

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL

**INVESTIGATION OF VARIOUS FLEXIBLE PAVEMENT
CONFIGURATIONS FOR GEOFOAM BLOCK HIGHWAY
EMBANKMENTS BY ACCELERATED PAVEMENT TESTING**



ABDULLAH YASİN BÜYÜKASLAN

**A THESIS OF MASTER OF SCIENCE
CIVIL ENGINEERING DEPARTMENT**

ADVISOR: PROF. DR. ABDULLAH TOLGA ÖZER

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LİSANSÜSTÜ EĞİTİM ENSTİTÜSÜ

GEOFOAM BLOK OTOYOL DOLGULARI İÇİN ÇEŞİTLİ
ESNEK ÜSTYAPI KONFIGÜRASYONLARININ
HIZLANDIRILMIŞ YOL TESTİ İLE İNCELENMESİ

ABDULLAH YASİN BÜYÜKASLAN

YÜKSEK LİSANS TEZİ
İNŞAAT MÜHENDİSLİĞİ ANABİLİM DALI

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*To my
My wife and daughter*

ABSTRACT

Geofoam is used in highway embankment constructions as a lightweight fill material to remediate settlement-related problems. The CBR value of a geofoam block is much smaller than the minimum allowable value for the foundation of the flexible pavements. Therefore, the pavement design for geofoam block highway embankments requires a special solution. The pavement stiffness can be increased by increasing the thickness of the base and sub-base layers. However, this technique is not preferred since it increases dead loads which may cause different internal stability-related issues. Constructing a 10-cm-thick reinforced concrete (PCC) slab is a widespread application to reduce pavement thickness. Despite being a standard application for traditional geofoam block highway pavement systems, it extends the time of completion due to the setting requirements of the concrete. In addition, plastic shrinkage and cracks may occur due to lack of curing which may cause failure to reach its design strength. Hence, lack of curing may cause significant structural problems in concrete quality and directly affect the performance of the pavement. Regardless of these disadvantages, fast and applicable alternative flexible pavement system for geofoam block highway embankments has not been developed yet.

The behavior of the five different flexible pavement configurations was evaluated in this study by constructing field-scale pavement models in a 3.75 wide and 50 m long test bed. The first configuration included a flexible pavement system composed of a PCC load distribution slab which is the industry standard for geofoam highway embankments, whereas the other four flexible pavement configurations didn't include a PCC slab. While one of these configurations was constructed using only a plant-mix base course the other three configurations included geosynthetic reinforcements. Each field-scale flexible pavement configuration was constructed as 3.75 m wide and 10 m long.

An accelerated pavement testing (APT) program was designed to apply traffic loads using a 3-axle, 37.72-ton truck. The surface deflection measurements were collected using laser profilometers prior and after the accelerated pavement test. In addition, the rut-depth of pavement surfaces was also measured using a straightedge. The number of passes and the speed of the truck were recorded via pneumatic traffic measurement hoses. Deformation characteristics of flexible pavement configurations quantified by falling weight deflectometer (FWD). The deflection bowls of FWD were used to estimate the modulus of elasticity and stiffness properties of the pavement layers by an iterative back calculation process. Plate load tests (PLT) and sand cone (SC) tests were used to evaluate deformation modulus and compaction efficiency of the granular base, respectively. The performances of five different flexible pavement systems for geofoam embankments were evaluated through an extensive APT program. Therefore, the effectiveness of alternative flexible pavement systems for geofoam embankment construction was evaluated by a field-scale testing program.

Based on an extensive field-scale testing effort, it was found that a flexible pavement configuration composed of 65 cm-thick granular base (Configuration #2) can be a viable alternative for the traditional flexible pavement system composed of a PCC load distribution slab (Configuration #1). Additional studies are required using various configurations to further investigate the efficiency of geosynthetic reinforcements in geofoam embankments flexible pavement configurations.

Keywords: Geofoam, Flexible pavement, Accelerated pavement testing (APT), Falling weight deflectometer (FWD), Field-scale testing.

ÖZET

Geofoam, otoyol dolgu inşaatlarında oturmayla ilgili sorunları gidermek için hafif dolgu malzemesi olarak kullanılmaktadır. Geofoam bloğunun CBR değeri, esnek yol üstyapıların temeli için izin verilen minimum değerden çok daha küçüktür. Bu nedenle, geofoam blok otoyol dolguları için esnek üst yapı tasarımı özel çözümler gerektirmektedir. Üstyapı rijitliği, temel ve alt temel tabakalarının kalınlığı artırılarak artırılabilir. Ancak bu teknik, ölü yükleri arttırarak geofoam blok yol dolgusunun iç stabilitesi ile ilgili farklı sorunlara neden olabileceğinden tercih edilmemektedir. Üstyapı kalınlığını azaltmak için 10 cm kalınlığında betonarme döşeme inşa etmek yaygın bir uygulamadır. Geleneksel geofoam blok otoyol sistemleri için standart bir uygulama olmasına rağmen, betonun priz alma süreci nedeniyle proje tamamlanma süresini uzamaktadır. Buna ek olarak, arazide kür eksikliğinden dolayı betonda tasarım dayanımına ulaşmasına engel olabilecek plastik büzülme ve çatlaklar oluşabilir. Dolayısıyla, kür eksikliği beton kalitesinde önemli yapısal sorunlara yol açabilir ve üstyapının performansını doğrudan etkileyebilir. Bu dezavantajlara rağmen, geofoam blok otoyol dolguları için hızlı ve uygulanabilir alternatif esnek üstyapı sistemi henüz geliştirilmemiştir.

Bu çalışmada beş farklı esnek üst yapı konfigürasyonunun davranışı arazide 3.75 m genişliğinde ve 50 m uzunluğunda bir test yatağı içerisinde inşa edilen modeller ile değerlendirilmiştir. İlk konfigürasyon, geofoam otoyol dolguları için uygulamada standartlaşmış olan betonarme yük dağıtma platformundan oluşan bir esnek üstyapı sistemini içermektedir. Diğer dört esnek üstyapı konfigürasyonunda betonarme yük yayma platformu yer almamıştır. Bu konfigürasyonlardan biri sadece plentmiks temel tabakası kullanılarak inşa edilmiş, diğer üç konfigürasyon ise geosentetik donatılar içermiştir. Her bir esnek üstyapı konfigürasyonu 3.75 m genişliğinde ve 10 m uzunluğunda inşa edilmiştir. Trafik yüklerini uygulamak için hızlandırılmış yol test (APT) programı tasarlanmıştır. Yüzey deformasyon değerleri, hızlandırılmış üstyapı testinden önce ve sonra lazer profilometreler kullanılarak ölçülmüştür. Ayrıca, esnek üstyapı yüzeylerinin tekerlek izi derinliği master ile de ölçülmüştür. Hızlandırılmış yol testinde kullanılan 3-aks, 37,72-ton kamyonun geçiş sayısı ve hızı pnömatik trafik ölçüm hortumları aracılığıyla kaydedilmiştir. Esnek üstyapı konfigürasyonlarının deformasyon özellikleri düşen ağırlık deflektometresi (FWD) ile ölçülmüştür. FWD ile elde edilen defleksiyon çanakları, üstyapı katmanlarının elastisite modülleri ve rijitlik parametrelerini elde etmek için iteratif geri hesaplamalarda kullanılmıştır. Granüler temel katmanının deformasyon modülü ve sıkıştırma verimliliğini değerlendirmek için de sırasıyla plaka yükleme (PLT) ve kum konisi (SC) testleri yapılmıştır. Geofoam dolgular için beş farklı esnek üstyapı sisteminin performansları ve verimlilikleri arazi ölçeğinde gerçekleştirilen kapsamlı bir APT programı ile değerlendirilmiştir.

Arazi ölçeğinde gerçekleştirilen kapsamlı test programına dayanarak, 65 cm kalınlığında granüler temel katmanından oluşan esnek bir üstyapı konfigürasyonun (Konfigürasyon #2), PCC yük yayma platformu ile inşa edilen (Konfigürasyon #1) geleneksel esnek üstyapılara bir alternatif olabileceği anlaşılmıştır. Geofoam blok otoyol dolgularının esnek üstyapılarda geosentetik donatıların verimliliğini araştırmak için farklı konfigürasyonların tasarlandığı ilave çalışmalar yapılması gerekmektedir.

Anahtar Kelimeler: Geofoam, Esnek üstyapı, Hızlandırılmış yol testi, Düşen ağırlık deflaktometre, Arazi ölçekli test.

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LIST OF SYMBOLS AND ABBREVIATIONS

c_u	: Undrained shear strength
E	: Young's modulus
E_v	: Strain (deformation) modulus
E_{ti}	: Initial tangent modulus
f_s	: Sleeve friction
I_c	: Soil behavior type index
k_s	: Modulus of subgrade reaction
M_R	: Resilient modulus
q_c	: Cone tip resistance
σ_e	: Elastic limit stress
S_0	: Standard normal deviate
σ_0	: Normal stress
δ	: Interface friction angle
u	: Pore water pressure
ν	: Poisson's ratio
W_{18}	: Predicted number of 80 kN equivalent single axle load repetitions
AASHTO	: American Association of State Highway and Transportation Officials
ADT	: The number of traffic load passes
APT	: Accelerated pavement test
ASTM	American Society for Testing and Materials
CBR	: California Bearing Ratio
CH	Inorganic clays of high plasticity, fat clays according to the USCS
CL	: Inorganic clays of low to medium plasticity, gravelly, gravelly clays, sandy clays, silty clays, lean clays according to the USCS
CMD	: Cross machine direction
CPTu	: Piezocone penetration test
CS or σ_c	: Compressive Strength
CS1	: Compressive Strength at 1% strain
CS5	: Compressive Strength at 5% strain
CS10	: Compressive Strength at 10% strain
DIN	: Deutsche Norm

EN	: European Norm
EPS	: Expanded polystyrene geofoam block
EPS12	: Expanded polystyrene geofoam block with a minimum density of 11.2 kg/m ³
EPS15	: Expanded polystyrene geofoam block with a minimum density of : 14.4 kg/m ³
EPS19	Expanded polystyrene geofoam block with a minimum density of : 18.4 kg/m ³
EPS22	Expanded polystyrene geofoam block with a minimum density of : 21.6 kg/m ³
EPS29	Expanded polystyrene geofoam block with a minimum density of : 28.2 kg/m ³
EPS39	Expanded polystyrene geofoam block with a minimum density of : 38.4 kg/m ³
EPS46	Expanded polystyrene geofoam block with a minimum density of : 45.7 kg/m ³
ESAL	: Equivalent standard axle load for 80 kN (18 kips)
F	: Force
FWD	: Falling weight deflectometer
GTU	: Gebze Technical University
HMA	: Hot mix asphalt
HWD	: Heavy weight deflectometer
KGM	: Türkiye General Directorate of Highways
KMO	: Northern Marmara Highway
KTŞ	: Technical Specifications of Turkish Highways
LEF	: Axle load equivalency factor
LWD	: Light weight deflectometer
MD	: Machine direction
N	: Blow counts of standard penetration test
NCAT	: National Center for Asphalt Technology of United States
NCHRP	: National Cooperative Highway Research Program
NPRA	: Norwegian Public Roads Administration
NRRL	: Norwegian Road Research Laboratory

PCC	: Portland cement concrete
PLT	: Static plate load test
PMAT	: Plant-mix sub-base course
PMT	: Plant-mix base course (for granular base)
PMT*	: Pressuremeter test
S	: Settlement
SC	: Sand cone test
SN	: Structural number
SBT	: Soil behavior type
SPT	: Standard penetration test
TBR	: Traffic benefit ratio
TIO	: Wheel path settlement
TS	: Turkish Standard
TUBITAK	: Scientific and Technical Research Council of Türkiye
USCS	: Unified soil classification system

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1. INTRODUCTION

One of the main problems encountered in the design of highway and bridge approach embankments can be considered as the inadequate soil-bearing capacity and excessive ground settlements. Traditional soil improvement techniques are readily available to mitigate bearing capacity and settlement problems. Although soil improvement techniques are successfully utilized to improve sub-surface profiles, these geotechnologies prolong the project's completion time and increase the initial investment costs depending on the soil depths to be improved [Duškov and Waarts, 2011; Farnsworth et al., 2008; Özer et al., 2012; Papacharalampous and Sotiropoulos, 2011]. As an alternative to soil improvement, lightweight fill technologies are considered to construct highway and bridge approach embankments.

Expanded polystyrene (EPS) block (geofoam block), EPS beads-soil mixtures, foam stabilized soil mixtures, porous concrete, coal ash/granulated slag, volcanic ash soil, light expanded clay aggregates, hollow structures (culverts made of concrete/reinforced concrete, steel, polymer-based materials), shells, wood chips and tire chips are used as lightweight fill material [Frydenlund, 1987; Miki H., 1996]. In addition, EPS can be categorized as an ultra-lightweight fill material, and it has been used as an alternative solution to ground improvement applications in numerous highway projects in the last 50 years [Aabøe et al., 2019]. EPS blocks, which have proven long-term performance under service loads, stand out with their mechanical, physical and strength properties compared to that of other lightweight filling materials [Aabøe et al., 2019].

Geofoam geotechnology is used in highway and bridge approach embankments as a lightweight fill material to remediate settlement-related problems. Despite the widespread usage of geofoam in embankments, designing pavement systems requires special solutions since the California Bearing Ratio (CBR) of geofoam block is much smaller than the minimum allowable values that of the traditional flexible pavement materials [Stark et al., 2004a]. The stiffness of pavement can be increased by increasing the thickness of the granular base layer; however, it is not preferred due to the additional dead load that will impose atop the blocks. The stiffness of the pavement can be increased with a stiffer material without increasing the layer thicknesses [Stark

et al., 2004a]. Therefore, a 10 to 15-cm thick reinforced Portland-cement-concrete (PCC) slab between geofoam blocks and unbound aggregate layers is recommended [Stark et al., 2004a]. In addition, PCC slab helps to transfer loads uniformly atop the geofoam blocks. However, despite its load distribution benefits, this solution has also some disadvantages. The construction of the reinforced concrete platform and the curing time directly affect the project duration. In addition, plastic shrinkage and structural cracks, which occur due to inappropriate or insufficient curing, prevent reaching its design strength [Özer and Akay, 2022]. Regardless of these disadvantages, fast and applicable alternative flexible pavement system for geofoam block highway embankments without PCC slab has yet to be developed.

In the construction of paved or unpaved pavements, geocell and geogrid applications are widely used to improve the performance of the pavement under traffic loads and reduce the pavement's total thickness [Giroud et al., 2021]. The traditional granular pavement materials and different geosynthetic reinforcement layers were used together to develop different flexible pavement systems without increasing the pavement thickness.

The behavior of five different flexible pavement configurations constructed atop prototype geofoam block highway embankment by accelerated pavement test (APT) with actual traffic loads was investigated. Full-scale test bed was constructed at the Gebze Technical University, Gebze, Kocaeli, Türkiye for this study. Five different flexible pavement configurations were constructed with EPS geofoam block sub-base. The testing program was designed to investigate alternative pavement systems for geofoam block embankments without PCC slab, which is the prevailing traditional practice in geofoam highway embankment constructions. Dynamic and repetitive traffic loads were applied using the full-scale APT. Through the APT, the rut measurement of the surface was recorded with a laser profilometer and straightedge. During the APT, pneumatic traffic measurement hoses were used to capture the number of vehicle passes and speed. A 3-axle, 37.72-ton truck was used to apply controlled wheel loading during the APT. The deformation characteristics of flexible pavement systems were quantified by falling weight deflectometer (FWD) and static plate load tests (PLT).

The background information about geofoam block geotechnology, PLT, FWD and APT was provided in the [Chapter 2](#) and the design of the pavement system was discussed

in the [Chapter 3](#). The study area and steps of the prototype construction were provided in the [Chapter 4](#) in detail. The material characterization of bedding sand, geofoam, concrete, geosynthetics, granular base (plant mix base material-PMT) and asphalt were given in the [Chapter 5](#). All the field-testing effort, including PLT, sand cone, FWD and APT were extensively discussed in [Chapter 6](#) and the major findings of the prototype testing program were summarized in [Chapter 7](#).



2. BACKGROUND INFORMATION

2.1. Expanded Polystyrene (EPS) Block (Geofoam block)

EPS is known as an insulation material due to its first and widespread use in the world however, it has been used in block form (geofoam block) as an embankment in road constructions since 1972 [Aab   et al., 2019]. The American Society for Testing and Materials (ASTM) defines geofoam block as a lightweight and closed-cellular geosynthetic used in geotechnical engineering applications [ASTM D4439, 2020]. Several test procedures used to quantify the physical and mechanical properties of geofoam blocks, and the ASTM standards specify both testing procedures and minimum product requirements [ASTM D2863, ASTM C165, ASTM C203, ASTM C303, ASTM D1621, ASTM D1622 ASTM C578, and ASTM D6817]. In addition, design guidelines for the use of geofoam blocks in highway construction have been published in Norway [NRPA, 1992] and in the United States [Stark et al., 2004a; 2004b].

Geofoam blocks are produced by granular polystyrene beads by a series of steps including pre-expanding, resting, and molding (**Figure 2.1**) [  zer, 2021]. The EPS blocks produced in a controlled environment which allows to achieve declared physical and mechanical properties of the product. In addition, the material is also suitable for recycling. Since it is a lightweight fill material, handling on-site does not require construction machines (**Figure 2.2**).



Figure 2.1: Production stages: a-b) Raw polystyrene, c) Pre-expansion, d) Molding, e) Resting for dimensional stability in the stock area of the molding facility [  zer, 2021].



Figure 2.2: Geofoam block handling on-site [Özer, 2021].

Common production dimensions of the product are provided by ASTM (**Table 2.1**) [ASTM D6817/D6817M, 2021]. On the other hand, these dimensions are not limitations, they may vary according to the mold dimensions of the manufacturer (**Figure 2.1d**). ASTM D6817 indicates the dimensional tolerance as $\pm 0.5\%$.

Table 2.1: Common production dimensions of Geofoam [ASTM D6817/D6817M, 2021].

Dimensional Property	Dimensions (mm)
Width	305 to 1219
Length	1219 to 4877
Thickness	25 to 1219

ASTM D6817 specifies the product requirements of geofoam blocks with densities varying between 11.2 kg/m^3 (EPS12) and 45.7 kg/m^3 (EPS46) (**Table 2.2**) [ASTM D6817/D6817M, 2021]. The density of traditional backfill material, which varies based on the project specifications, but in practice, it is generally between 1800 kg/m^3 and 2500 kg/m^3 . Hence, geofoam block is approximately 50 to 100 times lighter than

traditional earth fill. It is also called ultra-lightweight fill material when compared with other lightweight fill materials [Miki H., 1996] and it has high strength to density ratio [Elragi, 2000]. Due to these physical and mechanical advantages, geofoam allows the construction of highway embankments on soft soil sites by eliminating soil improvement against settlement and bearing capacity issues. The EPS geofoam block compressive resistance increases with the block density increases (**Table 2.2**). It is also possible to mold geofoam blocks with different densities other than listed in **Table 2.2**.

Due to its closed-cellular structure, water absorption capacity of geofoam blocks is very low therefore, long-term exposure to water does not have a negative effect in terms of dimensional and mechanical properties [Özer, 2020; 2021; Stark et al., 2004b]. The long-term effect of water absorption was evaluated for geofoam block embankment in Norway [Frydenlund and Aabøe, 2001]. The water content was measured up to 4 % by volume located in temporarily submerged areas and on the order of 10 % by volume located in permanently submerged areas [Frydenlund and Aabøe, 2001].

Table 2.2: Minimum requirements of EPS geofoam block [ASTM D6817/D6817M, 2021].

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

Compressive properties of geofoam blocks are measured based on ASTM D1621 (2016). A typical stress-strain curve for EPS20 can be found in **Figure 2.3** [Özer, 2021]. The typical stress-strain curve comprises four distinct zones:

Zone 1 - Initial linear zone: The limit of this initial linear response extends up to approximately 1 % - 1.5 % axial strain [Özer, 2021]. Elastic limit stress (σ_e) is called the compressive strength at 1% axial deformation (elastic limit strain) and initial tangent modulus (E_{ti}) is defined as the slope of this linear portion of the stress-strain curve at Zone 1 [Arellano and Stark, 2009; Stark et al., 2004b]. The design recommendations for geofoam block embankments limit the service loads at this stress [Horvath, 2010]. The elastic limit stress is the function of EPS density and increases with an increasing EPS density (**Figure 2.3**).

Zone 2 – Yielding zone: The yielding behavior of EPS is observed at Zone 2 and continue up to axial strain of between 3 % and 5 %. This zone can be considered as a transition between yielding and post-failure linear hardening zone (Zone 3).

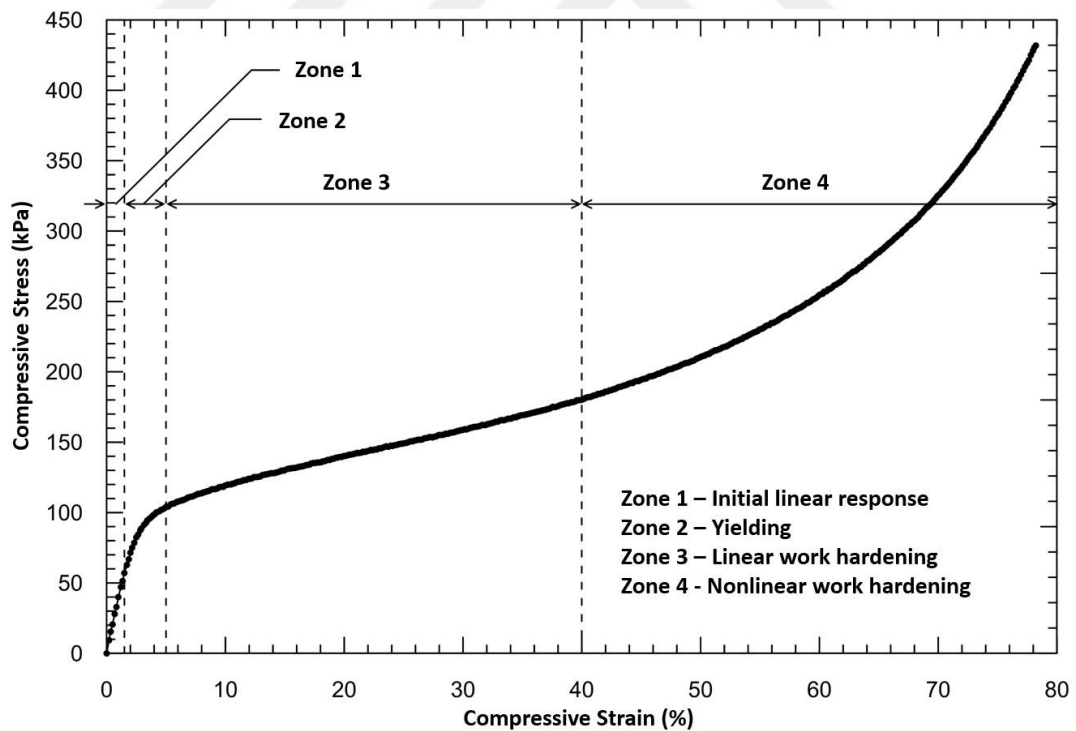


Figure 2.3: Monotonic loading test of EPS19 [Özer, 2021].

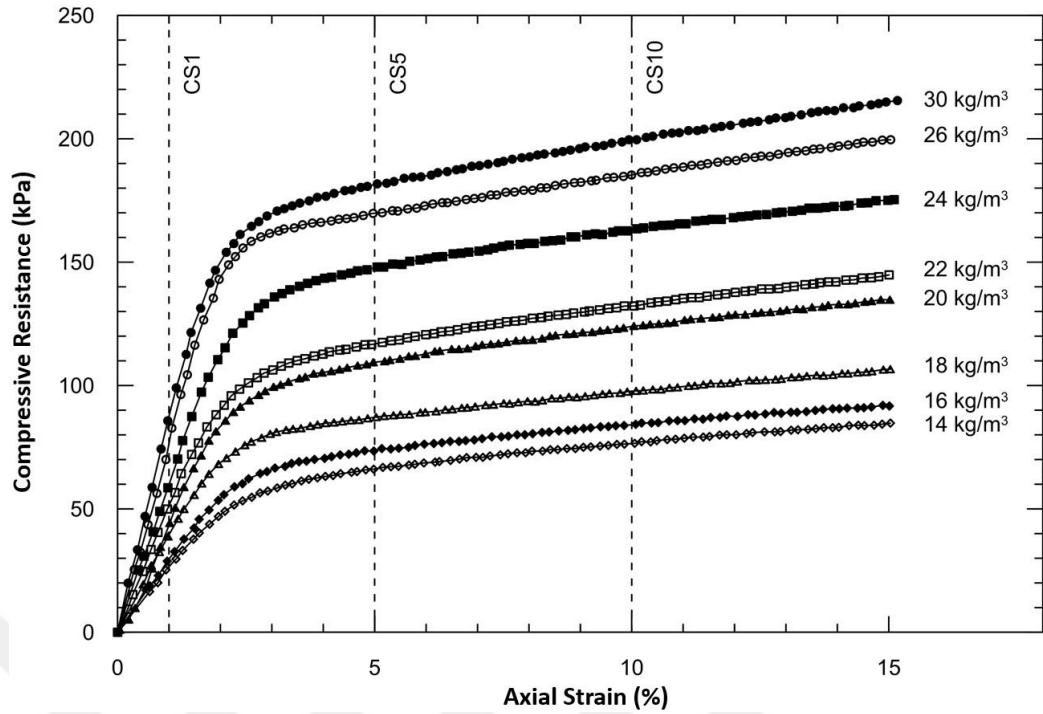


Figure 2.4: Stress – strain curves for EPS specimens with various [Özer, 2021].

Zone 3 – Linear strain hardening zone: This linear hardening behavior is observed at end of the yielding and extends to strain of between 30 % and 40 % (Figure 2.3) [Stark et al., 2004b].

Zone 4 – Non-linear strain hardening: The hardening that occurred in Zone 3 continues in Zone 4, but the stress-strain behavior is no longer linear (Figure 2.3) [Stark et al., 2004b].

Unlike soils and other traditional construction materials, EPS blocks do not exhibit a failure plane or rupture (Figure 2.2). Therefore, compressive strength of EPS defined as compressive stress as a function of compressive strain. The compressive strength is defined the strength at 5% strain (CS(5)) by Norwegian Public Roads Administration (NPRA) and at 10% strain (CS(10)) by ASTM [Özer, 2021]. Compressive strength at 2% strain (CS(2)), 5% strain (CS(5)) and 10% strain (CS(10)) for geofoam block with various densities are provided in TS EN 14933.

The interface shear strength properties of the interfaces in a typical geofoam block embankment, such as EPS-EPS, EPS-bedding sand, and EPS-concrete, needs to be quantified for both internal and external stability analysis [Özer, 2020]. Interface strength properties of these interfaces were extensively studied and quantified using

direct shear test apparatus [Ateş, 2016; Atmatzidis et al., 2001; Negussey et al., 2001; Özer et al., 2017; Özer and Akay, 2016; Sheeley and Negussey, 2000; Özer and Akay, 2022a; 2022b]. Interface friction angle (δ) for both EPS-EPS and EPS-dis-similar surfaces were recommended (**Table 2.3**) [Stark et al., 2004b].

Table 2.3: Interface friction angle for different system interactions [Stark et al., 2004b].

Interface system	Friction angle (δ) (degrees)
EPS – EPS	30
EPS – Nonwoven geotextile	25
EPS - Geomembrane	52
EPS - Sand	30

Özer et al. (2017) and Sheeley et al. (2000) were quantified interface shear strength properties of EPS-cast-in-place concrete (**Table 2.4**). Metal connectors, adhesives, interlocked EPS block configurations were proposed to enhance interface friction behavior of EPS-EPS [Barrett and Valsangkar, 2009; Amini, 2014; Özer and Akay, 2016; Özer and Ekincioglu, 2019].

Table 2.4: Friction angle of EPS – Concrete interface for different EPS densities.

Interface	Type of EPS	Interface Friction Angle (δ) (degrees)	References
EPS – Cast in place Concrete	EPS19	24.1	Özer et al., (2017)
	EPS29	37.2	
	EPS19	67.0	Sheeley et al., (2000)

The EPS geofoam block was first used to construct road embankment in 1972 by the NPRA (**Figure 2.5**) [Aabøe, 1987; Aabøe et al., 2019; Aabøe and Frydenlund, 2011; Alfheim et al., 2011; Frydenlund, 1987; Frydenlund and Aabøe, 2001]. This first implementation in the world was realized as part of the reconstruction of the approach

embankments of the Flom Bridge on National Highway 159 connecting Karihaugen (Oslo/Norway) and Lillestrøm (Skedsmo/Norway) [NRRL, 1987].

Geofoam technology was used to remediate total settlement in this project. The NRRL measured a total settlement of 80 cm in 1972, and 0.8 m to 1.2 m of the existing embankment was replaced with geofoam blocks to remediate settlement problems [Alfheim et al., 2011]. Geofoam blocks with a density of 20 kg/m³, which is 40 times lighter than traditional earth fill (1800 – 2500 kg/m³) and 20 times (1000-1200 kg/m³) lighter than other lightweight fill materials [Alfheim et al., 2011] were used. Thanks to this remediation project, the settlement rate of Flom Bridge approach embankment was decreased from 10 cm/year to 1 cm/year and further, it was reported as zero in 1987 [Aabøe, 1987]. Long-term measurement program showed that the blocks were performing under service loads per design [Aabøe, 1987; Aabøe and Frydenlund, 2011, Aabøe et al., 2019].



Figure 2.5: The first geofoam block embankment project – 1972, Norway [Aabøe and Frydenlund, 2011].

Following the success of the first implementation in 1972, the number of embankments constructed using geofoam geotechnology was increased in the following years. To date, geofoam block embankment projects have been completed in Europe - apart from Norway - in the Czech Republic, Denmark, Finland, France, Germany, Greece, Ireland, the Netherlands, Poland, Russia, Serbia, Sweden and the United Kingdom [Aabøe, 1987] and has been implemented in Türkiye since 2017 [Aabøe et al., 2019; Özer et al., 2017; Özer and Akınay, 2022].

Japan is the leader in geofoam block technology in Asia, where the first geofoam block embankment was constructed in 1985 [Miki G., 1996]. Approximately a total of 5.3 million m³ of geofoam blocks (85% of which were used for highway embankments) were used in Japan between the first application to 2010 [Kubota, 2011; Miki et al., 2001; Momoi and Kokusyo, 1996]. This momentum in Japan provided significant design and field experience about the future of geofoam block technology worldwide. In Asia, apart from Japan, geofoam blocks are also known to be used in China, Malaysia, Thailand, the Philippines, South Korea and Taiwan [Aabøe et al., 2019].

The first geofoam block highway embankment in the USA was constructed in 1989 as part of the reconstruction of a section of Highway 160 (Durango/Colorado) that failed due to a landslide [Bartlett et al., 2000]. Approximately 100,000 m³ of geofoam blocks were used in the rehabilitation and embankment widening project of the Highway I-15 (Salt Lake City/Utah), which was built on the highly compressible Lake Bonneville Clay [Özer, 2005], between 1998 and 2001 (**Figure 2.6**) [Farnsworth et al., 2008; Negussey et al., 2001; Newman et al., 2010].



Figure 2.6: Geofoam block embankment in Highway I-15, Salt Lake City/USA [Newman et al., 2010].

The first geofoam block roadway embankment in Türkiye was constructed in 2017 (**Figure 2.7**) [Özer et al., 2017; Özer and Akınay, 2019; 2022], and opened to traffic in April 2017, then the second geofoam road embankment opened to traffic in November 2017 [Özer et al., 2018]. The first highway geofoam embankment was constructed in the İstanbul Airport junction of the Northern Marmara Highway (KMO)

and opened to traffic in November 2019 [Özer, 2021; Özer and Akınay, 2022]. The geofoam block technology was selected due to existing buried infrastructures in these three projects. If the embankments were constructed using conventional compacted soil, stress increases would adversely affect the stability of these buried structures [Özer, 2021; Özer and Akınay, 2022]. As a result, highway embankments were constructed using geofoam blocks [Özer et al., 2017; 2018; Özer and Akınay, 2017; 2019; 2022]. The fourth geofoam block highway embankment project in Türkiye is the first geofoam block highway embankment project which was constructed by the General Directorate of Highways (KGM) [Özer, 2021]. The bedrock at this site is located approximately 38 meters below the ground surface and the local soil profile atop the bedrock consists of highly compressible elastic silts (MH), organic silts (OH) and high plasticity clays (CH). Two bridge approach embankments, 320 meters long and 14 meters wide, starting at a height of 2 meters at the beginning station and reaching to a height of 10 meters behind the abutment, were constructed in only 75 days using approximately 55,000 m³ geofoam blocks **(Figure 2.8)** [Özer, 2021]. Geofoam block geotechnology was not only eliminated the costly soil improvement alternatives in this site but also the speed of construction was also the other significant benefit [Özer, 2021].



Figure 2.7: First Geofoam block road embankment project in Istanbul / Türkiye [Özer and Akınay, 2019].



Figure 2.8: Geofoam block bridge approach road embankment in Kayseri / Türkiye [Özer 2021].

In addition to their use as highway and bridge approach embankments, geofoam blocks are also used to construct embankments atop culverts (**Figure 2.9**), lightweight fill atop buried pipes (**Figure 2.10**), railway embankments (**Figure 2.11**), green roofs (**Figure 2.12**), and retaining wall backfills [Bartlett et al., 2015; Li, 2014; Özer, 2020].



Figure 2.9: Geofoam block embankment constructed atop a hydraulic culvert in Utah / USA [Bartlett et al., 2015].



Figure 2.10: EPS Lightweight fill atop buried pipeline in Utah / USA [Bartlett et al., 2015].



Figure 2.11: Geofoam block railway embankment in Utah / USA [Li, 2014].



Figure 2.12: Geofoam block green roof application in Cesme / Türkiye [Özer, 2020].

2.2. Plate Load Test

Static plate load test (PLT) is used in engineering applications to quantify the stress-strain characteristics of soils in their natural or compacted state [DIN 18134, 2012; ASTM D1196/D1196M, 2021; Anyang et al., 2018; TS 5744, 2013; Adam et al., 2008]. The PLT is carried out to evaluate the deformation response of the soil to the applied loads on a rigid plate placed on the soil surface.

The PLT provides the load-settlement relationship, therefore, the strain modulus (E_v) and the modulus of subgrade reaction (k_s) are quantified. A typical PLT setup which conforms the standards [DIN 18134, 2012; ASTM D1196/D1196M, 2021] is shown in **Figure 2.13**.

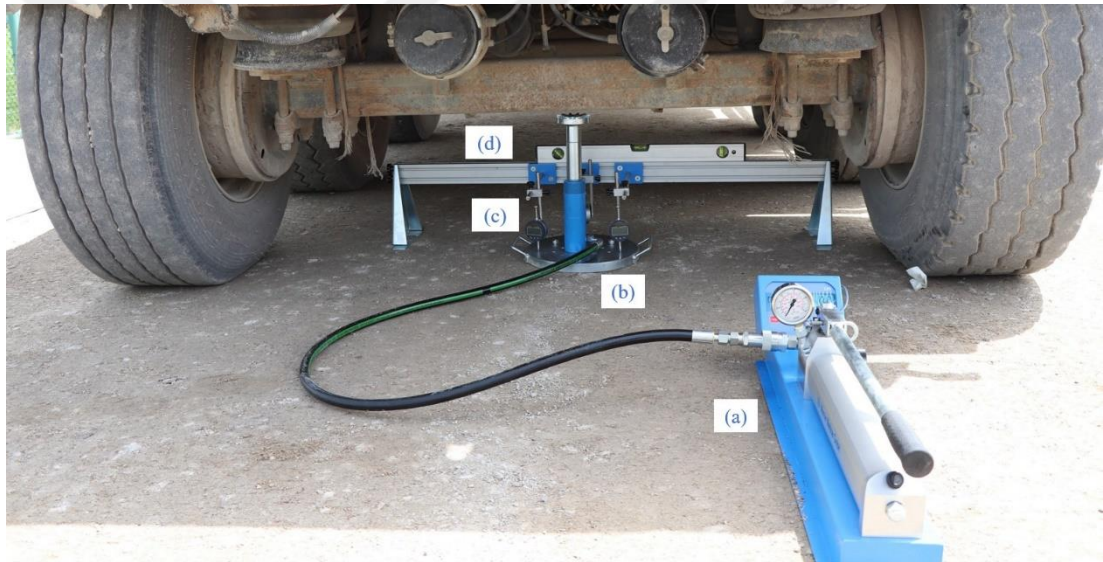


Figure 2.13: PLT set-up a) Hydraulic jack, b) Rigid plates, c) Dial gauges d) Deflection beam.

A reaction mass, against the direction of movement to hydraulic jack loading system, such as a truck which has a capacity greater than the maximum applied load, is required to transfer the applied loads to the soil (**Figure 2.13**). Loads are applied incrementally through hydraulic jack and magnitude of the loads are measured using calibrated gauge. Plates (300 mm to 762 mm in diameter) are used to transfer applied loads and displacements are captured by using a set of three dial gauges, with a resolution of 0.01 mm, fixed to a deflection beam (**Figure 2.13**).

The test surface needs to be levelled and cleaned. Intimate contact between the rigid plate and the surface must be ensured, for which sometimes a thin layer of dry sand can be used. The rigid plate is then placed on the prepared surface, and the loading jack is placed atop the plate (**Figure 2.13**). The dial gauges, positioned at an angle of 120° from each other, are fixed to the deflection beam.

After the test equipment is installed, preloading with 0.01 MPa is applied for 30 s to ensure complete seating of the plate [DIN 18134, 2012]. Then incremental loading is applied in at least six stages with approximately equal intervals of load increments until the maximum stress is reached and the unloading process is also completed in at least three stages [DIN 18134, 2012]. In the second cycle, the loading is only increased up to the penultimate stage of the first cycle [DIN 18134, 2012]. The settlement values are recorded from the gauges at each loading and unloading steps to plot a stress-settlement graph (**Figure 2.14**).

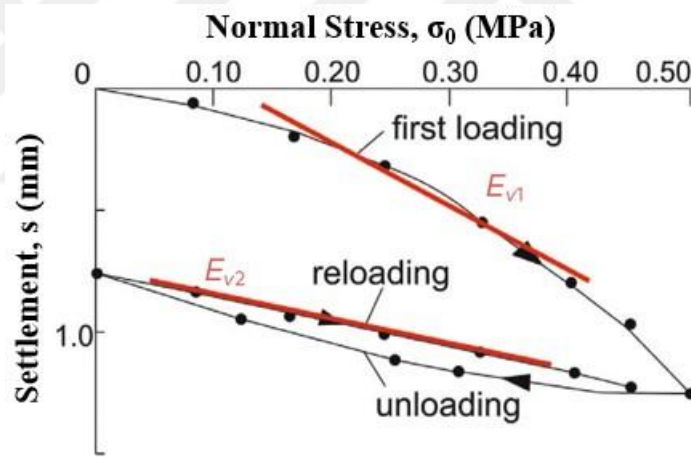


Figure 2.14: Typical stress – settlement graph of a PLT [Lehmann et al., 2020].

The stress-settlement curve of each loading stage should be generated as a second-order polynomial to calculate the strain modulus at each loading stage (E_{v1} and E_{v2} ; the strain modulus for the first and second stages, respectively). Therefore, the second order fitted equation for the curves of each loading stages is in the form of Equation 2.1 below.

$$s = a_0 + a_1 \cdot \sigma_0 + a_2 \cdot \sigma_0^2 \quad (2.1)$$

s (mm) is express the settlement value, a_0, a_1, a_2 (mm/MN²/m⁴) are constants and σ_0 (MPa) is the normal stress below the rigid plate.

