.14**) from the negative impacts of vertical permanent ground displacement: Its had two distinct advantages over alternative materials are (1) it's extremely low unit weight, and (2) it's clearly defined compressibility properties, which provide controlled compression and yielding while mostly preserving its original size and shape. Regarding the first, when compared to alternative materials, geofoam has a very low bulk density (12 to 46 kg/m³).

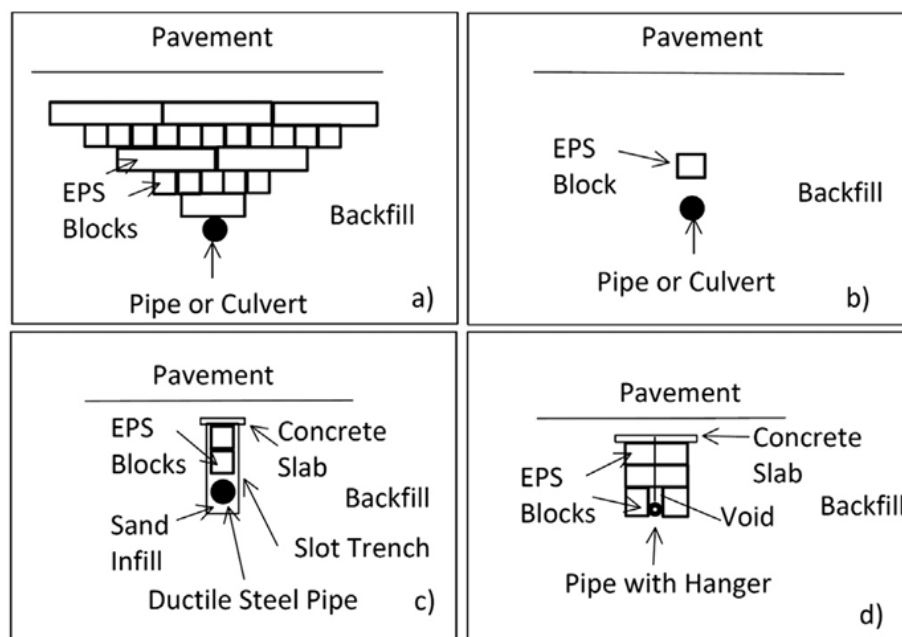


Figure 2.14: General techniques for protection pipelines and culverts from vertical ground displacement [Bartlett et al., 2015].

This property can greatly lessen overburden strains atop systems that are buried [Horvath, 1996; 1997]. [Vaslestad et al., 1993] presented the findings of three prototype experiments for concrete culverts with EPS geofoam placed above them. The 1.95 m diameter pipe with the 14 m high rock fill embankment served as the instrumented culvert in the first embankment. In the second embankment, a 1.71-meter-diameter pipe was placed beneath a 15-meter-high rock fill. In the third prototype instrumented case, a 2-meter-wide box culvert was excavated 11 meters below silty clay. Based on the outcomes of the three tests, the vertical stresses were reduced by 30% to 50% of the overburden stresses. The EPS geofoam has strains

ranging from 27 to 42%. Chapter 3 has further information about these case histories. The application of compressible inclusion over rigid culverts in Norway has decreased the cost. Conceptual drawings of some of the techniques evaluated, developed, and tracked in geotechnical literature.

2.2.8. Theater and Stadium Seating

In places like auditoriums, movie theatres, gymnasiums, and churches, tiered seating can also be created with geofoam blocks. Geofoam is an excellent choice for both new building and remodeling projects due to its strong compressive resistance and lightweight. Before a protective concrete layer is applied, chairs, stands and other attachments and finishes are put to complete the project (**Figure 2.15**).



Figure 2.15: Stair-step geofoam block placement for seating arrangements [Stark et al., 2012].

2.2.9. Airport Runway and Taxiway Embankment

The same applies true for airports, whether they have runways or taxiways; all that needs to be done is to provide an acceptable design so that the embankment can support the wheel loads from descending or ascending aircraft on soft soil sites with low

bearing capacity and high settlement potential can be constructed using geofoam blocks (**Figure 2.16**).



Figure 2.16: Conceptual geofoam block placement in an airport (Tolga, 2020).

2.2.10. Levees

Because of the patterns of river depositional flow, levees are typically constructed on compressible alluvial soils alongside rivers. These saturated, compressible soils settle over time as a result of both primary and secondary compression. As a result of this ongoing settlement, the levee must be raised more than once in order to offer the appropriate flood protection. In order to elevate levees to their former level, conventional soil fill is typically used.

The cycle of settling and rising is perpetuated by the additional weight from the levee raising [Bartlett et al., 2015]. To provide the volume required to restore the levee to its initial configuration, EPS geofoam may be deployed quickly. Additionally, using EPS as fill prevents or decreases additional stress as well as the cycle of settling and levee rising because it weighs around 1% as little as standard earth fills. Of course, geofoam needs to be covered with enough shielding soil. Using geofoam in a levee is shown in **Figure 2.17** conceptually.

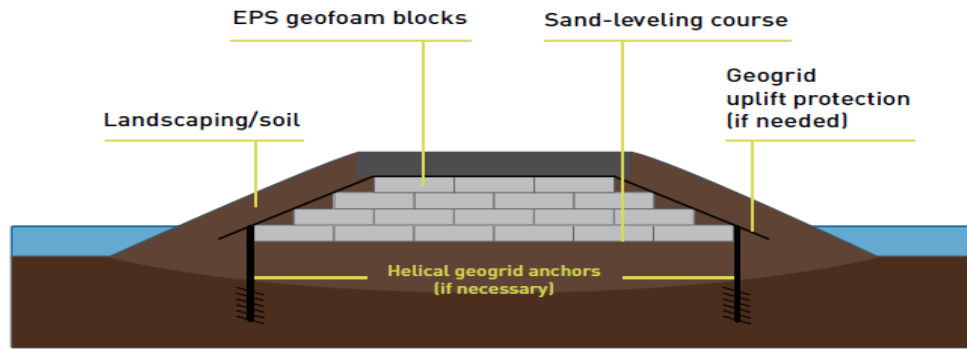


Figure 2.17: Levee repair using EPS geofoam [Stark et al., 2012].

2.2.11. Rock Fall/Impact Protection

In order to shield drivers from avalanche hazards in mountainous areas, avalanche sheds are periodically constructed on portions of roads where avalanche activity is common. In order to lessen the impact forces of falling boulders, such sheds typically feature a layer of earth material on the shed roof. The shed roof may be covered with geofoam blocks, a concrete slab, and soil to further lessen the impact loads. When big stones or pebbles contact the structure, geofoam will bend and absorb a major percentage of the dynamic energy, resulting in a significant reduction in the dynamic loads transmitted to the shed (**Figure 2.18**).

The concept was initially developed and tested in Japan [Konno et al., 2001]. The technique can also be used to shield other types of constructions from dynamic impacts.



Figure 2.18: Geofoam blocks applied for energy absorption on rock fall protection tunnel in Turkey (Courtesy of EPSDER, Turkey) [Özer, 2020].

2.2.12. Floating

Of course, floating piers and other such port and marina designs may make direct use of the buoyancy effect of geofoam blocks. For a long time, this phenomenon has been used; in Vancouver, British Columbia, Canada, a floating helicopter pad is supported by geofoam blocks (**Figure 2.19**).



Figure 2.19: Floating helipad supported by geofoam blocks (Vancouver, Canada) [Aabøe et al., 2012].

A floating vegetation island made of geofoam blocks is visible on one of Amsterdam's canals, demonstrating a comparable application of the buoyancy effect in the Netherlands (**Figure 2.20**).



Figure 2.20: Floating garden on one of the canals in Amsterdam, The Netherlands [Aabøe et al., 2012].

2.2.13. Noise and Vibration Damping

Road noise can be attenuated by constructing free-standing barriers or embankments made of geofoam. They can also be used as a cutoff wall between nearby structures and railroads or pavements, beneath railways or pavements, as part of the foundation of surrounding structures, or in other similar situations to lessen the transmission of ground-borne vibrations [Bartlett et al., 2012].

2.2.14. Seismic Applications

The low weight of geofoam significantly reduces the seismic forces applied to retaining walls, pipelines, underground buildings, and other structures since the seismic force is proportional to the mass of the system (i.e., force = mass times density).

3. USING EPS GEOFOAM AS A COMPRESSIBLE INCLUSION

The required technical characteristics of the material must generally be considered when choosing a suitable geofoam material, in addition to aspects like cost, durability, and environmental effect. The stiffness of the geofoam in the major direction of displacement is the most important feature for compressible inclusion applications. In order to calculate stiffness, the geofoam surface must be moved parallel to the direction of stress application. The normal tension applied to the geofoam surface is divided by this displacement to calculate the stiffness.

In other words, force per unit length cubed may be used to describe stiffness; this is best stated as force per length (or, more precisely, spring stiffness) per unit area of the geofoam. Theoretically, there is no difference between this and the widely used geotechnical metric, the coefficient of subgrade response. Experimental results indicate that block-molded expanded polystyrene (EPS) or products developed from it is the ideal geofoam material for compressible inclusions.

Block formed of EPS is the least expensive geofoam material overall in terms of cost per volume. It can also be manufactured to have a far lower rigidity than anything else. Moreover, unlike certain other foam materials, EPS does not use any fluorocarbon-family gases (CFC, HCFC, HFC) during the manufacturing process, which has been connected to the Earth's ozone layer's thinning [Horvath, 1993], nor does it release formaldehyde gas following installation, which poses a risk to health and safety.

3.1. Function of Compressible Inclusion

Any material that is much more compressible than adjacent or in contact with other materials, at least in one direction, is generally referred to as a compressible inclusion. In geotechnical applications, a compressible inclusion is often positioned between a below-ground structure and the surrounding ground (soil or rock). Because the soil's internal shear strength is triggered when the soil load is applied, the inclusion compresses and considerably redistributes the shear and normal stresses in the surrounding soil. The use of this idea can lower ground pressure in both horizontal and

vertical directions, contingent on the compressible inclusion's location, thickness, and stiffness.

For instance, compressible inclusions have been added to the backfill of retaining and basement walls to lower horizontal earth pressures [Reeves and Filz, 2000]. The inclusion is compressed selectively, which has a number of advantages. Most frequently, this results in a load on the structure that is much lower than it would be without the inclusion. Utilizing a compressible addition is frequently a more economical choice than building the structure to support the higher load. This is particularly true in the case of an existing structure that has to be upgraded to resist loads greater than those for which it was originally built, or that is in distress and needs to be repaired. A compressible inclusion may have extra functions and associated advantages depending on the material utilized. If taken into account, these extra features can increase how cost-effective utilizing an inclusion is [Horvath, 1997].

3.2. Applications

The usage of a compressible inclusion in geotechnical applications has been classified into three categories so far [Horvath, 1998]:

- (1) Behind retaining walls (**Figure 3.1a**).
- (2) Below structural (supported) slabs or grade beams that are part of the foundation (**Figure 3.1b**).
- (3) Above culverts, tunnels, and pipelines (**Figure 3.1c**).

The arrows denotes the major direction of displacement of the ground and compressible inclusion in **Figure 3**.

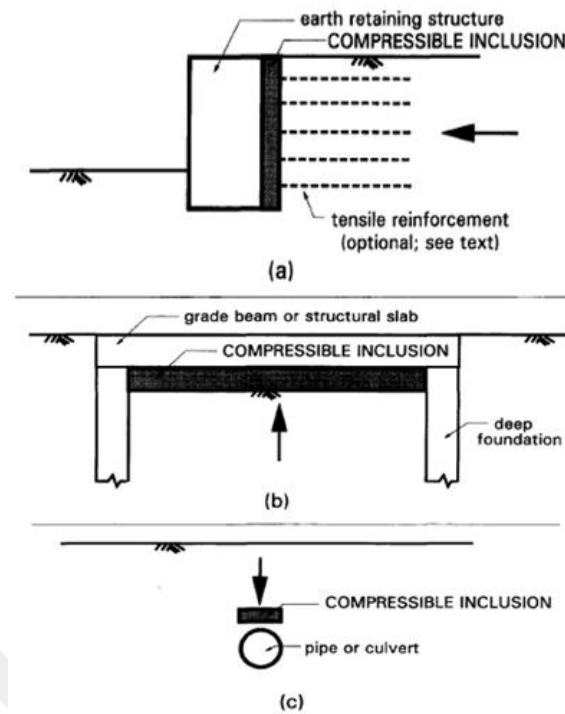


Figure 3.1: Categories of compressible inclusion applications a) Behind retaining structures b) Below structural slabs c) Atop pipes or culverts [Horvath 1998].

Geofoam can be utilized as cover material and backfill to protect pipes from the destructive effects of permanent ground displacement (PGD) due to its low weight and compressibility. PGD is the term used to describe any horizontal or vertical soil displacement brought on by mass movement processes or seismic events. Faulting, tectonic deformation, subsidence, liquefaction, land sliding, and other processes are responsible for PGD. EPS has a very high compressibility and an incredibly low mass density (12 to 46 kg/m³) in comparison to regular granular backfill materials. Geofoam can reduce the strains placed on underground pipes by acting as a lightweight backfill or as part of a pipeline cover system. [Sun et al., 2009] employed geofoam's low weight to minimize soil pressure on underground culverts, and [Negussey and Sun, 1996] used it to lessen static lateral earth pressure against buried walls. Additionally, it can be utilized to lessen the effects of neighboring embankment building on ground settling on underground structures [NRRL, 1992; Sanders, 1994; Duskov, 1997; Bartlett et al., 2000].

3.3. Design and Analysis for Compressible Inclusion

Applications with a compressible inclusion offer an indeterminate system that needs to be examined as a problem of soil-structure interaction that considers displacements and stresses. This is because, despite the fact that the structure is usually assumed to be stiff and unyielding, or at the very least inflexible, each element of the system the ground, inclusion, and structure has a unique stress-displacement behavior. As a result, finding the specific set of displacements of the ground, structure, and compressible inclusion that fulfill material continuity and force equilibrium is the necessary step towards solving the problem. The most useful general analytical tool for evaluating scenarios with a compressible inclusion is software that solves a continuum using the finite-element method. But as the usage of geofoam as a compressible inclusion becomes popularity, engineers have been able to create more straightforward solutions that may be applied in everyday practice. Simple mathematical models that may be solved manually or by a computer are examples of such simplified solutions, as are charts derived from parametric finite-element studies or physical modeling. The common factor is the stiffness of the geofoam compressible inclusion, regardless of whether a rigorous or simplified analysis is utilized in the design process [Horvath, 1993].

3.4. Pipeline Protection Using Geofoam

Underground utilities and pipelines are critical infrastructures, which exposed to high ground pressures. They have to be able to withstand pressures from within as well as lateral and overburden pressures. Under roads and highways, pipes are usually laid at shallow depths. If the pipe's capacity is exceeded by the demanding load, it may collapse or stop working because of a lack of strength. Evaluating subsurface pipeline performance under repeated loads has received a lot of interest in recent years [Oshati et al., 2012; Srivastava et al., 2013; Chaallal et al., 2015; Elshesheny et al., 2019a, b; Randeniya et al., 2020]. Until now, the protection of underground utilities has been accomplished through the use of lightweight fill materials, geosynthetics, and recycled materials to lessen and re-distribute the stress placed on them] or through the imperfect trench method [Tafreshi et al., 2012; Kumar and Nagaratnam, 2013; Hegde and

Sitharam, 2015; Tran et al., 2014; Bartlett et al., 2015; Moghaddas Tafreshi et al., 2020; Azizian et al., 2020.]

A lightweight EPS block cover effectively protects buried pipelines from the damaging effects of vertical offset caused by PGD resulting from normal faulting or mass movement processes (e.g., land sliding, liquefaction-induced lateral spread, etc.) according to full-scale experiments and numerical evaluations **[Lingwall, 2009]**.

Investigations demonstrate that when the pipeline runs through PGD, EPS dramatically lowers the soil pressures operating on the soil-pipeline system both vertically and horizontally **[Yoshizaki and Sakanoue, 2003]**. The horizontal strain on a 100-mm steel pipeline undergoing horizontal PGD significantly decreases when compared to sand backfill. Using both a geofoam cover system and a traditional sand cover, horizontal pipe displacements were measured in a box that measured 2 meters wide by 2 meters high by 3.1 meters long **[Yoshizaki and Sakanoue, 2003]**. Yoshizaki and Sakanoue tested horizontal pipe displacements using both a geofoam cover system and a standard sand cover in a box that measured 2 meters wide, 2 meters high, and 3.1 meters long. The pipe was moved a maximum of 150 mm before force-displacement measurements were made. When compared to a standard sand cover, the peak horizontal stress on the pipe for the geofoam cover system was reduced by 40 to 60 percent. Additionally, employing EPS backfill placed around the pipe elbow may considerably increase the resistance of pipeline elbows subjected to PGD **[Yoshizaki and Sakanoue, 2003]**. Additionally, **[Choo et al., 2007]** demonstrated how the addition of geofoam to the cover and trench sidewalls dramatically reduces the pressure on high-density polyethylene (HDPE) pipe. In order to protect infrastructures that pass under or beneath roads and railroads against deformations or ruptures, **[Bartlett et al., 2015]** provided an overview of the several EPS geofoam cover systems. The cover systems are listed as: post and beam EPS system, slot-trench cover system with EPS block put in slot, imperfect ditch method, and embankment configuration. The results indicate that lightweight cover systems with geofoam blocks protect buried steel pipes against faulting and long-term ground deformation; these techniques are covered in depth in this chapter.

3.5. Protection Systems

3.5.1. Embankment System

The first method is similar to that of traditional geofoam embankments; it is called the lightweight embankment system. The main difference is that in this case, the EPS blocks are used to reduce the overall stress on the buried infrastructure. This configuration is constructed to protect pipes and culverts from potential damage caused by the stresses that are often caused by both the different types of settlement and vertical ground displacement. **Figure 3.2** shows how a lightweight geofoam embankment is constructed to protect a concrete culvert from possible damage resulting from primary consolidation settlement in the compressible foundation. In order to accommodate a new commuter rail line, the embankment was widened and the use of geofoam in the broadened zone reduced soil loading on the culvert and foundation soils [**Figure 3.2; Bartlett et al., 2015**].



Figure 3.2: EPS geofoam embankment constructed over culvert system, Utah Transit Authority Frontrunner Commuter Rail System, Draper, Utah (photo courtesy of ACH Foam, Murray, Utah) [**Bartlett et al., 2015**].

It may not be essential to do thorough studies for this kind of system because the block placement plan in **Figure 3.3** considerably decreases the dead load placed atop the pipe/culvert. Additionally, this technique needs a substantial amount of right-of-way

area to be built, while being quite efficient at reducing weight. The construction and placement of the light-weight embankment configurations are likewise predicated on the assumption that there won't be any extra interferences or geometrical restrictions. This would not always be the case in metropolitan areas, where there might be numerous interferences or other pre-existing infrastructure.