The strain modulus is calculated according to Equation 2.2 where σ_{0max} is the maximum normal stress (MPa).

$$E_v = 1.5 \cdot r \cdot \frac{1}{a_1 + a_2 \cdot \sigma_{0max}} \quad (2.2)$$

The subgrade reaction modulus, k_s (MN/m³), is calculated from the normal stress value corresponding to 1.25 mm settlement (s^* in m) Equation 2.3.

$$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125} \quad (2.3)$$

PLT is frequently used to evaluate the compaction performance of road embankments and strain modulus of foundation bearing soils. Apart from natural and compacted soils, the plate load test is also widely used in the performance evaluation of unpaved road layers and natural soils reinforced with geosynthetics [DeMerchant et al., 2002; Consoli et al., 2003; Pokharel et al., 2010; Abu-Farsakh et al., 2016].

2.3. Falling Weight Deflectometer

The falling weight deflectometer (FWD) measures deflection values corresponding to stationary dynamic loads by simulating the wheel loads [Alavi et al., 2008]. In addition, the FWD is a non-destructive technique for quick and easy determination of the modulus of elasticity of the pavement layers. In addition, FWD is used to quantify structural strength, bearing capacity and remaining life analysis of pavement systems [Mehta and Roque, 2003]. The test procedure and requirements are provided in standards and specifications [ASTM D4694, 2020; Pierce et al., 2017; Alavi et al., 2008].

There are variety of deflectometer test equipment manufacturers (e.g., Dynatest (**Figure 2.15**), JILS, KUAB, Carl Bro), but all of them operate based on the same principle. Loads between 40-60 kN are applied in light weight deflectometer (LWD) tests for low-volume roads and roads reinforced with geosynthetics [Al-Qadi and

Hughes, 2000; Pasquini et al., 2013; Ahmedzade and Yılmaz, 2016; Abd El-Raof et al., 2018; White et al., 2019; Šiukščius et al., 2019; Alimohammadi et al., 2020].



Figure 2.15: Dynatest 8082 deflectometer with 320 kN capacity used in this study.

The test set-up is standardized by the ASTM D4694 (2020) and applies the dynamic test loads to the surface of the pavement by falling the weights which can vary from 30 kN to 320 kN depending on the capacities of the LWD or HWD equipment (**Figure 2.16a**). Load plates with a diameter of 300 mm to 450 mm is used to uniformly distribute the applied loads to the pavement surface (**Figure 2.16b**). Surface deformations are captured by geophones which are fixed to a beam to make sure that deformations are not affected by dynamic load vibrations (**Figure 2.16c**). Once the deformations are captured by the individual geophones located certain distances away from the load plate, then the deflection bowl can be constructed (**Figure 2.17**). The test is controlled by a software. The input parameters such as pavement temperatures, geophone positions, and dynamic load intensity are introduced to the software and the deflection values obtained by the test are captured through a data logger (**Figure 2.16d**).



Figure 2.16: FWD set-up: a) Loading system with weights, b) Geophones, c) Loading plate, d) Data logger and computer.

The deflection bowl (**Figure 2.17**) is used for back calculation which is an iterative process and helps to estimate the modulus of elasticity and stiffness properties of the pavement layers [Alavi et al., 2008]. The iterations for the back calculations are completed when the difference between the input predicted layer moduli and the data obtained from the deflection bowl due to the FWD test is close [Pierce et al., 2017]. Presently, back calculations are carried out using programs developed by researchers and equipment manufacturers. There are many software packages are available which are listed in the technical report FHWA-HRT-16-011 (**Table 2.5**) [Pierce et al., 2017].

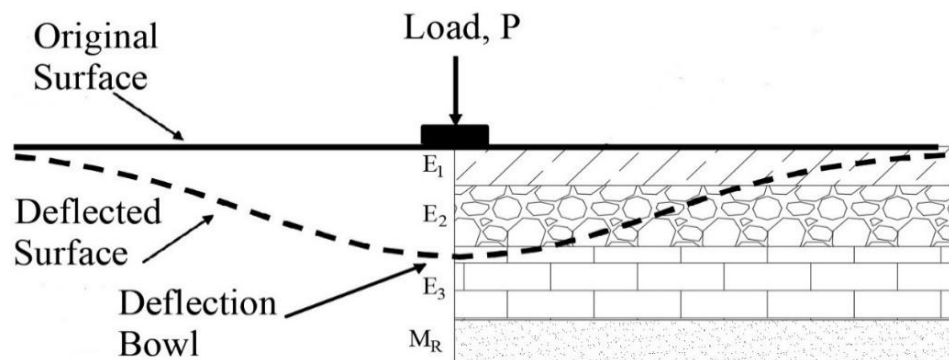


Figure 2.17: Typical FWD deflection bowl [Pierce et al., 2017].

Table 2.5: Software packages for back calculation of layer modulus based on deflectometer results [Pierce et al., 2017].

Software	Pavement type	Maximum number of layers
BAKFAA	Flexible and Rigid	Five
BISDEF [®]	Flexible	Three
BOUSDEF 2.0	Flexible	Four
CHEVDEF	Flexible	Three
COMDEF	Composite	Three
DPCONPAS	Rigid	Two
DIPLOBACK	Composite	Three
ELMOD [®] / ELCON 5	Flexible and rigid	Four (except rigid layer)
ELSDEF	Flexible	Three
EMOD	Flexible	Three
EVERCALC [®]	Flexible	Three (except rigid layer)
FPEDD1	Flexible	Four
ISSEM4	Flexible	Four
MICHBACK [®]	Flexible and composite	Three (except rigid layer)
MODTAG [®]	Flexible	Five
MODULUS 6.0	Flexible	Four (except rigid layer)
PADAL 2	Flexible	Four (except rigid layer)
PCASE 2.08	Rigid, flexible, composite	Five
RPEDD1	Rigid	Four
WESDEF	Flexible	Four (except rigid layer)

The following data must be introduced to perform back calculation to estimate layer moduli:

- Test location

- Diameter of the load plate
- Locations of geophones
- Pavement surface temperature (during test)
- Pavement temperature (during test)
- Ambient temperature (during test)
- Applied load
- Deflection values (captured by geophones)
- Type of pavement layers (hot mix asphalt-HMA, PCC, base, sub-base, subgrade)
- Thicknesses of pavement layers
- Poisson ratio for each pavement layer
- Density of pavement layers
- Seed modulus of pavement layers (Seed module is the initial module to initiate back calculation iterations)

In addition, the accuracy of the strain modulus and soil reaction modulus obtained by PLT was also compared with the data obtained by FWD experiments [Kim et al., 2007; Puppala, 2008].

2.4. Accelerated Pavement Test

The number of vehicles is increasing rapidly, and therefore the traffic density is also increasing. The growing traffic has become more important economically in terms of roadway pavement system development, innovation, preliminary design, implementation, performance, efficiency, and lifetime [Wu et al., 2012; Steyn et al., 2012]. The accelerated pavement testing (APT) methodology was developed to respond the required developments in pavement engineering research. APT is used to evaluate pavement performance under actual traffic by simulating dynamic and repetitive traffic loads which are applied in a controlled manner in a short period of time.

APT was first developed in the United Kingdom in 1912 to allow monitoring of the performance of flexible pavement systems and to observe accumulated traffic load damage over a much shorter period [Powell, 2012]. These experiments can be

performed on test roads or specially prepared test sections by full-scale accelerated loading test systems or using actual vehicles. Since the first introduction in 1912, many test facilities and test tracks have been developed for performing APT. Each of these the test facilities offer different features [Powell, 2012; Saeed and Hall Jr., 2003; Hugo and Martin, 2004] such as:

- Loading device system in terms of types of tires (e.g., single, tandem)
- Load magnitude (capacity of vehicles of APT facility or real vehicle)
- Load monitoring system (can be static scale by hydraulic pressure gauge, load cell, portable scales, load airbag pressure or weigh-in-motion)
- Load application speed (to simulate wheel speed)
- APT test vehicle's tire types
- Capacity of tire pressure
- Load carriage suspension system
- Suitability for lateral movement and wander
- Direction types and number simultaneously for different load applications

In 1912, UK Transport Research Laboratory investigated asphalt mixtures on concrete by the first APT Road Machine, which has a 13 kN load capacity and can reach a speed of 14 km/h in a 10 m diameter test track (**Figure 2.18**) [Metcalf, 2016; Powell, 2012]. In the following years, Road Machine 2, 1933 (with a load and speed capacities up 22.5 kN, 48 km/h, respectively) and Road Machine 3, 1963 (with a load capacity and speed capacities up to 68.5 kN and 25 km/h, respectively) were developed in the UK [Powell, 2012].

Nowadays, APT facilities are developed in various countries to provide economical, environmentally friendly, and durable pavement systems [Yang et al., 2021; Mshali and Steyn, 2020; Byron et al., 2003].

APT testing has recently been implemented for fully permeable asphalt pavements designed in four different sections under saturated conditions [Yang et al, 2021]. The purpose of this effort was to compare the performance and permeability of pavement systems subjected to traffic loads under fully saturated conditions. While examining the long-term performance of the pavement with porous structure in the laboratory, the

APT test vehicle is used to provide the long-term traffic load effect (**Figure 2.19a**) [Yang et al., 2021]. The dynamic response of dry and saturated asphalt pavement was investigated with an automated APT test equipment called NE-ALT (**Figure 2.19b**) [Wang et al., 2021].

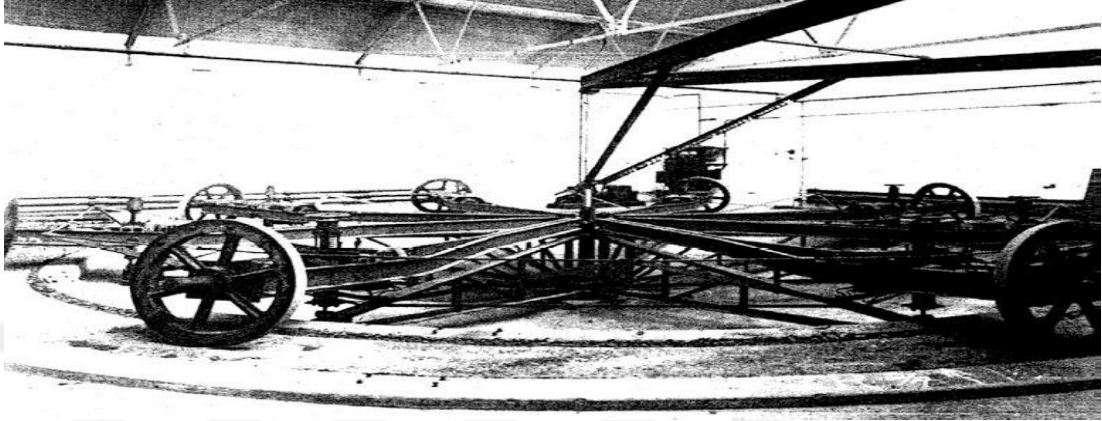


Figure 2.18: The first APT system developed by the National Physical Laboratory of UK [Metcalf, 2016].



Figure 2.19: a) An APT test machine with water sprinkler [Yang et al., 2021], b) NE-ALT APT test vehicle [Wang et al., 2021].

An airfield composite pavement system was evaluated in terms of temperature, surface bonding and load levels by full-scale APT [Ling et al., 2019]. Mobile Loading Simulator, called 66 (MLS66), is an APT test set-up with 75 kN axial load and 1.2 MPa tire pressure capacity was used (**Figure 2.20a**) [Ling et al., 2019].

The APT test is also used for six different pavement systems reinforced with a glass fiber grid with IFSTTAR (**Figure 2.20b**) equipment in Nantes, France [Nguyen et al.,

2020]. The pavements were subjected to 2.2-million-wheel passes with a load of 65 kN [Nguyen et al., 2020].

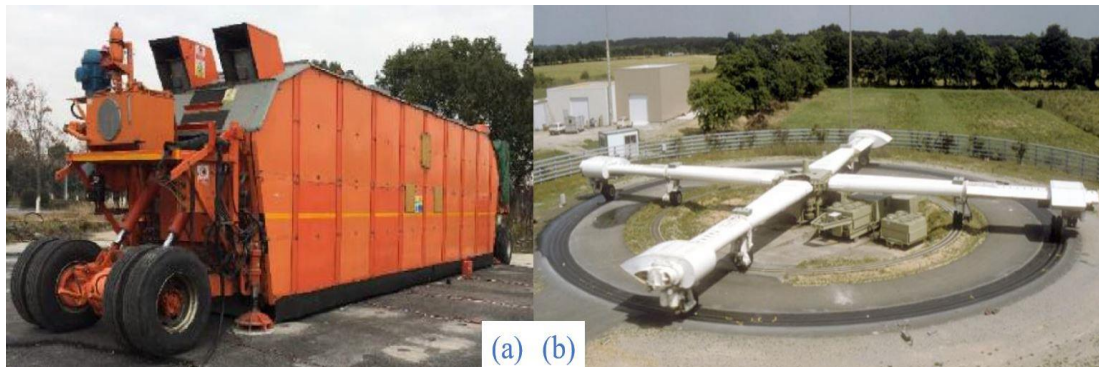


Figure 2.20: a) The MLS66 APT test vehicle [Ling et al., 2019], b) IFSTTAR system for APT [Nguyen et al., 2020].

In addition to load simulators and controlled test sites (Figures 2.18 – 2.20), APT testing is also carried out using actual vehicles. The US National Center for Asphalt Technology (NCAT) constructed a full-scale track APT facility in 2000 in Lee country, Alabama, USA (Figure 2.21). An actual heavy trailer truck with three trailers (Figure 2.22a) and box trailers (Figure 2.22b) are used for traffic loading along the 2.7 km track which consists of forty-six different 60 m long sections [Powell, 2012].



Figure 2.21: The full-scale APT track for innovation and research of pavement technologies in the US National Center for Asphalt Technology [Web 1, 2023].

Valero et al. (2014) investigated the performance of geogrid reinforced sections in full-scale traffic under realistic service conditions. To apply full-scale real traffic loads, they used an actual conventional 2-axle truck with each axle modified to apply a load of 80 kN (Figure 2.23).



Figure 2.22: NCAT APT actual vehicles, a) Triple trailer [Web 2, 2023], b) Box trailer [Timm and Priest, 2005].



Figure 2.23: Conventional 2-axle truck with each axle modified to apply a load of 80 kN [Valero et al., 2014].

It is seen that different magnitude and speed of loads were applied in various combinations for APT in the literature. These differences are due to the capacities and properties of the APT systems and the types of pavements investigated [Jersey et al., 2012; Howard and Warren, 2008; Aguiar-Moya et al., 2016].

Different APT setups have different speed capacities ranging from 5 km/h to 100 km/h [Aguiar-Moya et al., 2016]. Mshali and Steyn (2020) reported that deformation occurs very quickly at speeds below 30 km/h, while deformation occurs very slowly at speeds above 40 km/h. In addition, Corte et al. (1997) reported that increasing speed from 40 km/h to 50 km/h reduced rutting by 20-35%. Therefore, it is concluded that the speed of APT effort should be applied below 30 km/h to achieve high deformation levels in shorter period. The maximum speed of the "Road Machine 3" developed in 1963 is 25 km/h and the maximum speed of the Nottingham Road Pavement Testing Laboratory established in 1973 is 16 km/h. The Danish Road test machine developed in 1973 has a maximum speed of 30 km/h, and many of the systems that are still being developed, such as the Australian ALF and NE-ALT, have speed settings ranging from 0 to 30 km/h [Brown and Brodrick, 1999, Larsen and Ullidtz, 1998, Powell, 2012, Wang, 2021].

3. DESIGN OF PAVEMENT SYSTEMS

A layered structure, constructed atop a road embankment using selected materials with various physical and mechanical properties, is called pavement. The pavement carries traffic loads and distributes them to make sure that service loads does not exceed the bearing capacity of the subgrade [Kök, 2019]. The embankment and pavement should be constructed according to relevant design requirements to ensure that the embankment performs according to the design criteria throughout its service life. One of the most important parameters when designing the pavement system is the bearing capacity of the sub-base [Kök, 2019]. The California Bearing Ratio (CBR), a static penetration test, is used to obtain the bearing capacity in road constructions. The wet CBR value for the sub-base is required to be greater than 8% for flexible pavements [Kök, 2019].

The geofoam blocks used in the construction of highway embankments have much smaller CBR values than that of the compacted earth fill. CBR values of geofoam blocks range from 2% to 4% [Stark et al., 2004b]. Subbases with these low CBR values are classified as "very bad/bad-medium" [Kök, 2019]. Therefore, geofoam block embankments require special attention when designing their pavement systems.

Typical flexible pavements constructed to carry traffic loads consist of subbase, aggregate base and surface (bituminous base, binder and wearing course) layers. Aggregate base layers should be constructed from coarse-grained soils considering their physical, mechanical and hydraulic properties, such as stress transfer, durability, drainage conditions and freeze-thaw resistance. Based on these criteria, the maximum grain size for granular materials to be used in the construction of highway aggregate base layers were defined as 50 mm by AASHTO M147 (2017) to 75 mm by KGM (2023) and the maximum percentage of material passing sieve number 200 was defined as 0-12% as a type of granular materials (Types A-C) [KGM, 2023] to 2-25% as a function of grading requirements for soil-aggregate materials (Grading A-F) [AASHTO M147, 2017]. The surface layers of pavement consist of a bituminous base, binder and wearing course prepared traditionally from hot-mix asphalt (HMA). In areas with heavy vehicular traffic, a bituminous base, which is also called coarse-grained asphalt concrete, is constructed by mixing well-graded coarse-grained aggregate and bituminous binder in a plant. The binder layer, which is also called

medium-grained asphalt concrete, is constructed by mixing well-graded medium-grained aggregate with a bituminous binder in the plant. The last layer of the pavement, the wearing course, which is also called as fine-grained asphalt concrete, is prepared by mixing aggregate with a fine-grained aggregate with a maximum diameter of 19 mm and a bituminous binder in the plant.

The stress-strain behavior of typical aggregate layers is a function of the effective stress, and the stiffness of these layers is limited due to the small effective stresses induced by the dead load of the pavement structure [Stark et al., 2004b]. Pavement stiffness is defined by the ExD^3 , where E is the modulus of elasticity of the layer and D is the thickness of the layer [Stark et al., 2004b]. This relationship indicates two different situations. First, the layer thickness has a more significant effect on the stiffness performance than the material modulus of elasticity; therefore, the pavement system stiffness can be significantly increased by increasing the layer thickness. Second, using a material whose modulus of elasticity is relatively much larger than the modulus of elasticity of the typical granular base materials, the layer thickness can be reduced; therefore, the pavement system stiffness can be increased without increasing the total thickness of the pavement. In the case of the first possibility (increasing the granular base thicknesses), dead loads on the geofoam block highway embankment will increase, which will negatively affect the internal stability of the geofoam block highway embankment (increased block deformations and reduced stability). In order to prevent an increase in the dead load atop geofoam block highway embankment, second consideration (using a material with a relatively large modulus of elasticity) is traditionally preferred. The stiffness of the pavement system is increased by constructing a reinforced concrete platform between the geofoam block embankment and the granular base layer. A reinforced concrete platform, which also called as load distribution slab or Portland Cement Concrete (PCC) slab, was first applied in the construction of a geofoam block highway embankment in 1975 [NRRL, 1987]. The stiffness of a 10 cm thick PCC load distribution slab is equivalent to the stiffness of a 30-40 cm thick granular material layer [Aabøe, 1987; Refsdal, 1987]. In addition to increasing the system stiffness without increasing the overall thickness of the pavement, the PCC slab serves as the basis for separating the geofoam block embankment and granular base layers, as well as it creates a platform for the

installation of road hardware such as signaling, guardrail barriers and similar highway equipment.

The construction of a PCC load distribution slab with a minimum thickness of 10 cm between the geofoam block backfill and pavement was included in the design specification published by NPRA in 1992 [NPRA, 1992]. In subsequent years, the effect of the reinforced concrete load distribution slab on the stiffness of the pavement system was investigated using PLT in six different pavement section as shown in **Figure 3.1** [Duškov, 1997].

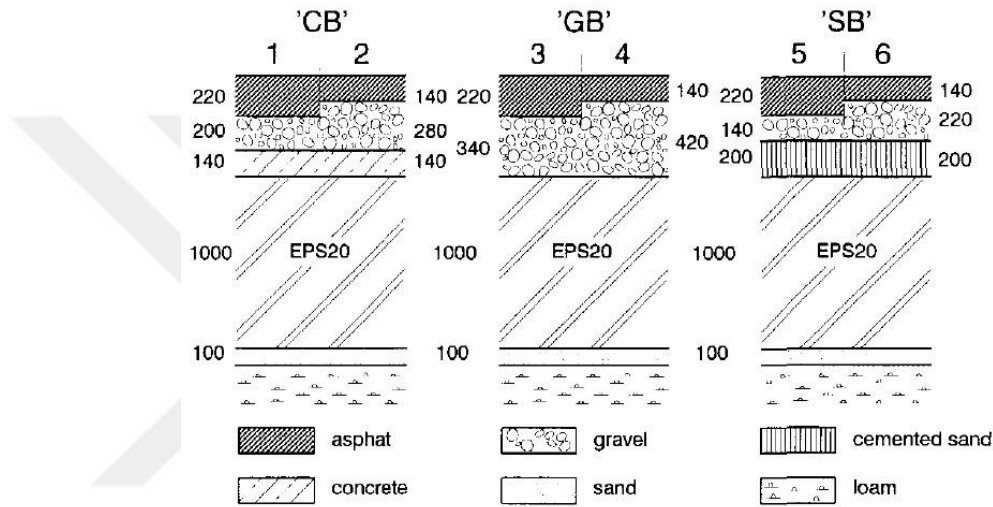


Figure 3.1: Six different pavement sections [Duškov, 1997].

The deformation modulus of the pavement constructed using only granular base atop the geofoam block embankment constructed using EPS20 with a total thickness of 34 to 42 cm was measured as 28 MPa (GB section in **Figure 3.1**). On the other hand, a deformation modulus of 136 MPa was measured for the pavement system with a 14 cm-thick PCC slab and granular base with a thickness of 20 to 28 cm (CB section in **Figure 3.1**) [Duškov, 1997].

The recommended design specification published by the American National Cooperative Highway Research Program (NCHRP) states that the pavement atop a geofoam block embankment should be at least 61 cm to minimize the effects of differential icing and solar heating [Stark et al., 2004a; 2004b]. The traffic load transfer mechanisms of flexible pavement sections with and without a PCC slab with a total thickness of 61 cm were investigated by series of analysis and design charts were published [Stark et al., 2004a].

The heaviest vehicle given in the Highways Technical Specifications is the H30-S24 standard truck and a pair of wheels on the rear axle of this vehicle causes a vertical load of 12 tons-force [KGM, 2023]. According to the NCHRP design charts, the vertical stress increase caused by a 12 ton-force (≈ 120 kN) wheel load on the geofoam block embankment surface was calculated as 66 kPa (**Figure 3.2**) for a section consisting of 43.2 cm thick subbase and base + 17.8 cm asphalt concrete, and 27 kPa for a section consisting of 10.2 cm thick reinforced PCC slab + 33 cm thick subbase and base + 17.8 cm asphalt concrete (**Figure 3.3**) [Stark et al., 2004a]. Therefore, using a 10 cm thick reinforced PCC slab reduces the vertical stress acting atop the geofoam block embankment surface more than twofold under the same traffic load without increasing the total thickness of the pavement (**Figures 3.2** and **3.3**). Due to its advantages, the construction of a reinforced PCC slab at least 10 cm thick (between the geofoam block embankment and the granular base layer) has become a standard practice for the pavement systems of geofoam block highway embankments [Aab   et al., 2019]. Reinforced PCC slab was also constructed in the pavement configurations of the first four geofoam block highway embankments in T  rkiye in between 2017 and 2019 (**Figure 3.4**) [  zer et al., 2017;   zer and Akınay, 2017;   zer et al., 2018;   zer and Akınay, 2019,   zer, 2020;   zer, 2021;   zer and Akınay, 2022].

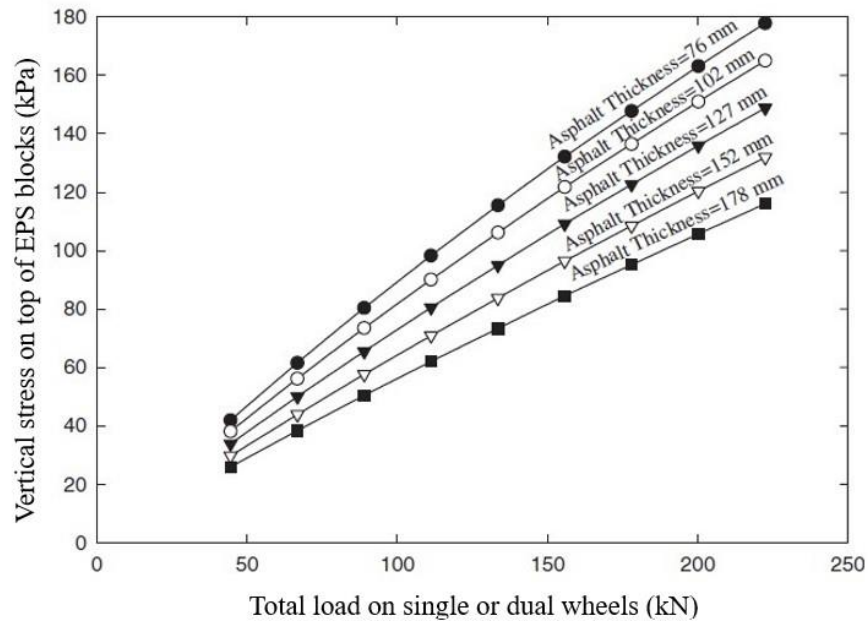


Figure 3.2: Vertical stress atop the EPS blocks due to traffic loads applied on 61 cm thick pavement system [Stark et al., 2004a].

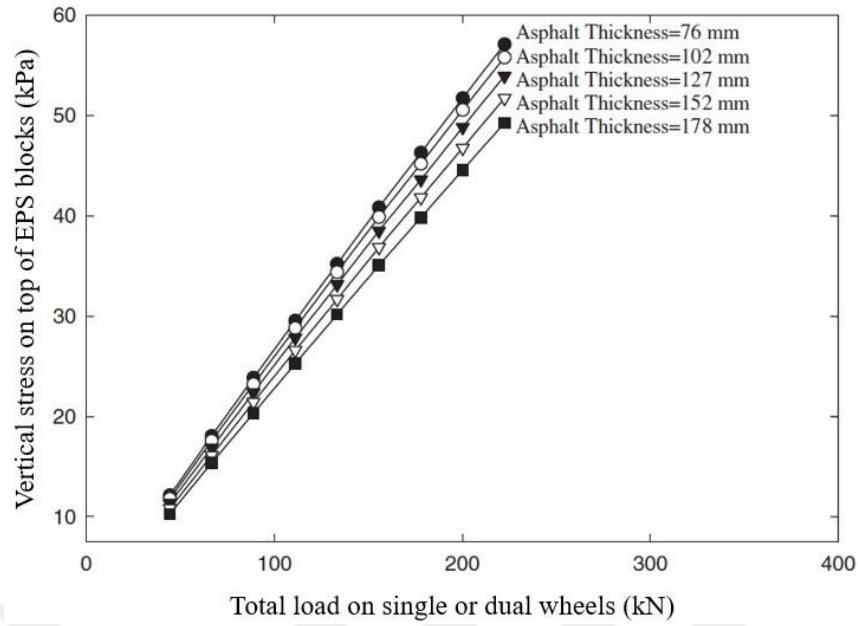


Figure 3.3: Vertical stress atop the EPS blocks due to traffic loads applied on 61 cm thick pavement system with a 10.2 cm PCC slab between pavement and EPS [Stark et al., 2004a].



Figure 3.4: Details of reinforced PCC slab atop EPS blocks in Boğazköprü Bridge, Kocasinan, Kayseri, Türkiye [Özer, 2020].

However, despite its load-distribution benefits, using PCC slab also has some disadvantages. The construction of the reinforced concrete platform directly affects the project duration due to the required curing time. In addition, plastic shrinkage and structural cracks, which occur due to inappropriate or insufficient curing, prevent

reaching its design strength [Özer and Akay, 2022]. Regardless of these disadvantages, fast and applicable alternative flexible pavement system for geofoam block highway embankments without PCC slab has yet to be developed.

In the construction of paved or unpaved pavements, geocell and geogrid applications from the geosynthetics family products are widely used to improve the performance of the pavement under traffic loads and reduce the pavement's total thickness [Giroud et al., 2021].

Geocell (**Figure 3.5a and 3.5b**) is a geosynthetic product manufactured from a polymer-based material (e.g., HDPE, polypropylene), that comprises three-dimensional, interconnected cells (in a honeycomb-like form). The geocell, due to its three-dimensional structure, provides lateral confinement to the infill; thus, it is used to increase the stiffness and bearing capacity of subbase-base and reduce permanent deformations under repetitive loads [Han et al., 2010]. The main benefits of geocell are limiting the lateral displacement of the embankment, providing a wider distribution of loads over a larger area due to its rigid form and combined structure [Han et al., 2010].

Geogrid (**Figure 3.5c and 3.5d**), is a geosynthetic product which is categorized into three different types according to its production technology. The first type of geogrids is produced by drawing a polymer-based material on which uniform holes are drilled by punching. The second type of geogrids is coated yarn geogrids, which are manufactured by knitting high-strength polyester yarns at the overlap points and coating the resulting grid structure with PE, PVC or bitumen. The third type of geogrids is generally produced by joining high-strength polyester or polypropylene stiffeners by laser or ultrasonic welding at overlapping points [Koerner, 2005].

Generally, biaxial and tri-axial geogrids with apertures to allow interlocking surrounding soil are used in aggregate base layers of flexible pavement systems. The behavior of unbonded granular layers that are reinforced with geocell or geogrid which are constructed on weak-soft soils with CBR values less than 8% has been investigated via static or dynamic PLT by various researchers [Al-Qadi and Hughes, 2000; Emersleben and Meyer, 2008a; Thakur, 2013; Kief et al., 2014; Pokharel et al., 2017; Inti 2018; Norouzi et al., 2019; Goud et al., 2020; Giroud et al., 2021]. Previous studies have shown the importance of the geogrid location and the granular base layer

thickness, which affect the vertical stress distribution caused by cyclic loading, to reduce permanent surface deformations and increase pavement service life [Haas et al., 1988].

The lateral confinement and membrane effect of the geogrid on the unbound granular material improve the vertical stress distribution [Leng and Gabr, 2002]. In addition, using a geogrid with a higher deformation modulus has a better load distribution effect and reduces permanent surface deformations [Leng and Gabr, 2002].

Abu-Farsakh and Chen (2011) reported that geogrid performed better when placed at the upper 1/3 of the unbonded granular base layer (compared to placement at the bottom or center of the base layer). Abu Farsakh et al. (2016) found that the unbound granular layer reinforced with two layers of geogrid (one at the bottom and one at the top 1/3 of the base) provided better performance.



Figure 3.5: Geosynthetics used in pavements, a-b) Geocell [Web 3, 2023], c-d) Geogrid [Web 4, 2023].

Emersleben and Meyer (2008a) reported that using a thicker geocell with a smaller mesh size provides a better performance than that of other geocells. In the reconstruction of a road on soft ground in Germany, using geocell resulted a 47%

reduction in the thickness of unbounded granular layer. According to static PLT effort on the gravel base course layer before the asphalt layer was constructed, the vertical stress on the subgrade was reduced by 50% with the geocell reinforcement [Emersleben and Meyer, 2008c]. The geocell provides horizontal and vertical confinement to the granular material. The beam and/or membrane effect occur in the granular soil layer constructed with a geocell under vertical loads. The beam effect is defined as a structural behavior that resists both bending and tension, whereas the membrane effect is defined as a behavior that resists only tension [Thakur et al., 2012]. The beam effect occurs in relatively thick sections and the membrane effect occurs in relatively thin sections, depending on the deformation level [Thakur et al., 2012]. The beam effect and/or the membrane effect increase the stress distribution angle and therefore transfer smaller vertical stresses to the base of the pavement [Asha and Latha, 2014]. Asha and Latha (2014) recommended that the geocell should be placed at the interface between the base of the pavement and the foundation and a granular backfill of sufficient thickness should be placed atop the geocell. The unreinforced layer thickness of 30 cm can be reduced by 50% using geocell and this reduction will reduce CO₂ emissions by 21 tons for a 1-kilometer road section [Suku et al., 2016].

Hedge and Sitharam (2013a; 2014) developed a model for investigating the behavior of sand reinforced with geogrid and/or geocell using the two-dimensional finite difference modelling technique using FLAC2D. The geocell-reinforced sand layer is modeled as an equivalent "composite geosynthetic-soil" layer. The results obtained by the analyses were in general agreement with the behaviors observed in the field measurements [Hedge and Sitharam, 2013b]. Hedge and Sitharam (2014) also modeled the behavior of sand reinforced with geogrid and/or geocell using the three-dimensional finite difference modelling technique. In three-dimensional modeling, the geogrid and geocell were modeled using the geogrid structural element from the software's (FLAC3D) design tools. It was observed that the results of modelling effort conformed with the field measurements [Hedge and Sitharam, 2014]. Following these studies, Hedge et al. (2016) modelled the behavior of geocell-reinforced sand with finite element modelling technique using the PLAXIS3D software. In the three-dimensional modeling, the geogrid structural element from PLAXIS3D design tools was used. Although the values obtained for the unreinforced section behavior were very close to the measured values, the reinforced section were generally not in

agreement with the measured values, in contrast to the analysis performed using FLAC3D [Hedge et al., 2016]. Hedge et al. (2016) argued that this difference was related with the shape of geocell which was modelled as a square instead of a honeycomb.

Sanjei and De Silva (2016) developed a geocell geometry in the form of a honeycomb using AutoCAD to eliminate the shape effect. They used this honeycomb model to analyze the behavior of geocell-reinforced sand using PLAXIS3D software. In these analyses, where the shape effect was eliminated, results were very close to that of the measured values [Sanjei and De Silva, 2016].

Various studies have been conducted based on laboratory and full-scale field experiments with geocell [Rajagopal et al. (1999; 2012; 2014), Kief et al. (2014), Asha and Latha (2014), Pokharel et al. (2009a; 2009b; 2009c; 2010; 2011; 2013; 2015; 2018), Han et al. (2008a; 2008b; 2010; 2011), Emersleben and Meyer (2008a; 2008b; 2008c) and Al-Qadi and Hughes (2000)] and geogrid reinforcements [Devi (2019), Han and Giroud (2012), Qian et al. (2011), Tingle and Jersey (2005), Giroud and Han (2004a; 2004b; 2012), Tingle and Webster (2003), Collin et al. (1996), Fannin and Sigurdsson (1996), US Army Corps of Engineers (1995), Giroud et al. (1985) and Steward et al. (1977)]. There is a consensus that geogrid reinforcement can improve the behavior of pavements under traffic loads and reduce the overall thickness of the pavement.

Despite the widespread use of both geogrid and geocell reinforcements in pavement configurations, a study has yet to be conducted to investigate flexible pavements for geofoam block highway embankments with geocell or geogrid reinforcements using field-scale prototype models.

Siabil et al. (2020) performed a series of laboratory PLT to investigate the behavior of geocell reinforced pavement section under repeated traffic loads where geofoam was used as a sub-base. Even though the study provided insight of the performance of geocell in the geofoam pavements, the results were not directly implemented since the traffic loads were simulated by an actuated plate loading instead of actual wheel loads. In addition, the boundary conditions of the experimental set-up were restrictive to mimic prototype behavior.

Therefore, a study that will investigate the applicability of different flexible pavement system under actual wheel loads with field-scale prototype geofoam block highway embankment is needed. In this thesis, a total of five different flexible pavement prototypes were constructed with an EPS geofoam block sub-base. The testing program was designed to investigate the performance of alternative pavement systems for geofoam block embankments without PCC slab, which is the prevailing traditional practice in geofoam highway embankment constructions. The reinforced PCC slab was also constructed as 10 cm-thick which is considered as the reference pavement system among the five different prototype pavement systems. The other four configurations were constructed without a concrete load distribution slab to investigate their applicability for prototype geofoam block embankments.

This pavement cross-section is considered as a reference thickness for the design of the first pavement configuration which includes a PCC slab (Configuration #1, **Figure 3.6**). The design of other four pavement systems did not include a PCC slab which is considered as the industry standard for geofoam block embankment. The first of these remaining four flexible pavement was designed using only granular base (Configuration #2, **Figure 3.6**) while other three flexible pavement configurations were designed using various geosynthetic reinforcements (Configurations #3, 4 and 5, **Figure 3.6**).

The AASHTO design method, which includes design requirements and design process, the pavement layer thickness is evaluated according to the serviceability-behavior relationship, considering the soil-bearing capacity, traffic load, regional climatic conditions and pavement layer properties (Equation 3.1). The design requirements include the set of input parameters for the design process are listed as follows:

Design variables: Time constraints, traffic, reliability and environmental impacts

Performance criteria: Serviceability

Material properties for structural design: Resilient modulus of the roadbed, material characterization and layer coefficients for pavement layers

Pavement structural characteristics: Drainage condition

Design variables: Time constraints, traffic, reliability and environmental impacts

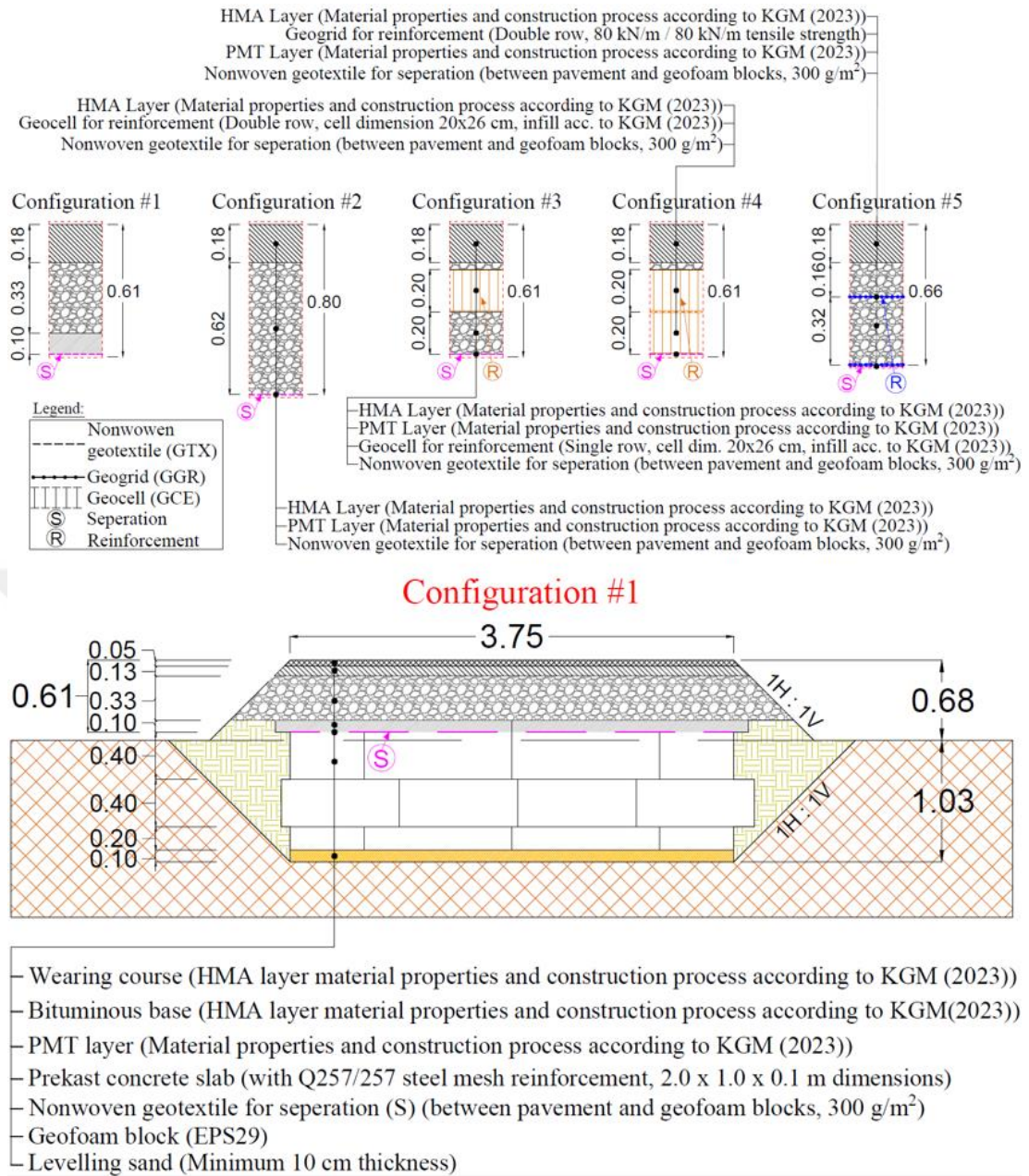


Figure 3.6: Cross-section of five prototype pavement configurations and detailed section of the Configuration #1 as reference pavement system.

Once the required inputs were determined, layer thicknesses are obtained using a two-step calculation. First, the structural number (SN), which is a number that converts the thickness of flexible pavement layers by using layer coefficients derived from the traffic analysis, subgrade conditions, drainage characteristics and ultimate serviceability is calculated using Equation 3.1, then the layer thicknesses calculated using Equation 3.2.

$$\log_{10} W_{18} = Z_R \cdot S_0 + 9.36 \cdot \log_{10}(SN + 1) - 0.20 + \frac{\log_{10} \left[\frac{\Delta PSI}{4.2 - 1.5} \right]}{0.40 + \frac{1094}{(SN + 1)^{5.19}}} + 2.32 \cdot \log_{10}^{M_R} - 8.07 \quad (3.1)$$

where;

W_{18} : Predicted number of 80 kN equivalent single axle load applications (ESAL)

Z_R : Standard normal deviate (Depends on reliability of pavement design)

S_0 : Combined standard error of the traffic prediction and performance prediction (Depends on type of pavements)

ΔPSI : Difference between the initial design serviceability index, p_0 , and the design terminal serviceability index, p_t (Depends on volume of roads)

M_R : Resilient modulus of sub-base (psi)

SN : Structural number (inches)

$$SN = a_1 \cdot D_1 + a_2 \cdot D_2 \cdot m_2 + a_3 \cdot D_3 \cdot m_3 \quad (3.2)$$

where;

a_1, a_2, a_3 : Layer coefficients for surface, base and subbase course materials, respectively

D_1, D_2, D_3 : Thickness (in inches) of surface, base and subbase course, respectively

m_2, m_3 : Layer drainage coefficients base and subbase, respectively

The layer coefficients can be calculated by Equation 3.3 according to the KGM (2008).

$$a_i = 0.0045 \cdot \sqrt[3]{M_{Ri}} \quad (3.3)$$

where;

a_i : Layer coefficient of the layer in concern

M_{Ri} : Resilient modulus of the layer in concern (psi)

The reliability value for local urban roads can be taken between 50% to 80% [AASHTO; 1993]. Stark et al. (2004a) analyzed the flexible pavement design of geofoam block embankment according to the AASHTO (1993) design method and published it in NCHRP Report 529. As a result of the analysis, the minimum structural number for the conditions provided in **Table 3.1** was calculated as 6.46 for the NCHRP Report 529 for the geofoam block embankment flexible pavements with 75% reliability. The parameters were selected based on AASHTO (1993) to use in SN calculations are given in **Table 3.1**.

Table 3.1: Parameters for structural number calculation at 75% reliability.

Design variables	Parameters from NCHRP Report 529	Parameters for Configurations #1-#5
ESAL		1,000,000
Initial serviceability index (p_0)		4.2
Terminal serviceability index (p_t)		2.0
Reliability level (%)		75
Standard normal deviate (Z_R)		-0.674
Overall standard deviation (S_0)		0.49
EPS block resilient modulus (psi)	725 (EPS19)	1160 (EPS29)
HMA, surface layer coefficient		0.44
Granular base layer coefficient		0.14
PCC slab layer coefficient*		0.50

*Only used in the designing Configuration #1.

In this study, the AASHTO (1993) design method for flexible pavement design and the NCHRP Report 529 by Stark et al. (2004b) were used for designing a flexible pavement for a geofoam block embankment.

The configurations were designed for 1,000,000 equivalent standard axle loads (ESAL) to represent traditional flexible pavement geofoam block highway embankment low traffic volume roads [Stark et al., 2004a; 2004b]. ESAL is the equivalent expression for different types of vehicles with different axle loads and number of axles converted into a constant single axle load, hence, the number of passes can be expressed in the same way [AASHTO, 1993; Giroud et al., 2021]. The standard axle load is equivalent to 80 kN [AASHTO, 1993; KGM, 2008; Giroud et al., 2021].

The first configuration (Configuration #1, **Figure 3.6**) was defined as the reference configuration since it represents typical industry standard pavement configuration with a PCC slab. This configuration was designed to comparatively evaluate the performance of the other geosynthetic-reinforced flexible pavement configurations (Configurations #3 - #5, **Figure 3.6**). The configuration #1, from bottom to top, comprised of a 10 cm thick PCC slab, 33 cm thick granular base (PMT material) and 18 cm thick HMA, respectively (**Figure 3.6**). The SN of this designed pavement section was calculated as 6.90 using Equation 3.2 with the parameters shown in the **Table 3.1**. This value is greater than the SN=6.46 calculated by Stark et al. (2004a) for minimum allowable cross-section flexible pavements of geofoam block embankments. Therefore, Configuration#1 is an acceptable design for an ESAL of 1,000,000. EPS29 was used as a sub-base layer in all configurations.

In addition, the minimum required SN was also calculated as 5.62 using Equation 3.1 for an ESAL value of 1,000,000 based on the parameters given in **Table 3.1** for the configurations. Hence, the SN for the Configuration #1 (**Figure 3.6**) was greater than both the minimum required SN which was recommended by Stark et al., 2004 and calculated using Equation 3.1, therefore, the Configuration #1 considered as a compatible reference section with the criteria provided in Stark et al. (2004b).

A non-woven geotextile was used for separation purposes atop the geofoam blocks (EPS29) (Configuration #1, **Figure 3.6**). In the construction of geofoam block highway embankments, the reinforced concrete load distribution slab is usually constructed as cast-in-place [Özer, 2020; 2021; Özer and Akinay, 2017; 2019; 2022; Özer et al., 2017; Özer et al., 2018; Özer et al., 2019; Özer and Ekincioğlu, 2019], whereas in this study the reinforced concrete load distribution slabs were precast and placed atop the blocks to ensure the non-destructive removal of geofoam blocks upon completion of the testing program.

Configuration #2 (**Figure 3.6**), like Configuration #1, was designed for the comparative purposes to evaluate the performance of geosynthetic reinforced flexible pavement configurations. Based on the section thicknesses designed for Configuration #2 (62 cm thick granular base and 18 cm thick HMA, **Figure 3.6**) and using the parameters in **Table 3.1**, SN of 6.54 was calculated using Equation 3.2. This calculated SN is not only greater than the minimum SN of 6.46 provided by Stark et al. (2004b) but also SN of 5.62 using Equation 3.1 for an ESAL of 1,000,000. Therefore, the proposed Configuration #2 is a compatible section for an ESAL of 1,000,000. A non-woven geotextile was used for separation purposes between the granular base and geofoam (**Figure 3.6**).

Configuration #3 consists of 43 cm thick granular base with a single row geocell reinforcement and 18 cm thick HMA layer (Configuration #3, **Figure 3.6**). Emersleben and Meyer (2008a; 2008b; 2008c) reported that a flexible pavement configuration with 70 cm thick granular base and 17.5 cm thick HMA has almost the same modulus of elasticity and deformation characteristics with a flexible pavement configuration consisted of a 40 cm thick, single row geocell reinforced granular base and 17.5 cm thick HMA pavement.

In the geocell reinforced section presented by Emersleben and Meyer (2008a; 2008b; 2008c), when the layer coefficients of HMA and the subbase are assumed as 0.44 and 0.14 (**Table 3.1**), respectively, the SN is calculated as 6.89 using parameters given in the **Table 3.1**. Since the calculated SN is greater than that of the SN of 6.46 [Stark et al., 2004a; 2004b], the proposed section for the Configuration #3 (**Figure 3.6**) is suitable for the 1,000,000 ESAL conditions. Calculated SN also greater than the SN of 5.62 using Equation 3.1.

Configuration #4 designed as 43 cm thick granular base reinforced with two-rows geocell reinforcements and 18 cm thick HMA layer (Configuration #4, **Figure 3.6**). This configuration was designed to investigate the effect of the number of geocell reinforcement rows for the pavement performance. A non-woven geotextile was considered for separation between the geofoam and granular base layer (**Figure 3.6**). Configuration #4 has the same SN as Configuration #3, because it has the same layer thicknesses at design section.

Configuration #5 design composed of 48 cm thick granular base with a two-rows of bi-axial geogrid reinforcements and 18 cm thick HMA layer (Configuration #5, **Figure 3.6**). Based on an APT program, Abu-Farsakh et al. (2019) reported that the traffic benefit ratio (TBR) can be used as 2 in a pavement section consisting of 45.7 cm thick granular base and 7.6 cm thick HMA consists of two rows of geogrid reinforcements placed at the bottom and upper 1/3 level of the granular base. Therefore, using layer coefficients of 0.44 and 0.14 for the HMA and granular base, respectively, the SN is calculated as 5.92 using parameters given **Table 3.1** for 48 cm thick granular base and 18 cm HMA pavement. ESAL of 500,000 (Equation 3.1) was calculated based on this SN. Abu-Farsakh et al. (2019) was recommended using TBR of 2 when two rows of geogrid reinforcement used. Therefore, ESAL value can be taken as 1,000,000 which makes the proposed Configuration #5 compatible with the criteria provided by Stark et al. (2004b). A layer of non-woven geotextile layer was also used for separation purposes between granular base and geofoam in this configuration as well.

The details of the study area, geotechnical characteristics of the site, and the construction sequence of the prototype configurations are presented in the next chapter in detail.

4. STUDY AREA AND PROTOTYPE CONSTRUCTION

The behavior of different flexible pavement systems on geofoam block highway embankments under actual dynamic and repetitive traffic loads was investigated with five different prototype configurations.

Prototype configurations were constructed at a 15 m x 100 m area located south of the soccer field in the south campus of the Gebze Technical University (GTU) (**Figure 4.1**). Site investigation program composed of standard penetration test (SPT) borings, pressuremeter (PMT*) and piezocone tests (CPTu) were performed to characterize the subsurface soil conditions. In addition, laboratory testing program were carried out to for classification purposes.

A standard penetration tests (SPT) to a depth of 16.5 m were performed according to the ASTM D1586/D1586M (2022) on 03.01.2020. Classification tests were carried out on the disturbed split-spoon samples collected from SPT drilling according to Unified Soil Classification System (USCS) at the Soil Mechanics Laboratory, GTU, Türkiye (**Figure 4.2**).

A layer of uncontrolled fill extending to a depth of 3 m below the ground surface consisting of poorly graded sand with gravel, concrete pieces and construction debris were encountered. Based on the information received from the GTU, this material was used as a levelling fill during the construction of the soccer field. Under this layer, lean clay (CL) with sand was encountered between 3.0 m - 4.0 m, fat clay with sand (CH) was encountered between 4.0 m - 7.5 m, and lean clay with sand (CL) was encountered between 7.5 m - 10.5 m. Beneath this layer; fat clay with sands and shell fragments (CH) were encountered up to the depth of termination of 16.5 m.



Figure 4.1: Site location map a) GTU south campus, b-c) The allocated area, approximately 15 m x 100 m, for the project at the south of the soccer field.

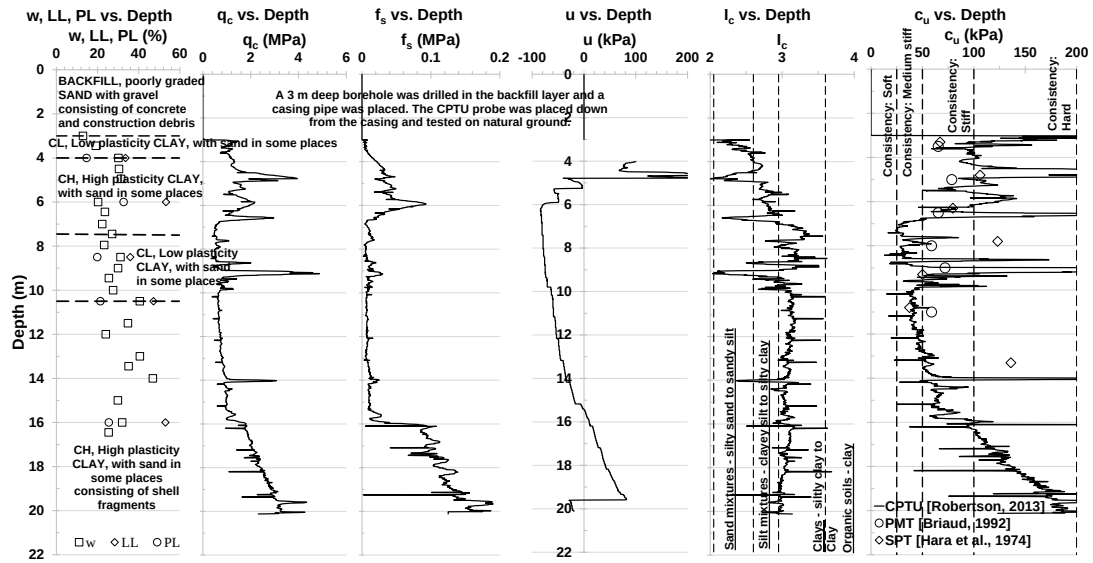


Figure 4.2: Site characterization: USCS soil types, w, LL, PL, CPTu, SBT Index and undrained shear strength profiles.

The groundwater level was measured 3.5 m below the ground surface, therefore 0.5 m below the natural ground surface. The SPT blow counts (N) of the clayey soils varied between 2 - 12 in the first 16.50 meters and their consistencies were very soft – soft - medium stiff - stiff. At 16.5 meters, SPT N of 32 was measured and the consistency was determined as hard according to the ASTM D1586/D1586M (2022).

Since the subsurface condition consists of lean-fat clays with sands and shell fragments, in situ tests were conducted to quantify the undrained shear strength (c_u). Pressuremeter (PMT*) and piezocone penetration tests (CPTu) were performed. PMT* were conducted at depths of 3.5 m, 5.0 m, 6.5 m, 8.0 m, 9.0 m and 11.0 m below the ground surface in the SPT borehole after the SPT was completed. CPTu performed to a depth of 20.12 m adjacent to the SPT borehole. The cone tip resistance (q_c), sleeve friction (f_s) and pore water pressure (u) were recorded at 1 cm interval (**Figure 4.2**).

The soil behavior type (SBT) index (I_c) was calculated from the CPTu measurements and the profile is provided in **Figure 4.2**. SBT types obtained from CPTu test results and physical classifications based on USCS were compatible (**Figure 4.2**). The undrained shear strength profile was obtained from SPT [Hara et al.,1974], CPTu [Robertson,2013] and PMT* [Briaud,1992] correlations. The undrained shear strength profile was comparatively provided in **Figure 4.2** based on in-situ tests correlations. The first 3 m of the natural ground was stiff to very stiff (average $c_u = 125$ kPa). The consistency of the clay is medium stiff in between 7 m - 8.5 m and 10 m - 14 m, with

average c_u of 35 and 45 kPa, respectively. The lowest c_u of 15 kPa was obtained (soft consistency) between 8.5 m and 10 m (**Figure 4.2**). The undrained shear strength was found to increase linearly with depth starting 14 m below the ground surface, and the consistency of the clays was found to increase from "stiff" to "very stiff" with depth (**Figure 4.2**).

It was found that the study area is a typical clayey soil site (**Figure 4.2**) where total and differential settlement can be the potential design issue for highway embankments. Consequently, the site was found to be a representative research site where prototype pavement system which has geofoam block subbase can be constructed.

All configurations were designed as 3.75 m wide and 10 m long (**Figure 3.6**). The schematic plan of configurations and direction of APT was designed as shown in **Figure 4.3**. Approach embankments were constructed to the east and west of the test bed to allow maneuvering of the truck to be used in the APT test.

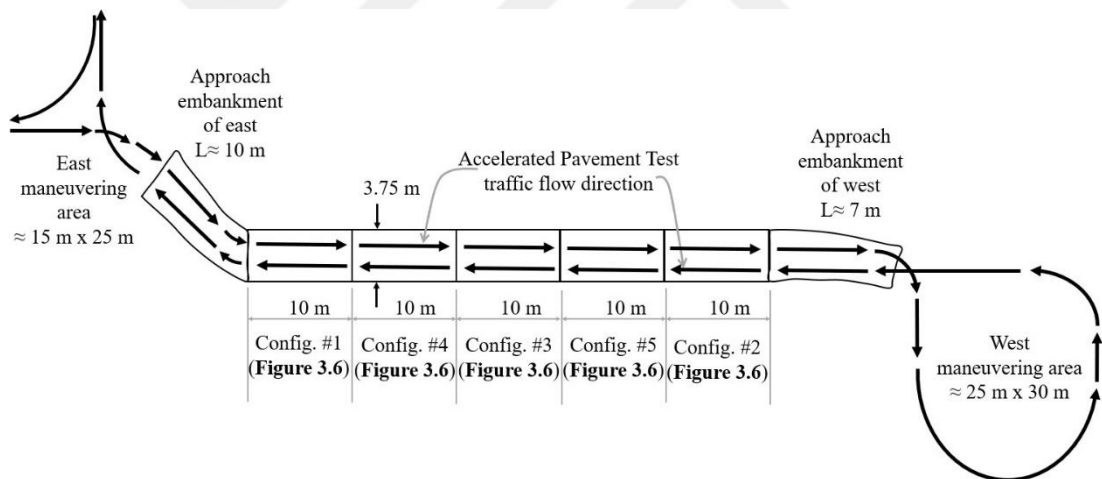


Figure 4.3: Schematic plan view of prototype constructions and APT test flow direction.

The Surveying Engineering Department of the GTU carried out entire surveying services in the study area (**Figure 4.4a-e**). Initially, surface elevations were collected to prepare excavation plan for the test bed (**Figure 4.4a**). The boundaries of the excavation plan were designed to construct the pavement sections (**Figure 3.6**) and marked to the site to guide excavation operations (**Figure 4.4b-d**). Since the thicknesses of the flexible pavement were the same for Configurations #1, #3 and #4, the excavation depth was 1.03 m, starting from the eastern end of the test bed (**Figure**

4.4e). Towards to the west, following these configurations, Configuration #5, which has a 5 cm thicker pavement section than Configurations #1, #3 and #4 (**Figure 3.6**), the depth of excavation was 1.08 m (**Figure 4.4e**). The last configurations located at the western end of the test bed, Configuration #2, has a depth of excavation of 1.22 m, which is 14 cm thicker than configuration #5 (**Figure 3.6**). These excavation depths was applied as an stair-step fashion along the bottom of the test bed to make sure the surface elevation of the HMA was the same for entire configurations (**Figure 4.4e**).

After the excavation plan was applied on site (**Figure 4.4**), the excavation was completed using an excavator mobilized to the site (**Figure 4.5a-f**). Following the completion of excavation (**Figure 4.5g**), construction of the approach embankments (**Figure 4.3**) located at the eastern and western ends of the test bed was started (**Figure 4.5h**).

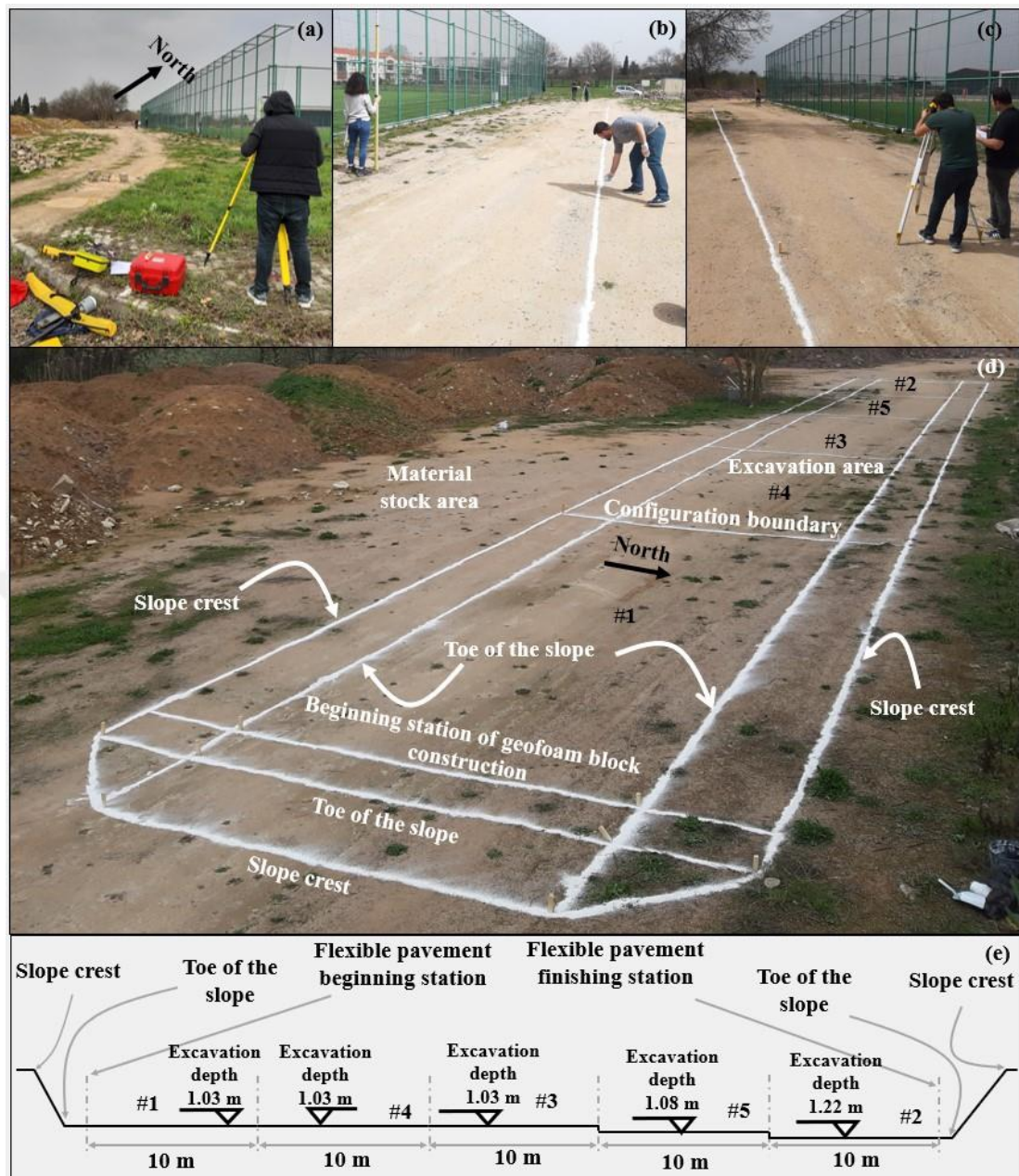


Figure 4.4: Project site marking and elevation controls, a) Measurement of surface elevations , b-d) Application of the excavation plan for the test bed, e) Longitudinal section of the excavation plan.

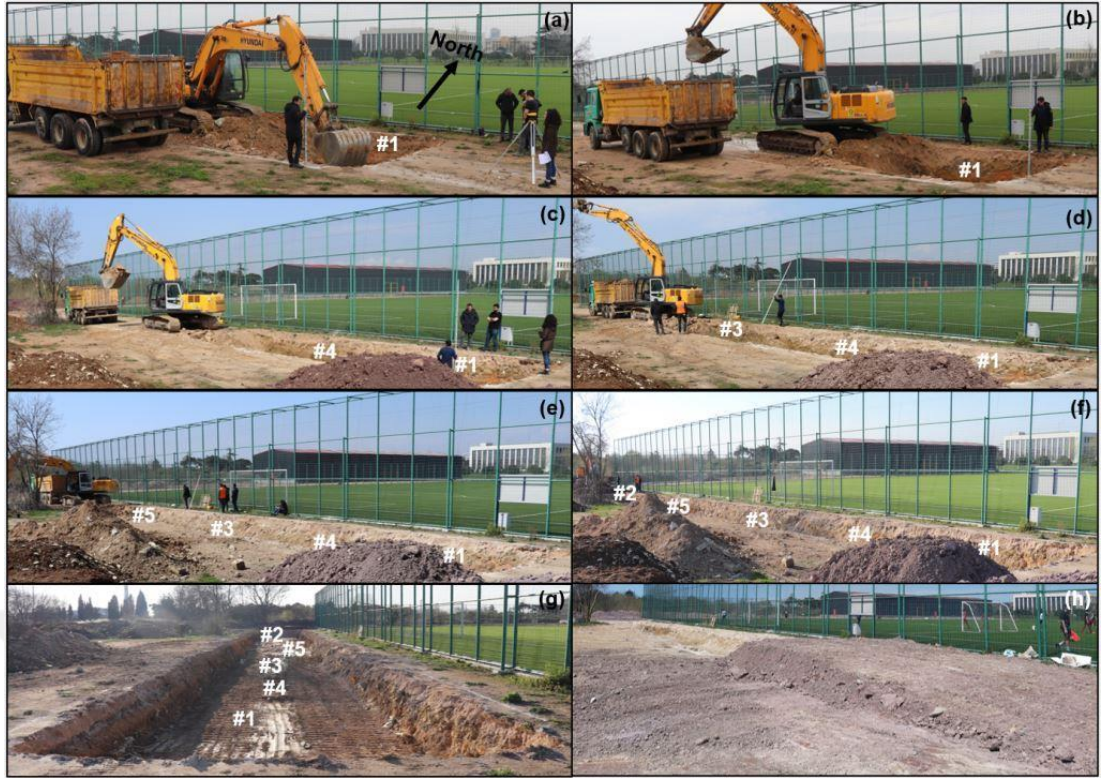


Figure 4.5: Excavation of the test bed, a-b) Excavation for Configuration #1, c) Excavation for Configuration #4, d) Excavation for Configuration #3, e) Excavation for Configuration #5, f) Excavation for Configuration #2, g) Completion of excavation, h) Construction of eastern approach embankment.

The first stage in the construction of traditional geofoam block highway embankments is placing the bedding sand [Stark et al., 2004a; Özer and Akınay, 2019; 2022; Özer, 2020; 2021]. The well-graded sand used for subgrade leveling was obtained from a quarry located in Tepeören/Tuzla/İstanbul. Approximately 10 cm thick bedding sand was laid at the bottom of the test bed with a wheeled backhoe-loader under the survey team's supervision (**Figure 4.6**).



Figure 4.6: Bedding sand delivery, stockpiling and placement a) Transported bedding sand stockpiled at the west of the test bed excavation, b) Laying of bedding sand for Configuration #1, #4 and #3 with a wheeled backhoe-loader, c) Setting up of total station for elevation control of bedding sand, d) Elevation control in Configuration #1, e) Elevation control in Configuration #4, f-g-h) Laying of bedding sand with a wheeled backhoe-loader in Configuration #4, #5 and #2, respectively, j) Completion of bedding sand placement.

Following the placement of bedding sand (**Figure 4.7a**), fine-tuning of the surface elevations was required to make sure that a tolerance of ± 1 cm for every 3 m of the alignment. An aluminum straightedge was used for the fine-tuning of the basal sand elevation (**Figure 4.7**).



Figure 4.7: Final surface levelling with a aluminum straightedge (a) Completion of bedding sand laying, (b) Elevation control of the bedding sand layer before the start of the leveling, c-j) Completion of the bedding sand placement per project tolerance.

Preparation of geofoam block placement plans is one of the crucial steps for the proper implementation of the project. There are some requirements to be considered when preparing the geofoam block placement plans. The block placement plan should be designed as the longitudinal axis of blocks in each row of geofoam blocks must be placed in a way that they are perpendicular to the longitudinal axis of the blocks at the above and below them [Özer 2020; 2021]. In addition, blocks in a row must be offset as a staggered fashion to prevent continuous vertical joint within geofoam block assemblage (**Figure 4.8**) [Özer, 2020;2021; Özer and Akay, 2016; 2022a; Özer et al., 2017; Özer and Ekinçioğlu 2019; Özer and Akınay, 2019; 2022]. Furthermore, the longitudinal axes of the geofoam blocks in the last layer should be placed perpendicular to the pavement axis [Özer 2020; 2021].

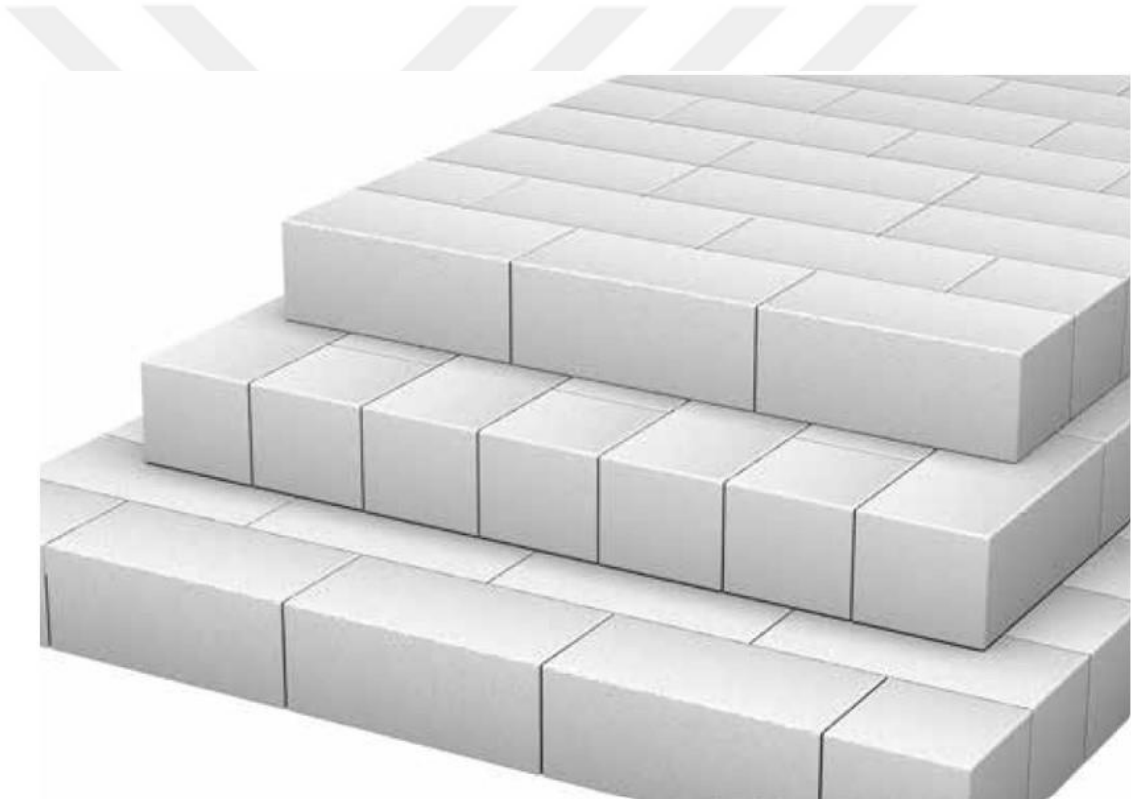


Figure 4.8: Typical geofoam block placement [Özer, 2020].

Geofoam block placement plans were designed considering these general rules. According to the block placement plan, a total of 1 m of geofoam block embankment was constructed using 20 cm thick geofoam blocks (on bedding sand) in the first row and 40 cm thick blocks in the second and third rows (**Figure 3.6**).

Geofoam blocks were labeled based on their dimension. Blocks with the dimensions of 20 cm high $\times$ 100 cm wide $\times$ 125 cm long (Type 1d), 20 cm high $\times$ 62.5 cm wide $\times$ 100 cm long (Type 1e) and 20 cm high $\times$ 50 cm wide $\times$ 125 cm long (Type 1f) were used for the first row. Block with the dimensions of 40 cm high $\times$ 100 cm wide $\times$ 125 cm long (Type 1a), 40 cm high $\times$ 62.5 cm wide $\times$ 100 cm long (Type 1b) and 40 cm high $\times$ 50 cm wide $\times$ 125 cm long (Type 1c) were used in the remaining rows (**Figure 3.6** and **Figure 4.9**). The geofoam block layout for configuration #1 is presented in **Figure 4.9**.

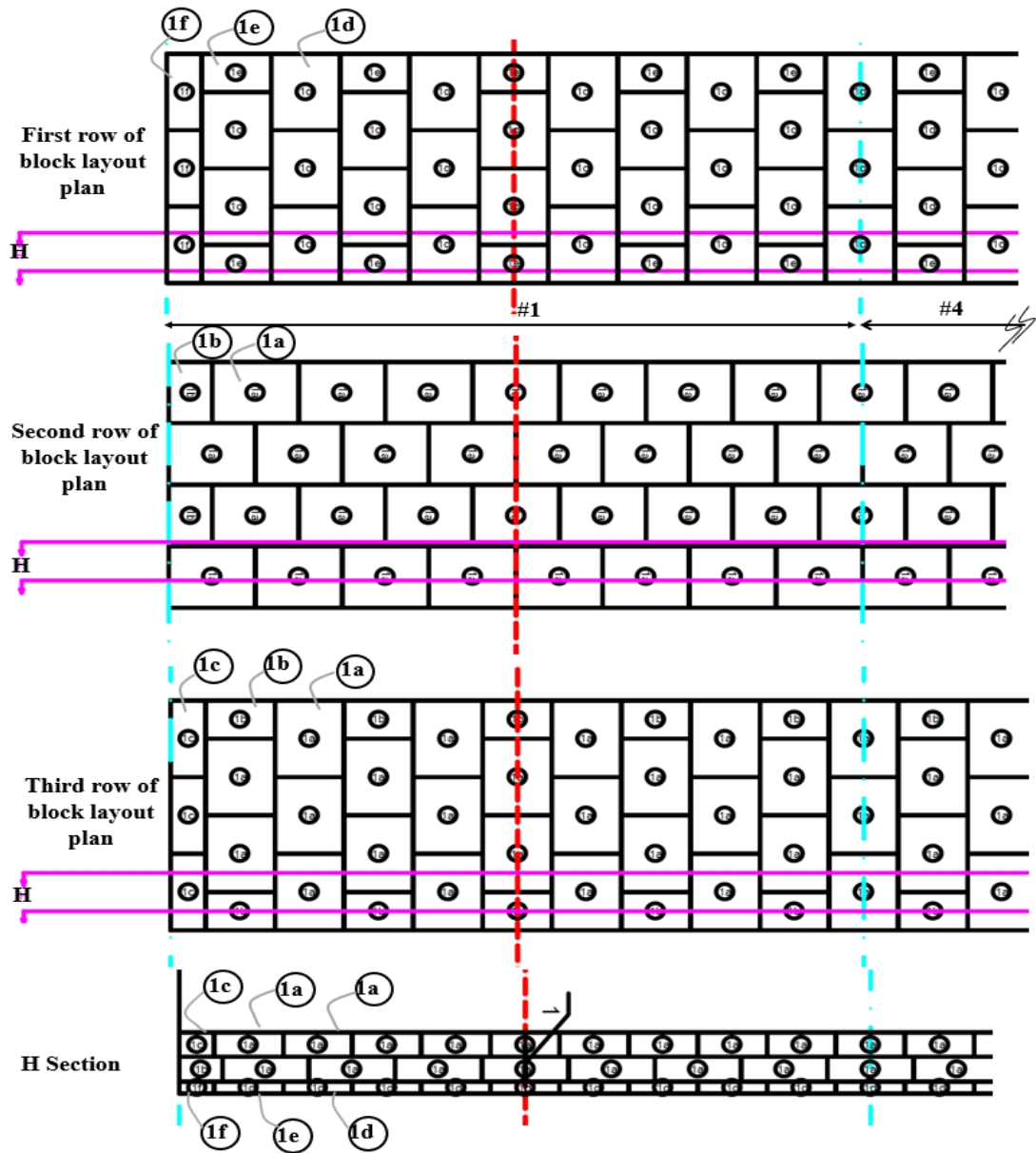


Figure 4.9: Geofoam block placement plan (Configuration #1).

Geofoam blocks were placed with gaps of no more than 2 cm in vertical-horizontal joints and with 1 cm tolerance according to project elevation. The height difference of the two side-by-side blocks in the same row was not exceed 5 mm and blocks located directly beneath the pavement was placed with a tolerance of ± 2 cm according to the project elevation (Figures 4.10, 4.11, 4.12, 4.13 and 4.14).

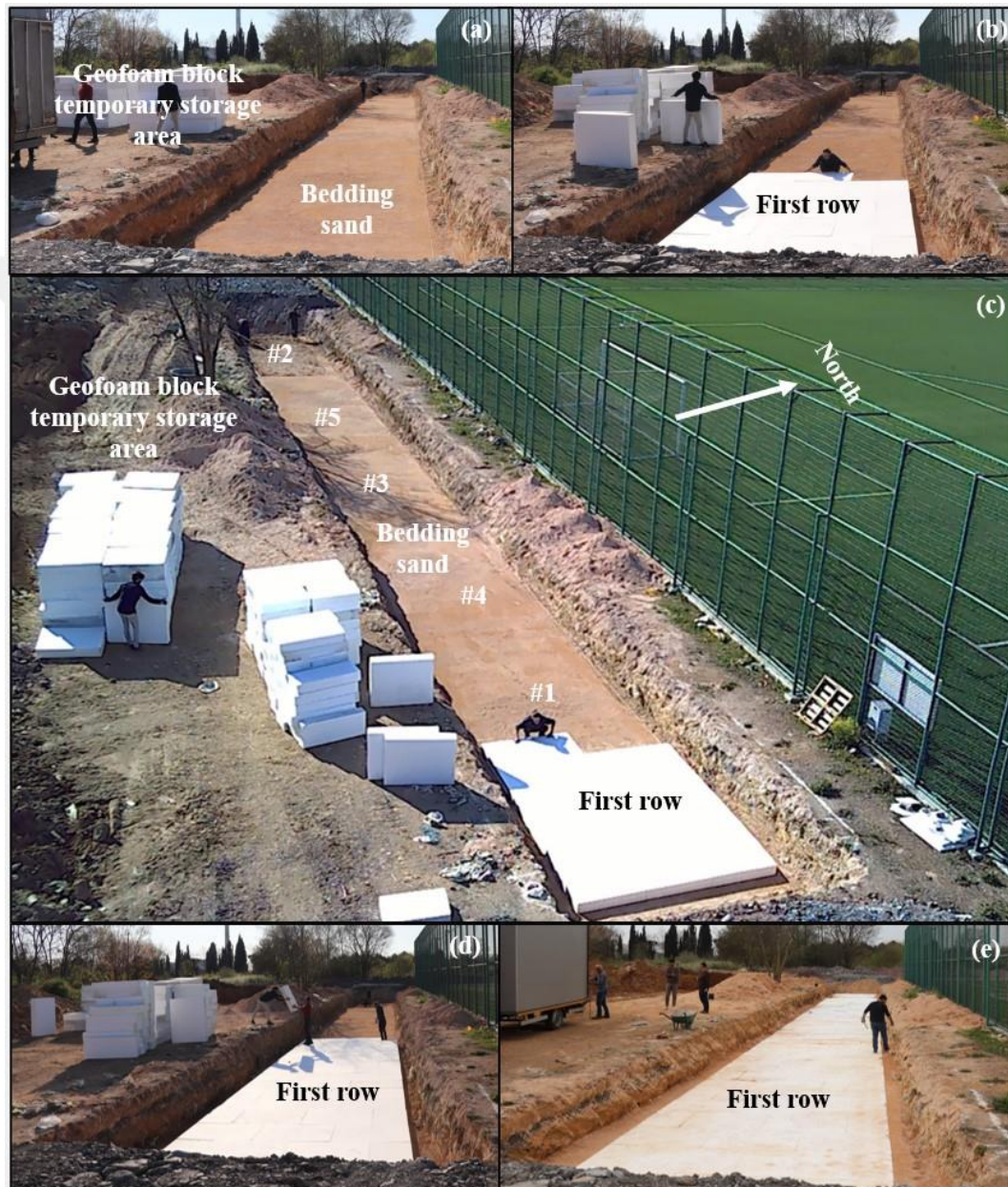


Figure 4.10: Geofoam block placement of the first row, a) Geofoam block temporary storage area, b) Beginning of first row block placement, c) First row block placement in Configuration #1 and placement of bedding sand in Configuration #5, d) Block placement of the first row in Configuration #4, e) Completion of the first row geofoam block placement.