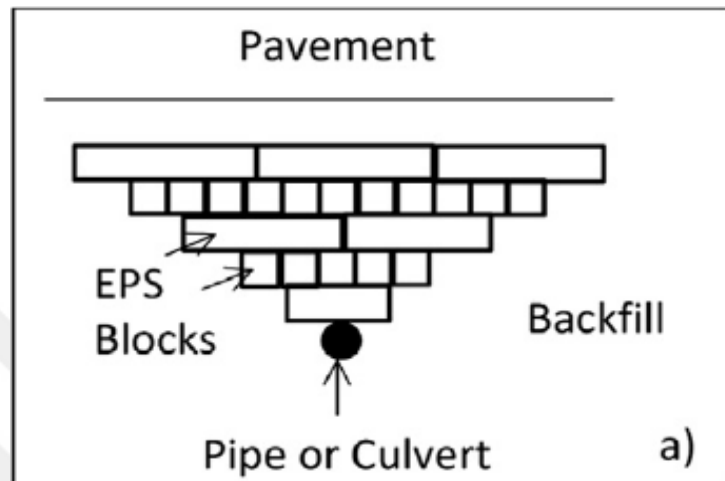


Figure 3.3: Light-weight cover or embankment constructed over pipe or culvert [Bartlett et al., 2015].

3.5.2. Slot Trench Cover System

In an urban Utah roadway, vertical offset associated with normal faulting resulted in high pressure natural gas pipeline system shear, bending, and vertical forces that were reduced by using the lightweight and compressible properties of EPS geofoam [Lingwall, 2011; 2014 and Bartlett et al., 2015]. Given the relatively shallow (2 to 3 m) burial of the intended pipeline, the construction technique was to place geofoam blocks on top of the pipe in the trench excavation. This was carried out on a section of road that ran parallel to the typical fault's hanging wall, or down-dropped, side. A lightweight, vertical "slot" within the trench composed of EPS cover system, with the goal of allowing upward relative movement of the pipeline after fault rupture. The bending tension that occurred in the pipe was predicted to be significantly reduced by the compression and uplift of the EPS block placed in the pipeline's slot, as opposed to any potential interactions with traditional trench backfill materials. Lastly, the design of the cover system needed to withstand actual traffic loads without perhaps

overstressing the EPS block or underlying pipe because the geofoam blocks would be placed beneath the pavement [Bartlett et al., 2015].



Figure 3.4: EPS slot-trench light-weight cover system constructed across the Wasatch fault in Salt Lake City, Utah [Bartlett et al., 2015].

3.5.3. Geofoam Post and Beam Cover System

Although the slot trench geofoam cover system worked well to lessen the pressures applied to pipe systems experiencing vertical offset, non-ductile or more sensitive pipe systems will not be able to benefit from this corrective approach, thus other solutions will need to be developed. For example, an interesting emergency situation arose that necessitated a modification to this concept.

A ruptured sewage pipe beneath a road necessitated a modification to the light-weight slot trench cover idea. Due to the dewatering of a thick layer of sand beneath the sewage pipe, there had been two occurrences of rupture. Deep fissures and fractures were seen in the adjacent and underlying earth, and the dewatering caused sinking in this region. At the lowest position of the damaged line during the initial repair, 0.25 m of consolidation and 0.15 m of horizontal displacement were detected.

It was clear that further settling in this location was to be expected and the pipeline would continue to suffer damage. This led to the evaluation and construction of a plan to create a 0.6 m headspace void above the pipeline. With this geofoam block structure, the pipe system wouldn't be subjected to any substantial soil loads until the foundation might settle. A hanger system with an adjustable rod was used to prevent the pipeline from potentially drooping under its own weight. The post and beam geofoam cover

Figure 3.5, [Bartlett et al., 2015] was designed and built under emergency circumstances, but it seems like the mechanism and design concept worked out well. One year after construction, there have been no further sewer line ruptures.

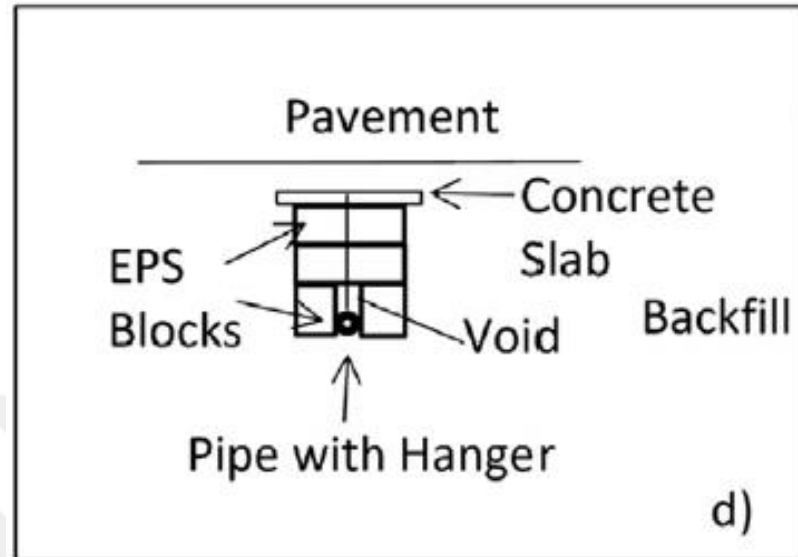


Figure 3.5: EPS post and beam system with head space void [Bartlett et al., 2015].

3.5.4. Imperfect Trench Method

According to Marston (1930), the earth load on an embedded culvert equals the weight of the soil column directly above it plus any shear loads that are transmitted to the soil column as a result of the soil column's deformation in respect to the nearby soil [ACPA, 1980]. The direction of relative deformation and shearing is determined by the installation procedure.

The phenomenon known as "negative arching" occurs when a portion of the surrounding soil weight is transmitted to the overlying soil prism by shear, causing the culvert loading for traditional (positive projecting) installations to be more than the weight of the overlying soil prism alone. Positive arching occurs when the culvert loading is less than that of the overlying soil prism and the overlying soil column settles more than the neighboring soil, as is the case for induced trench installations. Later, [Splanger, 1950] expanded on Marston's hypothesis and made significant contributions to our understanding of how buried pipes behave by illuminating the

significance of the relative settling of the soil prism over the structure and the outer soil prisms that are immediately around this central soil.

[Splanger, 1950] claimed that when the vertical displacement of the soil above the structure is greater than that of the adjacent soil, earth pressure on the pipe significantly decreases. This is attributed to a reverse arch-shaped deformation, which results in a radical alteration in the pipe pressure distribution. Positive arching is the term used to describe this phenomenon. The imperfect trench technique, often referred to as the induced trench method, was created because of positive arching [Splanger, 1950].

An example of an incomplete trench installation, also known as an induced trench installation (henceforth referred to as ITI), is shown in **Figure 3.6**. Imperfect trench installations utilize the high compressibility of lightweight materials such as baled hay, straw, or geofoam to generate positive soil arching without excavating trenches.

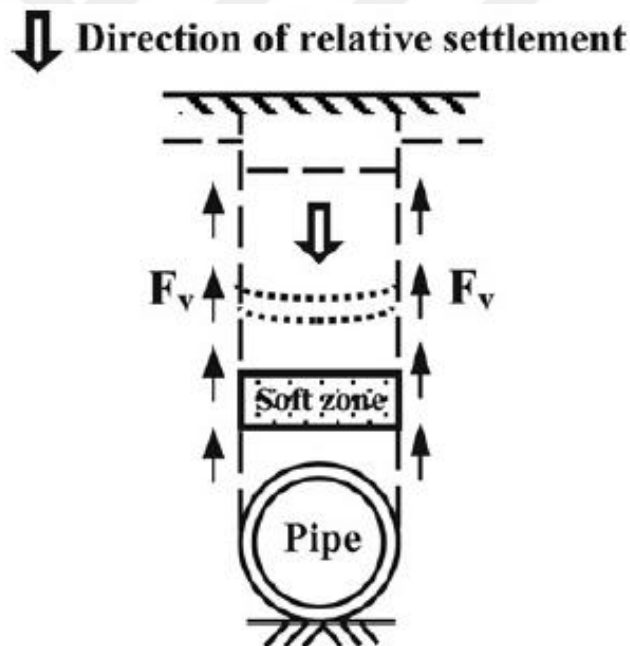


Figure 3.6: A typical installation which is called imperfect trench installation [Kang et al., 2007].

This soft material zone has been positioned above the buried pipes in induced trench installations, as illustrated in **Figure 3.7**, and successfully reduced both overburden and horizontal pressures [Kang et al., 2007; Kang et al., 2008].

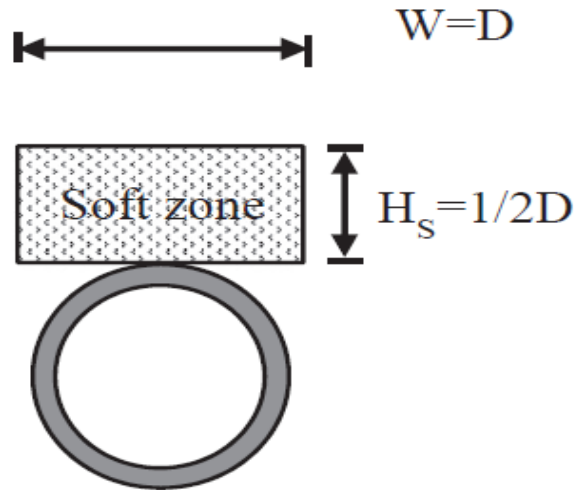


Figure 3.7: Notation for imperfect trench installations and geometries of soft zone [Kang et al., 2007].

The NPRA measured the decrease in vertical stress at four locations, and, the long-term measured vertical pressure above the pipe crown varied from 23 to 25% of the overburden pressure for installations employing cohesive soil backfill and granular backfill material [Vaslestad et al., 2011].

It was found that the type of soil that was used to construct the embankment significantly affected how well the induced arch performed. Installations of granular fill reduced the vertical pressure over the culvert more than installations of silty-clay embankments. Nevertheless, the culvert section's observed earth pressure was 124% greater than the overburden pressure. The final compression of the EPS geofoam block at the end of the embankment project ranged from 27% to 32% for concrete pipes with granular fill and 50% for cast-in-situ box culverts with cohesive fill [Kang et al., 2008]. Figure 3.8 depicts a typical construction plan for the incomplete trench method, per [NPRA, 2010].

The permitted EPS block must have a thickness of 0.5 m and a breadth of $1.5D$, where D is the diameter of the pipe. The block is about $0.2 D$ above the pipe's top. It is advised That there be at least 5 meters of earthen cover above the compressible inclusion, H , and that the granular backfill material be compacted to 97% of the average Proctor value. It is also advised to use a $D/3$ uncompacted zone at the pipe's base (Figure 3.8).

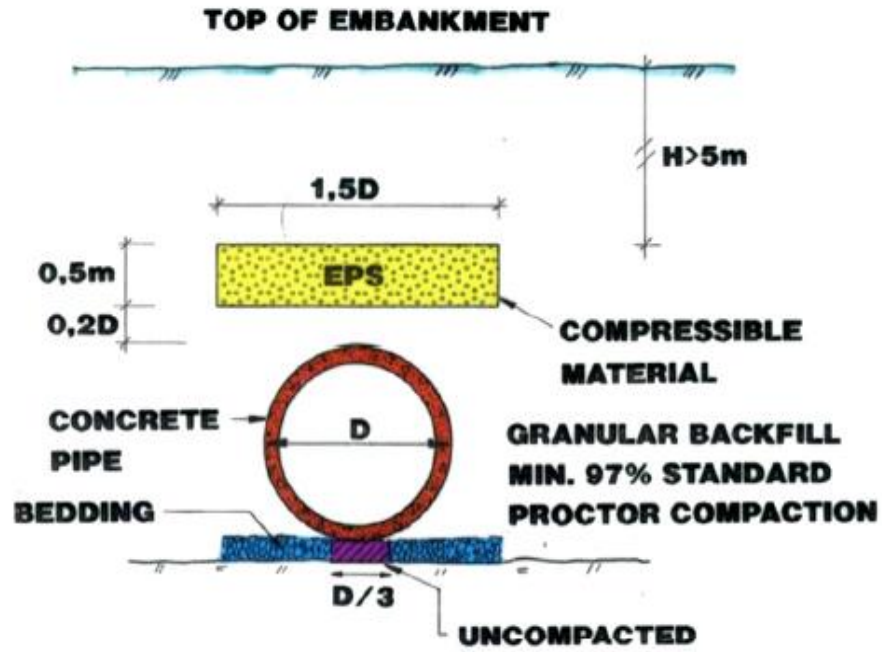


Figure 3.8: Recommended pipe and EPS layout for imperfect trench method from Handbook 16 [NPRA, 2010].

4. CASE STUDIES

There have been numerous case studies published using geofoam blocks as compressible inclusion atop the buried infrastructure. Some of the selected cases were summarized in this section. The schematic finite element model with soft material zone performed by [Kang et al., 2007] is shown in **Figure 4.1**

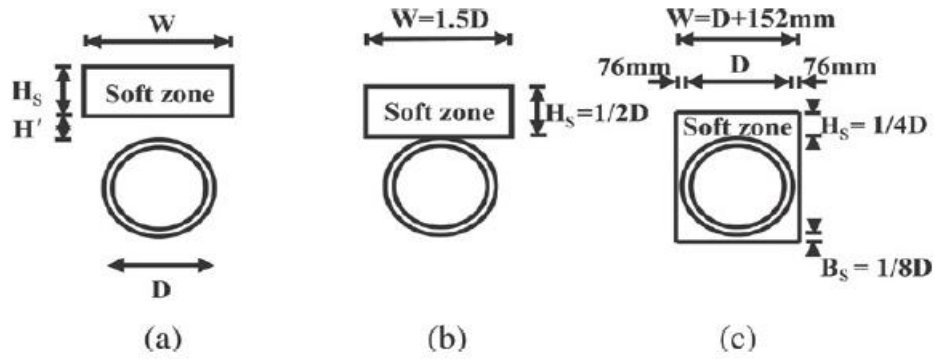


Figure 4.1: Notation for imperfect trench installations and geometries of soft zone:

(a) Notation, (b) soft zone geometry (conventional), and (c) soft zone geometry (Proposed) [Kang et al., 2007].

The best soft zone shape, as shown in **Figure 4.1 (c)**, is clearly the most efficient in lowering the maximum pipe wall stress, R_{ms} = maximum wall stress the key design parameter.

As seen in **Figure 4.2(a)**, after the height of the soft zone (H_s) divided by the pipe diameter (D) reaches to 0.25, the reduction rate (R_{ms}) for maximum wall stress essentially stays the same. Because the reduction rates (R_{ms}) indicated in **Figure 4.2(b)** do not significantly increase beyond the aforementioned suggested value, it is advised that the bedding thickness of soft material below invert, B_s , be 1/8 times the pipe diameter. The same decrease is shown in **Figure 4.2(c)** as a function of W , the breadth of the soft zone. The highest decrease rate of pipe wall stresses occurs when the breadth of the soft zone is considered to equal the pipe diameter + 1/32 times the pipe diameter, as illustrated in **Figure 4.2 (c)**. As illustrated in **Figure 4.2(c)**, the reduction rate actually decreases as the diameter of the soft material zone increases. This is due to the lack of lateral support produced by the soft side fill, which increases the bending moment at the spring line.

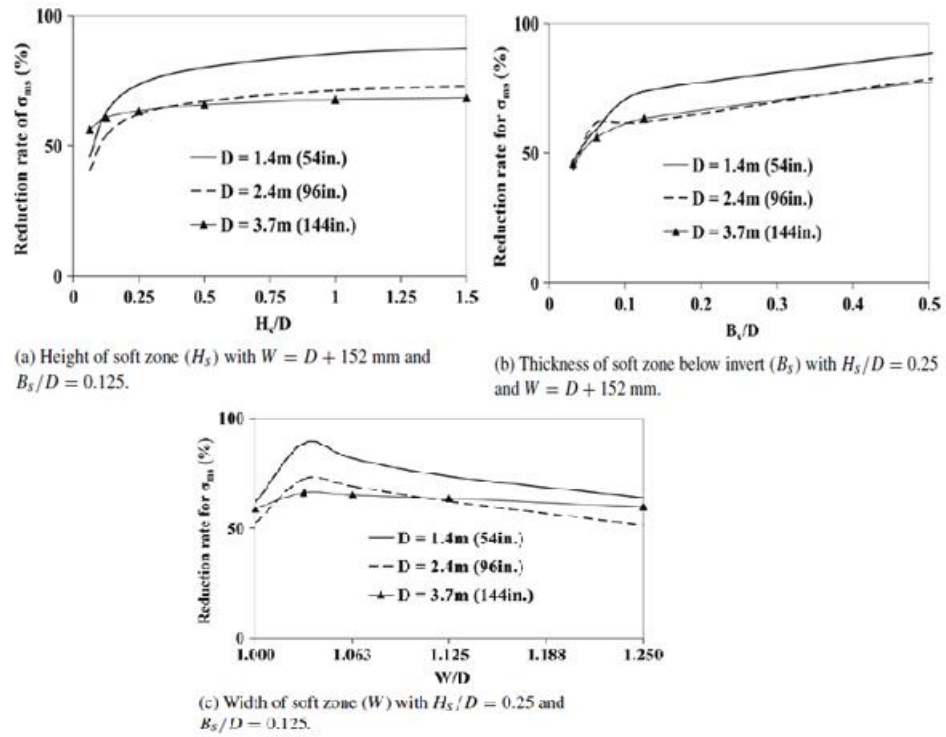


Figure 4.2: Optimization process of soft zone geometry ($R_{\max}$ = maximum wall stress; modulus of elasticity of the lightweight material = 345 kPa) [Kang et al., 2007].

The soft zone geometry is controlled by three parameters: width, W ; height, H_s ; and the distance from the top of the pipe to the bottom of the soft zone, H_0 , as illustrated in **Figure 4.1 (b)**. **Figure 4.3(a)** illustrates how the load is redistributed from the crown to the pipe's sides, creating the space, H_0 , between the top of the pipe and the bottom of the soft zone. Because of a shift in the distribution of soil pressure, **Figure 4.3(b)** shows that when a soft zone is constructed, the patterns of shear stresses on the pipe sidewall differ significantly from those that occur during embankment installation.

In the case of the embankment installation, a positive shearing force (in the clockwise tangential direction) develops above the spring line, while a negative shearing force (in the counter-clockwise tangential direction) develops below the spring line. Because these two opposing shearing forces were almost equal in magnitude, there was no discernible radial pressure rise at the bottom (invert) of the pipe, as shown in **Figure 4.3(a)**. However, as **Figure 4.3 (b)** illustrates, the soft zones altered the distribution of these shearing forces.

The radial pressure at about 25° from the inversion, as shown in **Figure 4.3(a)**, was substantially higher than that at the crown for the soft zone geometry depicted in **Figure 4.1 (b)**. By extending the soft zone to the bedding, as seen in **Figure 4.1 (c)**, the unwanted shearing stress distribution was changed. As can be seen in **Figure 4.3(b)**, the ensuing shearing stresses are significantly reduced. As a result, as seen in **Figure 4.3(a)**, the radial earth pressure on the pipe is greatly reduced. One of the study's key discoveries is the identification of the ideal soft material zone, which is shown in **Figure 4.1 (c)**. With the unwanted development of the shearing forces surrounding the pipe effectively eliminated, as illustrated in **Figure 4.4**, the negative impact of the totally bonded interface state on the earth load is greatly lessened.