Type 1d, Type 1e and Type 1f were transported to the site and stored in the temporary on-site storage area (**Figure 4.10a**). Geofoam block placement was started from configuration #1 and advanced to the west (**Figures 4.10b – 4.10d**). Geofoam block transportation and placement were carried out simultaneously in the entire first row placement (**Figure 4.10c – 4.10e**).

After the completion of the first row, the backfill of the area between the blocks and the slope side of the test bed excavation was backfilled with bedding sand which used for subgrade leveling sand (**Figure 4.11a**).

Upon the transportation of Type 1a, Type 1b and Type 1c blocks, block placement for the second row were started from Configuration #1 (**Figures 4.11b-g**) and advanced towards to west to the Configuration #2 (**Figure 4.11e**).

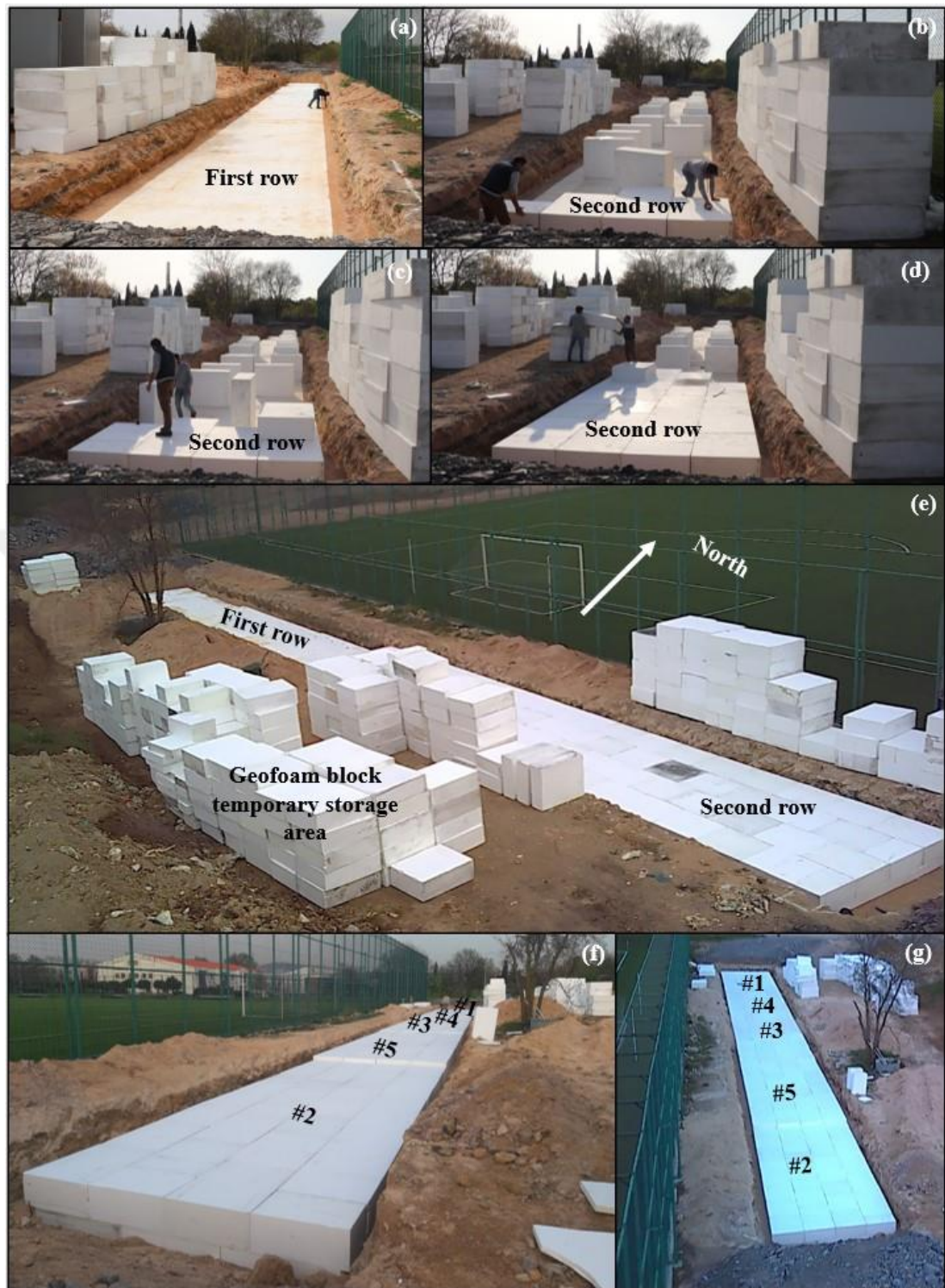


Figure 4.11: Blocks placement for the second row, a) Backfilling of the area between the blocks and slope side of the test bed excavation after the first row of geofabric blocks placement, b-d) Initiation of the second row placement plan from the Configuration #1, e) Progression of the second row placement to Configuration #3, f-g) Completion of the second row of the geofabric block assemblage.

Following the completion of the second row (**Figure 4.11f-g**), placement for the third row was started in the Configuration #1 (**Figure 4.12a**). Since the third row of blocks is the last row of geofoam block embankment, the longitudinal axes of the geofoam blocks in this row were placed perpendicular to the traffic flow direction (**Figure 4.12**). Configuration #1 is the traditional flexible pavement configuration therefore (Configuration #1, **Figure 3.6**) it was designed with a 10 cm thick PCC load distribution slab. In the construction of geofoam block highway embankments, the reinforced concrete load distribution slab is usually constructed as cast-in-place [Özer, 2020; 2021; Özer and Akinay, 2017; 2019; 2022; Özer et al., 2017; Özer et al., 2018; Özer et al., 2019; Özer and Ekincioğlu, 2019; Stark et al., 2004b], whereas in this study the reinforced concrete load distribution slabs were precast to make sure the non-destructive removal of geofoam blocks upon completion of the testing program. A nylon separation sheet was placed atop the blocks in the Configuration #1 to prevent adhesion bond between the concrete and the geofoam block interface (**Figure 4.12c**).

The wooden formwork, each 10 cm thick and 1 m wide x 2 m long, was installed atop the last row of blocks of Configuration #1 (**Figure 4.12d**). Q257/257 steel mesh reinforcements with 7 mm reinforcement diameter and 15 x 15 cm mesh opening size were placed in the panels formed after the formwork (**Figure 4.12e**). 2 pieces hook reinforcements for each panel were fastened to the steel mesh to lift the panels during removal operations after completion of the testing program. Following the placement of the reinforcement, class C30/37 concrete was poured into panels with a trans-mixer (**Figure 4.12f**).

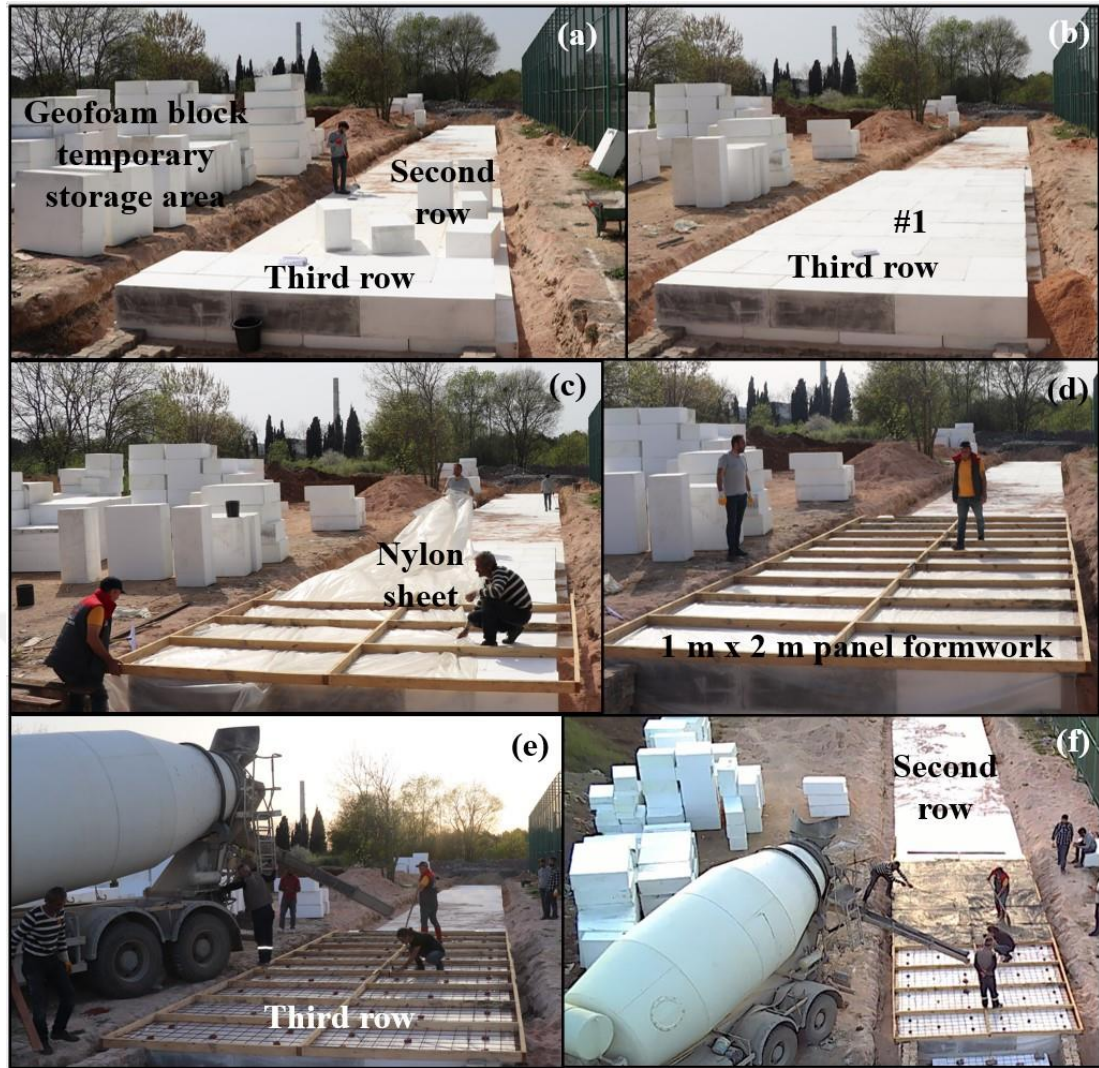


Figure 4.12: Third row block placement and casting PCC slab of Configuration #1, a) Beginning of the block placement for third row starting from Configuration #1, b) Completion of third row at Configuration #1, c) Nylon sheet placement atop the blocks, d) Formwork, e) Installation of steel mesh reinforcement, f) Casting of C30/37 class concrete.

One day after concrete pouring, the placement of the third row of geofoam blocks were continued from Configuration #4 (**Figure 4.13a**). Then, the block placements of Configurations #3, #5 and #2 were completed in turn (**Figure 4.13b-e**). While the block placements continued, the backfill of the area between the geofoam blocks and slope side of the test bed excavation was backfilled with bedding sand (**Figure 4.13e**).

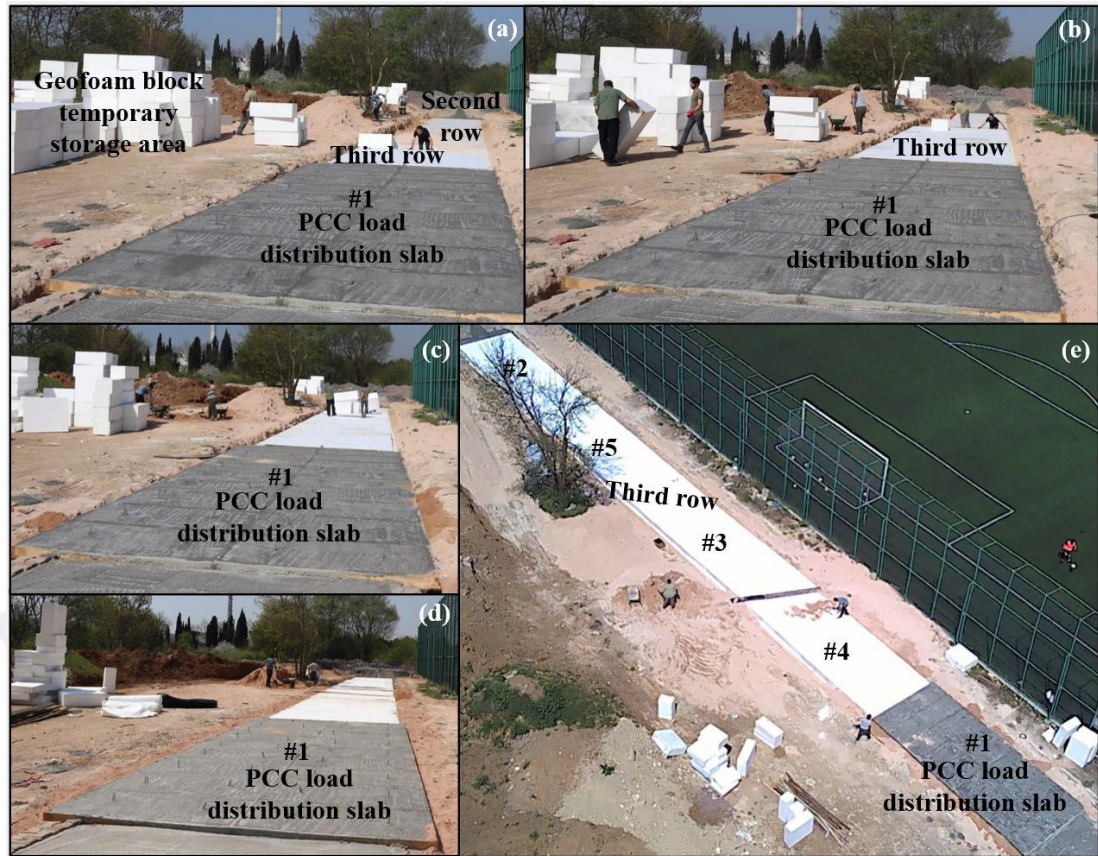


Figure 4.13: Blocks placement of the third row, a-d) Block placement at Configurations #4, #3, #5 and #2, respectively, e) Completion of geofoam block embankment and backfilling between slope of the test bed excavation and blocks in the third row.

After completion of the block placement according to the geofoam block placement plan (**Figure 4.9**), a layer of non-woven geotextile manufactured from polypropylene (PP) for separation purposes was placed at the interface of the granular base and geofoam blocks for flexible pavement in Configurations #2 through #5 (**Figure 4.14a-f**).

The flexible pavement of Configuration #4 consists of two rows and Configuration #3 consists of one row of geocells (**Figure 3.6**). The first row of geocell layer of Configuration #4 is placed atop the non-woven geotextile (**Figure 4.14c-d**).

The flexible pavement Configuration #5 consists of two rows of geogrids which was placed at the bottom and upper 1/3 level of the granular base (**Figure 3.6**). The first row of geogrid layer of the Configuration #5 was placed atop the non-woven geotextile (**Figure 4.14e-f**).

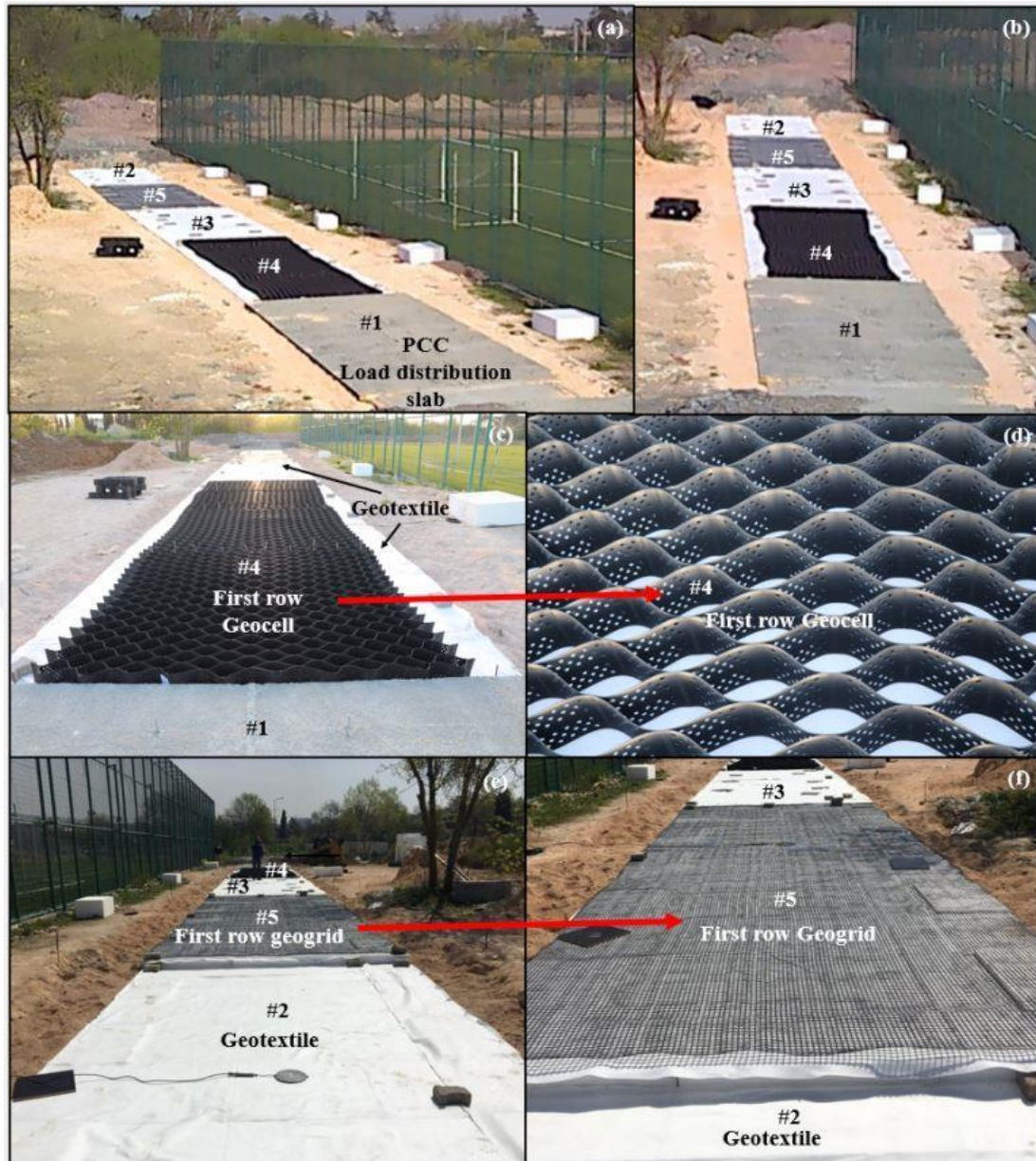


Figure 4.14: Placement of geotextile, geocell and geogrid, a-b) Placement of geotextile, first row geocell at the Configuration #4, first row geogrid at the Configuration #5, c-d) First row geocell at the Configuration #4, d-e) First row geogrid at the Configuration #5.

After the placement of the blocks and the placing of the first row of geosynthetics based on the cross-sections given in Figure 3.6, the laying-compaction of the granular base layer with plant mix (PMT) base course material was initiated.

During the placement of granular base, the construction equipment was not run directly atop the geofoam blocks or the PCC slab of Configuration #1 (**Figure 4.15**). Granular base is constructed using granular base course in two lifts. The first layer of the granular base course was placed atop the PCC slab in Configuration #1 (**Figures 4.15a**

and 4.15b) and atop the geofoam blocks in the other remaining configurations by pushing with a mini front-end loader (Figure 4.15c-i). Compaction was started after the first lift of granular base course was placed on the PCC slab and geofoam blocks with a thickness of at least 25 cm (Figure 4.15j). Therefore, a granular base course layer of approximately 20 - 25 cm thick was laid between the min front end loader and the geofoam blocks to avoid the direct contact between the tires and PCC slab of geofoam blocks (Figure 4.15c-i).



Figure 4.15: The first lift of granular base laying-compaction process, a-b) Placement of granular base atop the PCC slab in the Configuration #1 by pushing with a mini front-end loader, c-f) In-filling of geocell with granular base course at the Configuration #4, g-h) Laying of granular base at the Configuration #3, i) Laying of granular base at the Configuration #5, j) Compaction of loose granular base course material by a vibratory roller.

After the first layer of granular base course was laid and compacted (**Figure 4.15**), the second lift of the granular base course was laid at the project elevation with a roadway finisher. Before laying of the granular base course, the second row of geocells of the Configuration #4 (**Figure 4.16a** and **4.16b**) and the single row of geocells of the Configuration #3 were placed and granular base course was laid approximately 3-5 cm above the cell. Similarly, the second row of geogrid of the Configuration #5 was also placed and approximately 10 cm of granular base course was laid above it before the roadway finisher mobilized (**Figure 4.16c** and **4.16d**). Then, the second lift of the granular base course was laid with a roadway finisher in entire configurations according to the project elevation (**Figure 4.16d-g**) and compacted with a vibratory roller (**Figure 4.16h-j**).





Figure 4.16: Second lift of the granular base laying-compaction process, a-b) Second row geocell placement at Configuration #4 and granular base laying, c- d) Second row geogrid placement at Configuration #5 and granular base laying, e-g) Second layer granular base laying by finisher, h-j) Compaction of granular base layer.

Following the granular base laying-compaction process, bituminous hot mix asphalt (HMA) construction was started (**Figure 4.17**). The HMA was consisted of two layers: bituminous base and wearing course (**Figure 3.6**). The bituminous base was laid and compacted in two lifts with a roadway finisher. After the first layer was laid (**Figure 4.17a-c**), the asphalt was allowed to reach the required temperature for compaction (**Figure 4.17d**) and then compacted with a roller (**Figure 4.17e**). The second lift of bituminous base (**Figure 4.17f**) and wearing (**Figure 4.17g**) layers were laid and compacted and the HMA construction was completed (**Figure 4.17i**).



Figure 4.17: HMA laying-compaction process, a-c) Laying of first lift of the bituminous base, d) Temperature control prior to compaction, e) Compaction of first lift of the bituminous layer, f-g) Second lift of the bituminous base laying by a roadway finisher, h) Laying of wearing course layer, i) Completion of the HMA construction.

Before starting the accelerated pavement test (APT), reference boundary lines were painted on the HMA layer to ensure driving safety during the test. In addition, these boundary lines were intended to ensure that the wheels do not wander on the surface and follow the same direction during APT. Furthermore, the boundaries of each configuration were also marked on the HMA (**Figure 4.18a**). Core samples were then

taken from each configuration for determination of the thickness of the HMA. In addition, core samples were also used to check the compaction performance in the HMA layers (**Figure 4.18b-i**).

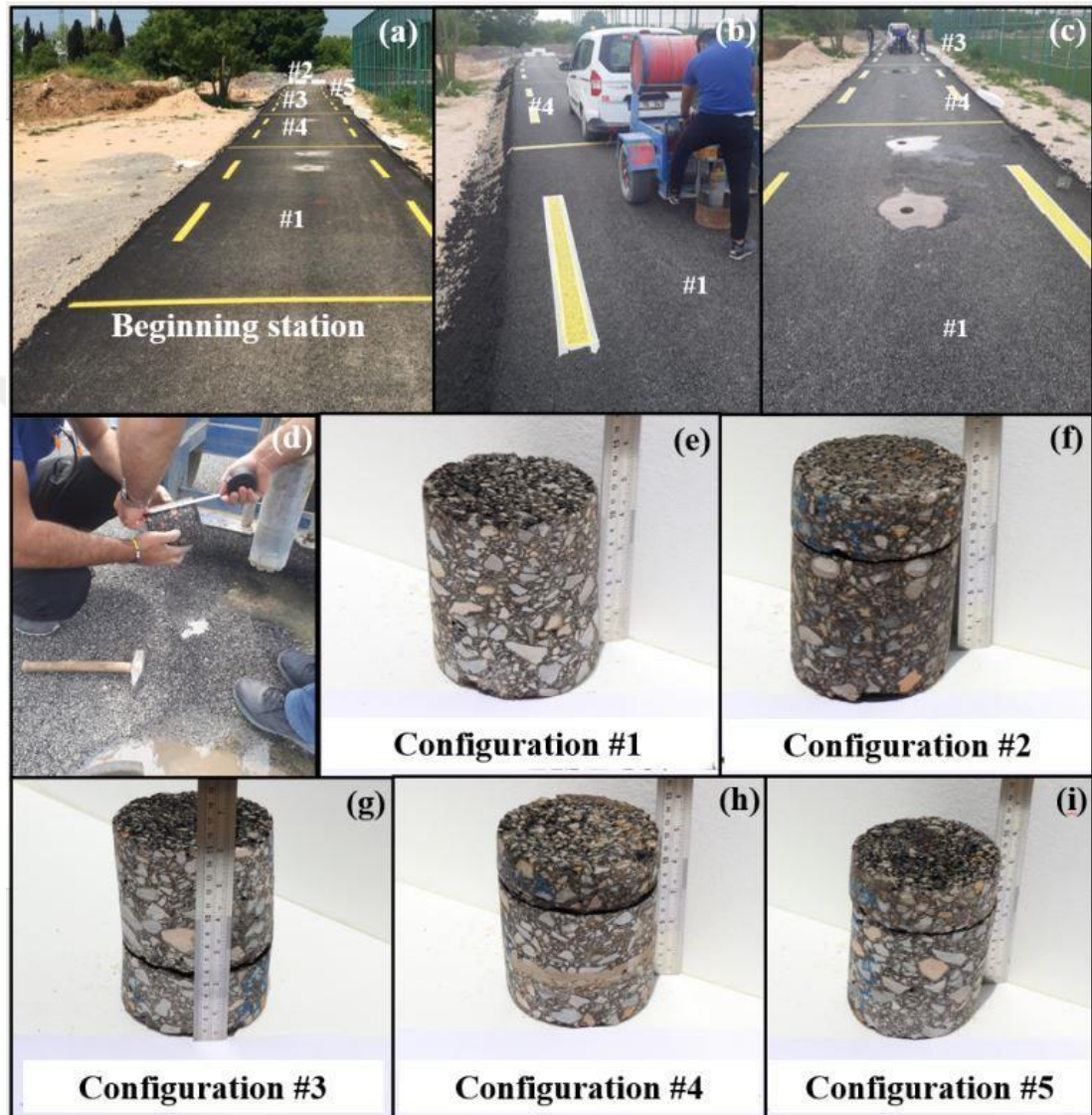


Figure 4.18: Painting of road boundary lines for APT and HMA core samples, a) Painting of road boundary lines, b-d) Collection of HMA core samples, e-i) Core samples.

Table 4.1 shows the constructed pavement thicknesses according to the core samples (**Figure 4.18 e-i**). The as-built sections of the completed pavement are given in **Figure 4.19**.

Table 4.1: As-built thicknesses of flexible pavement configurations.

Configuration no (East – West direction)	HMA thickness (cm)	Granular base thickness (cm)	PCC slab thickness (cm)
#1	13	38	10
#4	14	47	-
#3	15	46	-
#5	14.5	51,5	-
#2	15	65	-

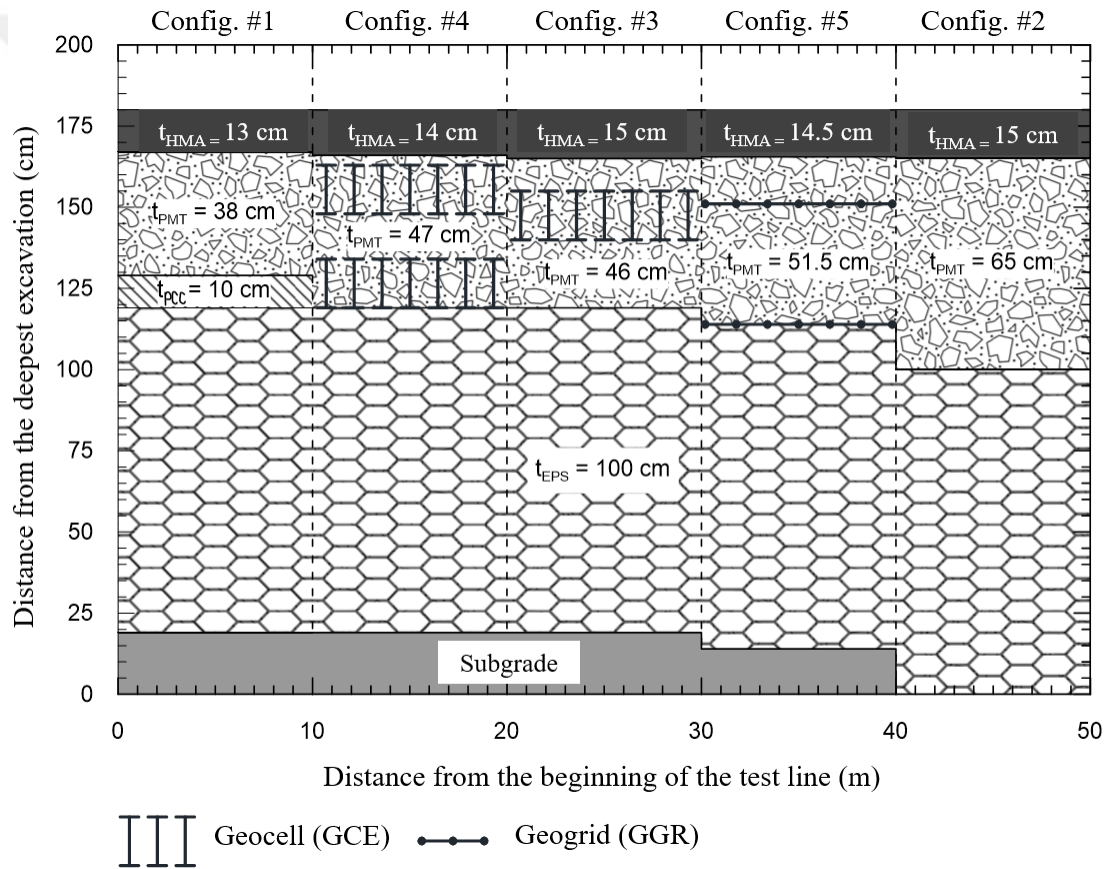


Figure 4.19: As-built section of EPS block embankment flexible pavements.

5. MATERIAL CHARACTERIZATION TESTS

5.1. Bedding Sand Classification Tests

The bedding sand (**Figure 4.6**) was obtained from a quarry located in Tepeören/Tuzla/Istanbul. Three different samples were taken from different deliveries for the classification and granulometric properties of the material. Grain size distribution and granulometric properties were given in **Figure 5.1** and **Table 5.1**, respectively.

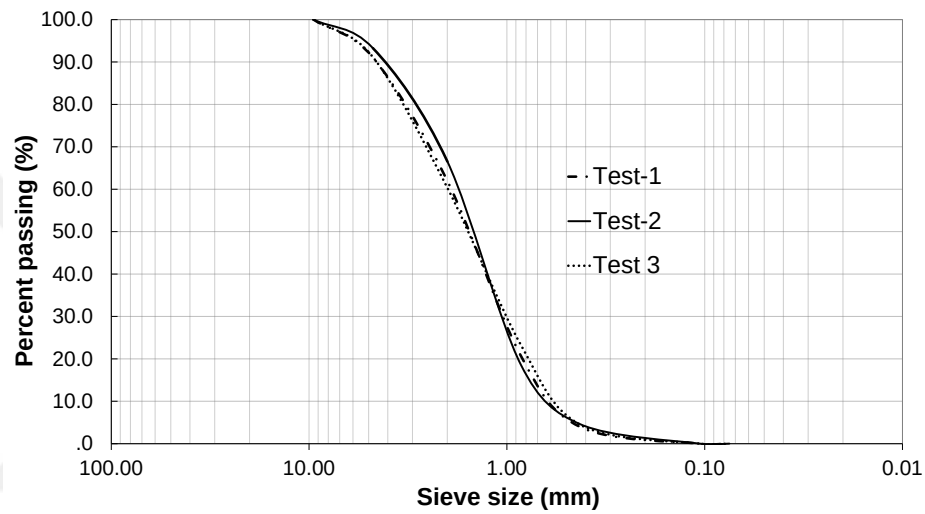


Figure 5.1: Grain size distribution of bedding sand.

Table 5.1: Granulometric properties of bedding sand.

Property	Description	Results			Range of results
		Test 1	Test 2	Test 3	
Soil classification	USCS	SP	SP	SP	-
Grain size distribution	Gravel (%)	9.00	6.80	8.90	6.80 – 9.00
	Sand (%)	91.00	93.20	91.10	91.91 – 93.20
Effective size	D_{10} (mm)	0.65	0.65	0.60	0.60 – 0.65
Average size	D_{50} (mm)	1.54	1.48	1.54	1.48 – 1.54
Uniformity coefficient	C_u (unitless)	3.05	2.70	3.30	2.70 – 3.30
Coefficient of gradation	C_c (unitless)	1.00	1.10	0.84	0.84 – 1.10

5.2. Physical and Mechanical Properties of Geofoam

Several test procedures are used to determine the physical and mechanical properties of geofoam blocks, and the ASTM standards specify both testing procedures and minimum product requirements [ASTM D2863, ASTM C165, ASTM C203, ASTM C303, ASTM D1621, ASTM D1622 ASTM C578, and ASTM D6817]. ASTM D6817 specifies the product requirements of EPS geofoam blocks with densities varying between 11.2 kg/m³ (EPS12) and 45.7 kg/m³ (EPS46) (**Table 2.2**) [ASTM D6817/D6817M, 2021].

EPS29 with a density of 30 kg/m³ was used for the flexible pavement sub-base (**Figure 3.6, Figures 4.10, 4.11 and 4.12**). Compressive strength tests were performed (**Figure 5.2**) according to the ASTM D1621 (2016) for two specimens taken from blocks delivered to the site at different times. The mechanical properties of EPS blocks were summarized in **Table 5.2**. In addition, the test results were compared with minimum requirements of ASTM D6817/D6817M (2021) for EPS29 block in **Table 5.2**.

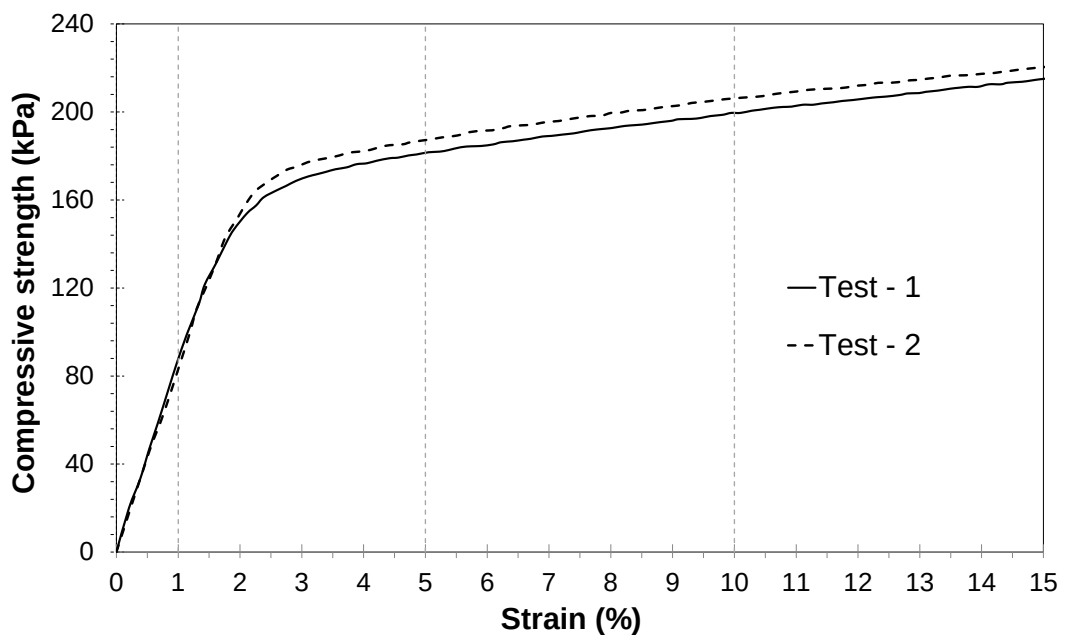


Figure 5.2: Stress – strain curves of EPS29 from this study.

Table 5.2: Geofoam blocks physical and mechanical properties from this study.

Properties	Definition	Test results of study blocks	ASTM D6817 definition	Minimum requirements of ASTM D6817
Density	γ (kg/m ³)	29.0 – 29.2	EPS29	28.8
Compressive Strength at 1% strain	σ_1 (kPa)	80.0 84.0		75.0
Compressive Strength at 5% strain	σ_5 (kPa)	181.0 – 187.0		170.0
Compressive Strength at 10% strain	σ_{10} (kPa)	200.0 – 206.0		200.0
Modulus of Elasticity	E_i (MPa)	8.0 – 8.4		-

5.3. Properties of PCC Load Distribution Slab

Precast PCC load distribution slabs were constructed in the reference section Configuration #1 (**Figure 4.12** and **4.13**), which was constructed to simulate a traditional geofoam block embankment flexible pavement. The PCC was designed with Portland cement 42,5-R. The PCC mix design was given in **Table 5.3**.

Table 5.3: PCC load distribution slab mix design.

Material	Amount (kg/m ³)
Cement – CEM I A 42,5 R	325
Water content	190
Aggregate (0-5 mm)	830
No 1 Aggregate (4-11.2 mm)	415
No 3 Aggregate (11.2-22.4 mm)	590
Chemical additive (Chryso Delta 2221)	3.9
Water / Cement ratio	0.58

At the end of the study, precast PCC load distribution slabs were removed from Configuration #1 (**Figure 5.3a**). Core samples were collected to quantify mechanical properties of PCC slab. Reinforcement detection device was used to select locations with no rebars to collect cores (**Figure 5.3b**). Then, core samples were collected (**Figure 5.3c-d**). Compressive strength tests were performed on the samples, and the test results were provided in **Table 5.4**.

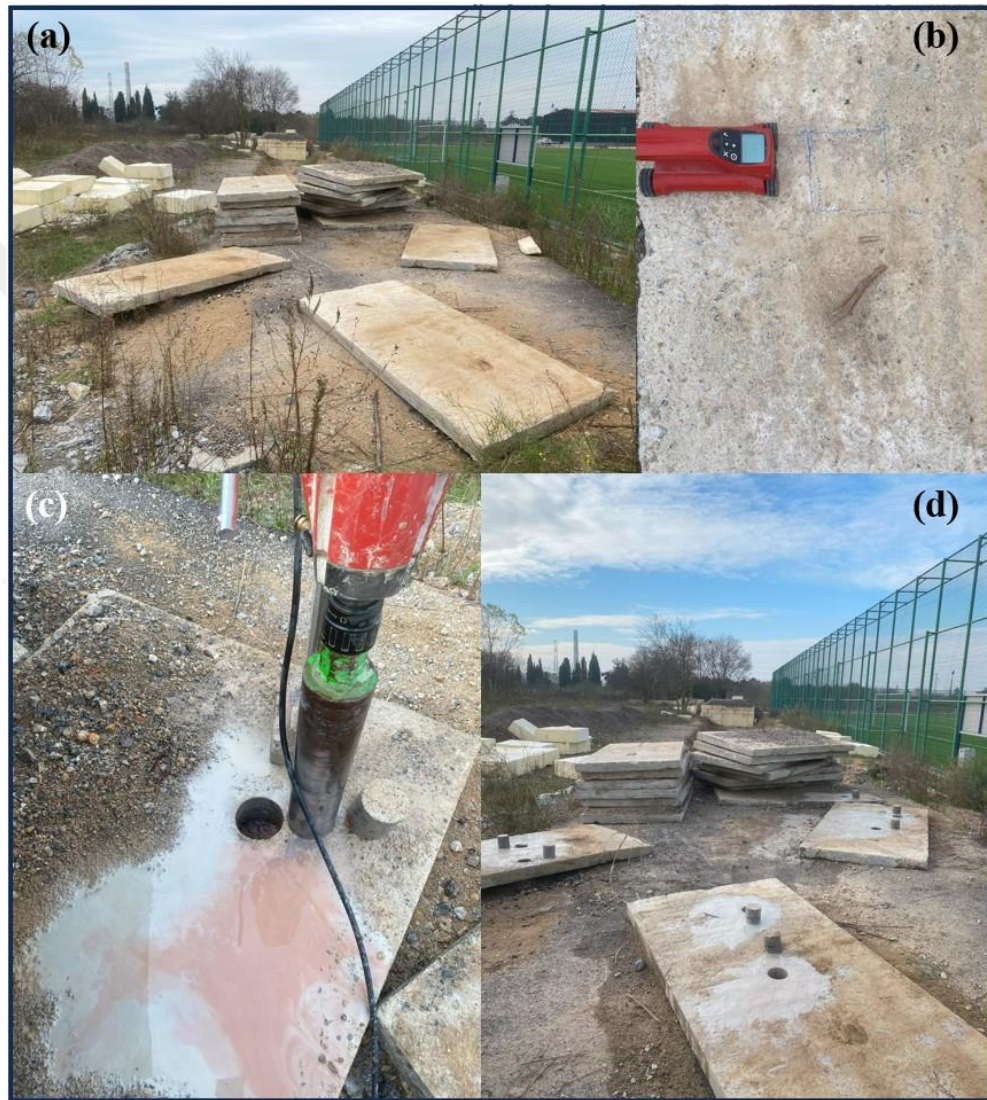


Figure 5.3: Core samples from PCC load distribution slab, a) Removed PCC slabs, b) Reinforcement pinpoint using a reinforcement detection device and marked, c) Coring process, d) Core samples.