[Kang et al., 2009] was conducted 3-D FE modeling using the Duncan/Selig soil models. The detailed protocols for using these models were developed using a general-purpose FE program called [Abaqus, 2016]. Originally developed in 2-D to provide important details on soil behavior in soil-culvert systems, the Duncan and Selig Soil Parameters were later included into two 2-D culvert-soil system FE programs, CANDE [Musser, 1989; Katona, 2015] and SPIDA [Heger et al.] Consequently, there are limitations when utilizing these soil models with all-purpose FE analysis programs like ANSYS or Abaqus. The outcomes of this investigation were also contrasted with those generated by earlier 2-D FE models, which comprised the soil models developed by Duncan and Selig.

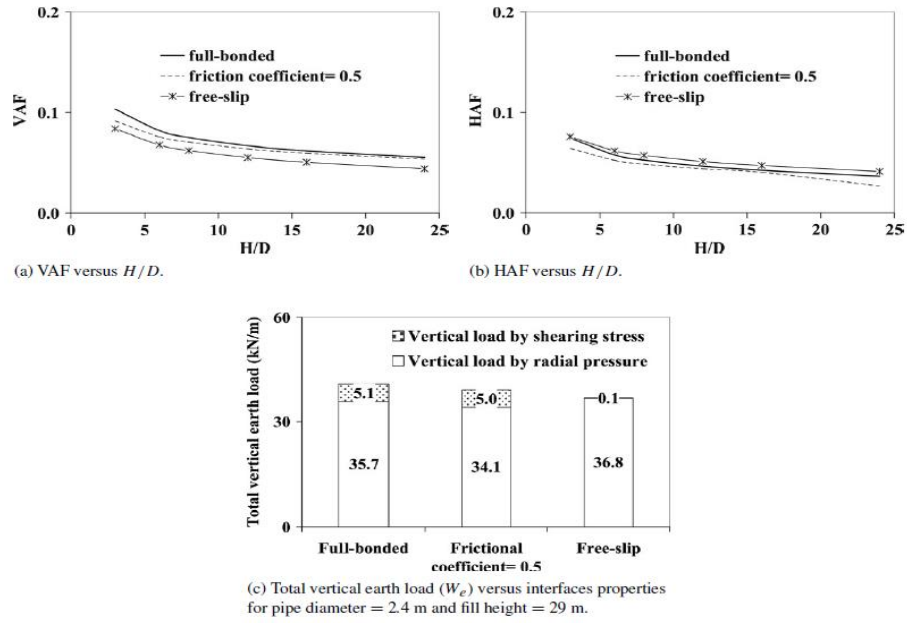


Figure 4.3 Effects of interface properties in ITI (parameters: modulus of elasticity of the lightweight material = 345 kPa; proposed soft zone geometry as shown in Figure 4.2(c)) [Kang et al., 2007].

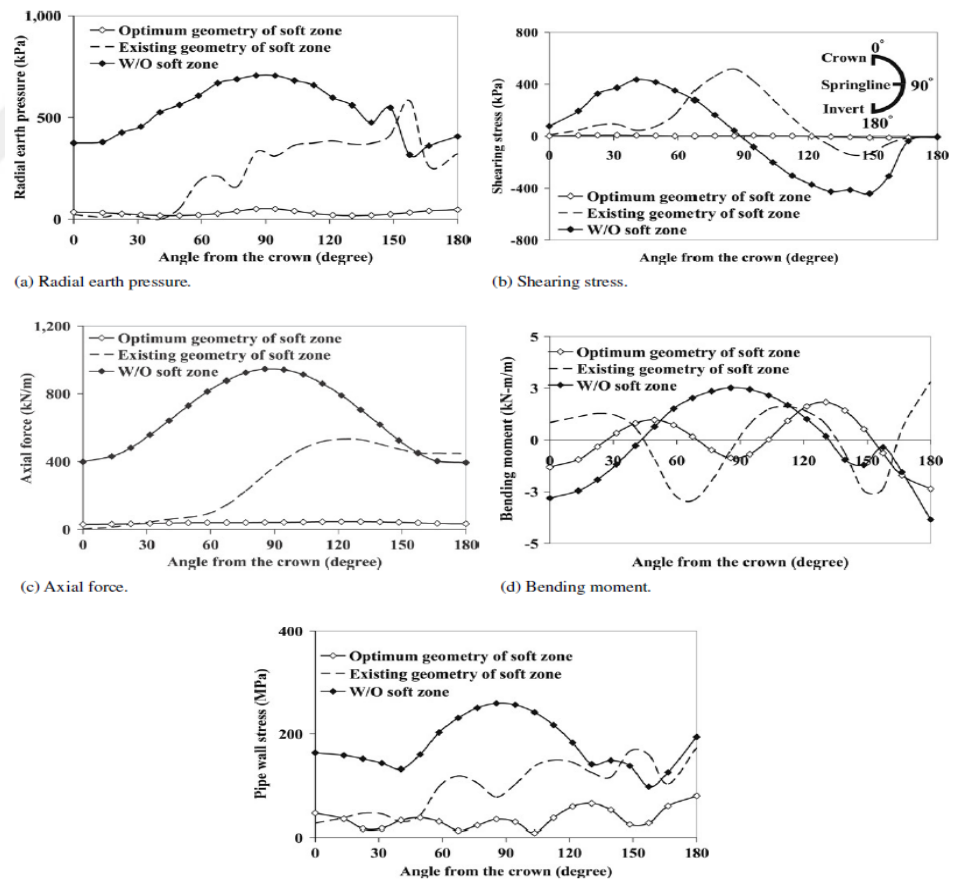


Figure 4.4: Imperfect trench installations versus embankment installation (parameters: pipe diameter = 2.4 m; fill height = 29 m; modulus of elasticity of the lightweight material = 345 kPa) [Kang et al., 2007].

Figure 4.5(a) depicts one of the 3-D FE models created for this investigation. The following are the key distinctions between prior studies [Kang et al., 2008] and the far more comprehensive model suggested here: (1) this is a three-dimensional model (not a plane strain approach); (2) the system is modeled in its entirety, not just a portion of it; (3) the concrete pipes are made of solid elements rather than beam elements; (4) all analyses use actual fill heights rather than comparable overburden pressure approaches; and (5) staged construction sequences are simulated. According to the proposed model, the stiff layer (existing soil) reaches 1.5 times the pipe diameter below the pipe invert, and the lateral limitations extend to three times the pipe diameter measured from the pipe's center [Kang et al., 2009].

The boundary conditions for the four upper and lower sides of the FE models are shown in **Figure 4.5(a)**. Abaqus represents pipes, soil, and soft materials through the use of eight-node solid components with reduced integration (C3D8R). The properties at the interface between pipes and soils are thought to be completely bound. The unit density of the concrete pipe used in this study is 23.5 kN/m³ (150 pcf), its elastic modulus is 28.2 GPa (4090 ksi), and its Poisson ratio is 0.15. The soil's elastic properties were evaluated using the Duncan and Selig soil models as a foundation. The elastic moduli of soft materials range from 345 kPa (50 psi) to 2756 kPa (400 psi), which are typical values for light materials such as EPS geofoam, bales of hay, sawdust, or leaves [Marston, 1922; ACPA, 2021]. Densities and Poisson's ratios of 0.14 kN/m³ (0.9 pcf) and 0.1, respectively, are assumed for the soft materials.

A FEA of a soil-structure system differs from a FEA of a straightforward continuum mechanics issue with linear elastic behavior in a number of respects. A nonlinear stress-strain connection exists in the soil. As a result, substantial load increments can cause sizable inaccuracies in the assessment of stress and strain inside a soil mass. Nonlinear soil characteristics are represented using the hyperbolic stress-strain relationship in the approximation nonlinear incremental analytic methodologies used by Kang et al., 2019. Additionally, it was vital to enable movement between the earth and the pipe's walls, which called for an interface element. According to [Kim and Yoo, 2005] and [McVay, 1982] the influence of interface behavior on soil-structure interaction was minimal. [McVay, 1982] also demonstrated that outcomes were plausible in the absence of numerical modeling of the compaction procedure. Therefore, in this study, interface behavior and the compaction process were not taken

into account. For the comparative examination of concrete pipes, four popular computer programs were employed. These applications include [ABAQUS, 2003] and MSC/NASTRAN 2005, generic finite-element software, [SPIDA ACPA, 1989] and an updated CANDE-89 [Musser, 1989] as well as widely utilized soil-pipe system software.

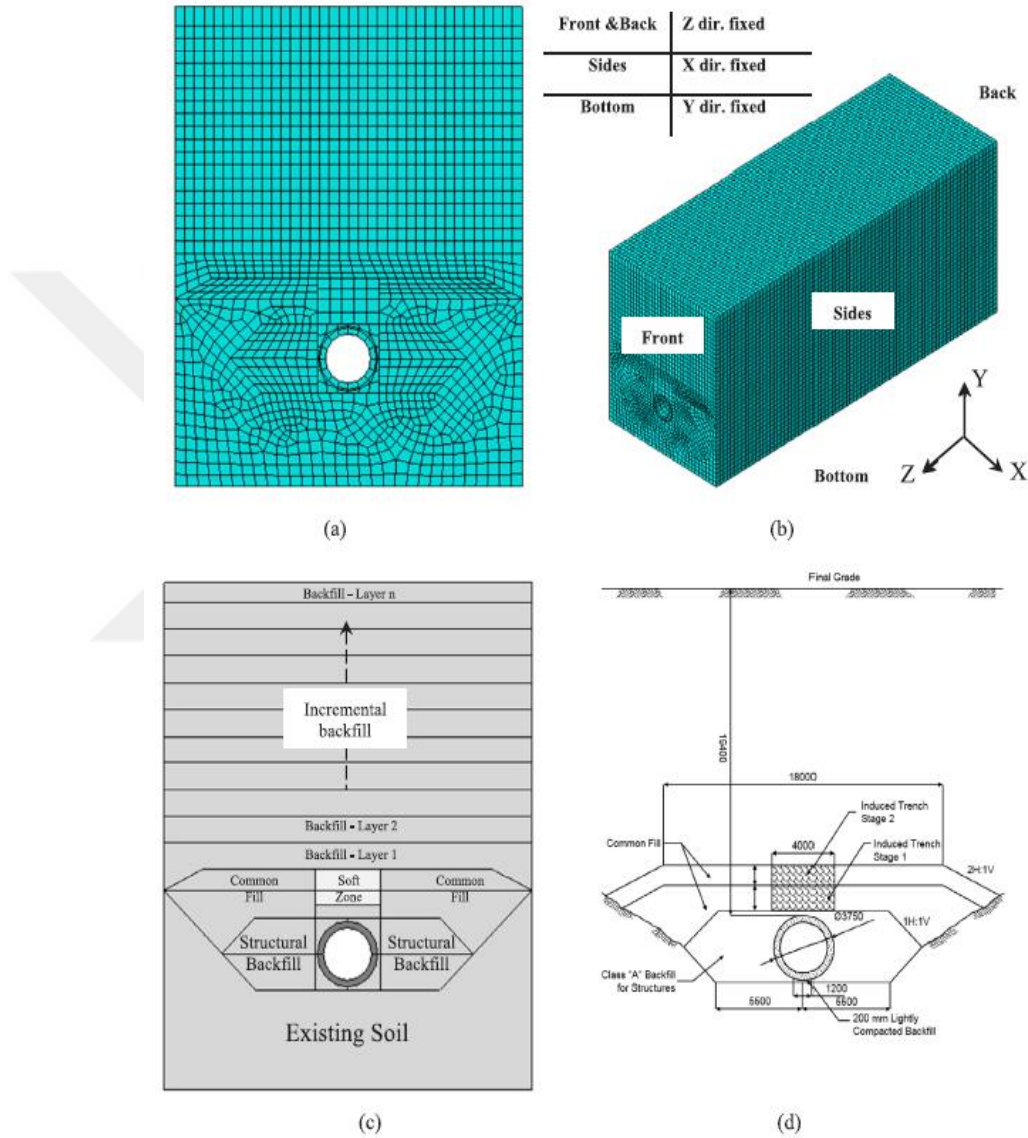


Figure 4.5: Schematic diagram of FE Models and Valsangkar's field instrumentation: (a) section view of FE model, (b) isometric view and boundary conditions of FE model, incremental layers of FE model, and (d) Valsangkar's field instrumentation [Kang, 2019].

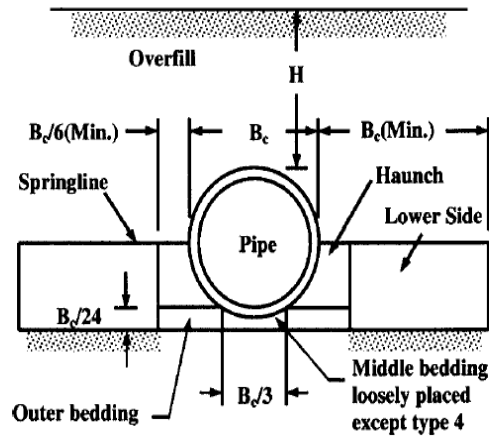


Figure 4.6: AASHTO standard embankment installations _AASHTO LRFD 2004 [Kang et al., 2019].

The results of finite-element modeling [McGrath and Selig, 1999] indicate that the use of uncompacted intermediate bedding, as illustrated in **Figure 4.6**, reduces both the loads on the pipe and the bending moments at the inversion. In order to provide a distinct vertical stress path around the pipe's bottom and support its haunch area, the outer bedding seen in **Figure 4.6** still has to be compacted.

The haunch region, shown in **Figure 4.6**, makes a substantial contribution to the support of the earth load. The bending moments are larger in the case of an uncompacted or untreated haunch, according to both field-testing and computer simulations. Therefore, in this investigation, numerical modeling includes the aforementioned installation aspects.

Figure 4.7 shows the schematic finite-element model's poor trench placement. The model only depicts half of the system due to symmetry. It is generally not necessary for lateral boundaries to extend beyond 2.5 pipe diameters from the pipe center, or 3 pipe diameters in the case of SPIDA and CANDE-89, respectively, according to experience.

The upper borders of SPIDA and CANDE-89 were not allowed to rise more than three and one and a half times the diameter of the pipe, respectively, above the crown of the pipe; at that point, an equivalent overburden pressure.

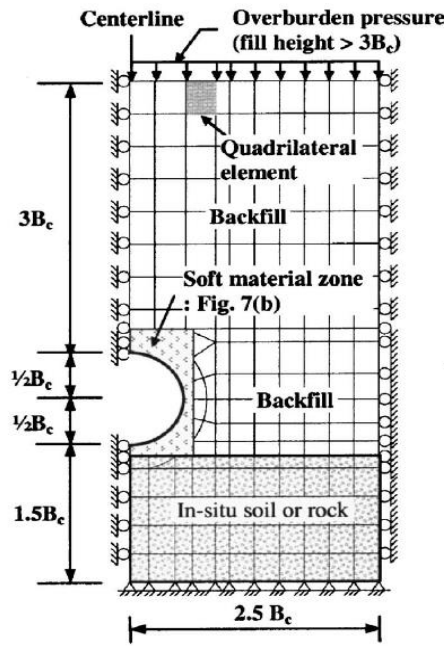


Figure 4.7: Schematic finite-element model imperfect trench installation B_c =width of the pipe; t =pipe wall thickness employed to represent the remaining soil weight [Kang et al., 2019].

The top and lateral limits do not have to extend more than 2.5 diameters from the pipe's center and three diameters from the pipe's crown, respectively, according to the FEA results of this study. Below the invert of the pipe, a hard layer 1.5 times thicker than the pipe's diameter was laid since dirt below this depth had no effect on the results.

A curved beam element is used to simulate the pipes. It was assumed that the unit weight of concrete pipe, Poisson's ratio, and modulus of elasticity E were each 25 GPa, 3,600 ksi, 0.20, and 23 kN/m, 3145 pcf, respectively. For the soil, a plane strain linear elastic element was utilized. Two stiffness parameters, tangent modulus and bulk modulus, were used to characterize the elastic characteristics of the soil.

Tarek et al., 2018 performed a laboratory testing. The loading structure for the steel tank has the following measurements: 1400 mm in length, 300 mm in width, and 1200 mm in height. **Figure 4.8** shows how UPVC pipe was installed in position to cross the tank sides. In order to explore the various overburden geofoam approaches, a number of tests were conducted. This approach used EPS blocks as an inadequate ditch overburden model. Sand was compacted to a height of 20 mm above the pipe after being poured over it. The procedure shown in **Figure 4.8** involved the placement of An EPS geofoam block on top of 550 x 100 mm of sand, followed by the addition of 300 mm of compacted sand fill over the pipe crown.

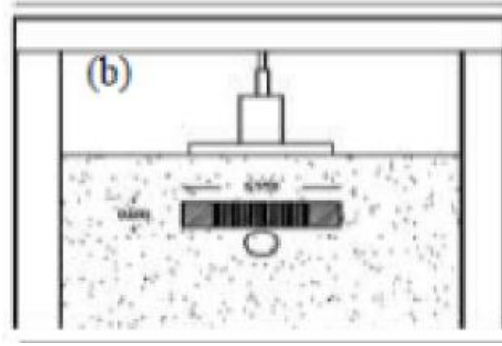


Figure 4.8: The experimental model Imperfect ditch method [Tarek et al., 2017].

The finite element program PLAXIS-3D was used to simulate and validate the experimental models in this work, as shown in **Figure 4.9**. The experimental model's measurements—1380 mm for length, 1000 mm for height, and 300 mm for width—match those of the computational model.

The pipe's six circular segment parts were arranged as plates, with a diameter of 110 mm and a wall thickness of 4 mm. The model was developed in three dimensions and characterized as a plane strain with fifteen node components. The Mohr-Coulomb plasticity model was used to undrain the sand, and the EPS model was a linear elastic model. For levels up to 180 kPa, this study forecasts surface displacement, crown, and spring line deformation that will be compared to experimental findings from surface stress.

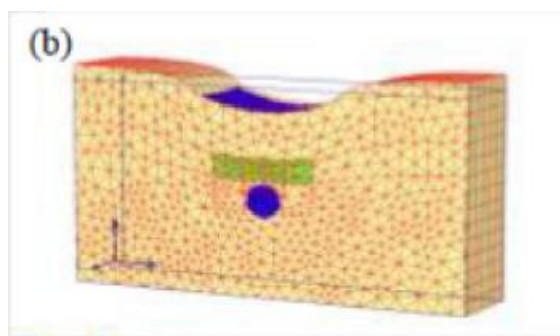


Figure 4.9: Numerical model in case of EPS block Imperfect ditch method [Tarek et al., 2018].

The verification of a three-dimensional finite element model created to mimic the scenario of a pipeline buried in an artificial trench [Beju and Mandal, 2017]. Using PLAXIS 3D, numerical analyses were carried out. When the pipe was positioned two

depths below the loading plate, the simulation was run both with and without jute-EPS geofoam. The goal was to create a finite element analysis model and material conditions that were comparable to those found in a laboratory. The same model geometry was first built in the finite element program across a surface area of (700 × 360 mm) in order to produce a surface to represent jute geotextile, a bottom volume of 110 mm height to represent the stone dust bedding, and a diameter of 110 mm and 360 mm length horizontal tube to symbolize the underground pipe, The PLAXIS 3D program's appropriate material models and attributes were then given to the complete geometry in order to accurately reflect the laboratory model. Mesh creation was the following stage; from the many mesh element distribution types available in PLAXIS 3D for this investigation, the medium mesh distribution was optimized. The mesh distribution (connectivity plot) for 2D burial depth and the three-dimensional finite element model of the pipe-fly ash system with and without jute geotextile-EPS geofoam scenario are shown in **Figure 4.10**.