Table 5.4: Compressive strength test results of precast PCC load distribution slabs.

Core sample no	Maximum load (kN)	Cylinder sample Dimension (mm)	Compressive strength* (N/mm ²)
1	141.04	84x84	20.87
2	130.79	84x84	19.35
3	126.76	84x84	18.76
4	129.90	84x84	19.22
5	130.69	84x84	19.34
6	134.03	84x84	19.83

*Since the core diameter/length ratio is 1, it is multiplied by a coefficient of CLF=0.82 [TS EN 13791].

5.4. Physical and Mechanical Properties of Geosynthetics

A non-woven geotextile was used for separation between the granular base and geofoam blocks in whole configurations (**Figure 3.6**). The physical and mechanical properties of the geotextile are presented in **Table 5.5** obtained from the manufacturer.

Table 5.5: Physical and mechanical properties of non-woven geotextile.

Property	Unit	Index test results
Weight	m ² /g	300
Thickness (under 2 kPa compressive stress)	mm	2.4
Tensile strength (Longitudinal/Transverse)	kN/m	14/17
Elongation at break (Longitudinal/Transverse)	%	Min. 50
Static puncture strength	N	2700
Apparent opening size	mm	0.077

The geocell was used for reinforcement purposes in Configuration #3 and #4 (**Figure 3.6** and **4.14**) and physical and mechanical properties were obtained from manufacturer (**Table 5.6**).

Table 5.6: Physical and mechanical properties of Geocell.

Property	Unit	Index test results
Cell internal dimensions	mm x mm	200 x 260
Cell height	mm	150
Average tensile strength	kN/m	8.3
Average maximum elongation	%	34.5

The bi-axial geogrid was used two rows for reinforcement purposes in configuration #5 (**Figure 3.6** and **4.14**) and physical and mechanical properties were obtained from manufacturer (**Table 5.7**).

Table 5.7: Physical and mechanical properties of bi-axial geogrid.

Property	Unit	Index test results
Max tensile strength (MD)	kN/m	>80
Max tensile strength (CMD)	kN/m	>80
Tensile strength at 2% strain	kN/m	>14
Tensile strength at 5% strain	kN/m	>29
Elongation at break (MD)	%	10 (±2)
Elongation at break (CMD)	%	10 (±2)
Apparent opening size (MD)	mm	30 (±3)
Apparent opening size (CMD)	mm	26 (±3)

5.5. Physical and Compaction Properties of Unbounded Aggregate

PMT Type-1 should be used if the PMT layer thickness is 15 cm or more [KGM, 2023]. The grain size distribution of the granular base layer, which conforms the requirements of the Type-1, is given in **Figure 5.4** in comparison with limits of KGM (2023).

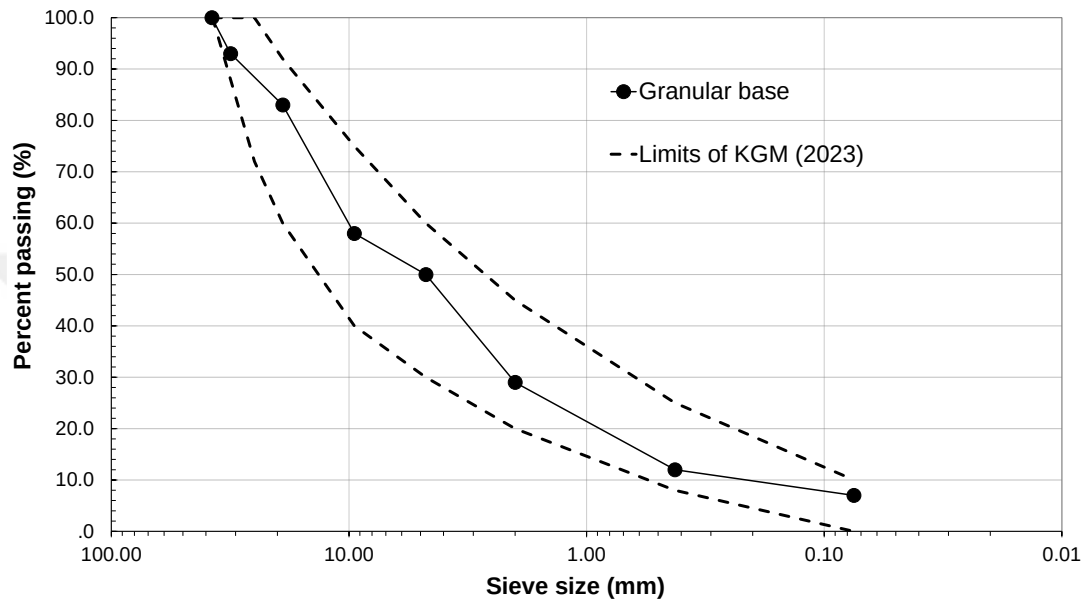


Figure 5.4: Comparison between the grain size distribution curves of granular base and limits of the KGM (2023).

Standard and modified Proctor tests also were performed on the granular base to evaluate the compaction performance. The test results were given in **Figure 5.5**. In addition, abrasion resistance and water absorption tests were carried out on samples by the PMT quarry. The test results were given in **Table 5.8**.

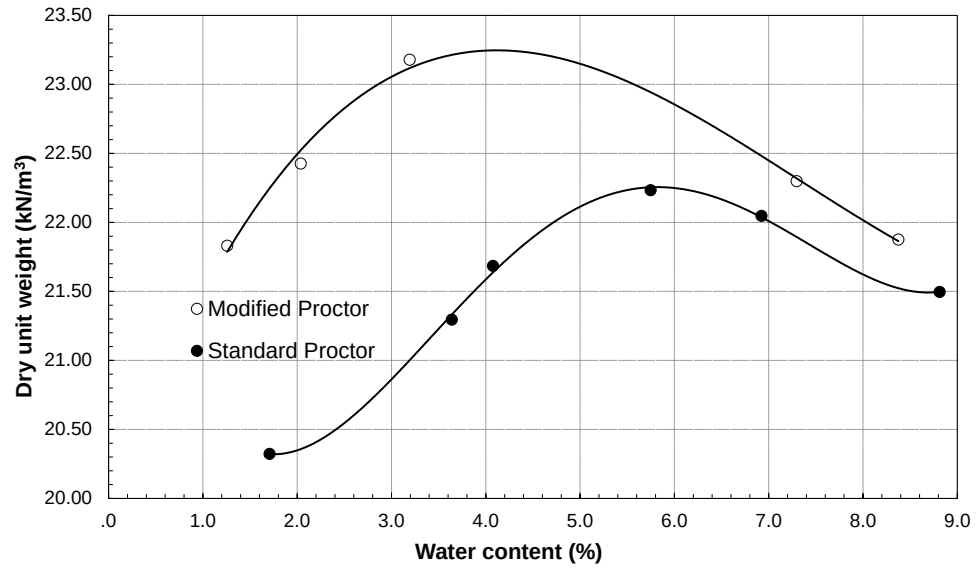


Figure 5.5: Water content versus dry unit weight curves for granular base.

Table 5.8: Compaction, abrasion and water absorption properties of granular base.

Property	Description & Unit	Result
Grain size distribution	Gravel; Sand; Clay (%)	50; 43; 7
Effective size	D ₁₀ (mm)	0.30
Average size	D ₅₀ (mm)	4.75
Uniformity coefficient	C _u (unitless)	35.00
Coefficient of gradation	C _c (unitless)	1.30
Water absorption	%	0.45 ≤ 3 KGM (2023) limit
Abrasion resistance (Los Angeles)	%	19 ≤ 35 KGM (2023) limit
Standard Proctor max dry unit weight	kN/m ³	22.30
Standard Proctor optimum water content	%	5.80
Modified Proctor max dry unit weight	kN/m ³	23.20
Modified Proctor optimum water content	%	4.00

5.6. California Bearing Ratio of Unbounded Aggregate

CBR test was performed according to the ASTM D1883 (2021). The penetration results and CBR curve were given in **Table 5.9** and **Figure 5.6**, respectively. The CBR specimen has a dry unit weight of 22.6 kN/m^3 which corresponds to the relative compaction of 97.4% of the modified proctor density (**Table 5.8**).

Table 5.9: CBR test results for granular base aggregate.

Penetration (mm)	Standard pressure (kg/m ²)	Applied load (kg)	Applied pressure (kg/m ²)	CBR (%)
0.625	-	558.3	28.9	-
1.250	-	906.6	46.9	-
1.825	-	1454.4	75.2	-
2.500	70.0	1799.5	92.9	132.9
3.750	-	2629.1	135.9	-
5.000	105.0	3295.5	170.3	162.2
7.500	-	4296.4	222.0	-
10.000	-	5098.5	263.5	-
12.500	-	5098.5	263.5	-

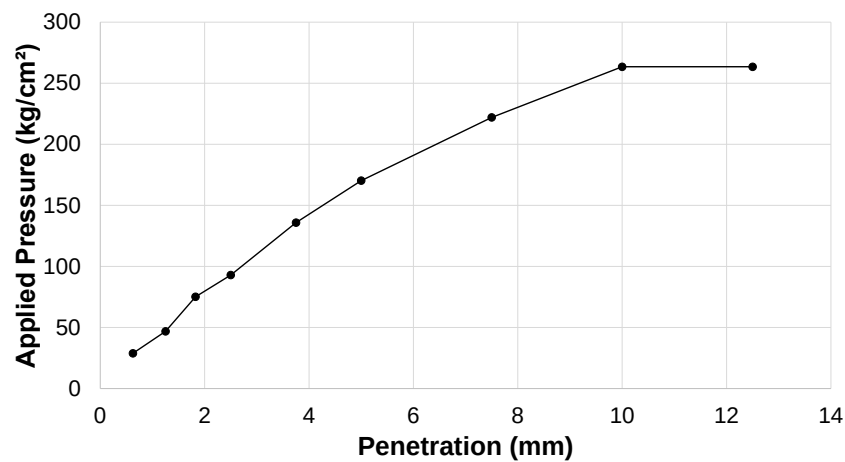


Figure 5.6: Applied pressure versus penetration curve of CBR test for granular base layer aggregate.

5.7. HMA Characterization Tests

The HMA layer of the flexible pavements consists of bituminous base and wearing course layers. The granulometry of the aggregates used in the bituminous base and wearing course layers are given in **Figure 5.6** and **Figure 5.7**, respectively, and compared with the limits of the KGM (2023).

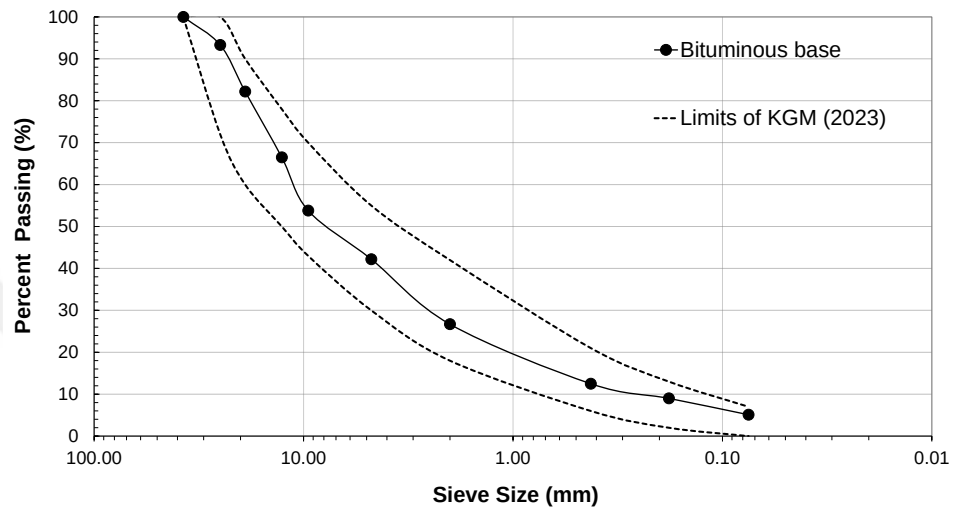


Figure 5.7: Comparison between the grain size distribution curves of bituminous base and KGM (2023).

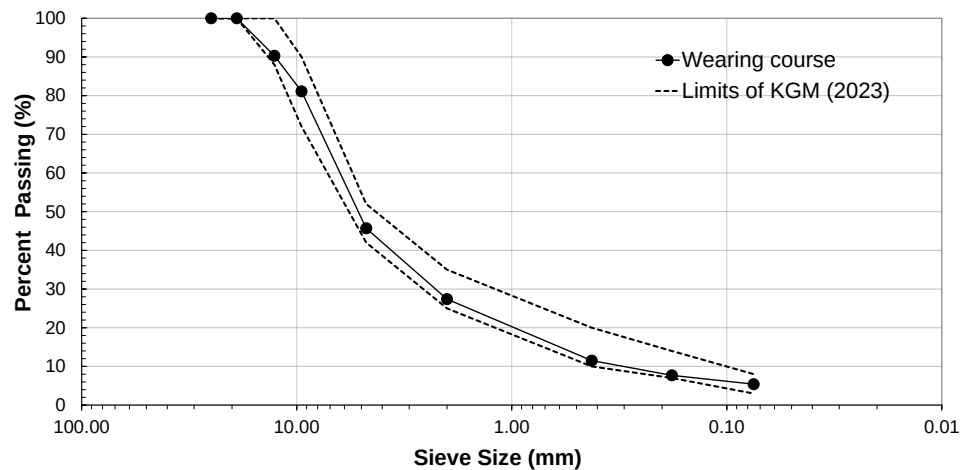


Figure 5.8: Comparison between the grain size distribution curves of wearing course and KGM (2023).

In addition, the bitumen ratio, void ratio, inter-aggregate voids, voids filled with asphalt, Marshall stability and flow of the bituminous base and wearing layers were depicted in **Table 5.9** with limits of KGM (2023).

Table 5.10: Properties of bituminous base and wearing course in comparison with KGM (2023) limits.

Property	Unit	Bituminous base		Wearing course	
		In this study	Limit of KGM (2023)	In this study	Limit of KGM (2023)
Bitumen ratio	%	3.70	3 – 5.5	4.86	4 – 7
Air void ratio	%	5.77	4 – 6	4.88	3 – 5
Inter-aggregate void ratio	%	13.46	12 – 14.5	15.23	14 – 16
Voids filled with asphalt	%	57.13	55 – 75	67.98	65 – 75
Marshall stability	kg	1144	Min 600	1188.5	Min 900
Flow	mm	3.60	2 - 5	3.27	2 - 4

After completion of the flexible pavement construction, core samples were taken from each configuration (**Figure 4.18**). Bituminous base and wearing course layers of the core samples were separated by cutting and air void ratio tests were performed. The test results were provided in the **Table 5.10** with the design criteria of the KGM (2023).

The bituminous base compaction performance in all configurations is close to each other and the average air void percentage is below the upper limit specified by KGM (2023) (**Table 5.10**).

Table 5.11: Air void ratio test from HMA core samples.

Configuration no (East – West direction)	Bituminous base Air void ratio (%)	Wearing course Air void ratio (%)
#1	6.76	8.91
#4	7.25	9.01
#3	6.30	7.06
#5	6.40	6.13
#2	6.07	7.94
Average	6.56	7.81
Standard deviation	0.46	1.23
Limits of KGM (2023)	Average Max 7.5%	Average Max 6.0% For Thin layers: Average Max 9.0%

6. FIELD TESTS AND MEASUREMENTS

Field tests were conducted to investigate the compaction performance and design performance of geofoam block embankment flexible pavement systems. While sand cone (SC) tests were carried out to investigate the compaction performance, static plate load (PLT) and falling weight deflectometer (FWD) tests were performed to evaluate the design performance. SC tests were conducted after the completion of the granular base in Configurations #1 and #2 (**Figure 6.1**). In addition, upon completion of the APT testing program, HMA layer was removed in 1 m x 1m area in each configuration and SC tests were performed at various depths to quantify compaction performance of the granular base (**Figure 6.1**). The SC and PLT test plan are shown in **Figure 6.1**. Furthermore, an APT test was performed with an actual truck and truck speed, number of passes were recorded with pneumatic pressure hoses. FWD tests were performed both prior and after APT.

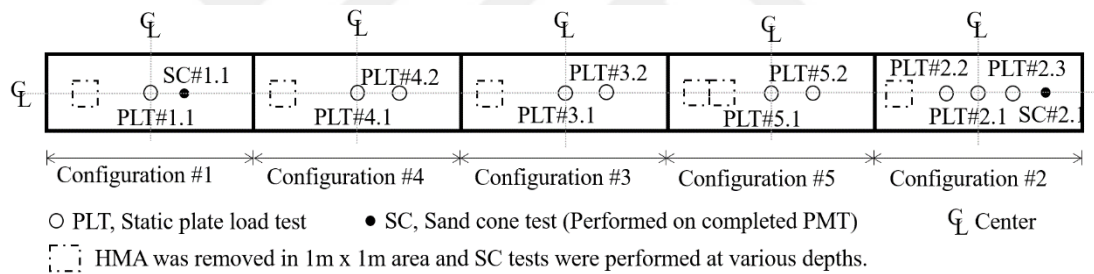


Figure 6.1: PLT and SC tests locations.

6.1. Sand Cone Tests

In order to investigate the compaction performance of the granular base, SC tests were carried out atop compacted granular base in Configuration #1 and Configuration #2 prior to HMA layer placement (SC#1.1 and SC#2.1 in **Figure 6.1** and **Figure 6.2a-b**). In addition, to determine the variation of the compaction performance along the granular base profile in each configuration, the HMA layer was removed upon completion of the APT program (**Figure 6.2 e-f**). In order to ensure that the granular base surface was not affected by the tire pressure loading of the APT, approximately 1 x 1 m area of the HMA was removed between the wheel paths (**Figure 6.2c**).

SC tests were conducted at different depths in the Configuration #1 (**Figure 6.2c**), Configuration #5 (**Figure 6.2d**) and Configuration #2 (**Figure 6.2e**). In the Configuration #4, the granular base over the geocell, which was approximately 3 to 5 cm thick, was carefully removed to reach the second layer of geocells. Since the cells of the geocell do not allow for traditional sand cone test, SC was performed in between the first and the second layer of geocell. For this purpose, the cells in the first row were removed and the test was performed at the bottom elevation of the first row geocell. The Configuration #3 was consisted single row geocell (**Figure 3.6**), therefore, as in the Configuration #4, in the Configuration #3, geocell was removed and a SC was conducted to determine the compaction performance of the granular base underneath geocell (**Figure 6.2f**).



Figure 6.2: Sand cone tests, a-b) Prior to HMA construction Configuration #1 and #2, respectively, c-e) SC tests performed at different depths in Configuration #1, #5 and #2 upon completion of APT, f) SC tests performed beneath geocell at the Configuration #3.

Additionally, a SC test was conducted to determine the unit weight of the granular base layer overlying the first layer of geocells in Configuration #3. A total of 14 tests were performed for entire configurations and the results are summarized in the **Table 6.1**.

Table 6.1: Relative compaction values based on sand cone tests according to standard and modified proctor test results.

Configuration no	Test no	Test date	Depth from top elevation of granular base (cm)	Sand cone test		Relative compaction $R = \gamma_{d,field} / \gamma_{d,max,lab}$	
				Dry unit weight of volume γ_d (kg/m ³)	Water content w (%)	Acc. to Standard Proctor (%)	Acc. to Modified Proctor (%)
#1	1.1	27.04.2022	0	20.60	2.80	92.3	88.8
	1.2	10.08.2022	5	18.10	3.30	81.1	78.0
	1.3	10.08.2022	14	16.50	3.30	74.0	71.1
	1.4	17.08.2022	27	16.00	5.40	71.7	69.0
#4	4.1	01.09.2022	19	16.70	4.30	74.9	72.0
#3	3.1	23.09.2022	0	16.70	2.90	74.9	72.0
	3.2	06.09.2022	28	16.10	5.10	72.2	69.4
#5	5.1	11.09.2022	0	19.60	3.10	87.9	84.5
	5.2	12.08.2022	14	19.10	2.30	85.7	82.3
	5.3	11.09.2022	29	16.30	4.30	73.1	70.3
#2	2.1	27.04.2022	0	19.70	3.00	88.3	84.9
	2.2	16.08.2022	0	20.40	4.20	91.5	87.9
	2.3	16.08.2022	15	18.20	3.00	81.6	78.4
	2.4	16.08.2022	28	16.10	3.50	72.2	69.4

The relative compaction (R) values given in **Table 6.1** were summarized based on standard (**Figure 6.3a**) and modified (**Figure 6.3b**) proctor results in the longitudinal profile to show the compaction efficiency (**Figure 6.3**).

During compaction processes, approximately 20 - 25 cm thick granular base material was loosely laid and compacted with a vibratory roller (**Figures 4.15 and 4.16**). In all configurations, the number of roller passes applied in each lift was similar. In the Configurations #1, #5 and #2, the relative compaction value (R) based on Standard Proctor, increased gradually from 70% atop the EPS surface to approximately 86% - 92% at the granular base surface (**Table 6.1 and Figure 6.3a**). Similarly, R value based on the Modified Proctor started from 69% near the top of the geofoam block assemblage and gradually increased to approximately 82% - 89% at the surface elevation of the granular base layer (**Figure 6.3b**).

In the geocell reinforced configurations (Configurations #4 and #3), unlike the other configurations, no gradual increase in R value was observed from the top elevation of geofoam block assemblage to the surface elevation of the granular base (**Figure 6.3**). In these configurations, R values were ranged from 72.2% to 74.4% and from 69.9% to 72% based on the standard and modified Proctor, respectively (**Figure 6.3**).

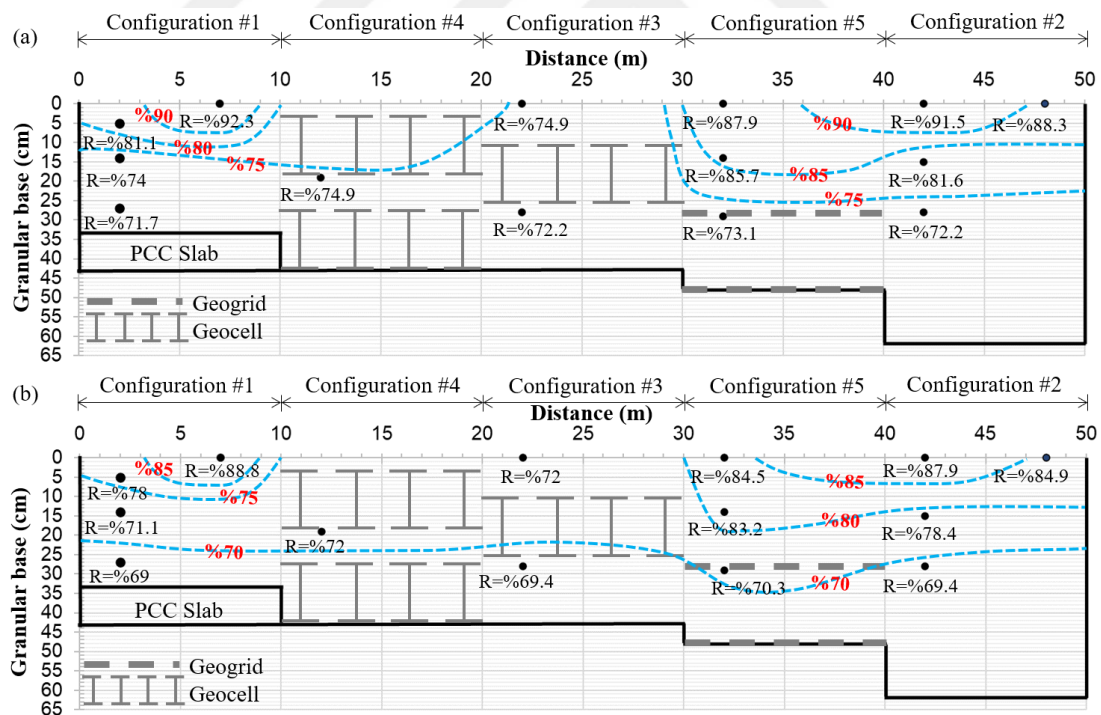


Figure 6.3: Evaluation of compaction performance, a) Variation of R values in longitudinal profile based on Standard Proctor, b) Variation of R values in longitudinal profile based on Modified Proctor.

Although equal energy (similar number of passes) was applied at each lift and the water content of the compacted soil was in the range of $\pm 1\%$ of the Modified Proctor optimum water content, the R of the first lift atop the surface of the geofoam blocks was 71% and 69% of the Standard and Modified Proctor value, respectively. This is associated with the energy absorbing nature of geofoam blocks. During the application of compaction energy, the geofoam blocks absorb the vibration, therefore, prevented to form a dense granular base layer close to the geofoam block surface. The R value gradually increased from the first lift atop the geofoam blocks to the surface elevation of the granular base in the Configuration #1, #5 and #2. In other words, while thickness of the compacted granular base material atop the geofoam blocks increased the relative compaction in the Configurations #1, #5 and #2 were increased from very loose-loose to dense at the surface of the granular base (**Figure 6.3**).

Similar behavior was also observed by Tafreshi et al. (2021) during the placement of various thicknesses of well-graded sand atop geofoam in the lab. They used a 2.2 m wide x 2.2 m long x 1.4 m high box. The sand was laid in 10 cm thick lifts, and each lift was compacted in a total of 5 passes with a vibratory roller. Total height of the sand column atop the EPS was varied between 30 – 60 cm. The distribution of the R value was ranging from dense to very dense from the layers near the EPS surface to the surface elevation [Tafreshi et al., 2021]. The difference in relative compaction values observed at the lifts near geofoam blocks at laboratory and field scale can be related with using different material (sand was used in laboratory, PMT was used in field scale) and the thickness of the lift was significantly different. However, both the laboratory-scale [Tafreshi et al. (2021)] and the field-scale compaction efforts showed that geofoam blocks absorb the vibration during compaction, which prevents the compaction quality of the first lift atop the EPS.

In the geocell reinforced configurations (Configurations #4 and #3), similar relative compaction values were obtained from the first lift atop geofoam blocks to the surface elevation of the granular base (**Table 6.1** and **Figure 6.3**). The presence of a geocell is a factor that challenges the compaction effort [Pokharel (2010)] and the dry unit weight of the compacted material inside the cell is an important factor affecting geocell performance [Pokharel, 2010; Thakur et al., 2012]. Although equal energy was applied at each compaction stages, lower relative compaction values were obtained in the geocell reinforced configurations compared to that of other configurations. A similar

behavior was observed for flexible pavements with and without geocell reinforcement flexible pavement constructed on high plasticity clay (CH) soil with CBR values ranging from 2.4% to 5.6% [Zipoli and Avesani Neto, 2022]. It was due to the stiffness of the geocell reinforcement walls, high lateral stresses are required to mobilize the confinement effect and therefore higher compaction energy (higher number of passes) must be applied to achieve relative compaction values close to obtained in unreinforced granular foundation layers during compaction in the field [Zipoli and Avesani Neto, 2022]. This confinement effect in geocell reinforcement is called "closed-confinement" due to the cellular structure of the geocell [Giroud et al., 2021].

A similar behavior was also observed by Pei and Yang (2018) for the geogrid-reinforced granular base layers. Although the granular base in contact with the geogrid is completely confined, the confinement of the granular base decreases as it moves away from the geogrid and this type of confinement effect is called "open-confinement" [Giroud et al., 2021]. It was indicated that the number of roller passes should be increased in order to interlock the geogrid reinforcement with the granular base and thus increase the confining effect [Pei and Yang, 2018].

6.2. Plate Load Tests

Static PLT were carried out after the compaction of granular base was completed to analyze load-settlement (stress-settlement) behavior of each configuration. The PLT were performed according to DIN-18134 (2012) standards (**Figure 6.4**) at the locations in **Figure 6.1**. The strain modulus (E_v) and modulus of subgrade (k_s) were quantified. The test results were also evaluated for quality control of the compaction effort.



Figure 6.4: PLT, a) A truck loaded to its capacity pulls up to the test location in the Configuration #2, b) Installation of PLT setup, c) Performing the PLT according to the defined loading and unloading protocol.

In the tests, two loading cycles were applied to a plate with a diameter of 30 cm and a height of 2.5 cm by a hydraulic jack. The rear axle of a truck loaded its capacity was used as a reaction beam for the hydraulic jack to transfer the loads to the granular base (**Figure 6.4b**). In the first loading-unloading cycle, vertical loads of 1 kN, 2 kN, 4 kN, 6 kN, 8 kN, 10 kN, 12 kN, 14 kN, 16 kN, 18 kN, 20 kN, 16 kN, 12 kN, 8 kN, 4 kN and 0 kN were applied and deformations were measured. After this cycle, the second loading stage consisting of 4 kN, 8 kN, 12 kN and 16 kN was applied. The vertical displacements at each loading stage were read by three digital displacement dial gauges placed at three different points on the 30 cm diameter plate. The average of the values obtained from these three displacement dial gauges were reported as the displacement corresponding to that loading stage. The initial load of 1 kN in the first loading stage corresponds to the approximate preload as defined in DIN 18134 (2012). In order to determine the stress-settlement curves, the displacement value corresponding to this initial loading level was defined as the reading corresponding to zero pressure [DIN 18134, 2012]. The typical stress-strain curve obtained from PLT is given in **Figure 6.5** and the calculation of modulus of elasticity and subgrade reaction modulus obtained from the test results are given in **Table 6.2**. All PLT results are presented in the **Appendix-A** and the results are summarized in **Table 6.3**.

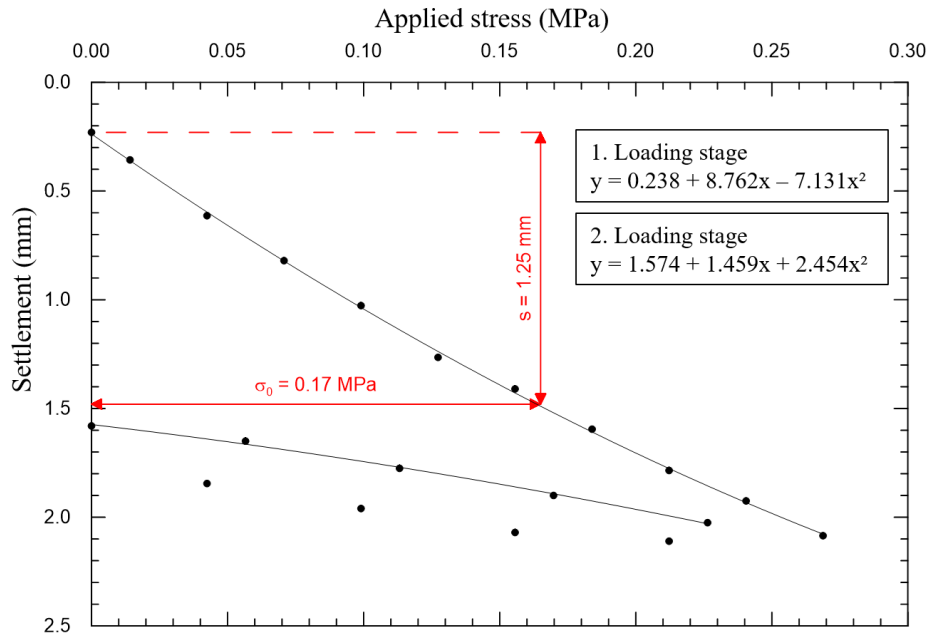


Figure 6.5: The PLT#1.1 (Figure 6.1) stress-settlement curves of Configuration #1.

Table 6.2: The calculations of modulus of elasticity and subgrade modulus for PLT#1.1 of Configuration #1 from stress-settlement curve of **Figure 6.5**.

Parameter	Unit	1. Loading	2. Loading
σ_{0max}	MPa		0.27
a_0	mm	0.238	1.574
a_1	mm/MN/m ²	8.762	1.459
a_2	mm/MN ² /m ⁴	-7.131	2.454
$E_v = 1.5r \frac{1}{a_1 + a_2 \sigma_{0max}}$	MPa	32.9	106.2
E_{v2}/E_{v1}	-		3.23
$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125m}$	MN/m ³		132,0

Table 6.3: PLT results.

Test no (Figure 6.1)	Configuration no (East – West direction)	E _{v1} (MPa)	E _{v2} (MPa)	E _{v2} / E _{v1} (Unitless)	k _s (MN/m ³)
PLT#1.1	Configuration #1	32.9	106.2	3.23	132,0
PLT#4.1	Configuration #4	11.7	37.9	3.22	26.1
PLT#4.2		13.8	44.8	3.25	33.4
PLT#3.2	Configuration #3	15.2	47.7	3.13	38.8
PLT#5.1	Configuration #5	20.6	67.0	3.25	70.5
PLT#5.1		26.2	76.4	2.92	102.5
PLT#2.2	Configuration #2	34.5	113.5	3.29	128.6

The deformation modulus and subgrade modulus' along the test bed alignment were given in **Figure 6.6** and **Figure 6.7** for Configurations #1, #4, #3, #5 and #2 from east to west direction, respectively.

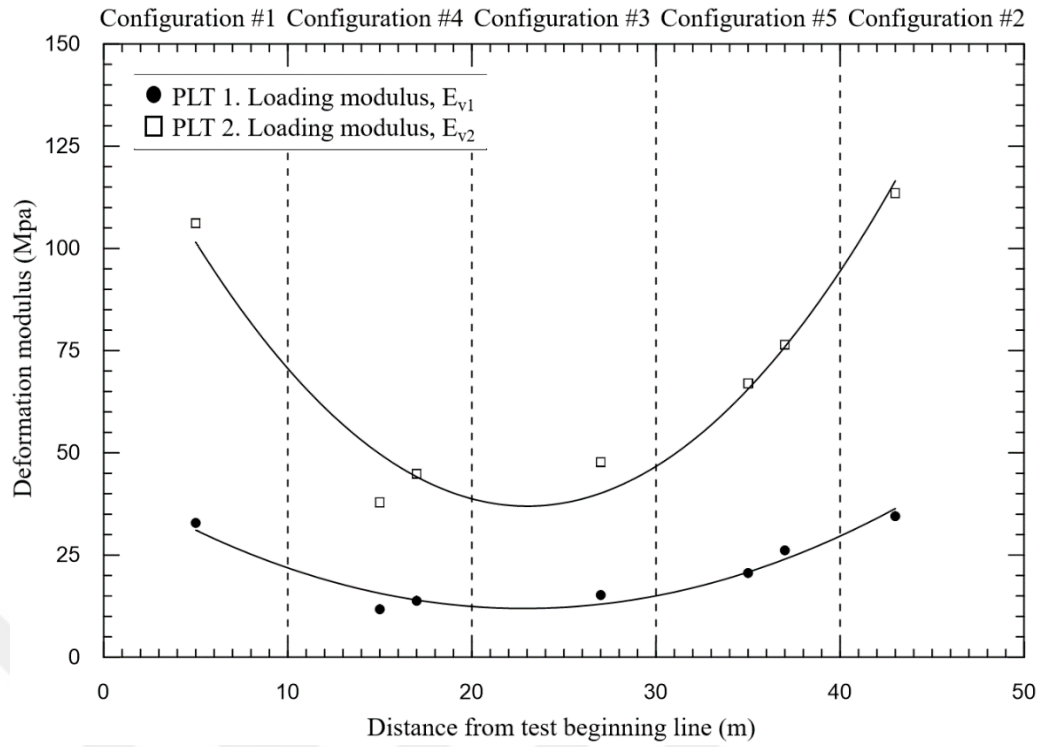


Figure 6.6: Deformation modulus (E_{v1} and E_{v2}) along the test bed.

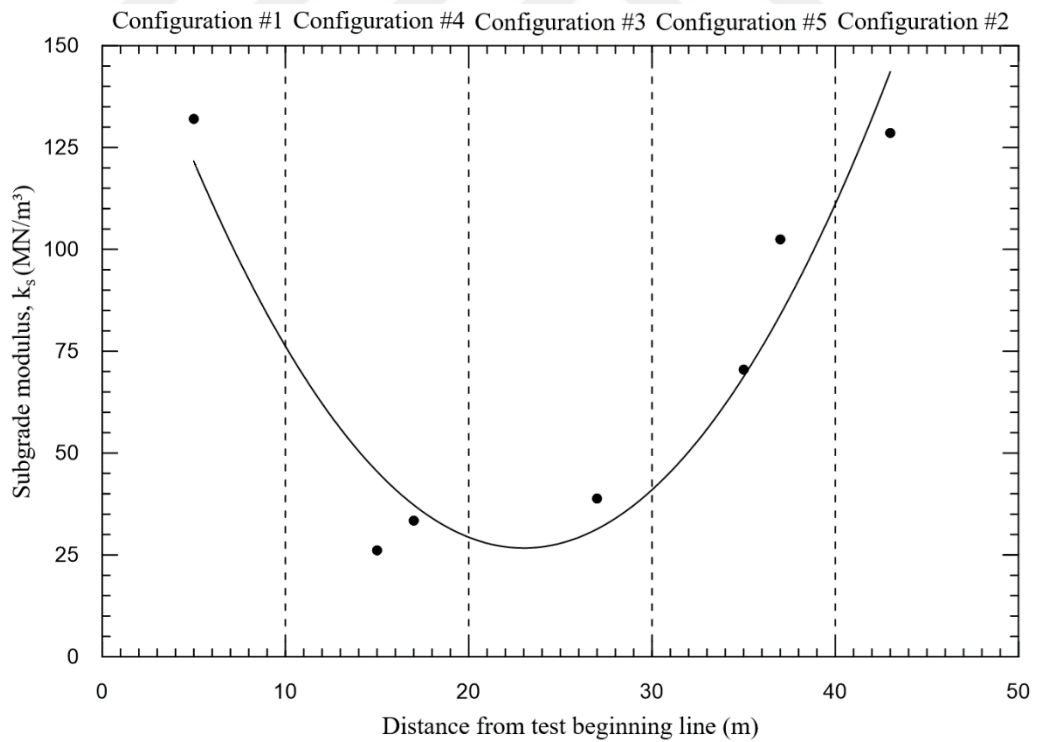


Figure 6.7: Subgrade modulus (k_s) along the test bed.

It was clearly seen that the deformation modulus and subgrade modulus obtained for the granular base are a function of the granular base thickness (PMT stiffness) (**Table**

6.3, Figure 6.6 and Figure 6.7). The highest deformation moduli were obtained in the Configuration #2 (65 cm thick granular base foundation, **Figure 4.18**) that was the highest granular base thickness used in the study.

Configuration #1, which was constructed to represent traditional geofoam embankment flexible pavement configuration, composed of a 10 cm thick PCC load distribution slab, 38 cm thick granular base and 13 cm thick HMA layer (**Table 4.1, Figure 4.18**).

The geofoam block highway embankment projects constructed in between 1975 and 1985 were indicated that the stiffness of a 10 cm thick PCC load distribution slab is equivalent to the stiffness of a layer of granular base material about 30 cm thick [Aab  , 1987; Refsdal, 1987]. In this point of view, it can be said that the total stiffness of 10 cm thick PCC slab and 38 cm thick granular base used in the flexible pavement of Configuration #1 is approximately equivalent to the stiffness of 68 cm thick granular base layer. The thickness of this granular base is close to the thickness constructed in Configuration #2. These stiffness equivalency between Configuration #1 and Configuration #2 are also seen from the PLT results (**Table 6.3, Figure 6.6 and Figure 6.7**).

Geocell reinforced Configuration #3 and Configuration #4 were designed using the results of Emersleben and Meyer (2008a; 2008b; 2008c). Emersleben and Meyer (2008a; 2008b; 2008c) reported that a 40 cm thick granular base reinforced with a single row of 20 cm thick geocells had almost equivalent modulus of elasticity and deformation behavior to a 70 cm thick unreinforced granular base.

In the design of Configuration #3, the effect of geocell reinforcement was not taken into account and layer thicknesses were calculated using the layer coefficients ($a_1 = 0.44$ for HMA and $a_2 = 0.14$ for granular base) given in the Turkish Highways Flexible Pavement Design Guide [KGM, 2008] for the design ESAL value. The thickness of the flexible pavement designed with this approach is 43 cm (**Figure 3.6**) and it was found that it was constructed as 46 cm in the in-situ (**Figure 4.18**).

On the other hand, the Configuration #4 was constructed with two rows of geocells to investigate the effect of the number of geocell reinforcements on the performance of the flexible pavement. In the completed construction, it was found that Configuration #4 was constructed with a 47 cm thick granular base (**Figure 4.18**).

Among the flexible pavements tested, the lowest deformation modulus and subgrade modulus were obtained in Configurations #3 and #4 (**Table 6.3, Figure 6.6 and Figure 6.7**). This can be explained by the inability to mobilize the beam effect of the geocell due to insufficient compaction caused by the geofoam absorbing the compaction performance.

When designing the Configuration #5, which has geogrid reinforcements, the section presented by Abu-Farsakh et al. (2019) was used as a reference (**Figure 3.6**). Abu-Farsakh et al. (2019) reported that geogrid performed better when placed at the sub-base level and at the upper 1/3 level of the unbonded granular base layer and TBR can be used as 2. Based on this cross-section, it was calculated that the pavement section consisting of 48 cm thick unreinforced granular base and 18 cm thick HMA could have equivalent stiffness with the reference configurations (Configurations #1 and #2) when the layer coefficients were taken as $a_1 = 0.44$ for HMA and $a_2 = 0.14$ for granular base according to KGM (2008). The thickness of the flexible pavement designed with this approach was 48 cm thick (**Figure 3.6**) and it was constructed as 51.5 cm thick in the prototype construction (**Figure 4.18**). The deformation modulus and subgrade modulus obtained in Configuration #5 are larger than Configurations #3 and #4, however, smaller than in the reference configurations (Configuration #1 and #2) (**Table 6.3, Figure 6.6 and Figure 6.7**). Although Configuration #5 was designed with equivalent stiffness of the reference configurations, it has clearly seen that the traffic benefit ratio was obtained less than 2, as recommended by Abu-Farsakh et al. (2019), for geogrid reinforced pavement configuration of geofoam block road embankments. While this TBR is ideal for a well-compacted granular base, it is found to be not suitable for geofoam embankments due to the fact that geofoam blocks absorb the vibration energy of roller during the compaction that prevents the efficient compaction of the first lift of atop geofoam blocks (**Figure 6.3**). Therefore, the well-known geogrid membrane effect was not fully mobilized since sufficient compaction performance was not achieved.

The E_{v2}/E_{v1} ratio represents the degree of compaction of the granular layer under the area of influence of a PLT [Wersäll et al., 2018; Lehman et al., 2020; Virsis et al., 2021]. The E_{v2}/E_{v1} ratio has been reported to range from 1 to 6 [Minažek and Mulabdic, 2014; Wersäll et al., 2018; Virsis et al., 2021]. The E_{v2}/E_{v1} ratio were provided in **Figure 6.8** for Configurations #1, #4, #3, #5 and #2 from east to west direction,

respectively. The E_{v2}/E_{v1} ratio ranged from 2.92 to 3.29 (**Table 6.3, Figure 6.8**), with an average of 3.19, homogeneously distributed among configurations (**Figure 6.9**) which indicated and similar compaction energy has been applied through the test bed. The German standard defines the allowable E_{v2}/E_{v1} ratio as a function of the relative compaction [FGSV, 2009]. The E_{v2}/E_{v1} ratio should be equal or less than 2.5 for $R \geq 98\%$ [FGSV, 2009]. The average value of the E_{v2}/E_{v1} ratio of 3.19 (**Figure 6.8**) for an average R value of approximately 80% which was measured based on SC tests (**Figure 6.3**), is expected considering the R versus E_{v2}/E_{v1} ratio provided in FGSV (2009). The linear relationship between deformation modulus was shown in **Figure 6.9**.

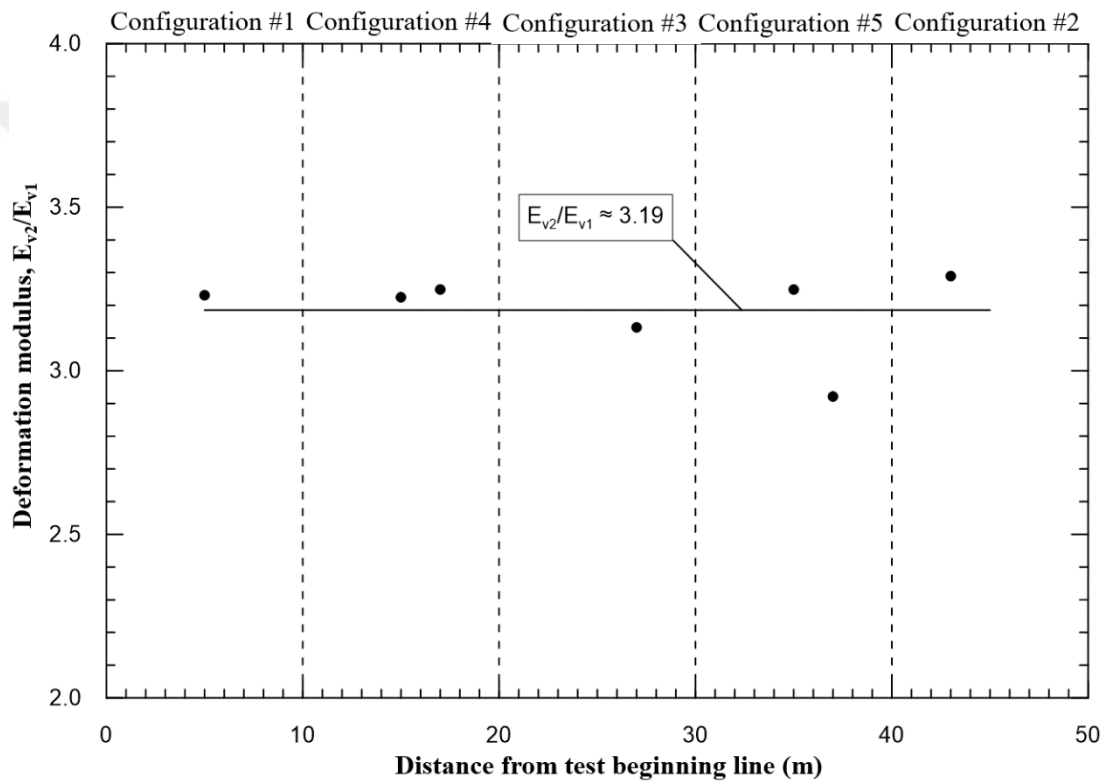


Figure 6.8: E_{v2}/E_{v1} ratio of PLT through the test bed.

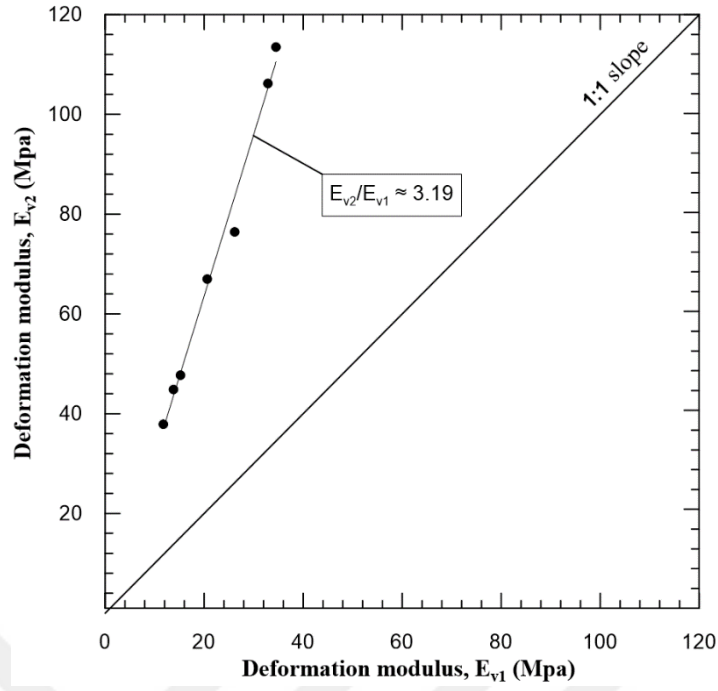


Figure 6.9: Relationship between E_{v1} and E_{v2} .

The subgrade modulus (k_s) was expressed as a function of the first loading stage's deformation modulus (E_{v1}) and the relationship was given in **Figure 6.10**.

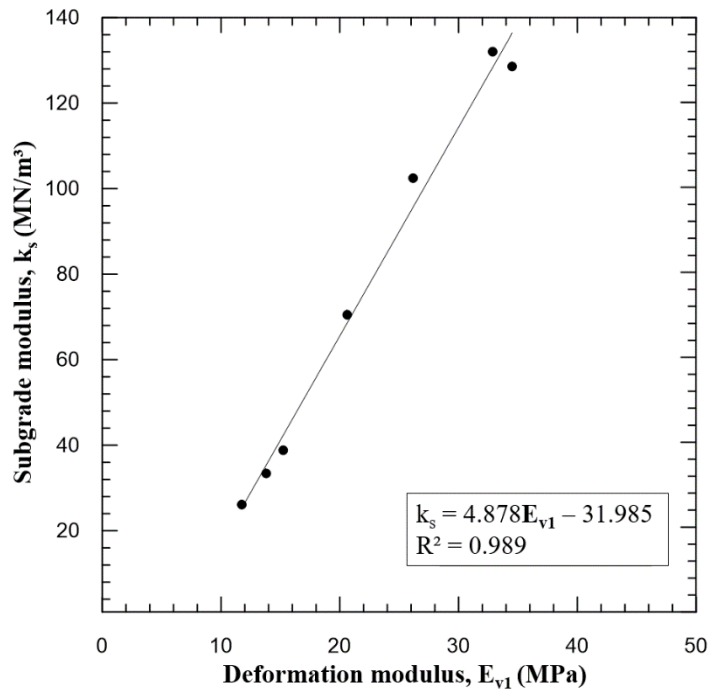


Figure 6.10: The relationship between first loading stage's deformation modulus (E_{v1}) and subgrade modulus (k_s) from PLT results.

The variation of deformation modulus versus granular base layer thickness (PMT stiffness) is provided in **Figure 6.11**. Unlike the other configurations, since Configuration #1 has a PCC load distribution slab at the base of the pavement, the measured deformation modulus for this configuration was not included in the regression analysis in **Figure 6.11**. The increase of the deformation modulus with increasing granular base layer thickness was clearly seen in **Figure 6.11**. The stiffness similarity between Configuration #2 and Configuration #1 was showed that a pavement configuration compose of only granular base with enough thickness would be an alternative for a traditional geofoam block highway embankment flexible pavement which is constructed using a PCC slab (**Table 6.3**, **Figure 6.6**, **6.7** and **6.11**). The relationship between granular base thickness and modulus of deformation was non-linear (**Figure 6.11**). Therefore, after the granular base reaches a threshold thickness, the modulus of deformation will no longer be affected by the granular base thickness (presence of EPS geofoam as subbase layer). Consequently, the deformation modulus of the granular base which increases linearly until a threshold thickness of the granular base is reached, then it became irrespective of the granular base thickness (**Figure 6.11**). When the granular base thickness and corresponding deformation modulus are considered, the granular base thickness of 65 cm (in Configuration #2) was found to be the approximate threshold (or greater) for flexible pavement configurations of geofoam block highway embankments that can be constructed without PCC load distribution slab (**Figure 6.11**). The relationship between subgrade modulus and granular base thickness (**Figure 6.12**) was similar to that of the relationship between deformation modulus and granular base thickness (**Figure 6.11**).

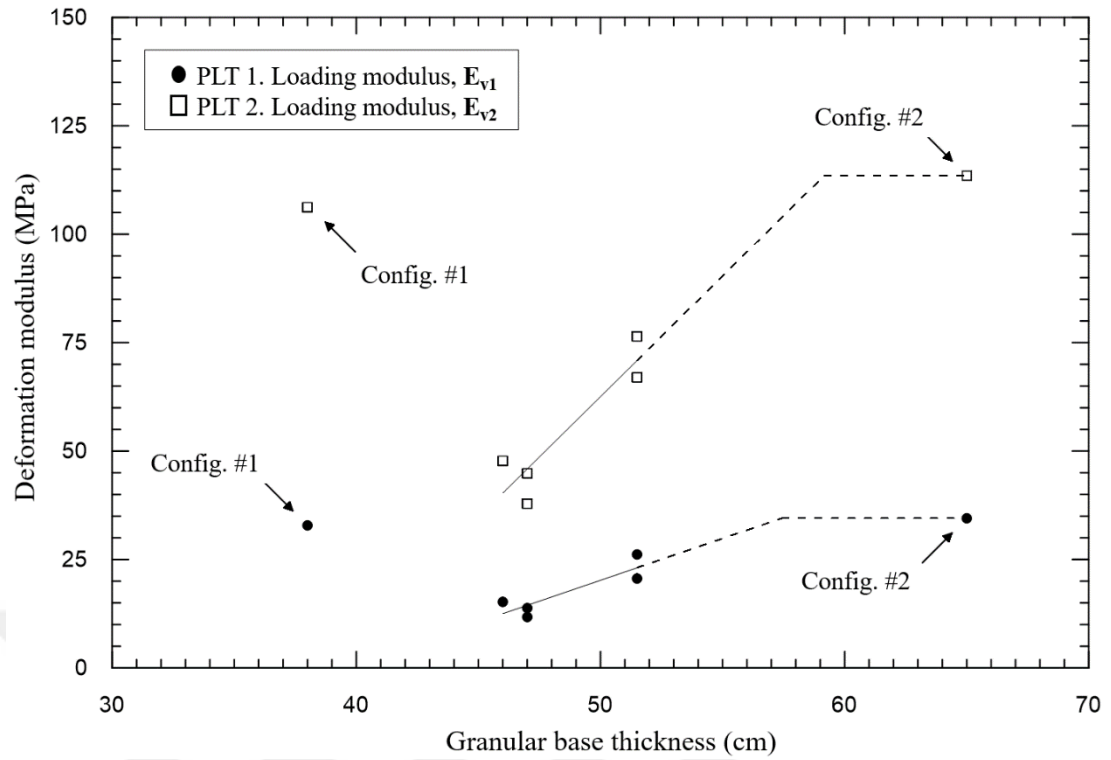


Figure 6.11: Relationship between the thickness of granular base and deformation modulus (E_{v1} , E_{v2}).

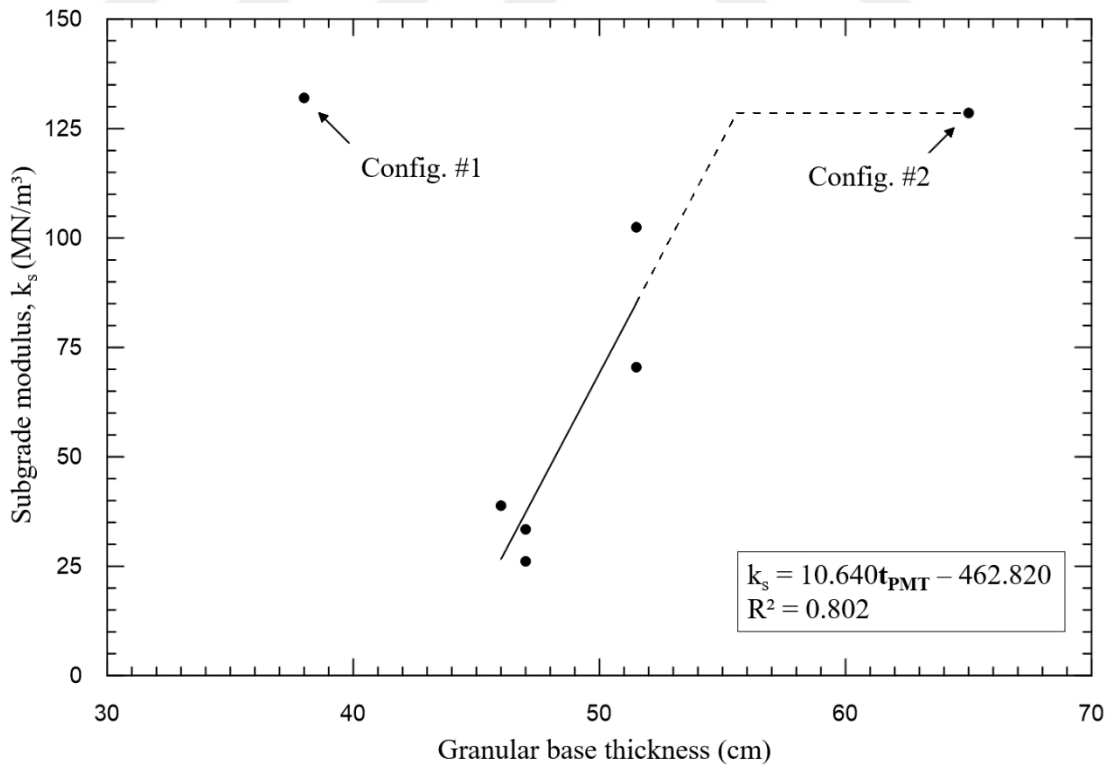


Figure 6.12: Relationship between the thickness of granular base (PMT material) and subgrade modulus (k_s).

6.3. Traffic Measurement by Portable Monitoring System

In order to record the number of truck passes and speed during APT, a MetroCount RoadPod VT model MC5900 portable vehicle counter and classifier system which uses two pneumatic tube sensors was installed in the HMA surface of the Configuration #3 prior APT started (**Figure 6.13**).

The pneumatic tube sensor (**Figure 6.13a**) capture the pressure change that occurs when every individual axle passes. They were fixed with wire clamps to the area outside the APT boundaries (North and South safety zones) (**Figure 6.13b** and **Figure 6.13c**). After one side of the hoses was fixed to the north safety area, it was laid on the surface of the road and stretched approximately 10% percent of the laying length and then the other end of the hose was fixed to the south safety area. After the hose was stretched and fixed, it was double checked to make sure that the tube is in full contact with the surface of the roadway (**Figure 6.13d**). Installation of the first hose was completed without gaps between the hose and the surface of the road and perpendicular to the direction of the traffic (**Figure 6.13d**). The second hose was installed 1 meter away from the first hose (**Figure 6.13e**). Guide holes to a depth of 2 to 3 cm on the HMA surface was drilled for driving 12 cm road nails into the HMA through guide holes (**Figure 6.13f**). Road bands with washers for two nails were placed atop the pneumatic hoses and then hoses were fixed with driving road nails (**Figure 6.13g** and **Figure 6.13h**). The vehicle counter and classifying system installation was completed using road tapes located 1 m apart from each other to make sure they will stay stable during APT (**Figure 6.13i**). According to the road description of the traffic counting station, the hoses were connected to the data logger (**Figure 6.13j**). The data logger was connected to the computer to retrieve the data from the traffic counting software and the system was ready for APT (**Figure 6.13k**).

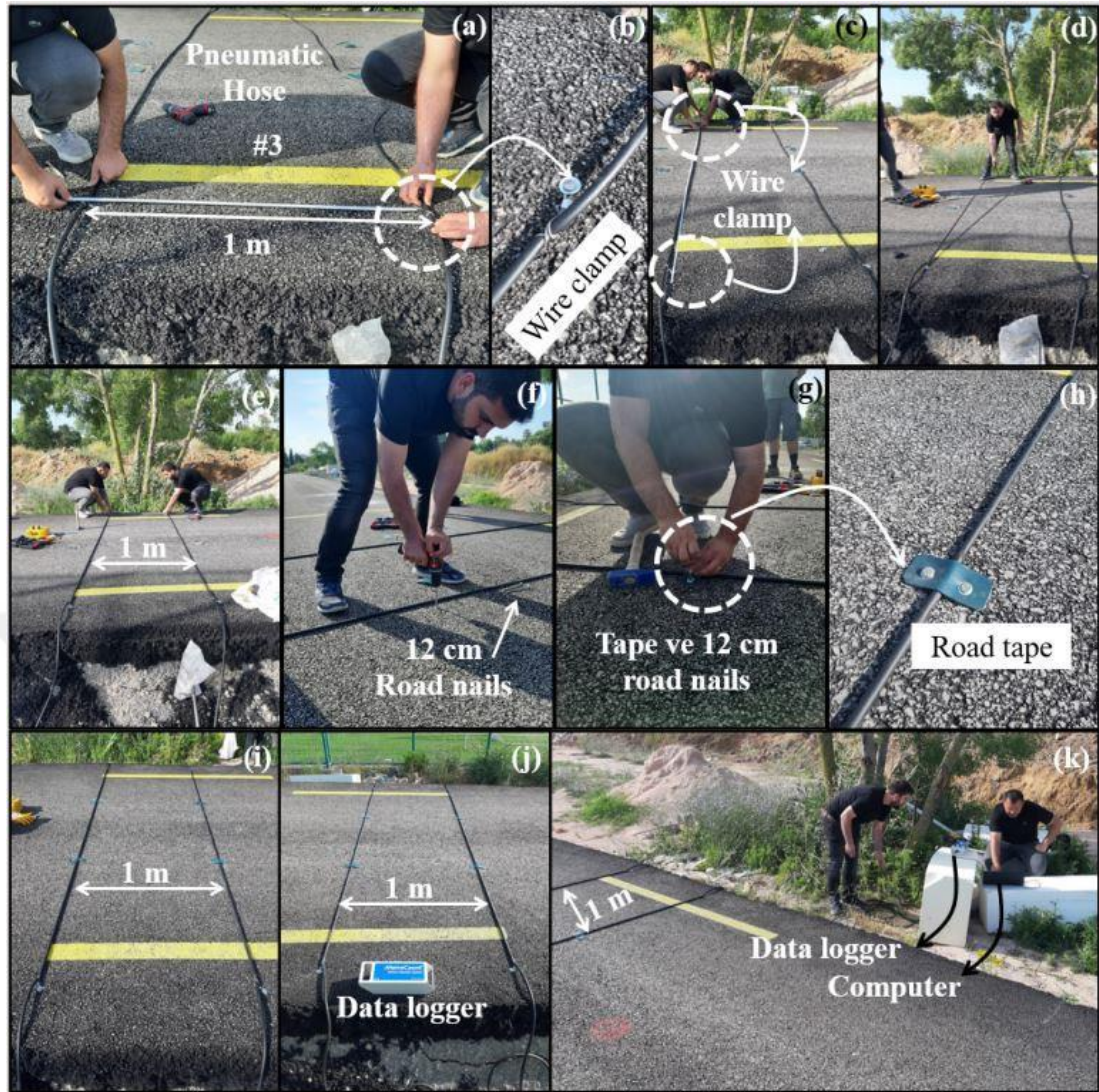


Figure 6.13: The installation process of vehicle counter and classifier system for APT.

The APT, performed by following the traffic flow directions (**Figure 4.2**), started on 17.06.2022 at 09:33 and continued for a total of four days and completed on 20.06.2022 at 16:36 (**Figure 6.14**). According to the data obtained from the vehicle counter and classifier system, the APT consisting of a total of 1299 passes (339 on the first day, 251 on the second day, 363 on the third day and 346 on the fourth day) was carried out (**Figure 6.14**). In addition, truck passing speeds during APT test ranged from 10.2 km/h to 23.7 km/h with an average speed of 16.5 km/h recorded by the vehicle counter and classifier system (**Figure 6.15**).

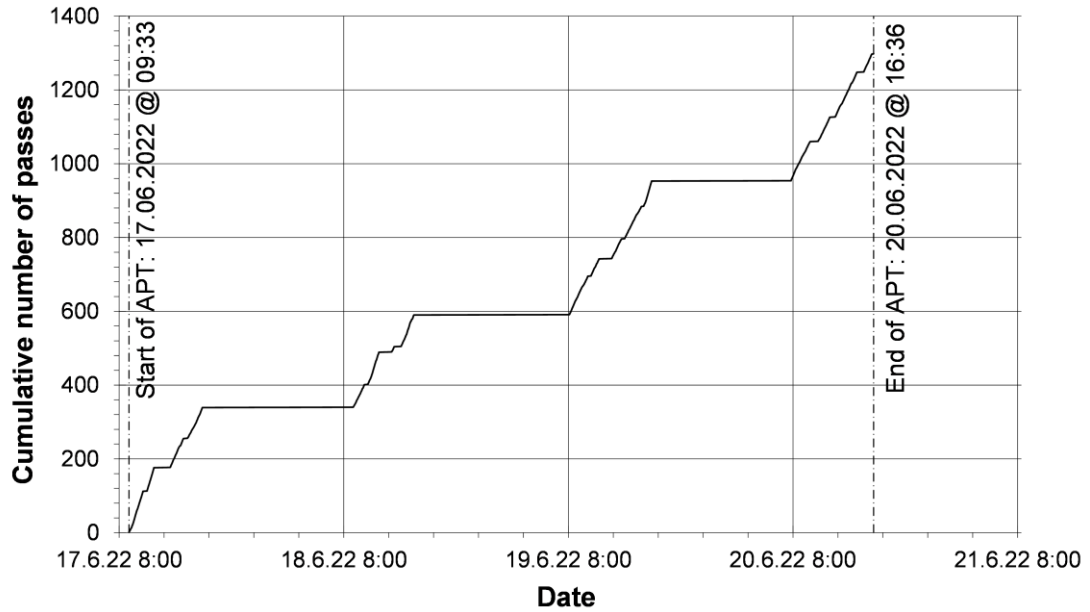


Figure 6.14: Cumulative number of APT tests passes versus date.

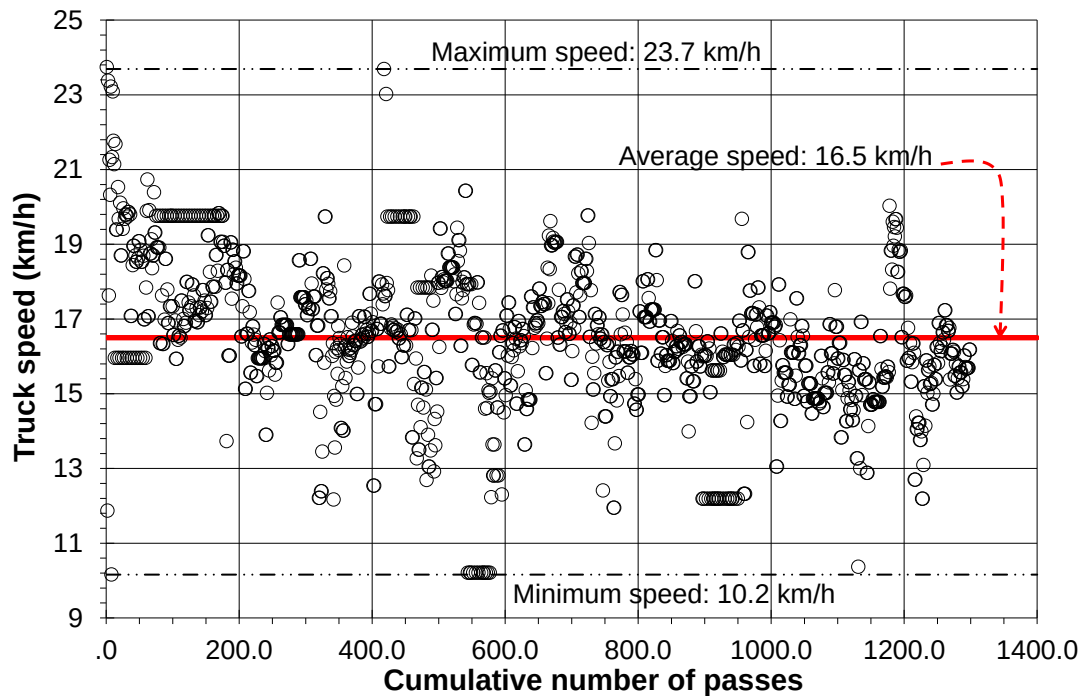


Figure 6.15: Truck speed during APT test period.

6.4. Accelerated Pavement Tests

3-axle Ford Cargo 2424 model truck with a total weight of 37.72 tons was used to apply controlled dynamic wheel loading (**Figure 6.16**). During the APT, the driving

speed of the truck was carried out based on the speed limits recommended in the literature. The recorded truck speeds were compatible with the speeds applied in the literature (**Figure 6.15**). APT program was carried out based on the traffic flow direction plan (**Figure 4.2**, **Figure 6.17** and **Figure 6.18**).



Figure 6.16: An actual 3 axle Ford Cargo 2524 model truck for loading of APT test.

Two different criteria were considered for the termination of APT loadings. The first criterion was considered as a maximum 30 mm rutting on the pavement surface based on the definition of high deterioration level of the Turkish Highways Flexible Superstructures Project Design Guide [KGM, 2008]. The rutting occurred on the pavement surface was regularly measured with an aluminum straightedge according to ASTM E1703/E1703M (2015) (**Figure 6.19**). The second criterion was considered as a function of ESAL based on similar studies in the literature. After 100,000 ESAL loading, tire and road surface abrasion forces are reported to increase significantly [Byron et al., 2004], therefore this ESAL value was considered as the second termination criteria in this study. Consequently, the termination criterion for the accelerated pavement test loading was set at 30 mm rutting or 100,000 ESAL.



Figure 6.17: Accelerated pavement test a-e) Loading in east-west direction, f-i) Loading in west-east direction.



Figure 6.18: Truck passage through pneumatic hoses during accelerated pavement test, a-h) Loading in east-west direction.

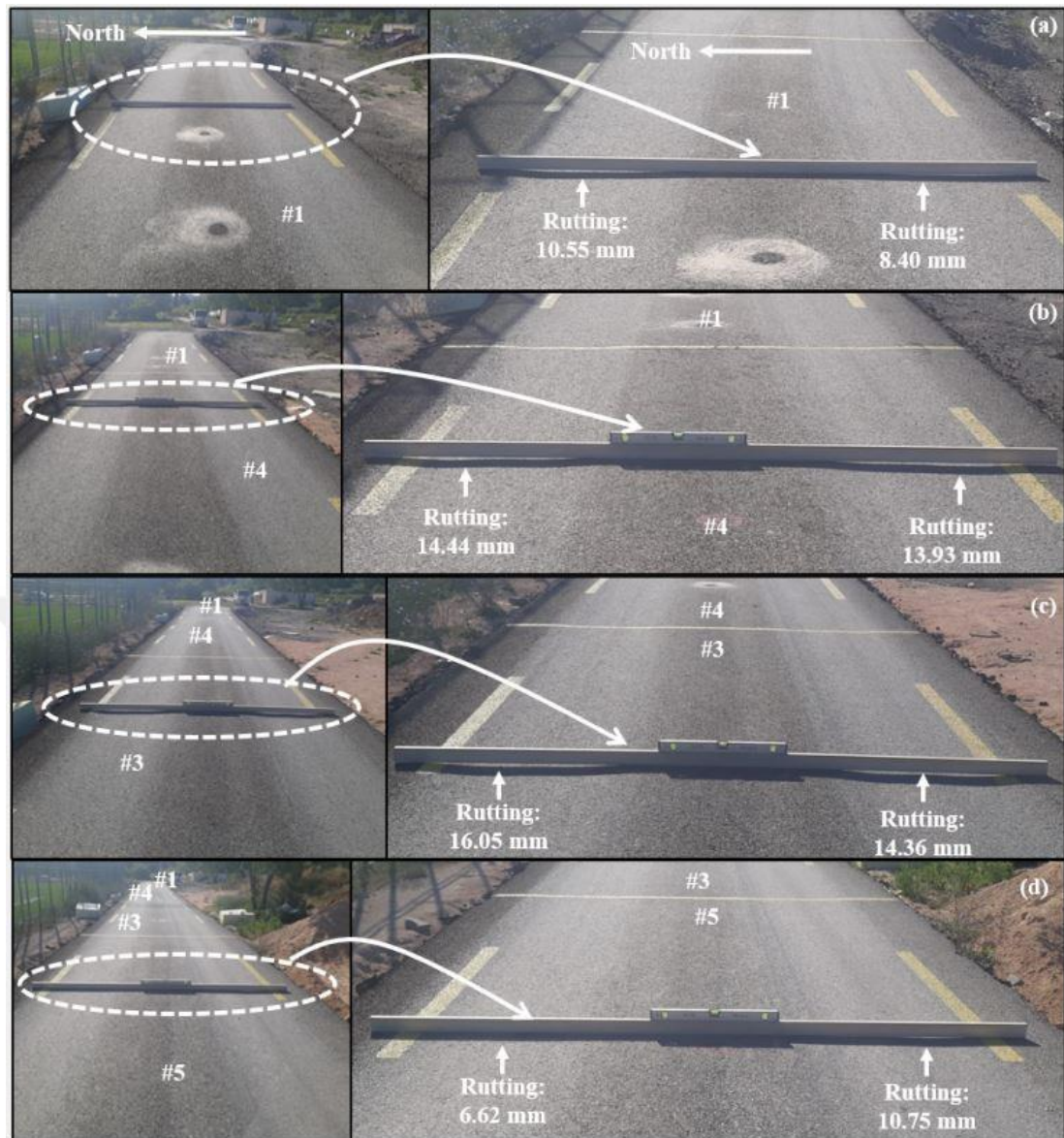


Figure 6.19: Rutting measurements during APT test.

During the APT, the actual and repetitive traffic loads affecting the pavement performance and causing deformations were calculated in terms of 80 kN (18 kips) equivalent standard axle load (ESAL) considering the Turkish Highways Flexible Pavements Design Guide [KGM, 2008] and AASHTO design method [AASHTO, 1993] (**Table 6.4**). The parameters required in ESAL calculations are the load acting on the flexible pavement, the number of traffic load passes during the test and the axle load equivalency factor (LEF) coefficients.

The loaded weight of the 3-axle Ford Cargo 2524 model truck (37.72 ton based on weighbridge measurement, **Figure 6.16**), which was used as a single type of vehicle in the APT test and classified as a heavy vehicle according to the KGM (2008)

guideline. The number of traffic load passes during the APT test was obtained from the MetroCount RoadPod VT MC5900 vehicle counter and classifier system (**Figure 6.13** and **Figure 6.14**). Axle load equivalency factor (LEF) coefficients were obtained by linear interpolation using the axle equivalency factors charts assuming final serviceability (p_t) of 2 and structural number (SN) of 6 in the design of pavement systems ([Chapter 3](#)). The truck axle details, load details, number of traffic load passes and 80 kN (18 kips) equivalent standard axle load (ESAL) calculations were given in **Table 6.4**.

Table 6.4: ESAL calculation for APT test.

Details of truck axle properties and loads for APT test			
Axle class	Single	Tandem	
Truck self-load + load (ton)	10.6	13.6	13.6
Truck self-load + load (kips)	23.3	59.9	
Axle load equivalency factor	3.1	12.5	
The number of traffic load passes	1299		
ESAL	20276.5		

The rutting was measured according to ASTM E1703/E1703M (2015) in entire configurations. The measured rutting and ESAL relationships were presented in **Figure 6.20**. KGM (2008) allowable rutting limit was also given in **Figure 6.20** to compare the performance of the configurations.

The ESAL (calculated in **Table 6.4**) and the variation of the rutting results during the APT are given in **Figure 6.21**.

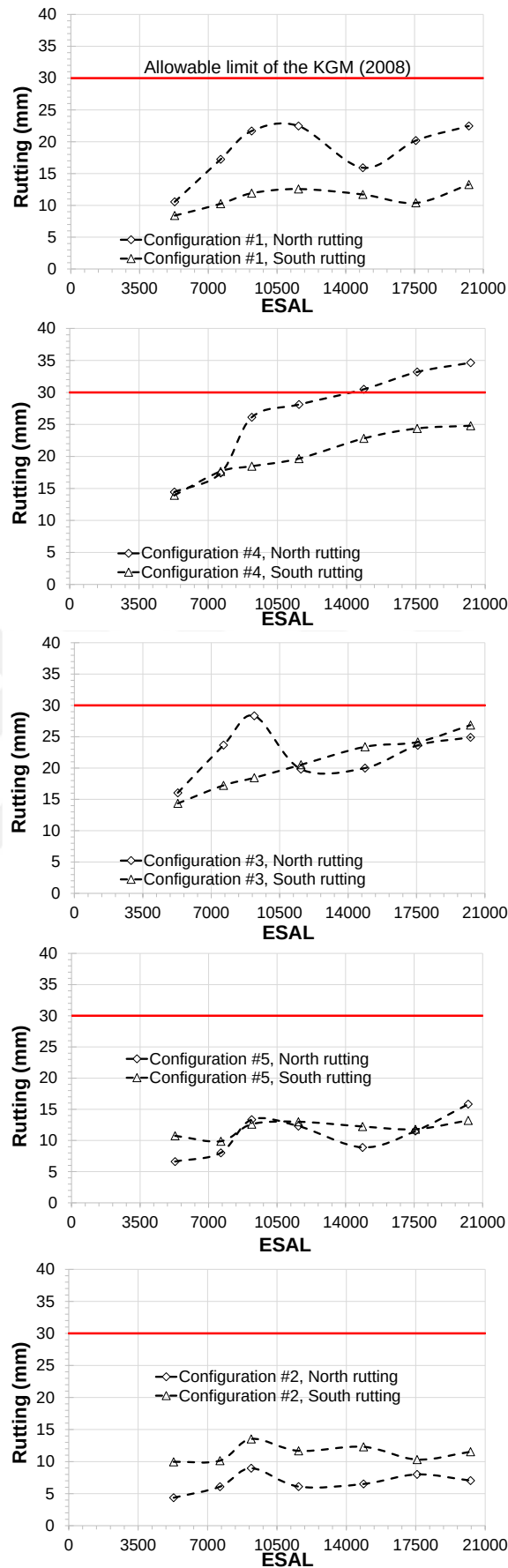


Figure 6.20: Rutting and ESAL relationship, a) Configuration #1, b) Configuration #4, c) Configuration #3, d) Configuration #5, e) Configuration #2.

In the Configuration #4, when the ESAL value was reached approximately 15,000 at the end of the third day of APT (total number of passes 953, **Figure 6.14**), the rutting at the northern wheel path was measured as 30.5 mm. This rutting value exceeded the allowable rutting limit by the KGM (2008) (**Figure 6.20b** and **Figure 6.21b**). Hence, the APT termination criterion (30 mm rut or 100,000 ESAL) was reached at the end of the third day. Traffic loading was continued one more day and the performance of the flexible pavements was recorded when an ESAL of 20,276 was reached (**Figure 6.14**, **Figure 6.20** and **Figure 6.21**).