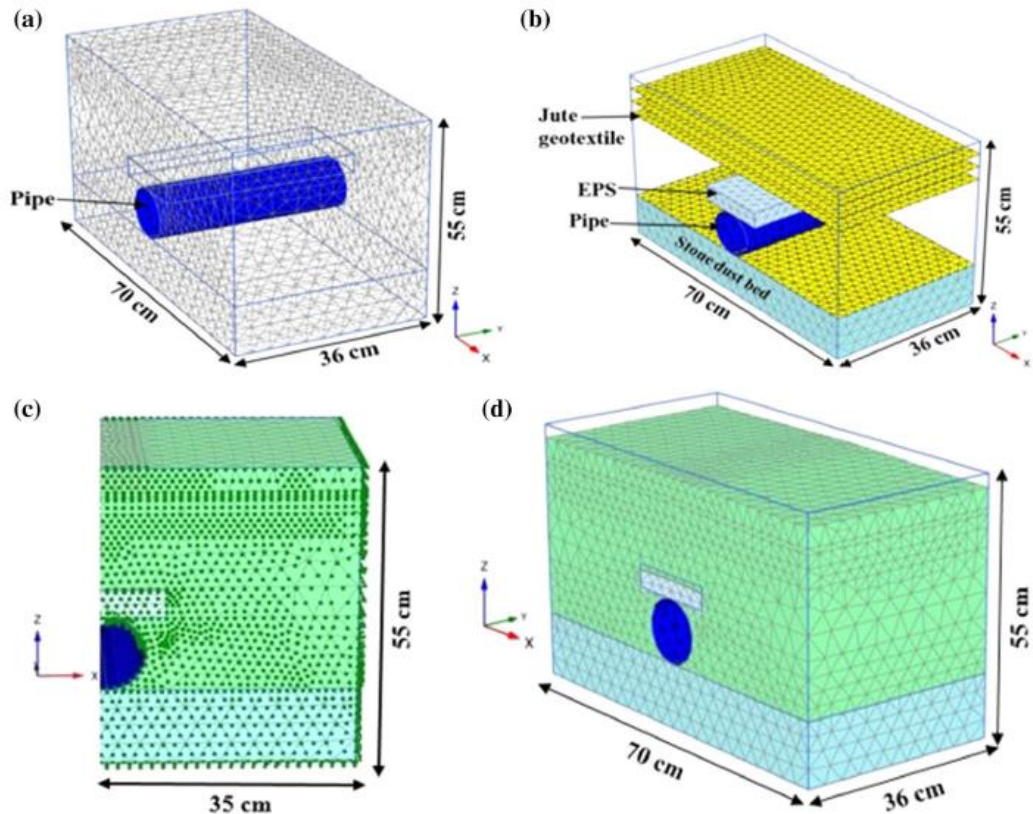


Figure: 4.10: 3D FE model of the pipe-fly ash system for 2D burial depth, a) skeleton view unreinforced case, b) skeleton view reinforced case, c) boundary conditions d) mesh distribution (connectivity plot) [Beju, 2017].

A Mohr–Coulomb nonporous material was used to mimic the EPS geofoam. Since EPS has stress-strain behavior that is similar to that of a fully plastic, linearly elastic material. Several scholars have created comparable models for EPS geofoam in PLAXIS [Ikizler, 2008, Abdelrahman, 2006, Padade, 2012]. For validation reasons, the experimental investigation employed EPS geofoam blocks with different densities (12, 15, and 20 kg/m³), 40 mm in thickness, and 1.5D in width.

For this investigation, the Poisson's ratio of EPS geofoam was determined using the empirical equation $\nu = 0.0056\rho + 0.0024$, where ρ and ν stand for the density of EPS geofoam (kg/m³) and Poisson's ratio, respectively [EDO, 1992].

The results of an unconfined compression test were used to calculate the EPS geofoam's elastic modulus. To simulate the behavior of the pipe, the linear elastic model was employed. In a prior investigation, similar values for the Poisson's ratio and modulus of elasticity were reported for pipes of the same size and type [Tafreshi, 2007]. It was determined that the pipe material's unit weight was 9.57 kN/m³. Both the stone dust and the fly ash infill materials were simulated using the Mohr–Coulomb model. The secant tangent-modulus, E'_{50} was determined by taking the elastic modulus of fly ash and stone dust. This modulus corresponds to 50% of the ultimate deviator stress.

4.1. Prototype Case

A full-scale test on concrete pipe with compressible layer of geofoam was instrumented by Vaslestad, 1990 was evaluated in detail. The construction, which is part of the drainage system for the new Euroroad 18 in Telemark County, Eidanger, is located beneath a 14-meter-tall rock embankment. The concrete pipe measures 1.6 m in diameter inside, 1.952 m in exterior diameter, and 0.176 m in concrete thickness. Two layers of steel reinforcement were used to strengthen the pipe. The diameter of the steel bars was 12 mm, and they were arranged with a 51 mm center-to-center spacing. This results in an area of 0.222 cm²/cm for reinforcing on both the inner and outer layers. 500 MPa is the steel's yield strength.

The concrete has a 50 MPa compressive strength. The overall length of the concrete pipe is 70 meters, composed of 1.5-meter-long pieces. The standard EPS-block

dimensions were 0.5 x 1.0 x 2.0 m. Six EPS test specimens indicated an average compression strength of 98.3 kPa. Density was found at 20.3 kg/m³.

The Standard Proctor curve and particle size distribution and proctor test result of the material utilized for backfill are depicted in **Figure 4.11**. Tests on the density of nuclear fields revealed an average of 100% Standard Proctor (15 tests). Layers of backfill that were compacted in 20 cm lifts.

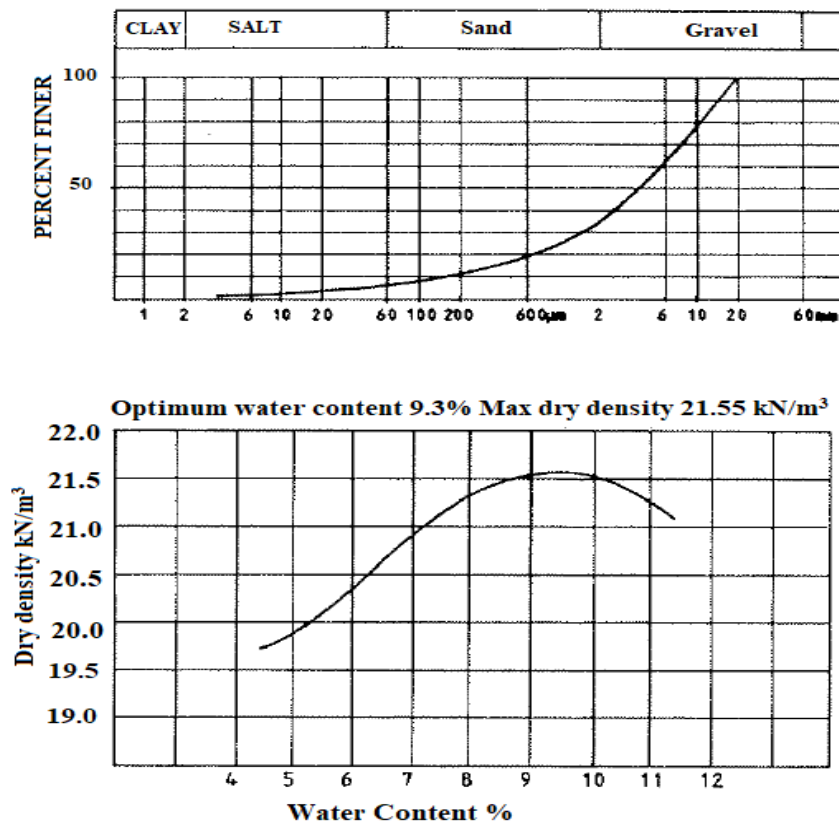


Figure 4.11: Particle size distribution and standard proctor curve for backfill material [Vaslestad, 1990].

The backfill extends 0.5 meters above the top of the pipe and 1 meter from the spring line. The rock-fill that made up the remaining fill of the embankment was stacked three meters thick and crushed over the course of six to eight passes using a six-ton vibratory roller. According to field experience, this is equivalent to 95–97% Standard Proctor. Construction was initiated in August 1988 and ended in June 1989. In September 1988, the embankment was filled to a height of five meters above the pipe. The remaining embankment was constructed using rockfill that was made available by the construction operation (**Figure 4.12**).

The pipe and the surrounding backfill were instrumented with hydraulic earth pressure cells of the Glötzl type to test the performance of the pipe and the EPS during construction and over time. Furthermore, settlement tubes were put on top of the pipe and geofoam to detect vertical deformation.

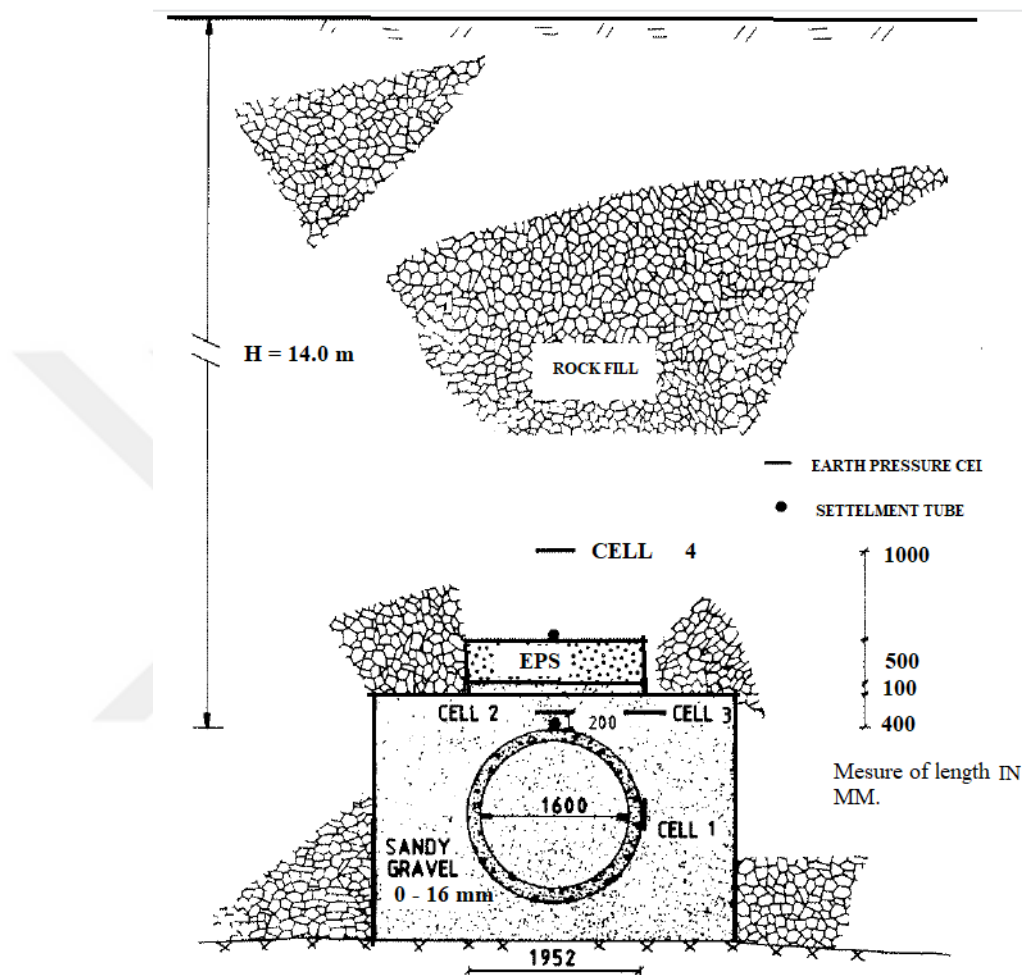


Figure 4.12: Geomtry of instalation of instromentation [Vaslestad, 1990].

The Glötzl-cells were 20x30 cm in size, with four cells employed. **Figure 4.12** depicts the position of the cells. One cell (No. 1) was put in the spring line to measure the horizontal ground pressure on the pipe.

One cell was placed in the spring line to measure the horizontal ground pressure on the pipe (No. 1). Two cells were placed 20 cm above the top of the pipe, one in the centerline (No. 2) and one above the springline (No. 3), in order to measure the vertical ground pressure. To determine the embankment's vertical earth pressure, the last cell (No. 4) was placed two meters above the pipe's top in the centerline. Temperature

adjustments were made after thermometers were used to measure the soil's temperature at the cell locations.

A profile gage, comprising a nearly horizontal plastic tube and a pressure transducer that can be dragged along the pipe, was used to measure the deformation of the EPS. By taking readings at several locations inside the tube, the complete vertical profile may be ascertained. Data for determining vertical deformation can be obtained from variations in the vertical profile over time. To verify the correctness of the whole profile gage, one plastic tube was placed on top of the pipe. The measured vertical earth pressure in cells no. 2 and 3 is shown in **Figure 4.13**. The values from the earth pressure cells for cells 2 and 3 are shown, and the fill height H above the pipe's top is displayed on the horizontal scale. Also displayed is the overburden pressure (γH -line). For the rock fill, an average unit fill weight of 20 kN/m^3 is utilized; for the EPS, it is 20 kN/m^3 . Up to a fill height of 4.7 m, the earth pressure in cells 2 and 3 is essentially linear. At this level cell 2's earth pressure is 62 kPa or 73% of the overburden. At this point, cell 3's ground pressure is 84 kPa which is equivalent to the overburden. Cell 2 has an earth pressure of 72 kPa and a fill height of 8.3 m. At this level, cell 3's earth pressure is 112 kPa or 72% of the overburden. By raising the fill height from 8.3 m to 13.7 m, there is no additional increase in earth pressure on cell 2. Cell 2's earth pressure is 67 kPa, or 25% of the overburden, at a full fill height of 13.7 meters. 43% of the total overburden is represented by the average earth pressure at the top of the pipe. **Figure 4.14** displays the observed vertical earth pressure on cell 4 in the fill, which is located 2 meters above the pipe's top.

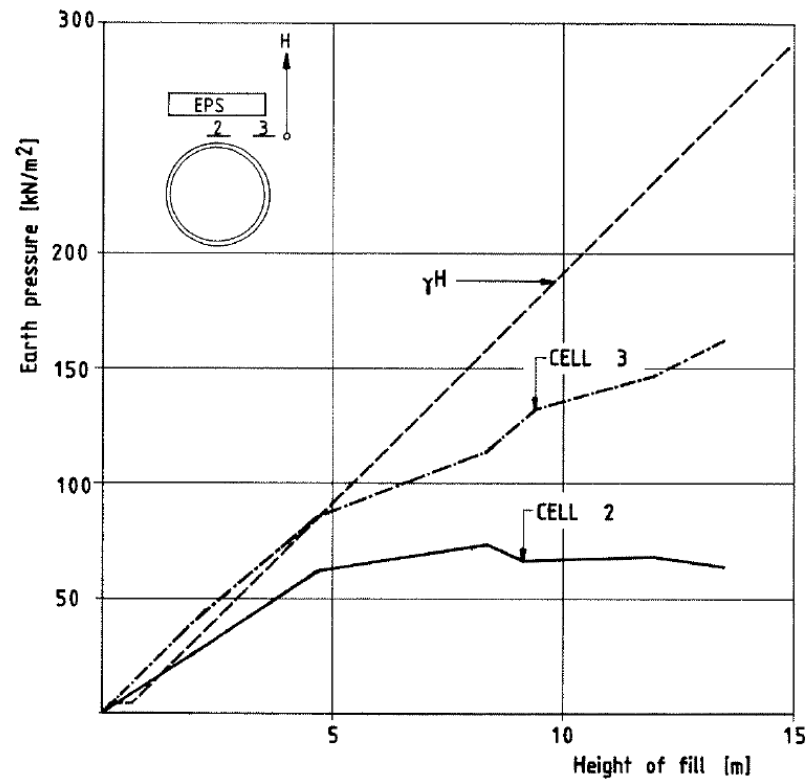


Figure 4.13: Measured vertical earth pressure above the pipe [Vaslestad, 1990].

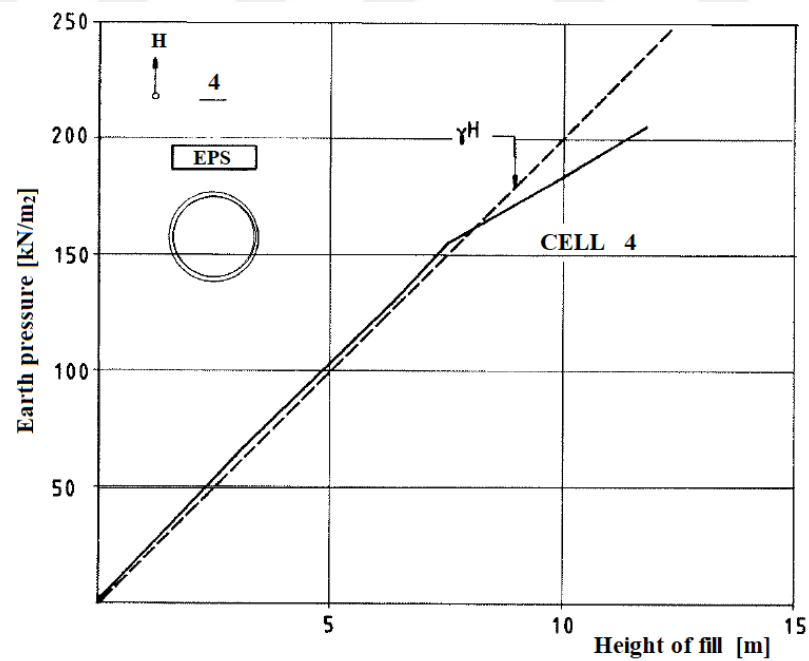


Figure 4.14: Measured vertical earth pressure on fill [Vaslestad, 1990].

Up to a fill height of 7.5 meters above the cell level, the observed vertical earth pressure on cell 4 is marginally greater than the overburden.

The measured earth pressure at full fill height is 205 kPa, or 86 percent of the overburden. **Figure 4.15** displays the observed horizontal earth pressure on the pipe spring line.

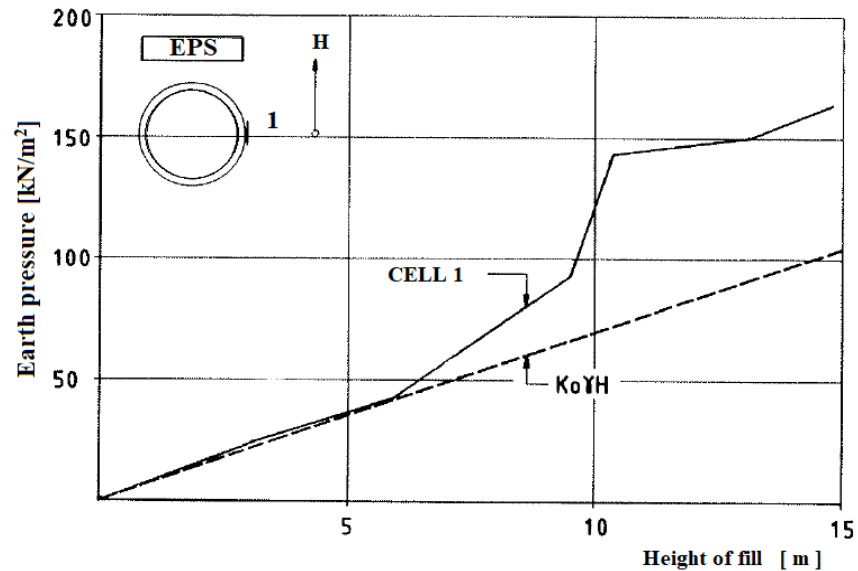


Figure 4.15: Measured horizontal earth pressure on the pipe [Vaslestad, 1990].

The figure also displays the computed horizontal at rest pressure ($K_0\gamma H$), with $K_0 = 0,35$ being the anticipated value.

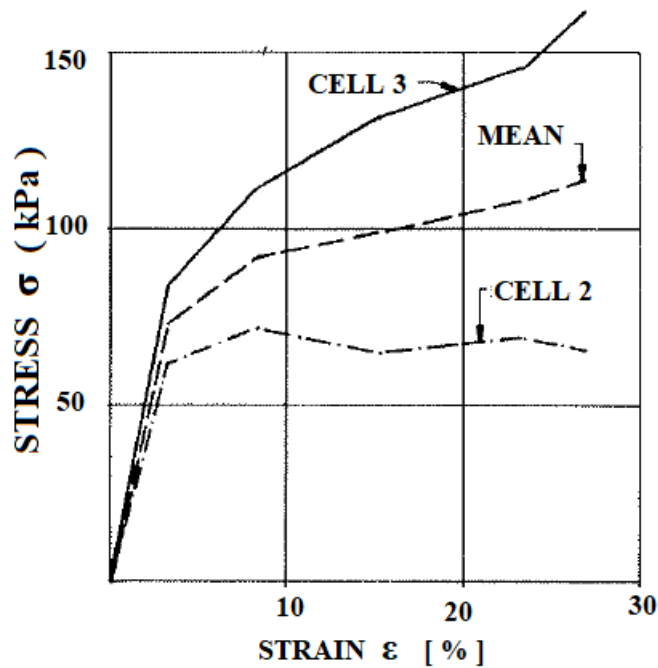


Figure 4.16: Stress-strain curve for EPS based on full-scale test [Vaslestad, 1990].

Figure 4.16 illustrate Stress-strain curve for EPS based on full-scale test . the fill height H is calculated at the cell level. The assured earth pressure is 43 kPa up to a fill height of 6,0 m above cell level, which is somewhat higher than the horizontal at rest pressure. The measured earth pressure is 94 kPa at fill height 9.5 m, and there is a significant rise in measured horizontal earth pressure to 144 kPa at fill height 10.4 m, which is 197% of the horizontal at rest pressure. Filling to 14.8 m above cell level raises the observed ground pressure to 164 kPa, which is 158 of the horizontal at rest pressure. This horizontal earth pressure is 55 of the overburden and corresponds to a horizontal earth pressure coefficient $K = 0.55$. Long-term earth pressure measurements show that there is no rise in earth pressure for six months following construction.

4.2. Experimental Results for Deformation

The measured vertical deformation of the EPS is shown in **Figure 4.17**. At a fill height of 4 m above the EPS, the distortion is 1.7 cm. At a fill height of 7.8 the distortion increases to 4.2 cm, and from this fill height to 8.3 the deformation increases to 7.7 cm. filling to 12.7 m raises the distortion to 13.6 cm, which is 27% of the EPS's initial thickness.

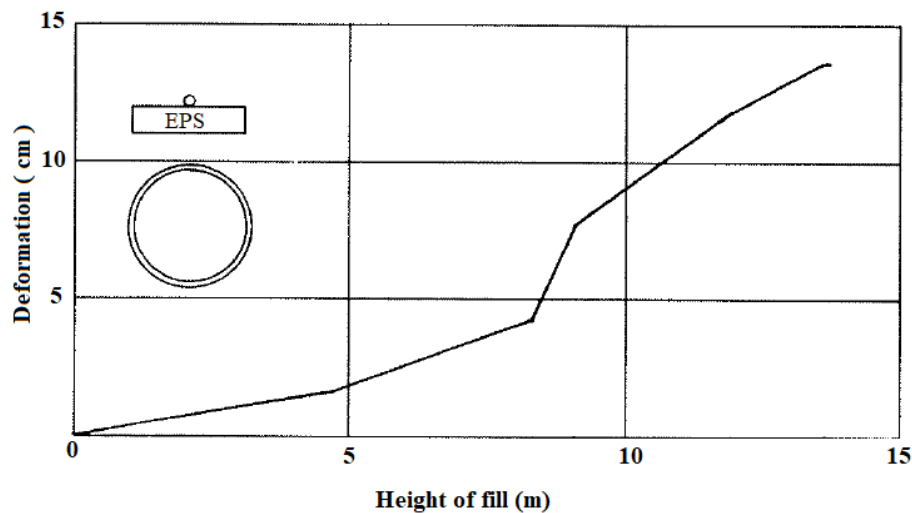


Figure 4.17: Deformation of the EPS [Vaslestad, 1990].

When these observed curves are compared to laboratory observed stress-strain curves **Figure: 4.18** it is clear that their shapes are similar.

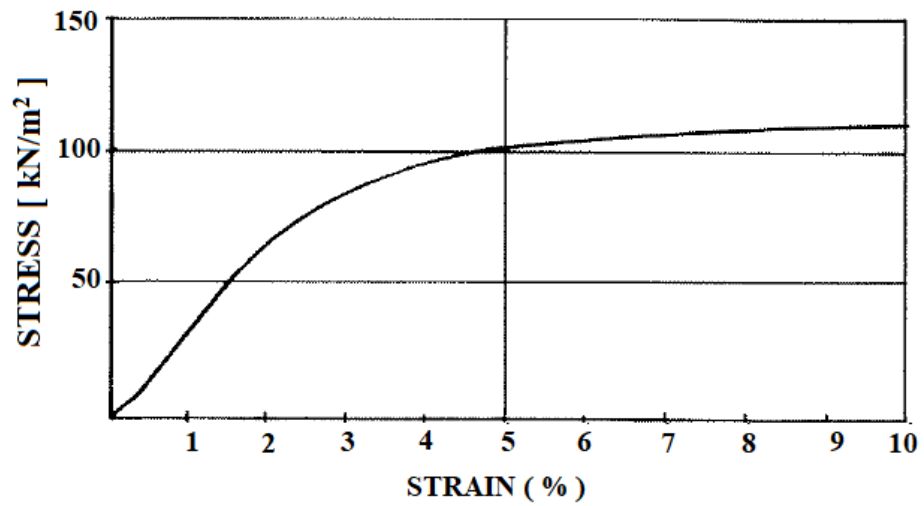


Figure 4.18: Typical stress – strain curve for EPS [Vaslestad, 1990].

Long-term measurements demonstrate that the vertical deformation of the EPS does not rise 6 months after construction is completed.

5. NUMERICAL ANALYSIS AND RESULTS

5.1. Field Installation

A well instrumented case study of [Vaslestad, 1990] was evaluated for verification and calibration. The field installation of a concrete pipeline, constructed in 1988, with 1.95-meter indiameter that sits below a 14-meter-tall rock-fill embankment at the section of Euro Road in Telemark country, Edanger. **Figure 5.1** shows the dimensions of the backfill zone as well as the instrumented cross section. The in-situ soil was removed and replaced with 0–16 mm sandy grave1.

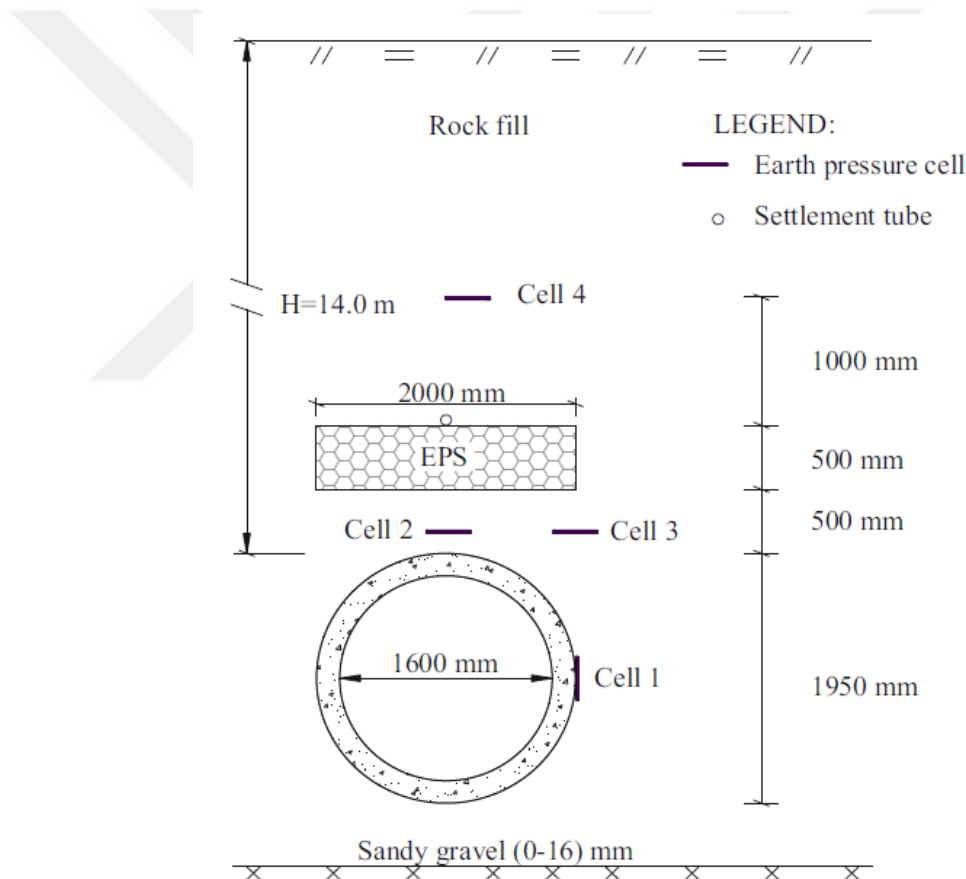


Figure 5.1: Locations of pressure cells and EPS geofoam above the pipe [M. Sayd et al., 1992].

After the in-situ soil was removed to bedrock, approximately 0.5 meters below the pipeline, and filled with 0–16 mm sandy grave1, the pipe was installed. The same material (zero to sixteen millimeters of sandy gravel) with a 97% Standard Proctor Compaction requirement. The backfill was one meter long, starting at the springline

and going 0.5 meters above the top of the pipe. The remaining fill of the embankment is composed of compacted rock-fill that has a Standard Proctor of 95–97% constructed in three-meter-thick lifts. The project began in August 1988 and was completed in June 1989. When the fill height was 8.3 meters in September 1988, the total pressure on cell 2 reached 72 kPa and it did not increase with a fill height reached to 13.7 m.

The earth pressure on top of the pipe, which made up 25% of the expected overburden pressure, was constant between 1996 and 2008 at 66–68 kPa [Vaslestad, 1990]. Around 140 mm is where the vertical deformation stabilizes. The data showed no further increase after more than a decade of measurement. 28% of the EPS block's original thickness is represented by the measured deformation [Vaslestad, 1990].

5.2. Numerical Modeling

To mimic the behavior of geofoam block compressible inclusion under overburden pressure, some studies have utilized numerical analysis [Bartlett et al. 2015; Abdollahi and Tafreshi, 2017; Beju and Mandal, 2017; Kang, 2019; Hassan, 2021]. The geofoam block was modeled by [Bartlett et al., 2015] using the finite difference program FLAC 2D. The Mohr-Coulomb (MC) constitutive model was used for the initial modeling. For stress levels below the MC failure envelope, this model use linear-elastic behavior. It was observed that the anticipated outcomes were mostly unaffected by the non-linear modulus applied to the geofoam. [Abdollahi and Tafreshi, 2017] evaluated the system's performance in safeguarding lifelines under static stresses using ABAQUS software and a 3D numerical technique. A Mohr-Coulomb constitutive model with elastic-perfectly plastic properties is used to predict soil behavior. Using the finite element program PLAXIS-3D, [Hassan, 2021] employed a numerical model to simulate and validate the experimental model findings. It was determined that the EPS block was undrained by applying the elasto-plastic Mohr-Coulomb model.

[Beju and Mandal, 2017] validated a three-dimensional finite element model developed to simulate the case of a buried pipeline in an induced trench condition. The EPS geofoam was modeled using a Mohr-Coulomb nonporous material.

In this analysis, a comprehensive finite element modeling effort was done to calibrate [Vaslestad, 1990] field study by employing approximation modeling and analyzing

the function of geofoam inclusion in minimizing soil pressure imparted to underground pipes. Following [Beju and Mandal, 2017] and [Hassan, 2021] an attempt was made to approximate numerical modeling in order to replicate the behavior of EPS. The possibility of a freely accessible linear elastic model and soft soil (SS) model, which are used to assess effectiveness of geofoam inclusion to reduce the soil pressure applied to the buried pipes. The model geometry is provided in **Figure 5.2**.

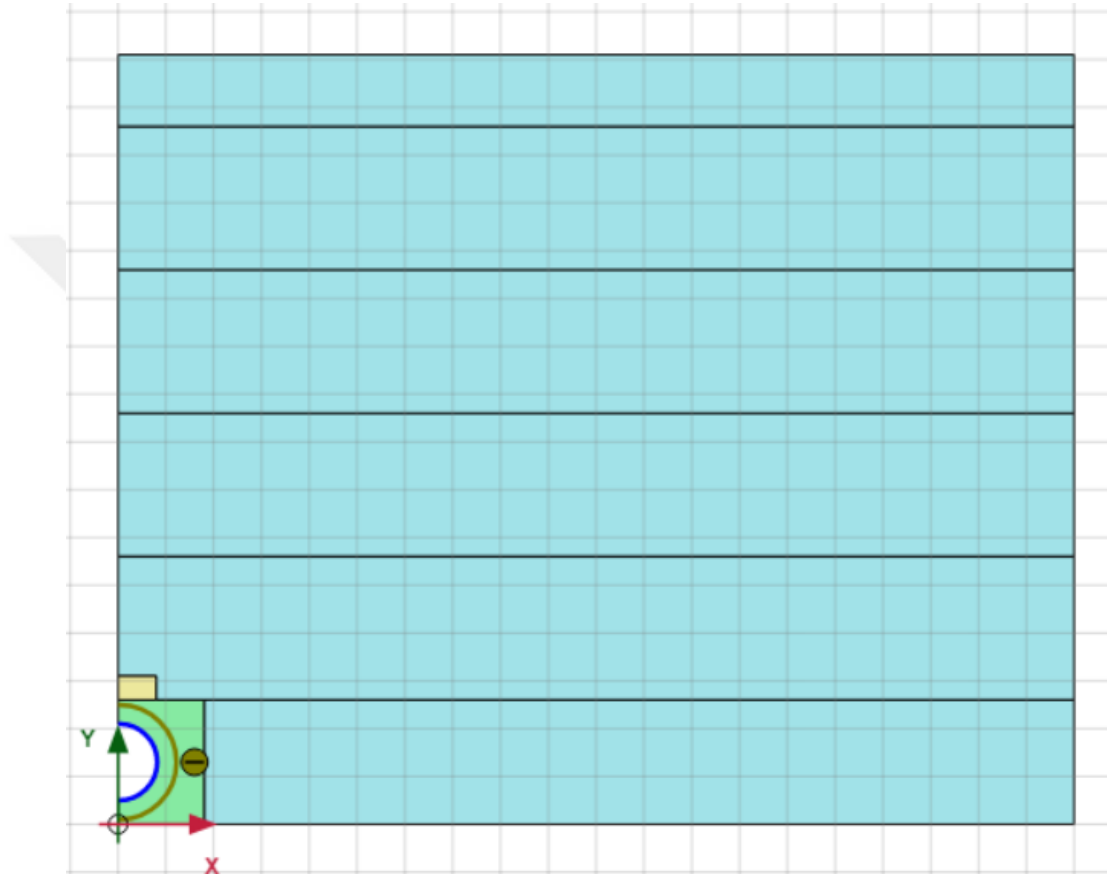


Figure 5.2: Cross-sectional view of PLAXIS model geometry.

A medium mesh was used for all layers and a fine mesh around the pipe due to the increased concentrations of stress at this area, as shown in **Figure 5.3**.

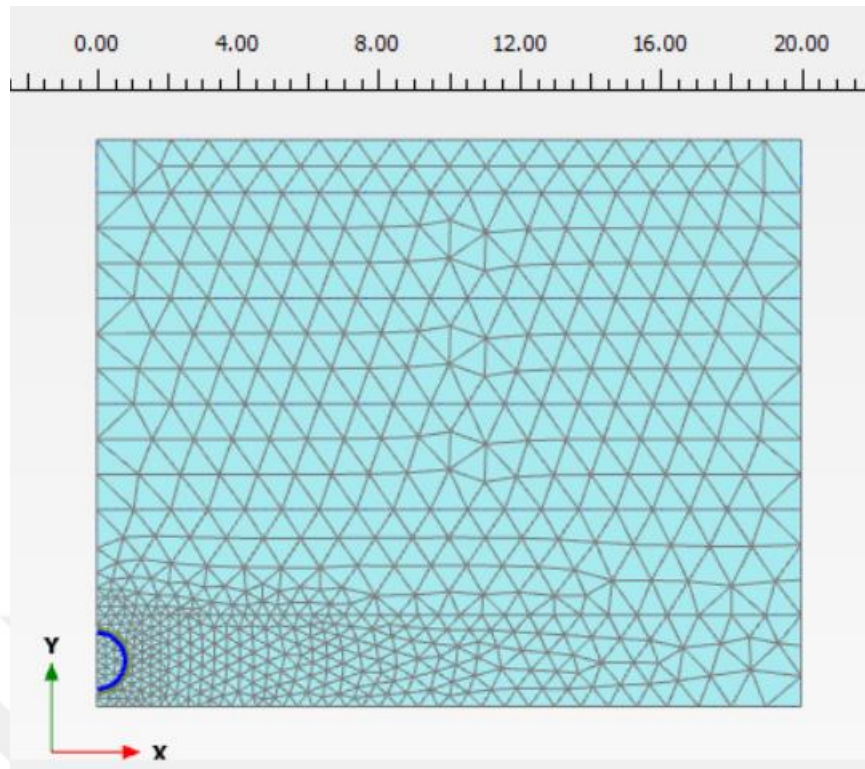


Figure 5.3: Cross-sectional view of mesh generation.

The establishment of the geofoam block embankment was modeled using PLAXIS 2D's "staged construction" function, which allows for a realistic simulation of the construction process (**Table 5.1**). Gravity loading was used at the initial stage, which is a type of plastic calculation. Because geofoam blocks are lightweight, strains resulting from their self-weight would be minimal. As a result, the models' initial condition was the geometry of the embankments following the insertion of the geofoam blocks (**Table 5.1**).

Table 5.1: Construction stages of numerical modeling.

Loading type	Calculation type	Stage	Phase
Initial phase	Completion geofoam block placement	Plastic	Gravity load
1	Activate first lift of 3m embankment	Elastic	Staged Construction
2	Activate second lift of 3m embankment	Plastic	
3	Activate third lift of 3m embankment		
4	Activate fourth lift of 3m embankment		
5	Activate fifth lift of 2m embankment		

A linear elastic constitutive model for EPS was used for the first 3-meter embankment above the pipe to represent the elastic part in the stress-strain curve of geofoam, as shown in the **Figure 5.4**, whereas a soft soil model was applied for the remaining layer to represent a plastic behavior. For soil layers, hardening soil constitutive model was used to mimic soil behavior.