After the accelerated pavement test loading completed at the first day, the rut depths measured. In addition, rut depths were also measured twice a day on the following days. After the first day of APT, rut depths were not increased in Configuration #2 and Configuration #5, they exhibited stable behavior regardless of the number of loading passes (**Figure 6.20c**, **Figure 6.20d**, **Figure 6.21c** and **Figure 6.21d**).

A similar behavior was also observed in the southern wheel path of the Configuration #1 (**Figure 6.20a** and **Figure 6.21a**). The rutting in the northern wheel path of Configuration #1 was also stable after ESAL of 9,000 (**Figure 6.20a** and **Figure 6.21a**).

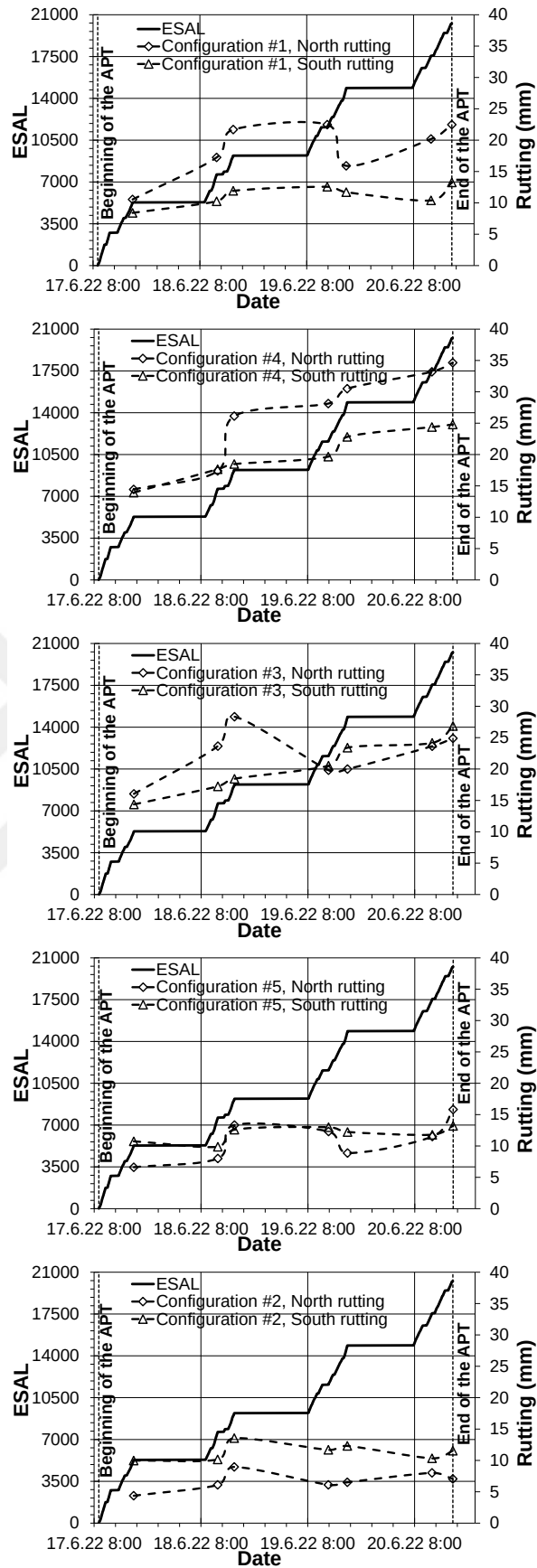


Figure 6.21: The relationship between ESAL and rutting, a) Configuration #1, b) Configuration #4, c) Configuration #3, d) Configuration #5, e) Configuration #2.

In the Configuration #4, the measured rutting increased until it reached the allowable limit value of 30 mm and continued to increase even after reaching the allowable limit on the northern wheel path. On the other hand, it become stable in the southern wheel path after the third day of APT (**Figure 6.20b** and **Figure 6.21b**).

At the end of the APT loading, the physical conditions of Configuration #1 and Configuration #4 were presented in **Figure 6.22** and the other configurations were presented in **Figure 6.23**. Tensile cracks in the HMA located in the northern safety zone of the Configuration #1 resulted in yielding towards the slope surface of the pavement (**Figure 6.22a** and **Figure 6.22b**). At the eastern boundary of the Configuration #4, rutting along the northern wheel path reached up to 34.64 mm (**Figure 6.22b** and **Figure 6.22c**), and the widths of the alligator cracks (**Figure 6.22d**) were measured from 5 mm (**Figure 6.22e**) to 1.5 mm (**Figure 6.22f**), towards the west. Tensile cracks along the safety zone of the Configuration #4 advanced to the safety zone and resulted yielding of the HMA which covered the entire northern slope surface of the Configuration #4 (**Figure 6.22g**). A significant amount of HMA material was lost from the safety area of the Configuration #4 (**Figure 6.22g**).

In the northern safety zones of Configurations #3, #5 and #2, tensile cracks were occurred and HMA material yielded onto the slope surface (**Figure 6.23a**, **Figure 6.23b** and **Figure 6.23c**). The amount of HMA yielded in these safety zones were observed less than that of the Configuration #4. The rutting path in these configurations were shown in **Figure 6.23d**. Similar tensile cracks were also observed on the southern slopes of these configurations (**Figure 6.23e** and **Figure 6.23f**).

As a result of exceeding the APT test termination criterion (30 mm) in Configuration #4 (**Figure 6.20** and **Figure 6.21**) and the subsequent progression of tensile cracks in the HMA in the northern safety zone of the Configuration #4, the driving comfort was impaired, and adversely affected the driving safety. At this point, the APT program was terminated.



Figure 6.22: Physical conditions at the end of the APT test for Configurations #1 and #4, a-b) Tensile cracks at the crest of the northern safety zone of the Configuration #1 and HMA yielding towards to the slope-side, c-f) Rutting path of Configuration #4 and alligator cracks at the north-west of the Configuration #4, g) HMA in the safety zone of Configuration #4 were yielded along the northern slope-side of the pavement.



Figure 6.23: Physical conditions of Configurations #3, #5 and #2 at the end of the APT , a-c) Tensile cracks at configurations and HMA yielding towards to the slope-side of the pavement, d) Wheel rut path, e-f) HMA in the safety zone of configurations yielding and spreading along the slope-side.

6.5. Falling Weight Deflectometer Tests

Since the 1980s, Falling Weight Deflectometer (FWD) has been used to non-destructively determine moduli of elasticity and stresses of soil layers and to analyze structural strength, bearing capacity, remaining life, new and required layer thickness [Mehta and Roque, 2003; Alavi et al., 2008]. The FWD tests were performed prior and

after the accelerated pavement test according to the ASTM D4694 (2020) standard (Figure 6.24).

FWD tests were conducted twice prior the initiation of the APT. The first series of the tests were performed in the morning and the second series of the test were performed in the afternoon to consider the effect of temperature of HMA. Tests were performed at the center and the north wheel path of each configuration to capture the effect of APT loading along the wheel path. The test points were marked on the road surface after the first series and the tests in the afternoon were carried out at the same locations.

In addition, after the APT program was completed, FWD tests were conducted twice, similarly that of test series prior to the APT. Tests were performed in the northern and the southern wheel paths. In addition, tests were also performed at the center of each configuration. FWD tests performed after APT program was completed in the center and northern wheel paths were conducted at the same locations with FWD tests performed prior to APT. The FWD tests performed on the south wheel path were also carried out in the same cross-section as the center of the configuration.



Figure 6.24: Falling weight deflectometer (FWD), a-b) Setup of FWD equipment, c) Geophones, d-e) Applying the load and recording the load-displacement curves with the software.

Loads between 30-60 kN were applied in FWD tests for low volume roads or reinforced roads with geocomposites [Izevbekhai and Pederson, 2010; Al-Qadi and Hughes, 2000; Pasquini et al., 2013; Ahmedzade and Yılmaz, 2016; Abd El-Raof et al., 2018; White et al., 2019; Šiukšcius et al., 2019; Alimohammadi et al., 2020]. Therefore, the loading sequence selected compatible to the published literature.

Generally, 4 or 6 drop loads were applied as a function of the configuration strengths. In the high-strength configurations, loading sequence was composed of approximately 30, 40 and 60 kN was applied twice, while in the low-strength configurations loading sequence was composed of 30 and 40 kN was applied twice to avoid exceeding the maximum deformation limit of the FWD device [Pierce et al., 2017]. The loads were applied on a diameter of 30 cm plate of FWD device.

Deflections at 0 cm, 20 cm, 30 cm, 45 cm, 45 cm, 60 cm, 60 cm, 75 cm, 90 cm, 120 cm, 150 cm and 180 cm away from the load application point were measured (**Figure 6.24d** and **Figure 6.24e**) using a total of 10 geophones (**Figure 6.24c**) for each loading level (**Figure 6.24a** and **Figure 6.24b**) and deflection bowls were obtained. The locations of the geophones and the typical deflection bowl were given in **Figure 6.25**.

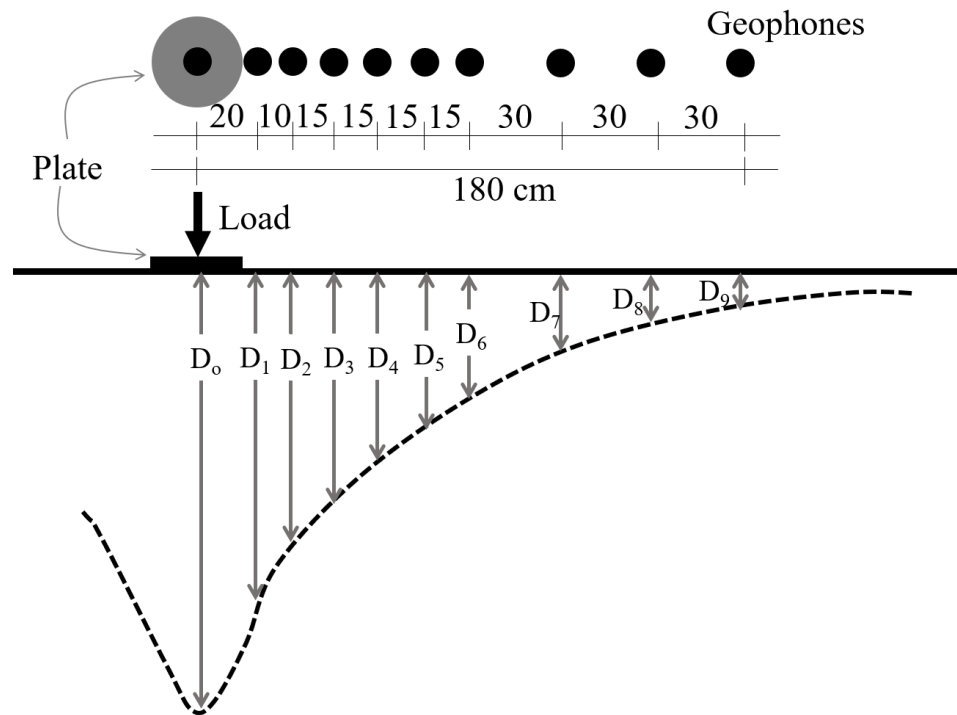


Figure 6.25: Locations of FWD geophones and typical deflection bowl.

The deflection curves obtained prior APT in the FWD test performed on 16.06.2022 at 14:10 in the center of the Configuration #2 were given in **Figure 6.26**. A total of 50 FWD tests were performed, 20 prior the APT test and 30 after the APT test. Deflection curves obtained from all FWD tests were given in **Appendix-B**. The list of FWD tests, applied vertical loads, maximum deflection (D_0) values and temperatures measured on the surface and half-depth of HMA, along with the ambient temperatures at the time of the test were summarized for Configuration #2 (**Figure 6.26**) in **Table 6.5**. The results for entire Configurations are summarized in **Appendix-C**.

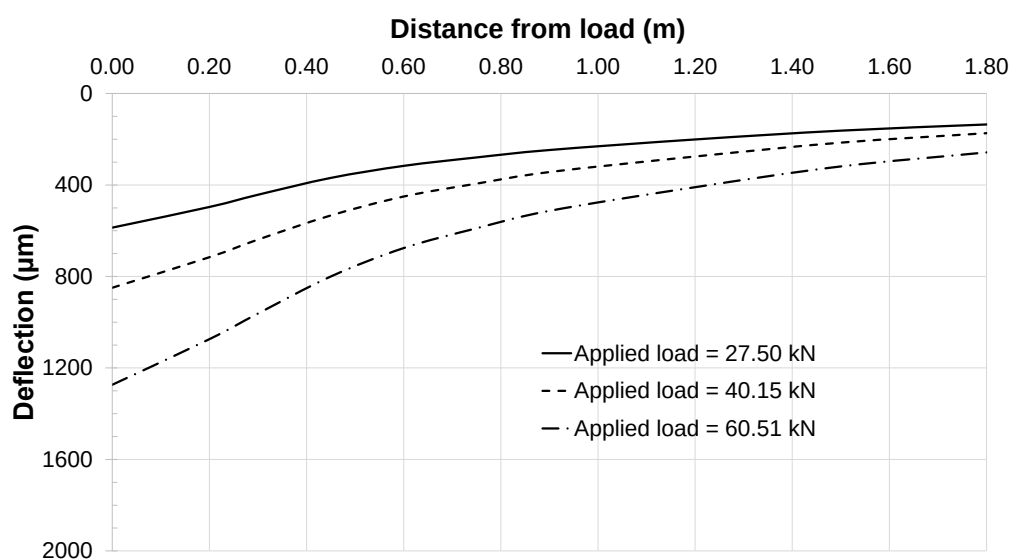


Figure 6.26: FWD deflection curves at 14:10 on 16.06.2022 in the center wheel path of the Configuration #2.

Table 6.5: FWD tests prior APT, loading sequence, maximum deflections, pavement and ambient temperatures for Configuration #2 at 14:10 on 16.06.2022 in the center.

Config. no	Date	Time	Location	Applied load level (kN)	Max. Deflection D_0 (μm)	Temperatures ($^{\circ}\text{C}$)		
						Pavement	Pavement surface	Ambient
2	16.06 2022	14:10	Center	27.50	586.60	41.10	43.45	32.22
				27.28	575.10			
				40.15	849.30			
				40.36	849.80			
				60.51	1273.30			
				60.44	1286.60			

Dynatest ELMOD (Evaluation of layer moduli and overlay design) software which allows the use of finite element method, Linear Elastic Theory and equivalent thickness methods, was used for back calculation of layer modulus based on FWD results. Deflection curves, typical layer moduli, Poisson's ratios and layer thicknesses were used as input parameters. The HMA which consisted of wearing course and bituminous base layers, was modeled as a single layer as recommended by ELMOD software. The granular layer and the reinforced layers with geogrid or geocell were also modeled as a single layer [Vennapusa et al., 2018; Gallage et al., 2015; Al-Qadi and Hughes, 2000]. Typical moduli for asphalt, granular or reinforced base, and geofoam layers were used as 2000 MPa, 200 MPa and 20 MPa respectively [Huang, 2004]. In the same order, the Poisson ratios of the layers were used as 0.35, 0.35 and 0.10. The EPS block Poisson's ratio of 0.10 is also used in the literature for back-calculation of FWD test [Duškov, 1997].

In this study, Odemark's equivalent thickness method (MET) was used to back-calculate the layer modulus. In this method, the two-layer system is converted into a layer of equivalent thickness with the properties of the underlying layer. [Ullidtz, 1987].

The output page of ELMOD was given in **Figure 6.27** that was performed of applied sixth load level in the center wheel path of the Configuration #2 at 14:10 on 16.06.2022 (**Figure 6.26** and **Table 6.5**). The back calculation modulus obtained from the analysis for Configurations #2, #3, #4 and #5 were summarized in **Table 6.6**. ELMOD software requires the stiffness of the layers to decrease downwards starting from the top layer. However, in Configuration#1, the 10 cm thick PCC load distribution slab between the granular base and the geofoam blocks limits the use of ELMOD's principle of stiffness reduction from top to bottom. Therefore, back analysis using ELMOD was not performed for Configuration#1. Before each FWD test, internal and surface temperatures of HMA were measured and temperature correction to a reference temperature of 25 °C were performed for the reported values.

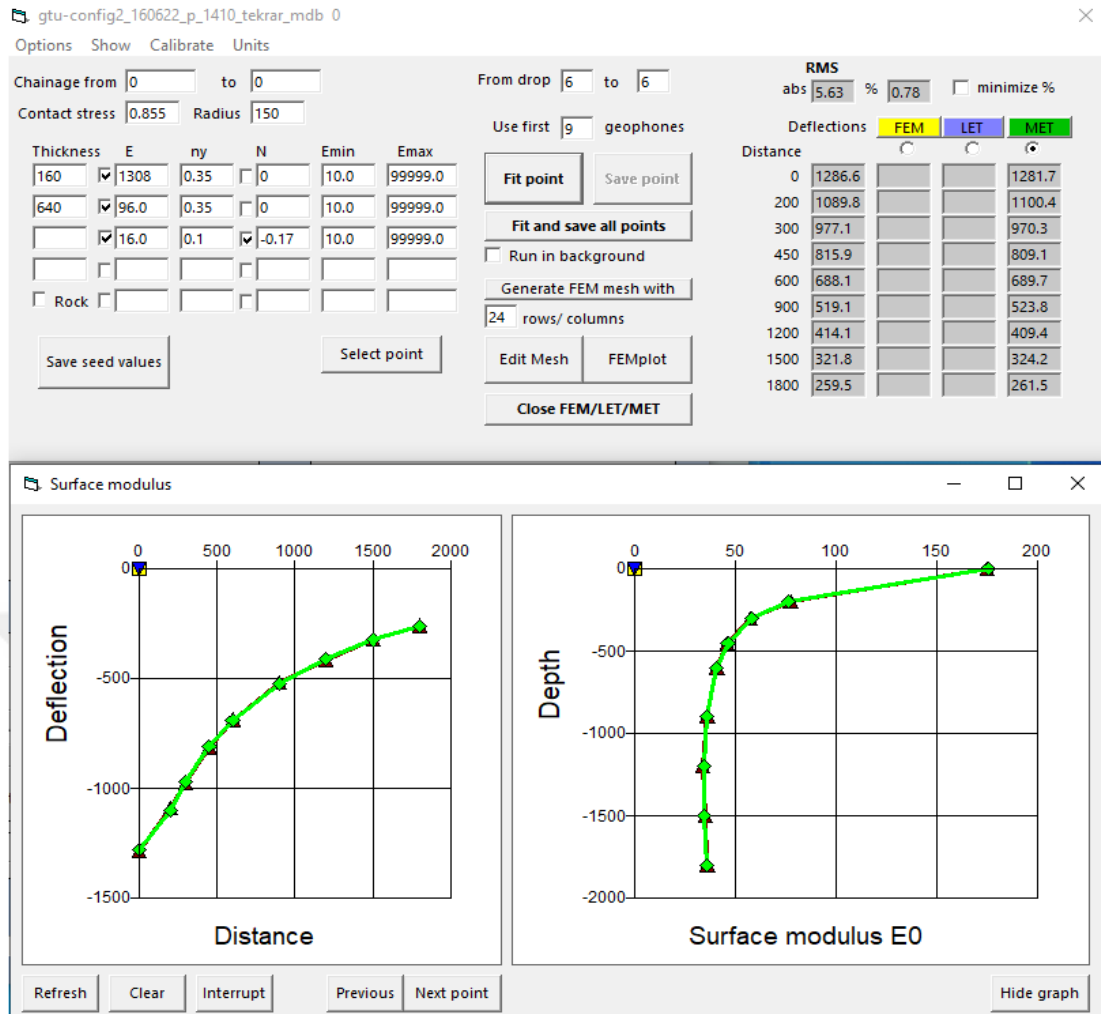


Figure 6.27: Output of ELMOD for applied sixth load level at 14:10 on 16.06.2022 in the center wheel path of the Configuration #2.

The moduli of elasticity obtained by back calculation using the FWD deflection measurements of the test performed under the vertical load of 60.44 kN (Table 6.5) for Configuration #2 in Figure 6.27. Similarly, moduli of elasticity values were obtained by back calculation for all FWD tests.

The average of the back-calculated values were temperature corrected (according to 25 °C) and summarized in Table 6.6 per configuration. Deformation moduli of PLT and FWD test measured prior the APT were compared in Figure 6.28.

Table 6.6: Deformation moduli (E_{FWD}) obtained by back analysis with ELMOD using FWD deflection curves.

Configuration no (East – West direction)	<u>Prior APT test</u>			<u>After APT test</u>		
	E_{FWD}			E_{FWD}		
	HMA (MPa)	Granular base (MPa)	EPS (MPa)	HMA (MPa)	Granular base (MPa)	EPS (MPa)
Configuration #4	772	31	11	753	35	10
Configuration #3	977	32	10	986	27	9
Configuration #5	1255	63	13	1576	59	12
Configuration #2	1948	113	18	2639	101	17

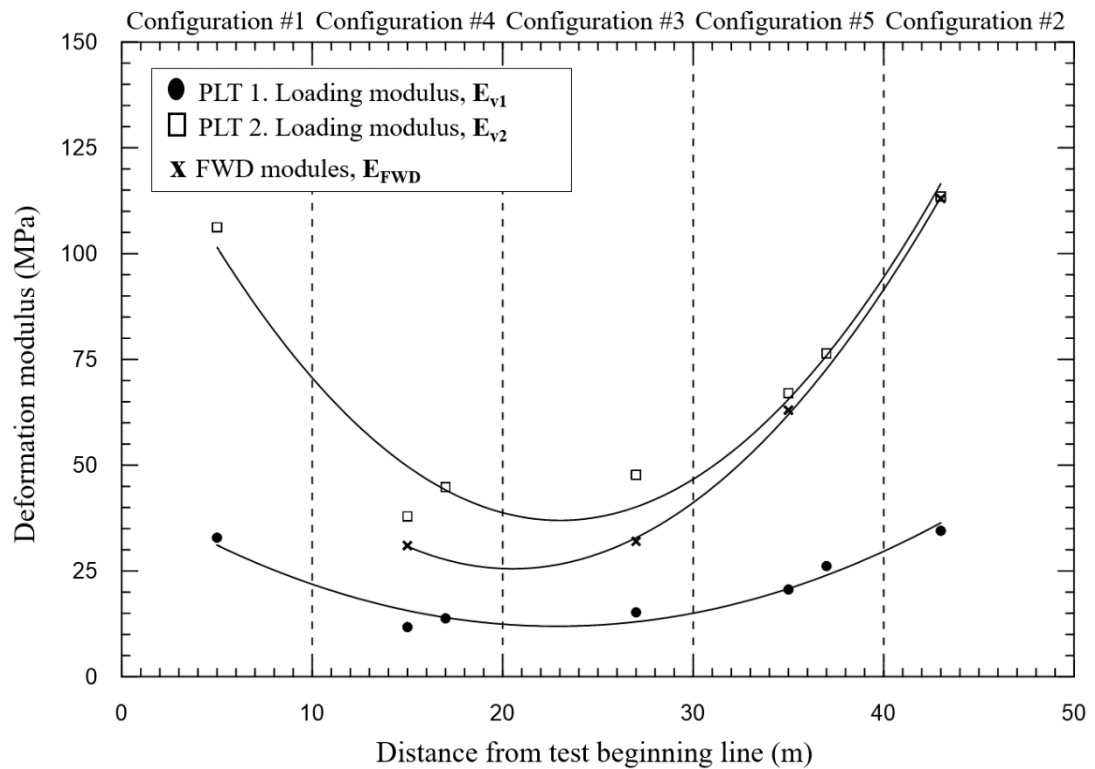


Figure 6.28: PLT deformation moduli (E_{v1} and E_{v2}) and deformation moduli determined by FWD (E_{FWD}) for each configuration.

The relationship between the deformation moduli obtained from the back analysis of FWD (E_{FWD}) and the E_{v2} deformation moduli of the PLT was depicted in **Figure 6.29**.

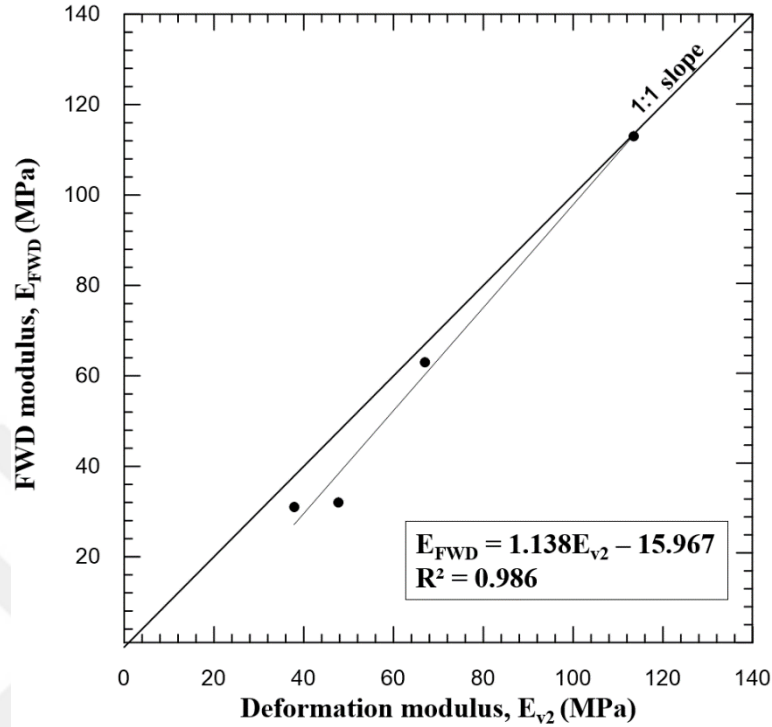


Figure 6.29: Relationship between the E_{FWD} and E_{v2} .

Similar to the PLT deformation moduli (**Figure 6.11**), the deformation moduli of FWD measurements (E_{FWD}) were also increased with thicker granular base thickness (**Figure 6.30**). The relationship between granular base thickness and E_{FWD} is non-linear and once the thickness of granular base reaches a threshold value, the modulus of deformation will not be influenced by the granular base thickness (in other words, the presence of the EPS geofoam block layer). Therefore, as observed in PLT results; the modulus of deformation of the granular base which increases with a trend that can be considered linear until the threshold thickness of the granular base is reached, and then become independent of the granular base thickness (**Figure 6.30**). Similar to that of PLT results, when the granular base thickness and corresponding FWD deformation modulus was considered, the granular base thickness of 65 cm (in the Configuration #2) was found to be threshold (or greater) thickness (**Figure 6.30** and **Figure 6.11**).

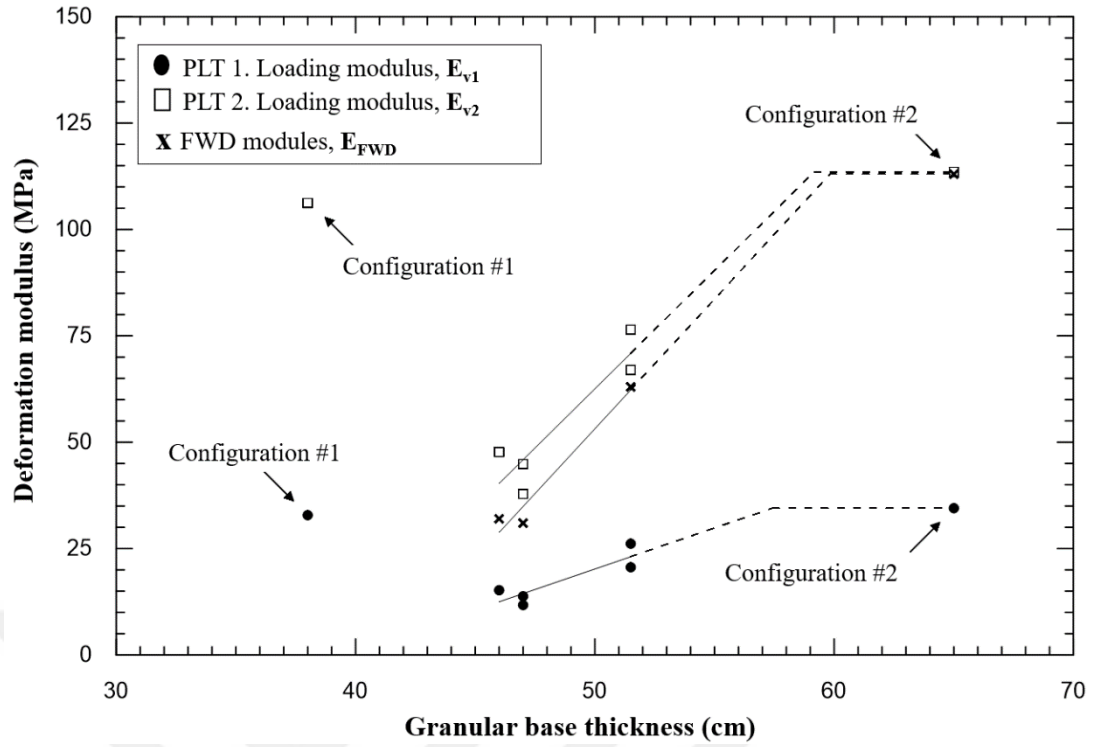


Figure 6.30: Relationship between granular base thickness and the modulus of deformations (E_{v1} , E_{v2} , E_{FWD}).

6.6. Profilometer

In addition to measuring rutting with a straightedge according to ASTM E1703/E1703M (2015) a laser profilometer was also used. Laser profilometer captured surface features (rutting) both before and after the accelerated pavement test (**Figure 6.31**). The difference between the two measurements (before and after APT) was used to report the wheel path settlement (TIO). The Dynatest profilometer collected the profile of the flexible pavement configurations every 10 cm with a laser and accelerometer system. (**Figure 6.31a** and **Figure 6.31b**). Before starting the experiment, the laser profilometers were calibrated using a reference beam (**Figure 6.31c** and **Figure 6.31d**). After the calibration was completed, the calibration beam was removed (**Figure 6.31e**) and the measurement system was ready for data collection. Data were collected every 10 cm in all configurations (**Figures 6.31f - k**), starting from the eastern end of the test bed (**Figure 6.31f**). The maximum TIO values and station information obtained from the collected data were summarized in **Table 6.7**, along with the TIO values (**Figure 6.20**) measured according to ASTM E1703/E1703M (2015) for comparison purposes.

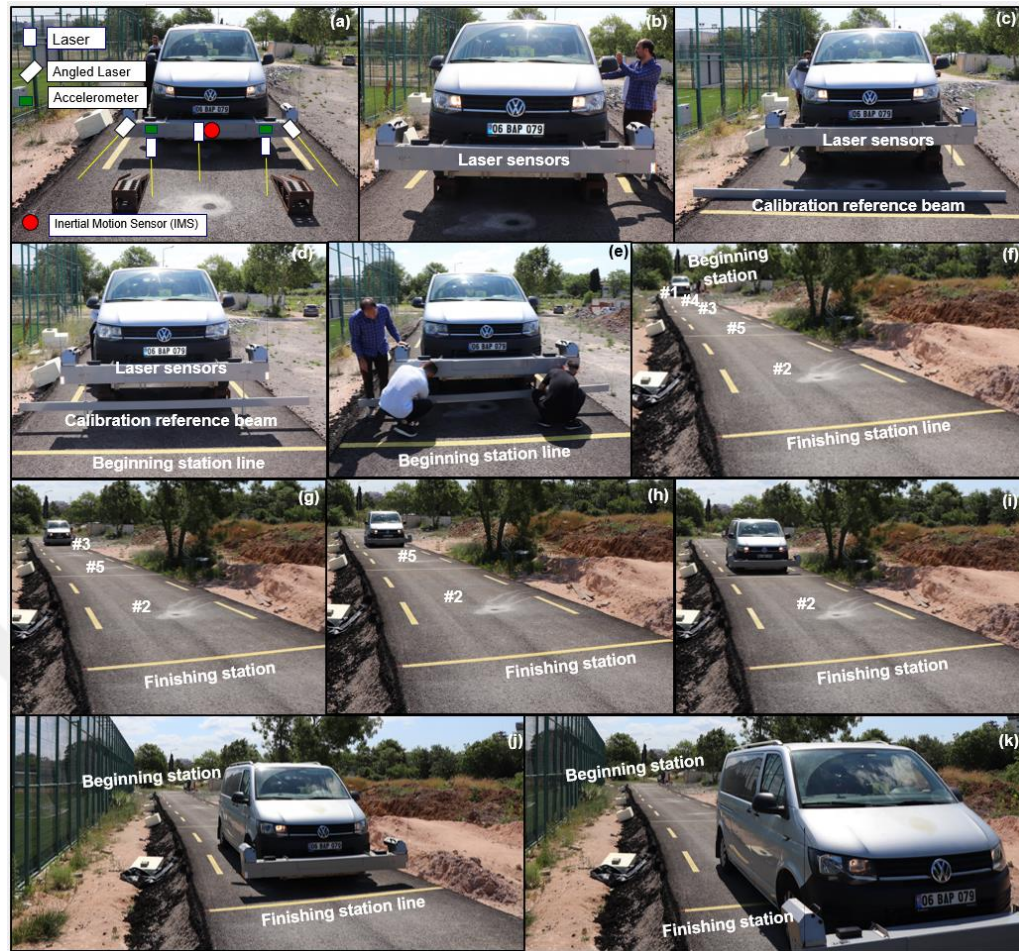


Figure 6.31: Measurement of surface features with profilometer a-b) Laser locations, c-e) Calibration of the sensors using a reference beam, f-k) Measurement of surface features with profilometer.

Table 6.7: Comparison of surface features measured by profilometer with values measured according to ASTM E1703.

Config. no (East – West direction)	Distance from beginning station / Max. TIO (mm)		According to ASTM E1703 Max. TIO (mm) (Figure 6.20)	
	North wheel path	South wheel path	North wheel path	South wheel path
#1	10.0 m / 12.6 mm	8.7 m / 16.1 mm	22.47 mm	13.26 mm
#4	16.7 m / 16.7 mm	10.2 m / 16.8 mm	34.64 mm	24.80 mm
#3	23.2 m / 23.6 mm	24.6 m / 17.5 mm	24.9 mm	26.86 mm
#5	38.9 m / 13.8 mm	31.1 m / 7.9 mm	15.83 mm	13.22 mm
#2	40.3 m / 7.4 mm	47.6 m / 4.2 mm	7.04 mm	11.54 mm

There were differences in some configurations between the surface features measured by profilometer and the values measured according to ASTM E1703 (**Table 6.7**). While the profilometer collected data at every 10 cm, the rutting measurements using a straightedge were performed at the maximum rutting spots which were identified visually. Therefore, the laser profilometer might have not collected data at these spots. Despite this principal difference, the measured TIO values are close to each other except for the rutting of Configurations #1 and #4 (**Table 6.7**).



7. CONCLUSION

An extensive prototype modelling effort was completed to investigate the performances of various flexible pavement systems for geofoam block highway embankments. A total of five different flexible pavement configurations were constructed in a 3.75 wide and 50 m long test bed. Configuration #1 included a flexible pavement system composed of a PCC load distribution slab which is the industry standard for geofoam highway embankments, whereas the other four flexible pavement configurations didn't include a PCC slab. While Configuration#2 was constructed using PMT and the other three configurations were reinforced with geosynthetics. PMT of Configurations #3 and #4 were reinforced with single row and two rows of geocell reinforcement, respectively, and two-rows of bi-axial geogrid reinforcements were used in Configuration #5.

The increase of E_{V2} and E_{FWD} with increasing granular base layer thickness was clearly seen. The stiffness similarity between Configuration #1 and Configuration #2 showed that a pavement configuration compose of only granular base with enough thickness would be an alternative for a traditional geofoam block highway embankment flexible pavement which is constructed using a PCC slab.

The well-known beam effect of geocell reinforcement was not mobilized, therefore, the lowest E_{V2} and k_s were obtained in Configurations #3 and #4. This can be related with insufficient compaction caused by the geofoam absorbing the compaction performance. Therefore, the weak stiffness and the high rutting for Configurations #3 and #4 indicated. Hence, increasing the compaction efficiency and selecting the gradation criteria of base course for geocell reinforced flexible pavements atop of geofoam block highway embankments need to be further investigated.

E_{V2} and k_s obtained in Configuration #5 are larger than that of Configurations #3 and #4, however, smaller than Configurations #1 and #2. Although Configuration #5 was designed with equivalent stiffness of Configurations #1 and #2, it has clearly seen that the TBR was less than 2. While this TBR is ideal for a well-compacted granular base, it is found not to be suitable for geofoam embankments. This is simply due to the fact that geofoam blocks absorb the vibration energy of roller during the compaction that prevents the efficient compaction of the first lift of atop geofoam blocks. Therefore,

the well-known geogrid membrane effect was not fully mobilized since sufficient compaction performance was not achieved.

The relationship between granular base thickness and E_{V2} and E_{FWD} is found to be non-linear and once the thickness of granular base reaches a threshold value (65 cm in Configuration #2), they didn't influence by the granular base thickness.

Consequently, this study was proved that the thickness of granular base layer is the crucial design parameter for flexible pavements constructed with EPS block highway embankments and that these systems can be constructed without PCC load distribution slab. Further studies need to be performed to research the efficiency of the geosynthetic reinforcement in flexible pavement configurations of geofoam block highway embankments.

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BIOGRAPHY

Abdullah Yasin Büyükaslan completed his primary and high school education in Konya / Türkiye. In 2010, he was eligible to study undergraduate education at the Department of Civil Engineering of the Faculty of Engineering at Balıkesir University. He completed his Bachelor's degree in 2014. He began his professional career after graduating. He worked as a technical office engineer in a private company between 2014-2015 and as a field engineer in a different private company between 2015-2016 in Istanbul / Türkiye. Between 2016 and 2021, he worked as a field engineer for a private Turkish company involved in a railway project in Ethiopia. In 2021, he applied for a master's degree at Gebze Technical University and was eligible for education while working on the Ethiopian Railway project. At the same year, while continuing his master studies, he started at a company that performing controlled in Gebze - Darıca subway project in Gebze/Kocaeli. He still works for this company on a subway project in Gebze.

APPENDICES

APPENDIX-A: PLATE LOAD TESTS RESULTS

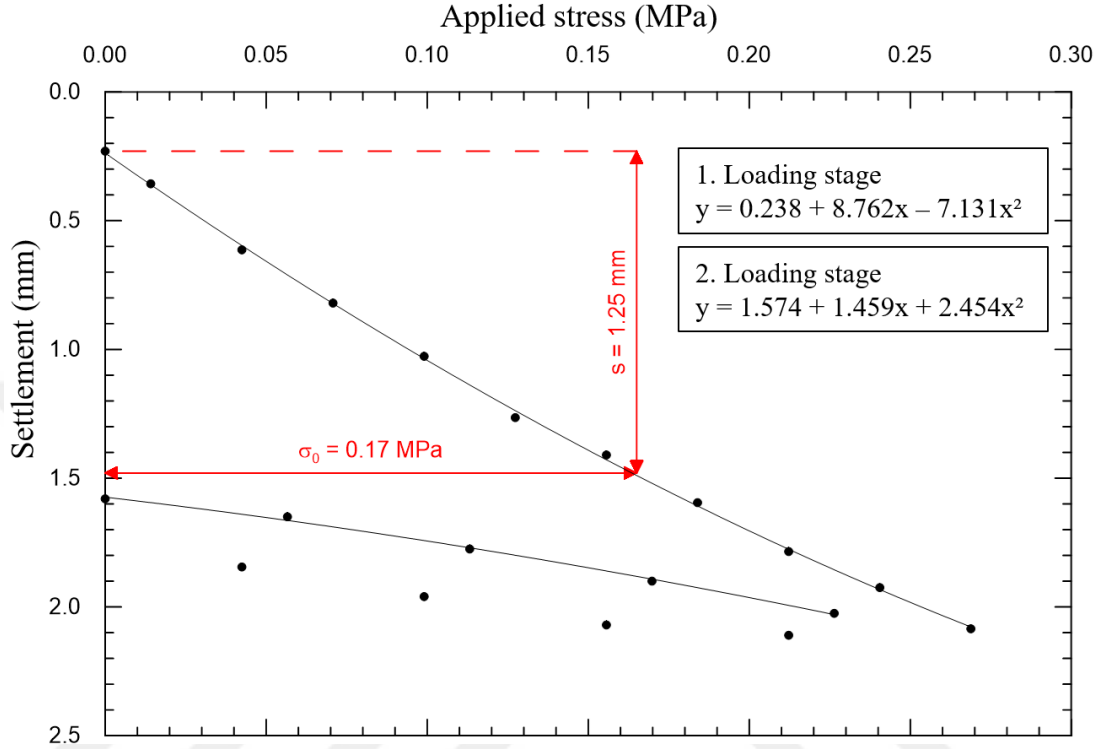


Figure A.1: The PLT#1.1 (Figure 6.1) stress-settlement curves of Configuration #1.