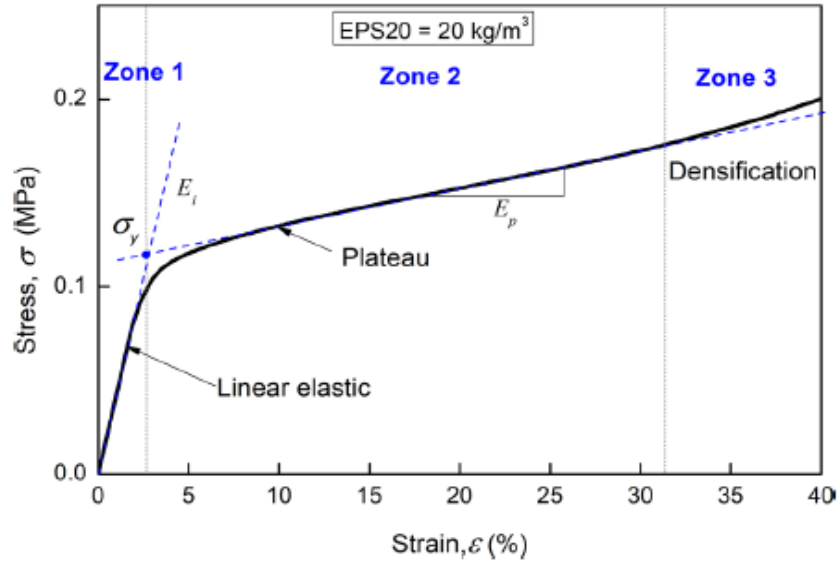


Figure 5.4: Stress–strain behavior of an EPS specimen with a density of 20 kg/m³ [Malai and Youwai, 2021].

According to Stark et al. (2004), the EPS density and EPS initial tangent modulus have a linear relation that may be experimentally described by Equation (2):

$$E_{ti} = 450 \rho_{EPS} - 3000 \quad (2.2)$$

where;

E_{ti} = is the initial modulus at 1%

ρ_{EPS} = is EPS density

Within the vertical stress range, up to the **elastic** limit stress of the EPS block, the EPS-sand interface shear behavior is completely frictional and generally correlates to the internal friction of sand [Xenaki and Athanasopoulos 2001; Özer and Akay 2021]. As so, there will be a possible shear plane in the sand. Therefore, no interface for EPS-sand has been specified in numerical models.

$$c = 1.3226\rho_{EPS} + 8.1416 \quad (2.3)$$

Equation (3) utilized to determine the cohesion of the EPS for the modeling process

As a function of the apparent density of geofoam blocks [Meguid et al., 2019] EPS density and cohesion show a linear trend; in contrast, the results on friction angles are relatively dispersed, with the majority falling within a limited range of around 10°. As

a result, for modeling reasons, the internal friction angle for geofoam was set at 10°, Independent of its density [Özer and Akinay, 2022]. Table (5-2) presents the model parameters and material models that were employed in the numerical modeling for geofoam.

Table 5.2: Model parameters and material models for EPS20.

Parameter	EPS20	EPS20
Material Model	Soft soil	Linear elastic
E_{ti} (Kpa)	-	6000
γ	0.2	0.2
λ	0.1250	-
κ	0.03	0.2
ν_{ur}	0.1	0.1
ϕ	10	0
ψ	-	0
c (kPa)	35	-

The study used a hardening soil model to study soil behavior, which is complex due to multiphase materials. It found that soil behavior is elastic in small stresses, with nonlinear stress-strain relationships. However, soil stiffness is recoupable under unloading conditions. The study found that variations in stiffness can be observed after pre-failure non-linearities, starting with modest shear stresses, which contradicts linear-elastic Mohr-Coulomb models [Rafa Obrzud, 2018]

The commonly observed order of magnitudes of the "static" modulus E_s can be used to approximate the stiffness moduli in the absence of experimental data. With the assumption that the behavior of the soil is in accordance with the stress-strain relation given by Equation (4) and that $\epsilon_1 = 0.1\%$ the "static" modulus may be expressed as follows:

$$q = \frac{\epsilon_1}{a + b\epsilon_1} \quad (2.4)$$

The internal friction was estimated from previous literature. Table 5.3 and Table 5.4 presents the model parameters and material models that were employed in the

numerical modeling for soils and concert pipe. Table 5.3: Input parameters for calibration in PLAXIS 2D.

Table 5.3: Input parameters for calibration in PLAXIS 2D.

		Unit	Sandy gravel	Rock fill
Material model	Model		Hardening soil model	Hardening soil model
Type of material Behaviour	Type	—	Drained	Drained
Young's Modulus	E_{50}^{ref}	—	40000	55000
	E_{oed}^{ref}	—	40000	55000
	E_{ur}		80000	110000
EPS unit weight	γ	(kN/m ³)	21.5	20
Poisson's Ratio un/reloading	ν_{ur}	—	0.2	0.2
Friction angle,	Φ'	(°)	36	38
Cohesion,	C'	(kpa)	5	3
Dilatancy angle	Ψ	(°)	6	8

Table 5.4: Input parameters for Reinforced concrete pipe In PLAXIS 2D.

Material	Unit	Reinforced concrete pipe
Material type	—	Elastic
ν	—	0.2
W	Kn/m/m	25
EA	Kn/m	6.512E6
EI	Kn m ² /m	16.81E3

5.2.1. Results of Verification

The results of the PLAXIS 2D indicated that the selected parameters were closely mimicked the recorded pressures at the field , as **Figure 5.5** shows the shape of the model after completing the calculations, where we can clearly see the area of positive arching above the pipe.

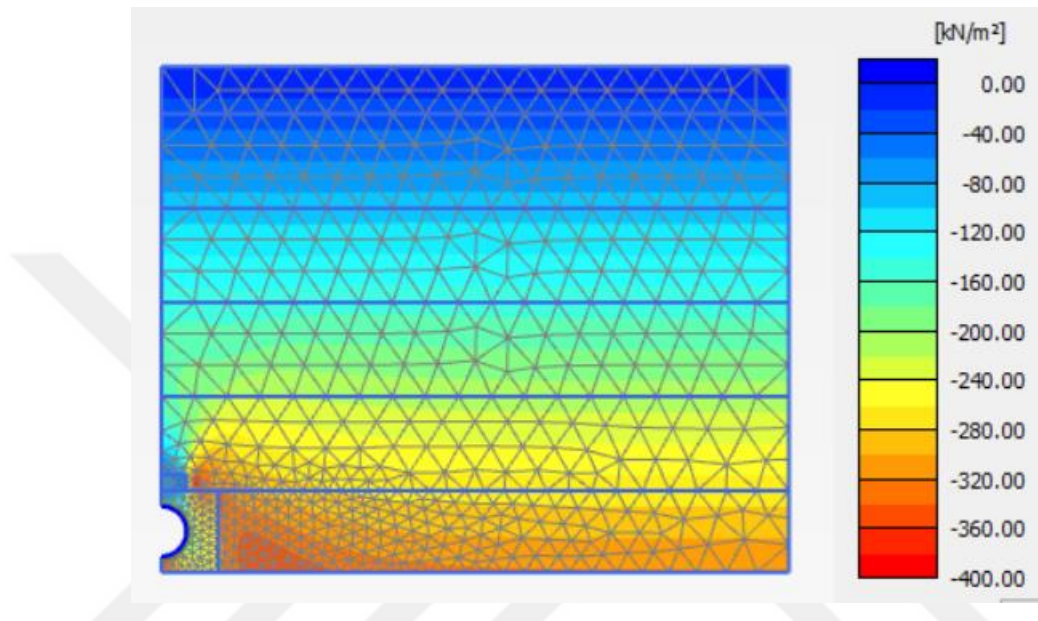


Figure 5.5: PLAXIS 2D model.

By comparing the calibration results for cell No. 2, which is located 20 cm above the pipe in the crown area, and cell No. 3, which is located above the spring line, with the field results of the [Vaslestad, 1990], we can observe the close match between model results and the field measurements (**Figure 5.6**).

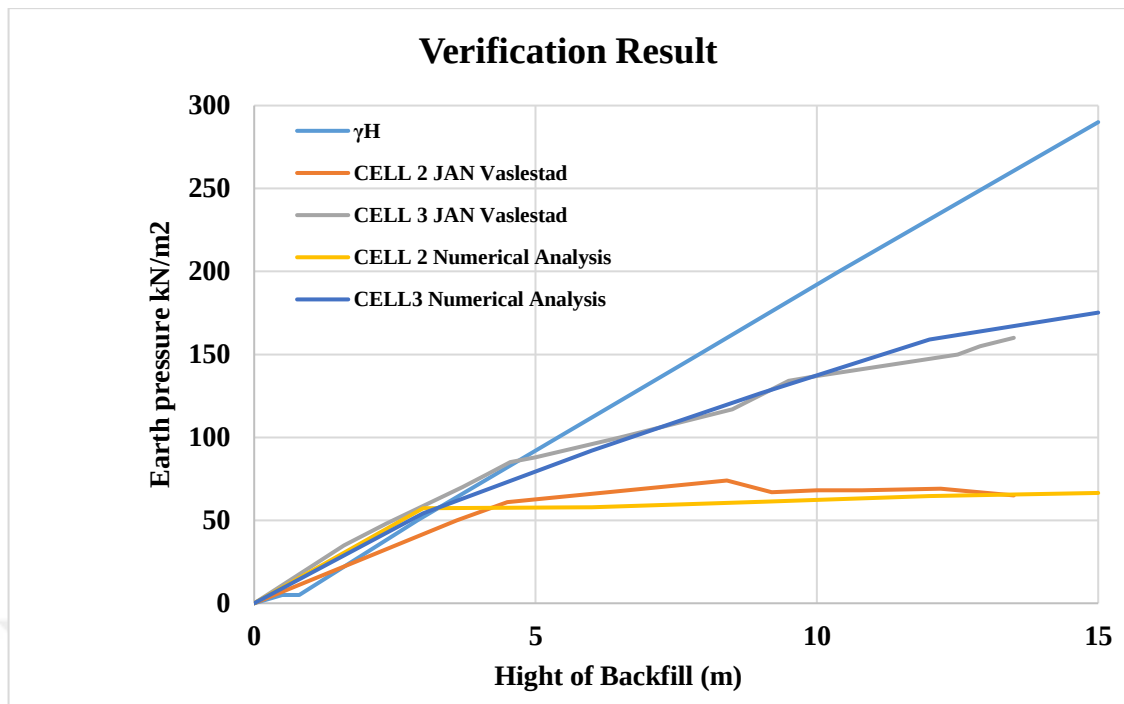


Figure 5.6: Numerical calibration of the field measurements.

5.3. Parametric Numerical Analysis

The 2D numerical analysis using the PLAXIS-2D finite element program to examine the impact of backfill height, thickness, density and geofoam width on reducing soil pressures were performed. To provide insight into how each element affected the soil pressure atop the pipe, **Table 5.5** displays the parameters (width, thickness, density, and backfill height) of the geofoam that was employed in the parametric modeling process.

Table 5.5: Variables in the models (width, thickness, density, and backfill height).

Model number	EPS density	EPS density	EPS density	EPS width (m)	EPS Thickness (m)	EPS backfill height (m)
1	16	20	24	1D	0.5	14
2	16	20	24	1.5D	0.5	14
3	16	20	24	2D	0.5	14
4	16	20	24	1D	1.0	14
5	16	20	24	1.5D	1.0	14
6	16	20	24	2D	1,0	14
7	16	20	24	1D	2,0	17
8	16	20	24	1,5D	2,0	17
9	16	20	24	2D	2,0	17
10	16	20	24	1D	0,5	17
11	16	20	24	1,5D	0,5	17
12	16	20	24	2D	0,5	17
13	16	20	24	1D	1,0	20
14	16	20	24	1,5D	1,0	20
15	16	20	24	2D	1,0	20
16	16	20	24	1D	2,0	20
17	16	20	24	1,5D	2,0	20
18	16	20	24	2D	2,0	20

5.3.1. Results and Discussion

5.3.1.1. Effect of The Width.

After performing analyses on the model with a density of 20 kg/m^3 , the width of the geofoam was changed to 1D, 1.5D, and 2D. It turns out that the pressures at the spring line, minimizes as the width of the EPS block increases. In contrast, the crown area, experiences an increase in pressure as the width of the geofoam increases. This behavior was independent of the density of geofoam blocks. The results were similar when 16 kg/m^3 and 24 kg/m^3 blocks were used as shown in the **Figure 5.7**.

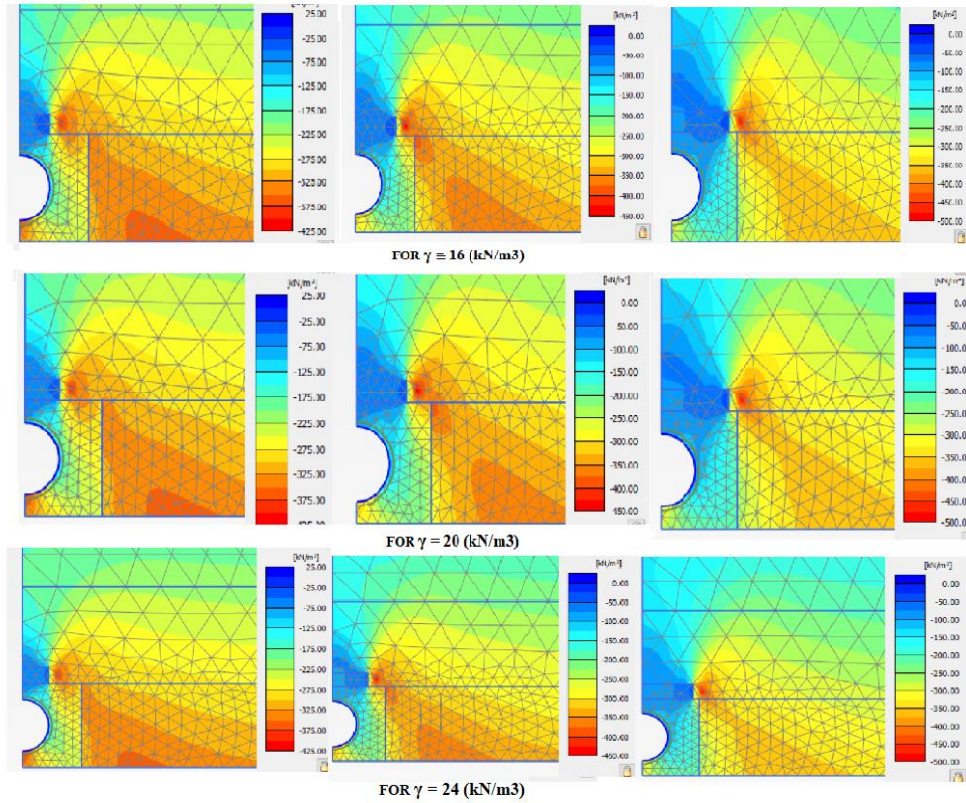


Figure 5.7: Effect of increasing the width of the EPS at different densities.

Calculated pressures at the crown and spring line are summarized in **Figures 5.8** and **5.9** as a function of backfill height and W/D ratios for EPS20. Overburden pressure, $\square H$, was provided in these figures as comparison purposes.

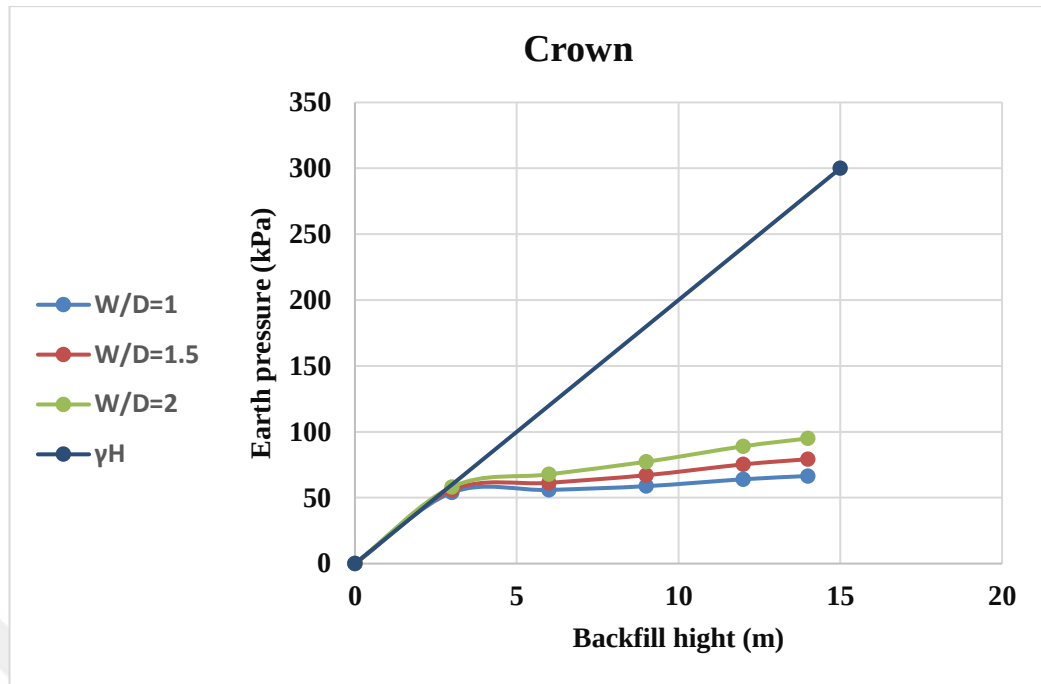


Figure 5.8: Effect of W/D ratio at Crown for EPS20.

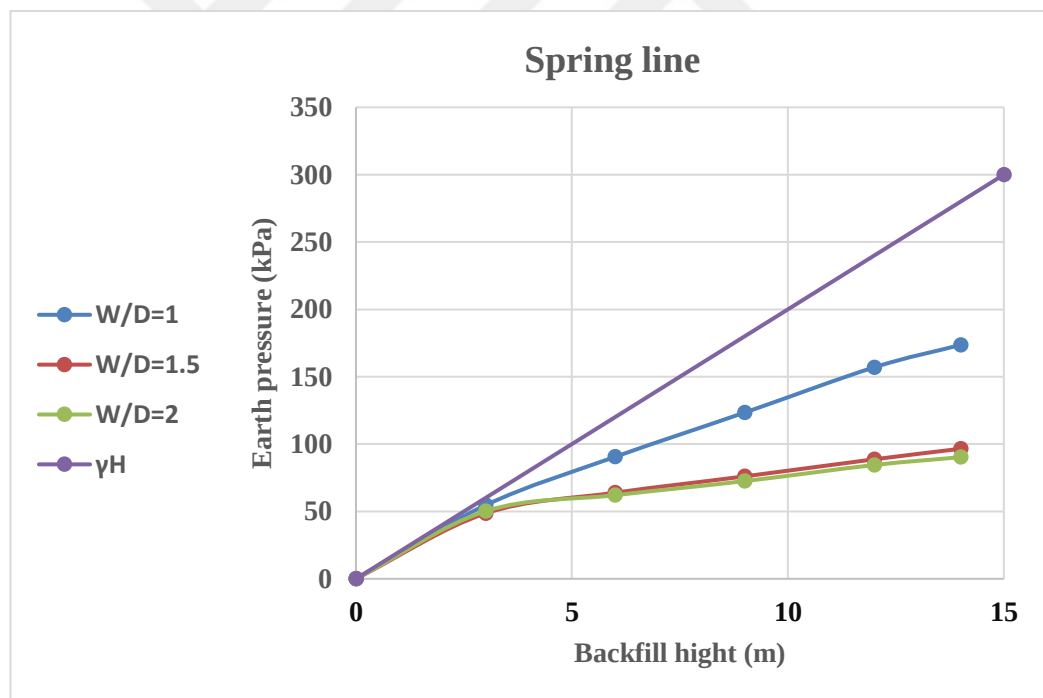


Figure 5.9: Effect of W/D ratio at spring line for EPS20.

From **Figure 5.8**, we can see the effect of the width of the geofoam on the pressure exerted on the crown of the pipe and **Figure 5.9** at the spring line. It turns out that the ideal width for the design was 1.5 D.

Figure 5.10 and **Figure 5.11** illustrate that the earth pressure reduction factor which defined as the ratio of simulated pressure to the overburden pressure if there is no compressible inclusion, is dependent on dimensions of geofilm inclusion. As the width increases, the earth pressure reduction factor decreases. However, extending the width beyond 1.5D does not yield any significant economic benefit.

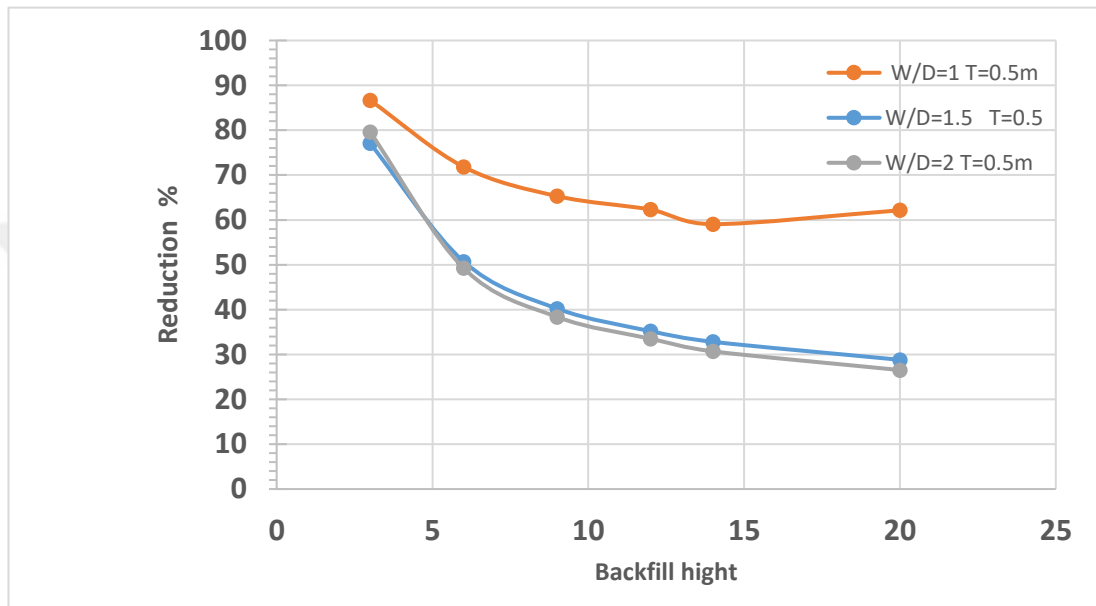


Figure 5.10: W/D reduction factor for different W/D ratios for T = 0.5 m at the spring line.

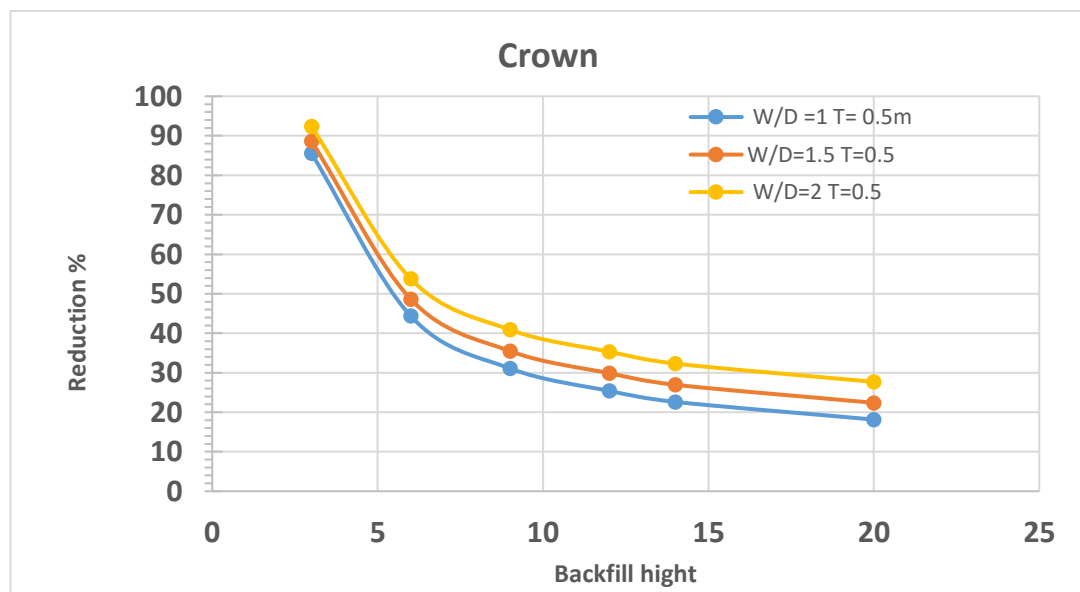


Figure 5.11: Reduction factor for different W/D ratios for T = 0.5 m at the crown.

5.3.1.2. Effect of the EPS Thickness and Backfill Height

The modeling procedure was run using two distinct EPS thicknesses, 0.5 m and 1 m. It was found that the pressure applied to the spring line and the pipe crown decreased as the thickness of the geofoam increased as shown in Figure 5.12.

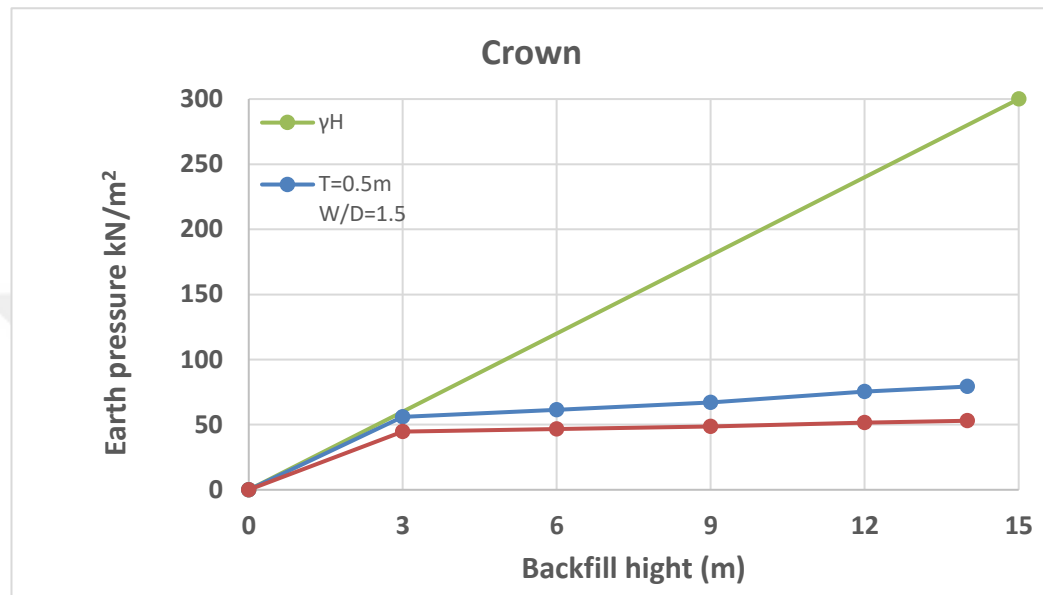


Figure 5.12: Effect of geofoam thickness at the crown.

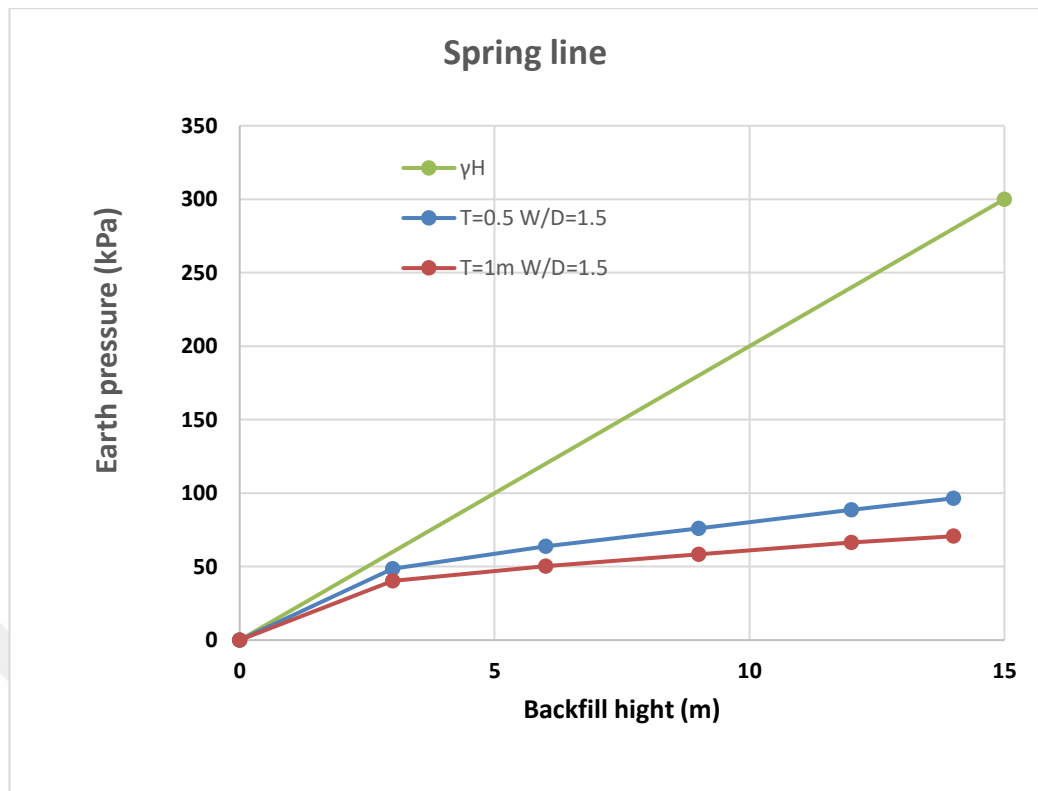


Figure 5.13: Effect of geofoam thickness at the spring line.

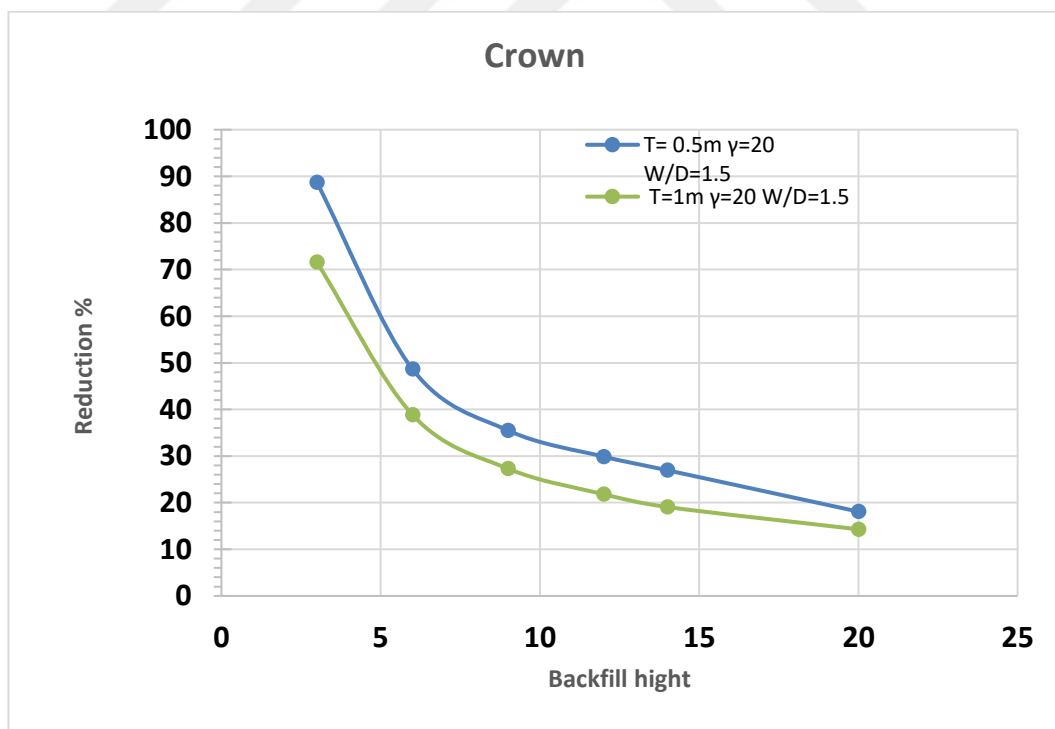


Figure 5.14: Reduction factor for $T=0.5\text{m}$ and 1m for $W/D = 1.5$ at crown.

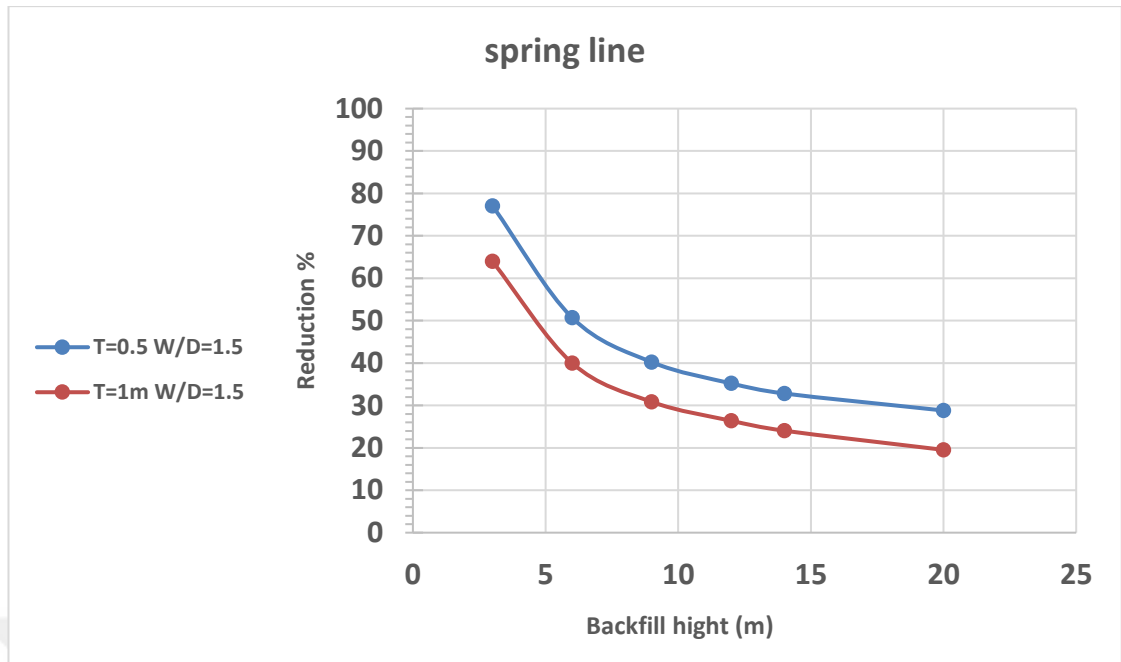


Figure 5.15: Reduction factor for T=0.5m and 1m for W/D = 1.5 at spring line.

We notice from **Figure 5.14 and 5.15** that increasing the thickness of the geofoam from 0.5 m to 1 m does not greatly affect the pressure reduction factor. Therefore, it is not economical to increase the width of the geofoam by more than 0.5 m for the range of thicknesses that were modeled in this study. It is also noticed that increasing backfill height does not affect the pressure applied to the pipe.

5.3.1.3. Effect of Unit Weight

Three densities (16, 20, and 24 kg/m³) were utilized in this investigation to determine how density affected the pressure above the pipe in order to determine the most cost-effective design. **Figures 5.16 and 5.17** show that the reduction factor for the spring line and the crown area, respectively, increases with decreasing density. Therefore, for a more economical design, it is recommended to use a density of 16 kg/m³ Among the ranges of stiffnesses tested in this study.

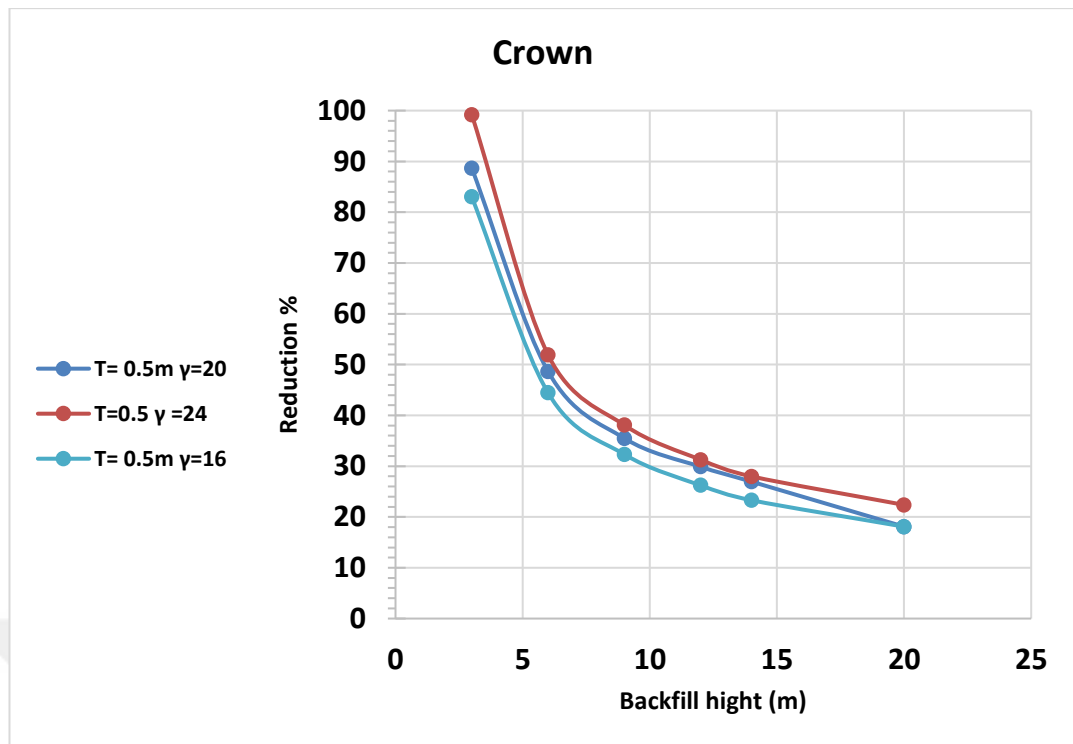


Figure 5.16: Pressure reduction factor of (16, 20 and 24) kg/m^3 at crown.

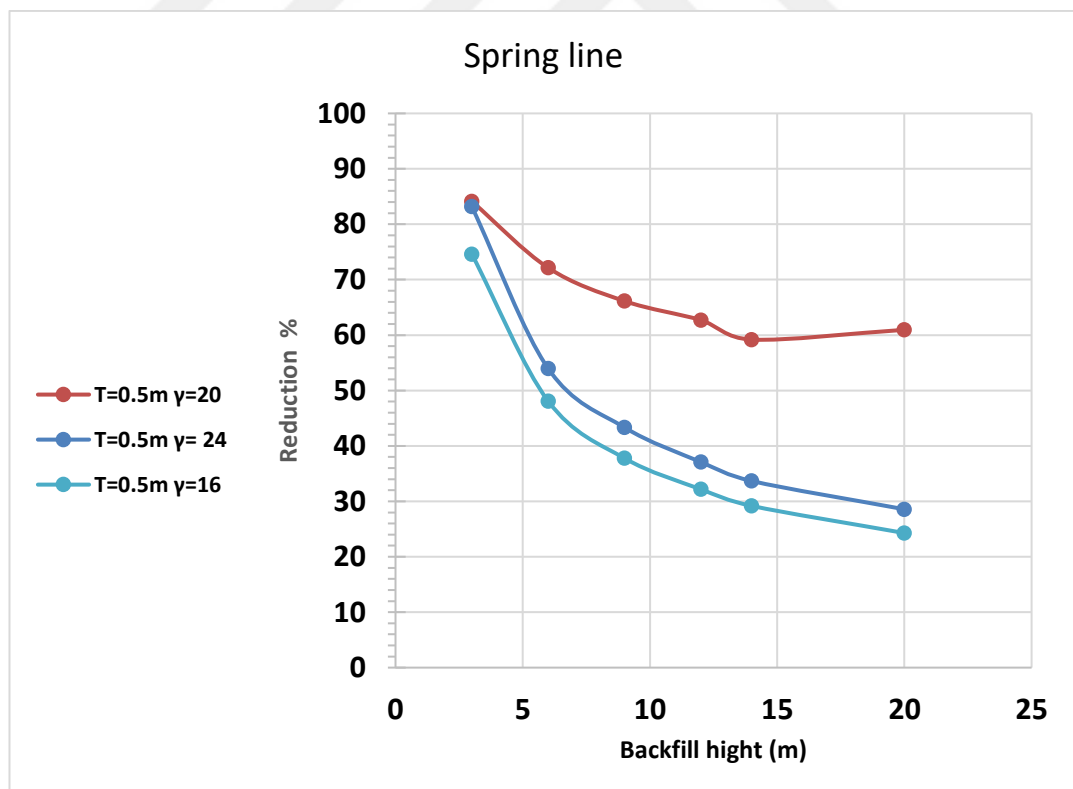


Figure 5.17: Pressure reduction factor of (16, 20 and 24) kg/m^3 at spring line.

6. CONCLUSION

In this study, a historical review of the existing literature on the effectiveness of geofoam to reduce the stress on buried pipes using the imperfect trench method were summarized. In addition, a comprehensive finite element modeling effort aimed at evaluating the role of geofoam in mitigating soil pressure applied to underground pipes. The performance of buried pipelines under service loads based on a well-documented case study was calibrated, followed by extensive numerical analyses to assess the impact of backfill height, geofoam thickness, and width on the reduction of soil pressures. The findings of this study contribute a numerical modeling approach that facilitates economical solutions for the design of buried pipes. The key conclusions drawn from the study are as follows:

1. The calibrated numerical model accurately represents the site's conditions, as its results closely matched the recorded prototype data.
2. Three densities (16, 20, and 24 kg/m³) were analyzed to understand the impact of the geofoam stiffness on the pressure atop the pipe and to identify the most cost-effective design. The findings indicated that the earth pressure reduction factor for both the spring line and the crown decreased as the density decreased. Consequently, a density of 16 kg/m³ is recommended for a more economical design for the range of stiffnesses tested in this study.
3. The modeling procedure was conducted using two different EPS thicknesses, 0.5 m and 1 m. Results showed that increasing the thickness of the geofoam led to a decrease in pressure on both the spring line and the pipe crown. However, from an economic standpoint, the optimal thickness for design is 0.5 m for the range of thicknesses used in this study.
4. Three different width-to-diameter ratios of the geofoam (1D, 1.5D, and 2D) were tested. It was observed that as the EPS block width increased, the pressure at the spring line decreased, while the pressure at the crown increased. Based on these findings, the optimal design width was determined to be 1.5D.

5. Three backfill heights were examined: 14 m, 17 m, and 20 m. It was observed that increasing the backfill height does not significantly increase the pressure exerted on the pipe.
6. These results underscore the potential of numerical modeling to offer practical and economical solutions for buried pipe designs, with specific recommendations on the use of geofoam for stress reduction.



REFERENCES

A. Corporation (2011), GeoGripper Plate. American Foam Manufacturers.

Aabøe R, Frydenlund TE (2011) 40 years of experience with the use of EPS geofoam blocks in road construction. Paper presented at the 4th international conference on geofoam blocks in construction applications, Lillestrøm, Norway, 6-8 June 2011.

Abaqus 2016. Dassault systemes; 2016.

Abdelrahman GE, Elragi AF (2006) Behavior improvement of footings on soft clay utilizing geofoam. The 10th Arab Structural Engineering Conference, 13–15 Nov, Kuwait, pp 333–338.

American Concrete Pipe Association (ACPA). ACPA concrete pipe handbook. Vienna (VA); 2001.

Attapole Malai1 · Sompote Youwai1, stiffness of expanded polystyrene foam for different stress states international journal of geosynthetics and ground engineering (2021) 7:80.

Attapole Malai12017, Sompote Youwai2 and Nattaporn Jaturabandit3.

B. Plastics, Styropor – Highway Construction/Ground Insulation: Rigid Styropor Foam as a Lightweight Construction BASF Material for Highway Foundations, BASF, Ludwigshafen, Germany, 1998.

Bartlett S F, Lingwall B N, Vaslestad J. (2015) "Methods of protecting buried pipelines and culverts in transportation infrastructure using EPS geofoam" Geotextiles and Geomembranes. Pp1- 12.

Bartlett, S.F., Lingwall, B. N., Trandafir, A. C. And Lawton E. C. (in press). "Protection Of Steel Pipelines from Permanent Ground Deformation Using EPS Geofoam," in Increasing the Seismic Resilience of Natural Gas Systems - Select Topics of Interest, ASCE Technical Council and Lifelines and Earthquake Engineering, 33 p.

Bartlett, S.F., Lingwall, B.N., & Vaslestad, J. (2015). Methods of protecting buried pipelines and culverts in transportation infrastructure using EPS geofoam. Geotextiles and Geomembranes, 43(5), 450-461.

BASF Corp., (1997) "Styropor Technical Information" Technical Information. BASF Corp., Germany.

BASF. (1998). "Styropor technical information." Ludwigshafen, Germany.

Beinbrech G. Hillmann R. (1997) EPS in road construction—Current situation in Germany. Geotext Geomembranes 15(1–3): 39–57.

Choo, Y. W., Abdoun, T. H., O'Rourke, M. J. And Da, H. (2007). "Remediation for buried pipeline systems under permanent ground deformation," *Soil Dynamics and Earthquake Engineering* 27 (2007) 1043-1055.

D. Arellano, Guidelines for the Use of Expanded Polystyrene (EPS)-Block Geofoam for the Function of Lightweight Fill in Road Embankment Projects. University of Illinois at Urbana-Champaign, 2005.

E.L. Santiago, An investigation of expanded polystyrene geofoam mechanical. EDO (1992). Expanded polystyrene construction method. Riko Tosho Publishers, Tokyo (In Japanese).

Elragi, A., D. Negussey, and G. Kyanka 2000. "Sample Size Effect on the Behavior of EPS Geofoam". In Proceedings of the Soft Ground Technology Conference. May 28 - June 2, 2000 | Noordwijkerhout, the Netherlands, 2001 American Society of Civil Engineers.

"EPS Substitution" Proceedings of the International Symposium on EPS Construction Method, Tokyo, Japan, pp. 201-211.

Eriksson, L., and Trank, R., (1991) "Properties of Expanded Polystyrene, Laboratory Experiments" Swedish Geotechnical Institute, Sweden.

Frydenlund, T. E., and Aabøe, R. (1996). "Expanded polystyrene - The light weight solution." Proceedings of International Symposium on EPS construction method (EPS TOKYO'96), Tokyo, Japan, 32-46.

G. Beinhreich, R. Hillmann, EPS in road construction—current situation in Germany, *Geotext. Geomembr.* 15 (1) (1997) 39-57.

Heger F, Liepins A, Selig ET. SPIDA: an analysis and design system for buried Concrete pipe. In: Proc int conf advances in underground pipeline engineering ASCE. P. 143-154.

Horvath, J. S. (1993c). Computer software for load-deformation and geothermal Analyses in problems involving geosynthetics. *Geotextiles and Geomembranes*, 12(5), 425-433.

Horvath, J.S. (2005). "Expanding the Use of Expanded Polystyrene (EPS) Geofoam in Practice," Boston Society of Civil Engineers Section, ASCE Geo-institute engineering Seminar, 29th October, 2005, Waltham, MA, U.S.A.

Horvath, J.S. 2010. Lateral pressure reduction on earth retaining structures using geofoams: Correcting some Misunderstandings. In Proc., ER2010: Earth Retention Conference 3, ASCE, Reston, VA.

Huntsman Corporation, Texas, USA, Technical Bulletin, No. 7-7.7.

Huntsman, (1999i) "Huntsman Expanded Polystyrene Performance Characteristics.

Hussein, M.G. (2015). On the numerical modeling of buried structures with compressible inclusion. Geo-Quebec, (8). Retrieved from.

Ikizler SB, Aytekin M, Nas E (2008) Laboratory study of expanded polystyrene (EPS) geofoam used with expansive soils. Geotext Geomembr 26(2):189–195.

J.S. Horvath, Expanded polystyrene (EPS) geofoam: an introduction to material behavior, Geotext. Geomembr. 13 (4) (1994) 263–280.

John S. Horvath Geotextiles and Geomembranes 15 (1997) 77-120 © 1998 Elsevier Science Limited Printed in Great Britain.

Junsuk Kang, Frazier Parker, Chai H. Yoo. Soil–structure interaction for deeply buried corrugated steel pipes Part II: Imperfect trench installation. AL 36849-5337, USA. 2007 Auburn University.

Jutkofsky, W., S., Teh, S., J., and Negussey, D., (2000) “Stabilization of an Embankment Slope with Geo foam” Transportation Research Board, 79th Annual Meeting, Paper No. 00-1315, Washington, D.C., USA.

Kang J, Han TH, Kang Y, Yoo C. Short-term and long-term behaviors of buried Corrugated high-density polyethylene (HDPE) pipes. Compos B Eng 2009; 40(5):404–12.

Kang J, Parker F, Yoo C Y, Yoo C. Effects of frictional forces acting on sidewalls of Box culverts. Int J Numer Anal Meth Geomech 2008; 32(3):289–306.

Kang J, Parker F, Yoo C. Soil-structure interaction and imperfect trench installations for deeply buried concrete pipes. J Geotech Geoenviron Eng 2007; 133(3):277–85 ASCE.

Kang J, Parker F, Yoo C. Soil-structure interaction for deeply buried corrugated steel pipes part II: imperfect trench installation. Eng Struct 2008; 30(3):588–94.

Katona MG. CANDE-2015; 2015. www.candeforculverts.com.

Kawashima Y, Atsushi, N, Kawagishi Y et al (2001) Construction of high eps embankment in heavy snowfall region. Paper presented at the 3rd international conference on EPSgeofoam blocks in construction applications, Salt Lake City, USA, december 10-12, 2001.

Kim, H., Choi, B., & Kim, J. (2010). Reduction of earth pressure on buried pipes by EPS geofoam inclusions. Geotechnical Testing Journal, 33(4), 304-313.

Konno H, Nakono O, Kishi N, et al (2001) A practical design method of three-layered absorbing system. Paper presented at the 3rd international conference on EPS geofoam blocks in construction applications, Salt Lake City, USA, December 10-12, 2001.

Marston A. Second progress report to the joint concrete culvert pipe committee. Ames (IA): Iowa Engineering Experimental Station; 1922.

Marston, A., 1930. The theory of external loads on closed conduits in the light of the latest experiments. Bulletin 96 (Iowa Engineering Experiment Station, Ames, Iowa).

Medne O. Research on Expanded Polystyrene Product Manufacturing Process. Summary of Doctoral Thesis.-R.:RTU, 2011.

Moghaddas Tafreshi SN, Gh Tavakoli Mehrjardi, Moghaddas Tafreshi SM (2007) Analysis of buried plastic pipes in reinforced sand under repeated-load using neural.

Musser SC. CANDE-89 user's manual. FHWA-RD-89-169. Federal Highway Administration, U.S. Dept. Of Transportation; 1989.

NCHRP Report 647. Recommended design specifications for live load distribution To buried structures. Washington, D.C.: TRB, National Cooperative Highway Research Program; 2010.

Negussey, D. (1997). Properties and applications of geofoam. Society of the Plastics Industry, Inc., Washington, DC.

Negussey, D., Anasthas, N., and Srirajan, S. (2001). "Interface friction properties of EPS geofoam." Proc.EPS Geofoam 2001: 3Rd International Conference, Salt Lake City, Utha, USA.

Negussey, D., and Sun, M., (1996) "Reducing Lateral Pressure by Geofoam.

Neupane R (2015) Expanded polystyrene geofoam embankment for support of railways and bridges. Ph.D Dissertation, University of Utah, SLC, UT, USA.

NPRA, 2010. Håndbok 016: Geoteknikk i vegbygging (in Norwegian). Norwegian Public Roads Administration.

O'Brian AS (2001) Design and construction of the UK's first polysterene embankment for railway use. Paper presented at the 3rd international conference on geofoam blocks in construction applications, Salt Lake City, USA, December 10-12, 2001.

Özer AT (2016) Laboratory study on the use of EPS-block geofoam for embankment widening. Geosynth Int 23(2): 71-85.

Özer AT, Akay O (2015) Interface shear strength characteristics of interlocked EPS-Block Geofoam. J Mater Civ Eng 28(4):04015156.

Özer, A. T. (2020). Geofoam: The use of EPS block in civil engineering applications. ISBN 978-6056-9266-1-7.

Özer, A. T. (2021). Geofoam: The use of EPS Blocks in Civil Engineering.

Özer, A. T. and Akınay, E. (2022). Construction of geofoam block embankments atop existing transportation infrastructure. Geosynthetics International.

Padade AH, Mandal JN (2012) Behavior of expanded polystyrene (EPS) geofoam under triaxial loading conditions. *EJGE* 17:2543–2553.

Properties. Universidad Politecnica Puerto Rico (Puerto Rico), 2014.

Rafa l F. Obrzud (2018) the hardening soil model -a practical guidebook.

Sheeley, M. (2000). “Slope Stabilization utilizing geofoam.” Master’s Thesis, Syracuse University, Syracuse, New York, USA.

Spangler MG. A theory of loads on negative projecting conduits. *Proceedings of the Highway Research Board*, vol. 30. Washington (DC): Transportation Research Board; 1950. P. 153–61.

Spangler, M.G., 1950. A theory on loads on negative projecting conduits. *Highw. Res. Board* 30, 153–161.

Stark TD, Arellano D, Horvath JS, Leshchinsky D (2004) Geofoam Applications in the design and construction of highway Embankments. *NCHRP Web Doc* 65:24.

Stark, T.D., Bartlett, S.F., and Arellano, D., 2012, Expanded polystyrene (EPS) geofoam applications and technical data: The EPS Industry Alliance, 1298 Cronson Blvd., Suite 201, Crofton, MD 21114, 36 p.

Sun, L., Hopkins, T.C., Beckham, T.L., 2009. Reduction of Stresses on Buried Rigid Highway Structures Using the Imperfect Ditch Method and Expanded Polyesterene (Geofoam). Kentucky Transportation Center, Report No. KTC-07-14-Spr228-01-1f. University of Kentucky, Kentucky, USA, pp. 49.

Suzuki, Y., Nishimura, A., and Kuno, T., (1996) “Design and Construction of Road Embankment on Steep Hillside by EPS” *Proceedings of the International Symposium on EPS Construction Method*, Tokyo, Japan, pp. 265-273.

T. Stark, D. Arellano, J. Horvath, D. Leshchinsky, *Guideline and Recommended Standard for Geofoam Applications in Highway Embankments*, Transportation Research Board, 2004.

Van Dorp, T. (1988). “Expanded polystyrene foam as light fill and foundation material in road structures.” *International Congress on Expanded Polystyrene*, Milan, Italy.

Vaslestad J, Johansen TH, Holm W. Load reduction on rigid culverts beneath high fills: Long term behavior. *Transportation Research Record*, 1415, Washington (DC): Transportation Research Board; 1993. P. 58–68.

Vaslestad, J., Sayd, M.S., Johansen, T.H., Wiman, L., 2011. Load reduction and arching on buried rigid culverts using EPS Geofoam. Design method and instrumented field tests. In: *Proceedings of EPS 2011*, *Proceedings of EPS 2011*, June 6th e 8th 2011 at Lillestrøm, Norway, vol. 1 paper 36, 10 pp.

Yoshizaka, K., and Sakanoue, T. (2003) . “Experimental Study on Soil-pipelineinteraction Using EPS Backfill,” ASCE, Pipelines 2003 , Baltimore USA, July2003, 1126-1134.

Yoshizaki K. And Sakanoue, T. (2003). “Experimental Study on Soil-Pipeline interactionusing EPS Backfill,” Proceedings of the Pipelines 2003 International Conference on Pipeline Engineering and Construction, Baltimore, MD, July 13-16, 2003, pp. 1126-1134.

Youwai S, Kongkitkul W, Sripobink T, et al (2011) Application of EPS for remedial work of bridge bearing unit on Bangkok Soft Clay: A case study. Paper presented at the 4th international conference on geofoam blocks in construction applications, Lillestrøm, Norway, 6-8 June 2011.



BIOGRAPHY

Zahraa ALMUSAWI, as a highly skilled and experienced geotechnical engineer, is dedicated to delivering innovative and effective solutions to complex engineering challenges. With two years of experience in the field, she has a deep understanding of geotechnical engineering principles, including advanced soil mechanics, advanced foundation design, and slope stability analysis. She has a proven track record of successful project completion on time and within budget. In addition, she possesses strong communication skills and is able to work collaboratively with clients, contractors, and regulatory agencies to ensure project success. Overall, she is passionate about applying her knowledge and skills to create safe, efficient, and sustainable infrastructure that meets the needs of communities and society as a whole. Zahraa holds a Master of Science degree in Geotechnical Engineering from Gebze Technical University, Kocaeli, obtained in 2024, and a Bachelor of Science degree in Civil Engineering from Thi Qar University, Thi Qar, completed in 2021.

PUBLICATIONS AND PRESENTATIONS FROM THE THESIS

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