Table A.1: The calculations of modulus of elasticity and subgrade modulus for PLT#1.1 of Configuration #1.

Parameter	Unit	1. Loading	2. Loading
σ_{0max}	MPa		0.27
a_0	mm	0.238	1.574
a_1	mm/MN/m ²	8.762	1.459
a_2	mm/MN ² /m ⁴	-7.131	2.454
$E_V = 1.5r \frac{1}{a_1 + a_2 \sigma_{0max}}$	MPa	32.9	106.2
E_{V2}/E_{V1}	-		3.23
$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125m}$	MN/m ³		132,0

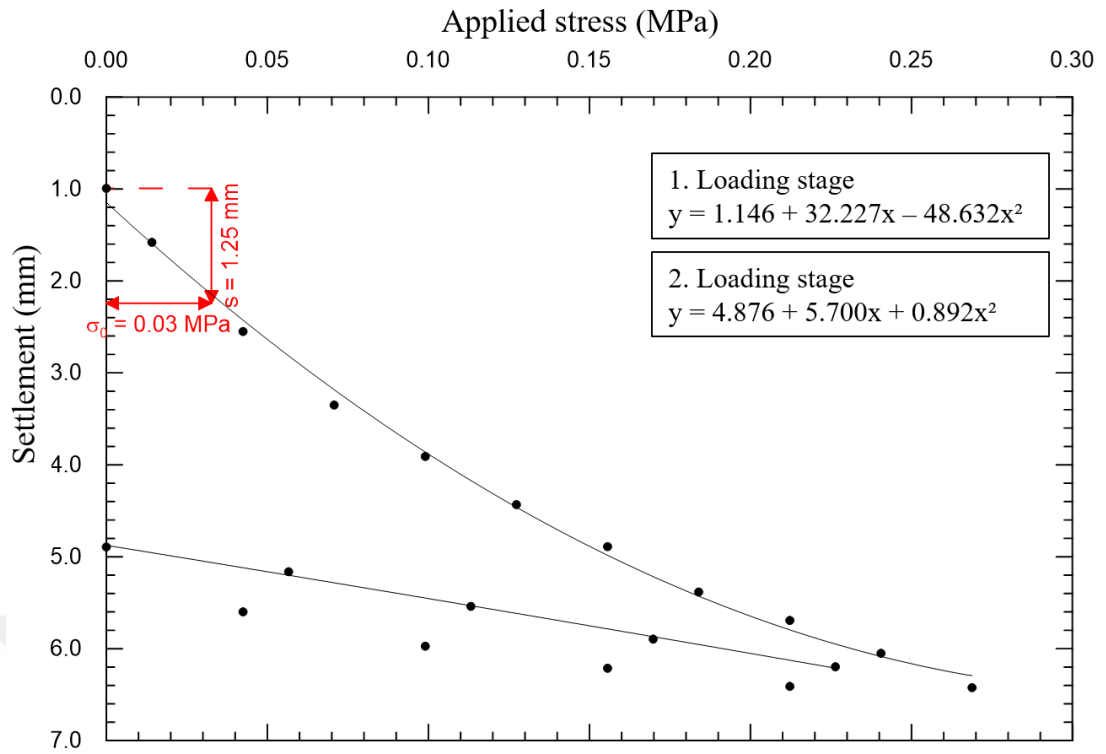


Figure A.2: The PLT#4.1 (Figure 6.1) stress-settlement curves of Configuration #4.

Table A.2: The calculations of modulus of elasticity and subgrade modulus for PLT#4.1 of Configuration #4.

Parameter	Unit	1. Loading	2. Loading
σ_{0max}	MPa	0.27	
a_0	mm	1.146	4.876
a_1	mm/MN/m ²	32.227	5.700
a_2	mm/MN ² /m ⁴	-48.632	0.892
$E_V = 1.5r \frac{1}{a_1 + a_2 \sigma_{0max}}$	MPa	11.7	37.9
E_{V2}/E_{V1}	-	3.22	
$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125m}$	MN/m ³	26.1	

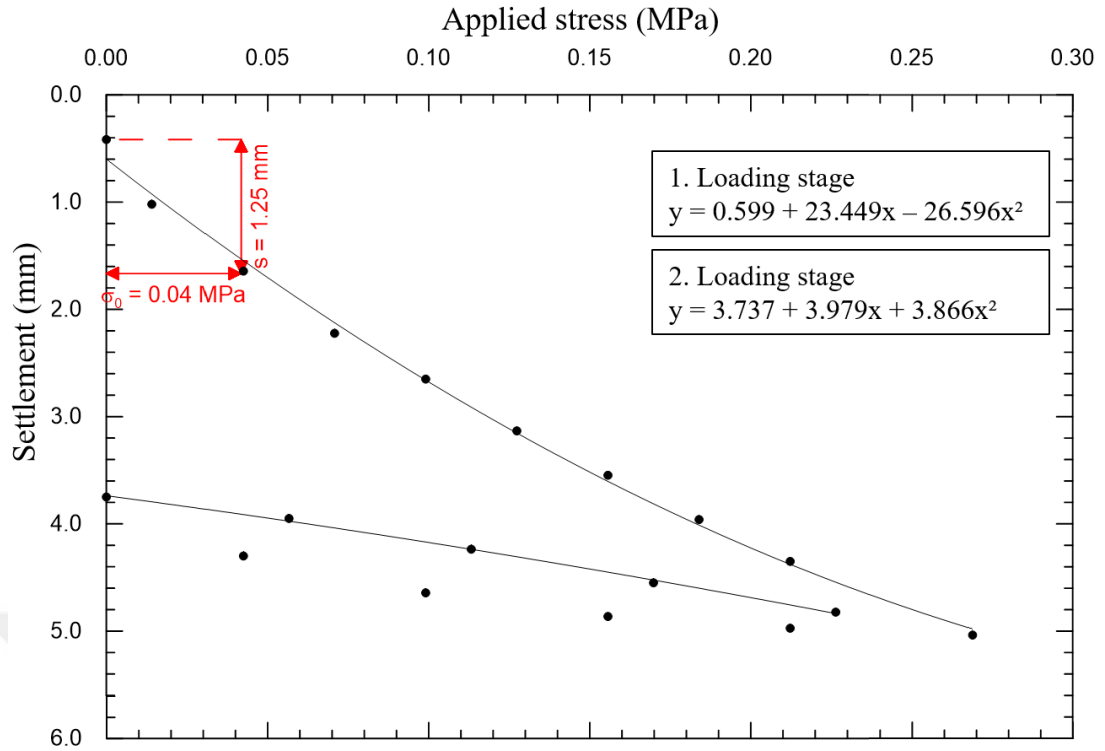


Figure A.3: The PLT#4.2 (Figure 6.1) stress-settlement curves of Configuration #4.

Table A.3: The calculations of modulus of elasticity and subgrade modulus for PLT#4.2 of Configuration #4.

Parameter	Unit	1. Loading	2. Loading
σ_{0max}	MPa	0.27	
a_0	mm	0.599	3.737
a_1	mm/MN/m ²	23.449	3.979
a_2	mm/MN ² /m ⁴	-26.596	3.866
$E_V = 1.5r \frac{1}{a_1 + a_2 \sigma_{0max}}$	MPa	13.8	44.8
E_{V2}/E_{V1}	-	3.25	
$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125m}$	MN/m ³	33.4	

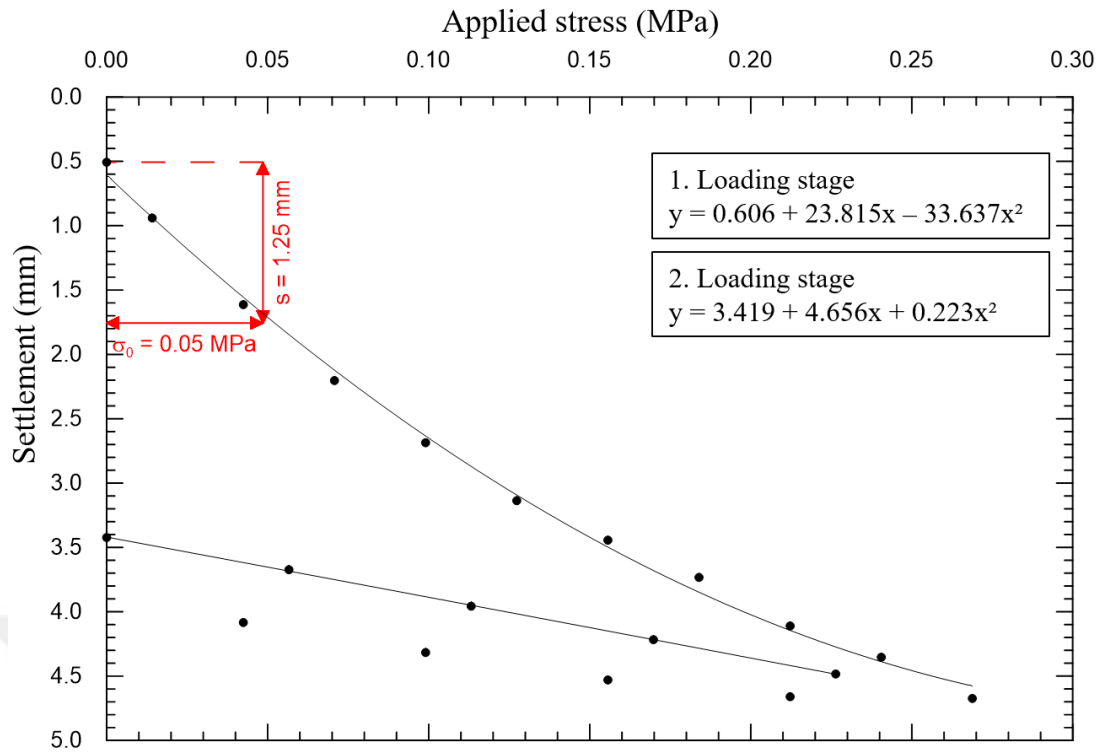


Figure A.4: The PLT#3.2 (Figure 6.1) stress-settlement curves of Configuration #3.

Table A.4: The calculations of modulus of elasticity and subgrade modulus for PLT#3.2 of Configuration #3.

Parameter	Unit	1. Loading	2. Loading
σ_{0max}	MPa	0.27	
a_0	mm	0.606	3.419
a_1	mm/MN/m ²	23.815	4.656
a_2	mm/MN ² /m ⁴	-33.637	0.223
$E_V = 1.5r \frac{1}{a_1 + a_2 \sigma_{0max}}$	MPa	15.2	47.7
E_{V2}/E_{V1}	-	3.13	
$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125m}$	MN/m ³	38.8	

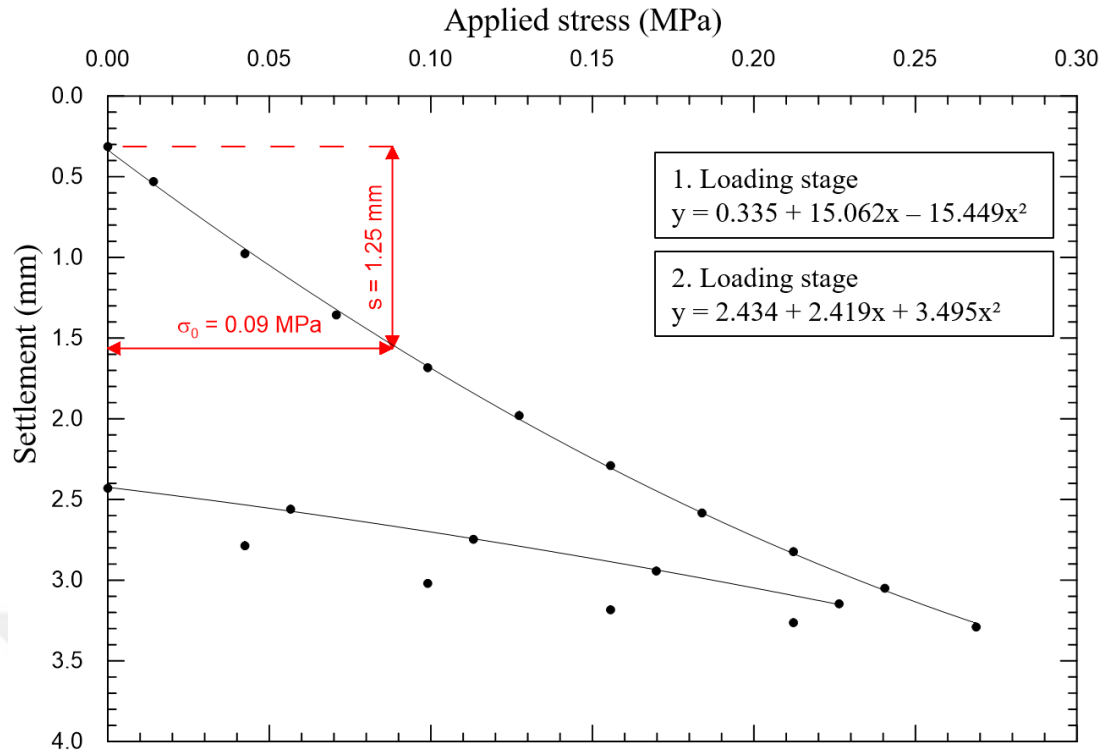


Figure A.5: The PLT#5.1 (Figure 6.1) stress-settlement curves of Configuration #5.

Table A.5: The calculations of modulus of elasticity and subgrade modulus for PLT#5.1 of Configuration #5.

Parameter	Unit	1. Loading	2. Loading
σ_{0max}	MPa	0.27	
a_0	mm	0.335	2.424
a_1	mm/MN/m ²	15.062	2.419
a_2	mm/MN ² /m ⁴	-15.449	3.495
$E_V = 1.5r \frac{1}{a_1 + a_2 \sigma_{0max}}$	MPa	20.6	67.0
E_{V2}/E_{V1}	-	3.25	
$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125m}$	MN/m ³	70.5	

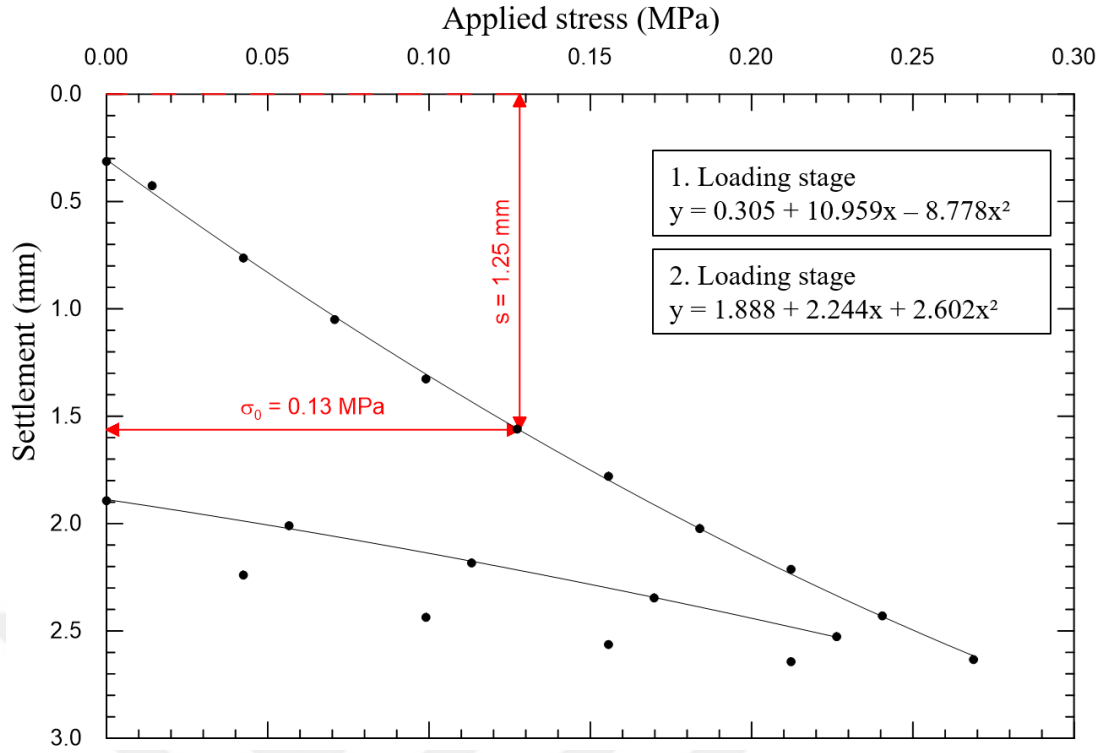


Figure A.6: The PLT#5.2 (Figure 6.1) stress-settlement curves of Configuration #5.

Table A.6: The calculations of modulus of elasticity and subgrade modulus for PLT#5.2 of Configuration #5.

Parameter	Unit	1. Loading	2. Loading
σ_{0max}	MPa	0.27	
a_0	mm	0.305	1.888
a_1	mm/MN/m ²	10.959	2.244
a_2	mm/MN ² /m ⁴	-8.778	2.602
$E_V = 1.5r \frac{1}{a_1 + a_2 \sigma_{0max}}$	MPa	26.2	76.4
E_{V2}/E_{V1}	-	2.92	
$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125m}$	MN/m ³	102.5	

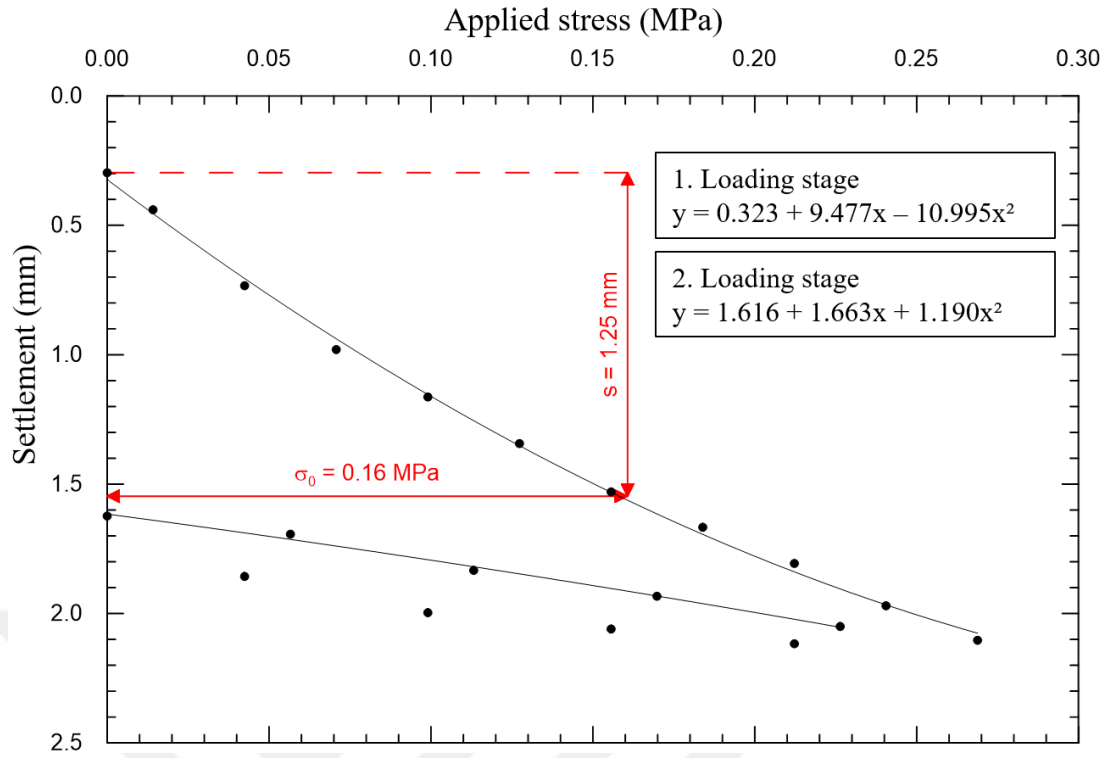


Figure A.7: The PLT#2.2 (Figure 6.1) stress-settlement curves of Configuration #2.

Table A.7: The calculations of modulus of elasticity and subgrade modulus for PLT#2.2 of Configuration #2.

Parameter	Unit	1. Loading	2. Loading
σ_{0max}	MPa	0.27	
a_0	mm	0.323	1.616
a_1	mm/MN/m ²	9.477	1.663
a_2	mm/MN ² /m ⁴	-10.995	1.190
$E_V = 1.5r \frac{1}{a_1 + a_2 \sigma_{0max}}$	MPa	34.5	113.5
E_{V2}/E_{V1}	-	3.29	
$k_s = \frac{\sigma_0}{s^*} = \frac{\sigma_0}{0.00125m}$	MN/m ³	128.6	

APPENDIX-B: FWD TEST GRAPHS

Falling weight deflectometer graphs prior accelerated pavement test:

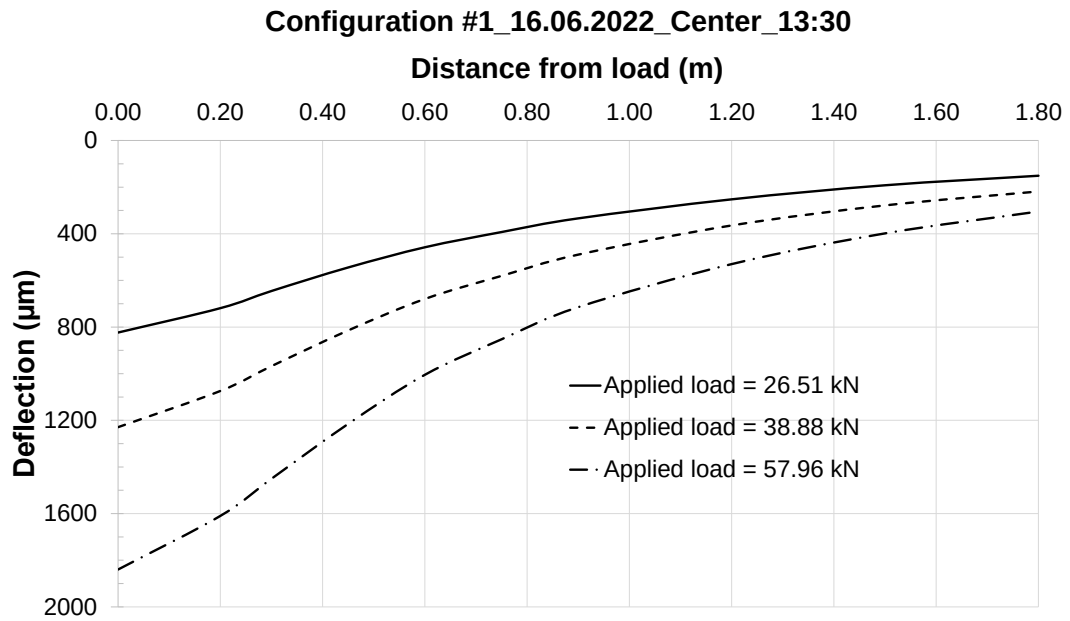


Figure B.1: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

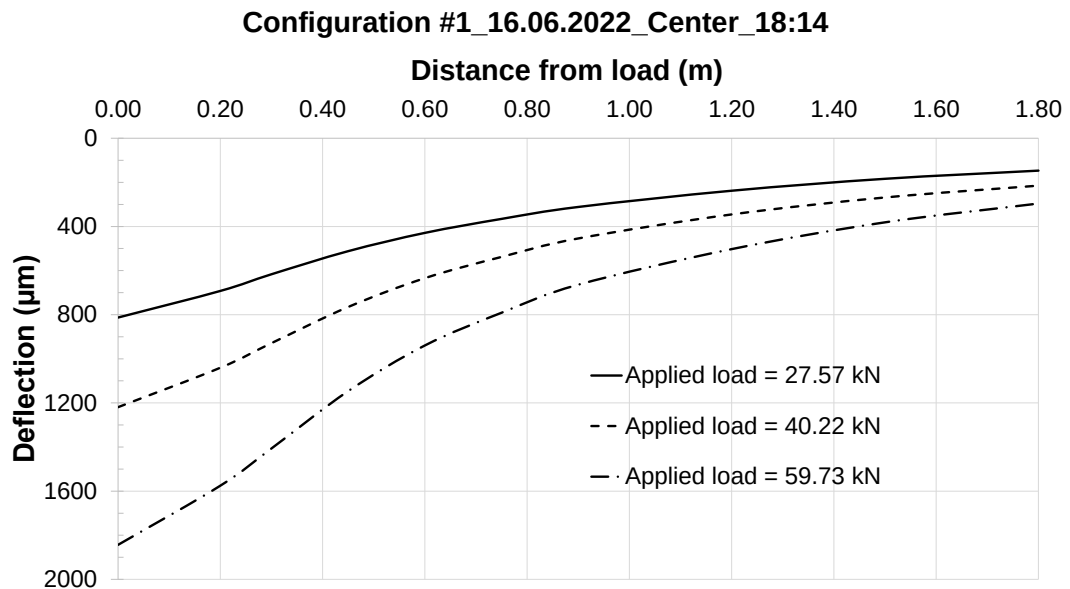


Figure B.2: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

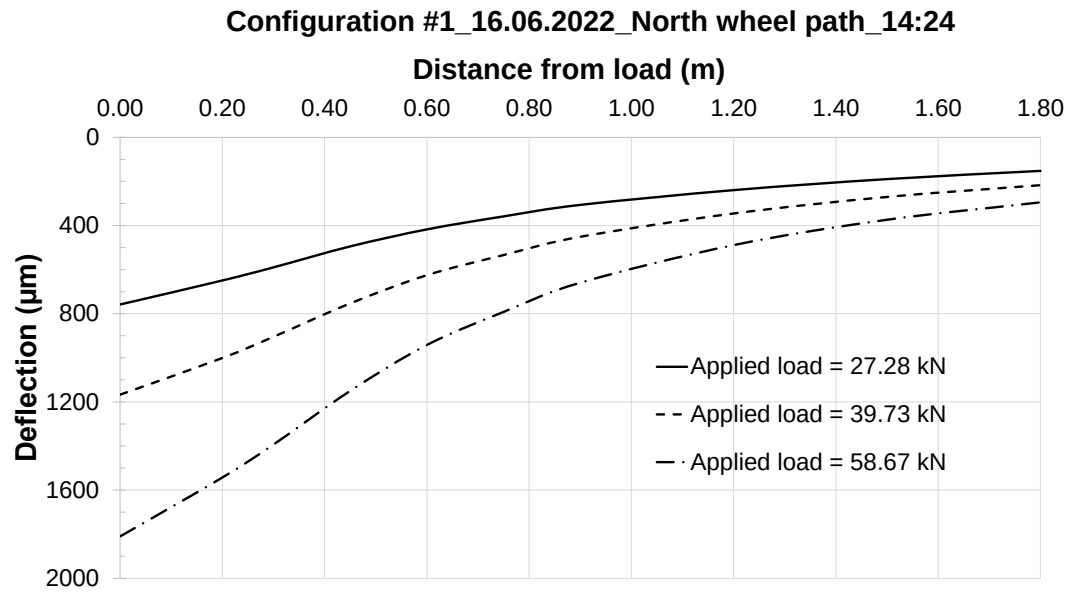


Figure B.3: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

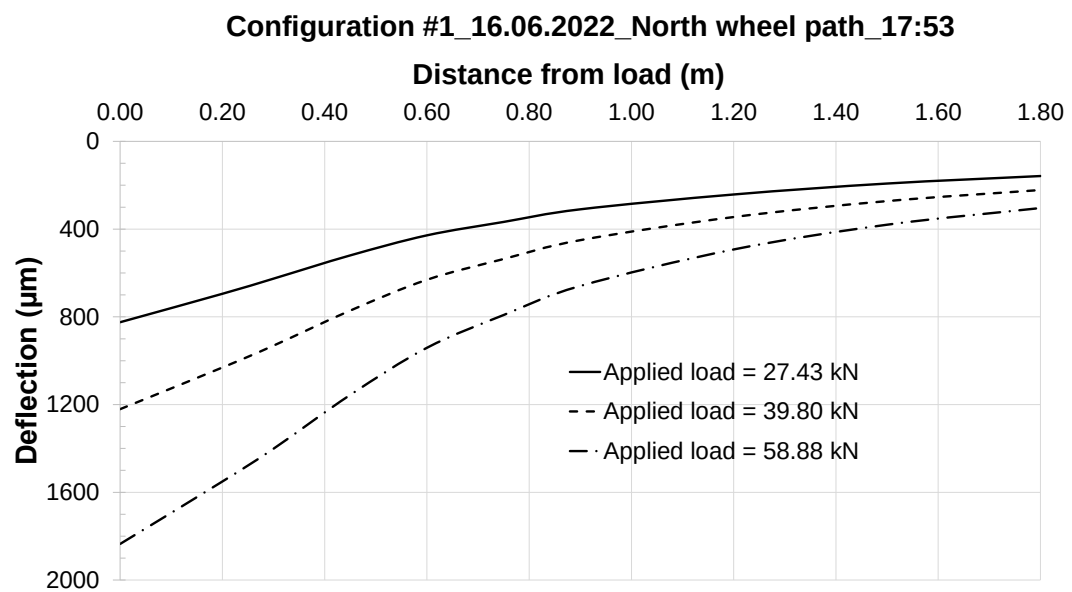


Figure B.4: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

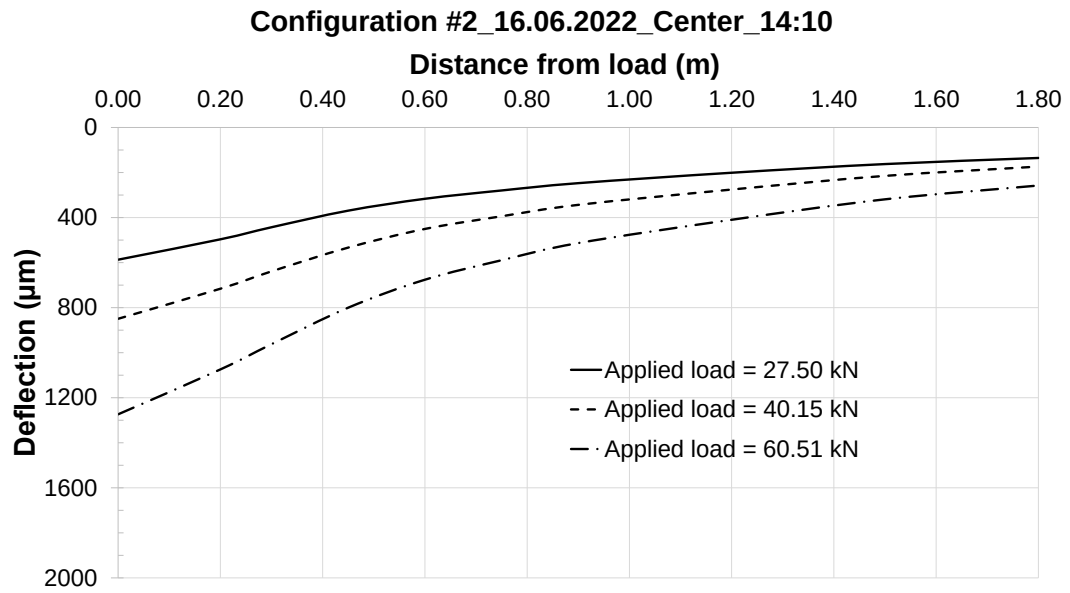


Figure B.5: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

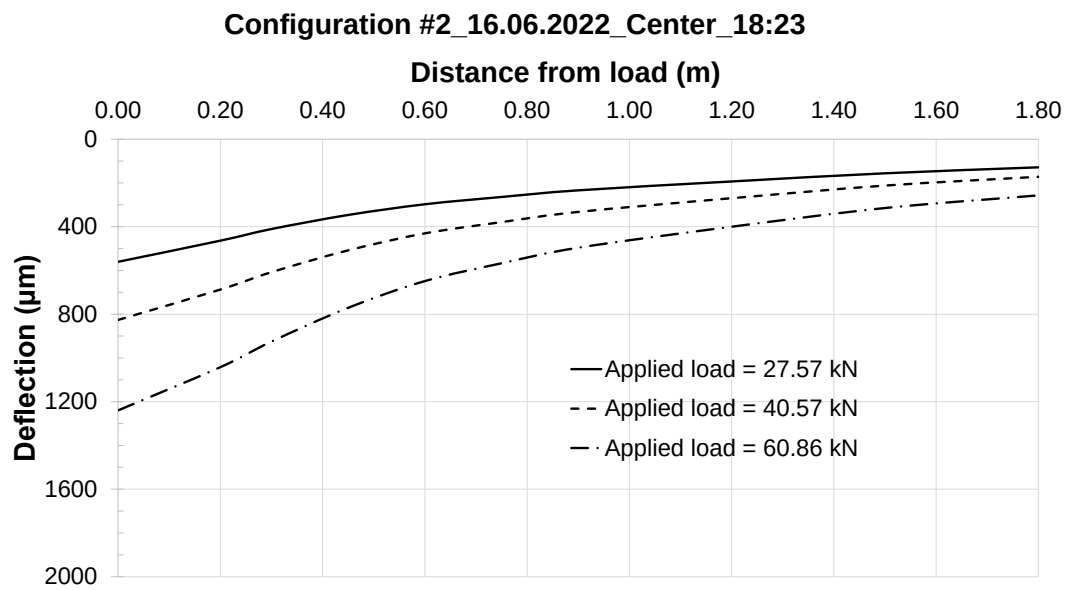


Figure B.6: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

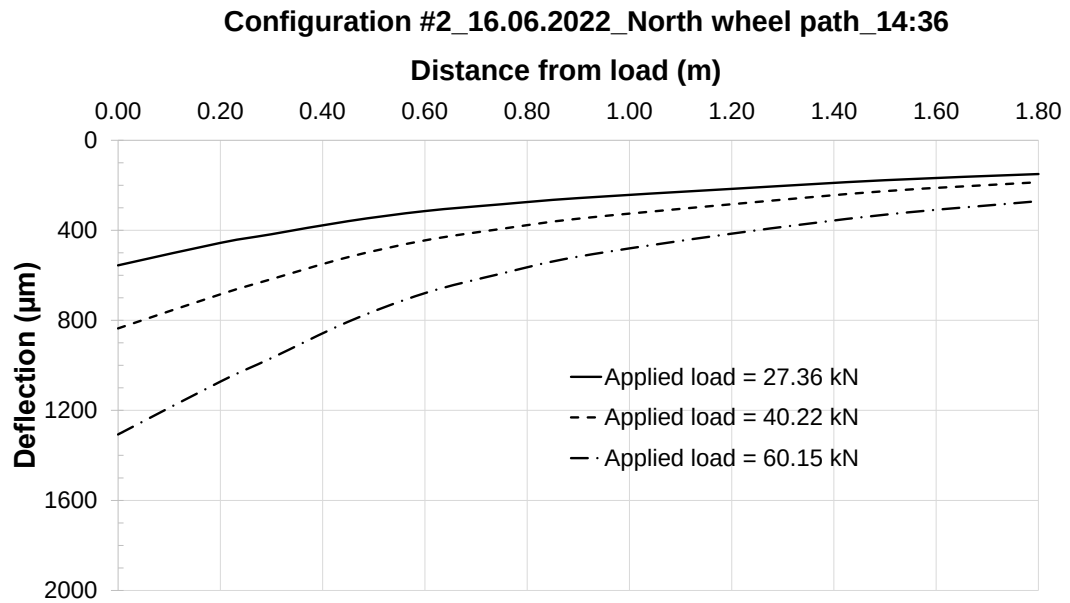


Figure B.7: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

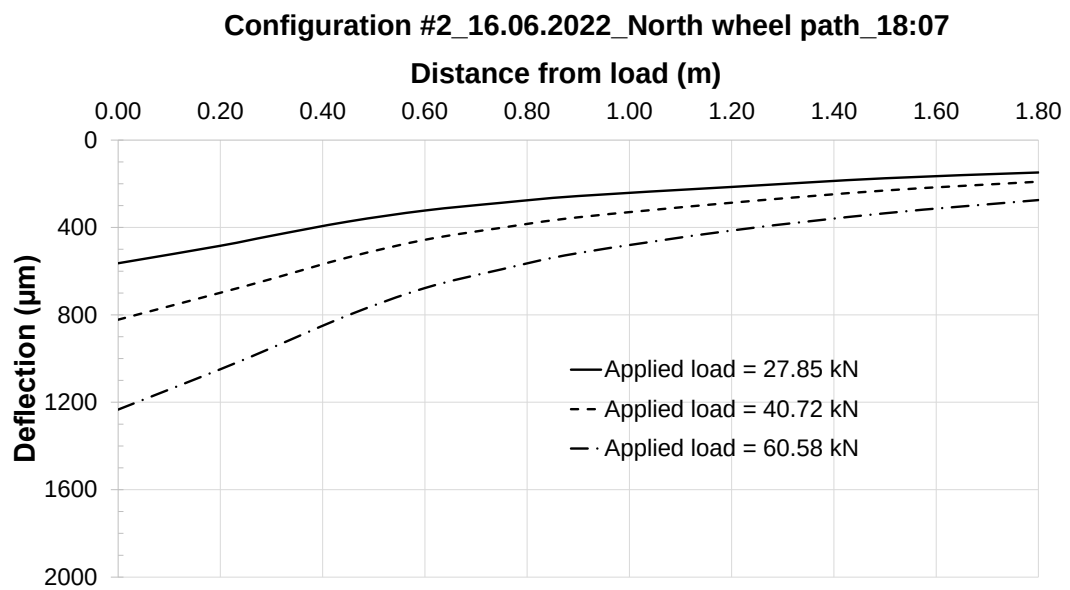


Figure B.8: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

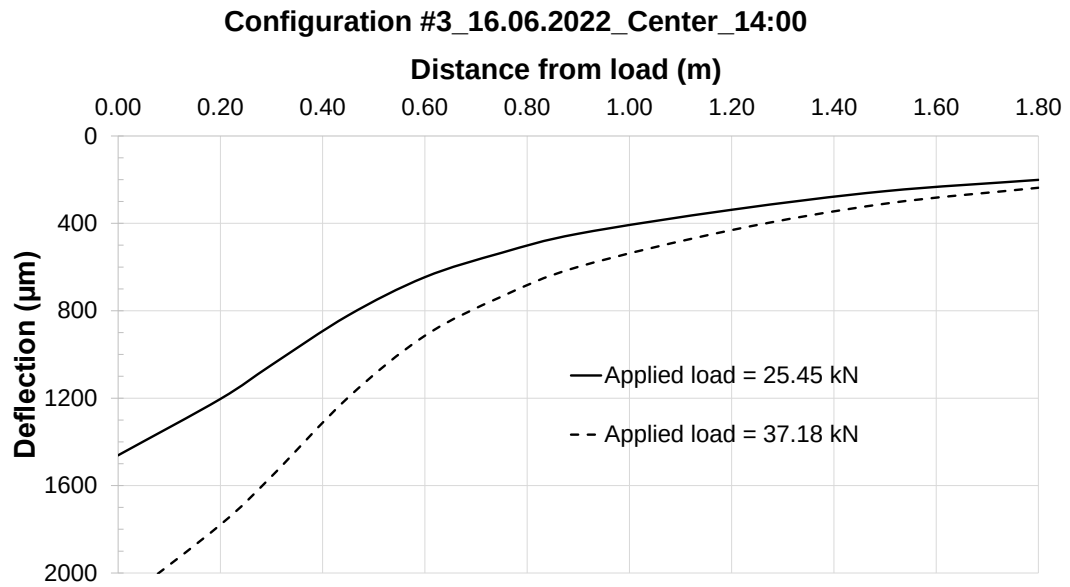


Figure B.9: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

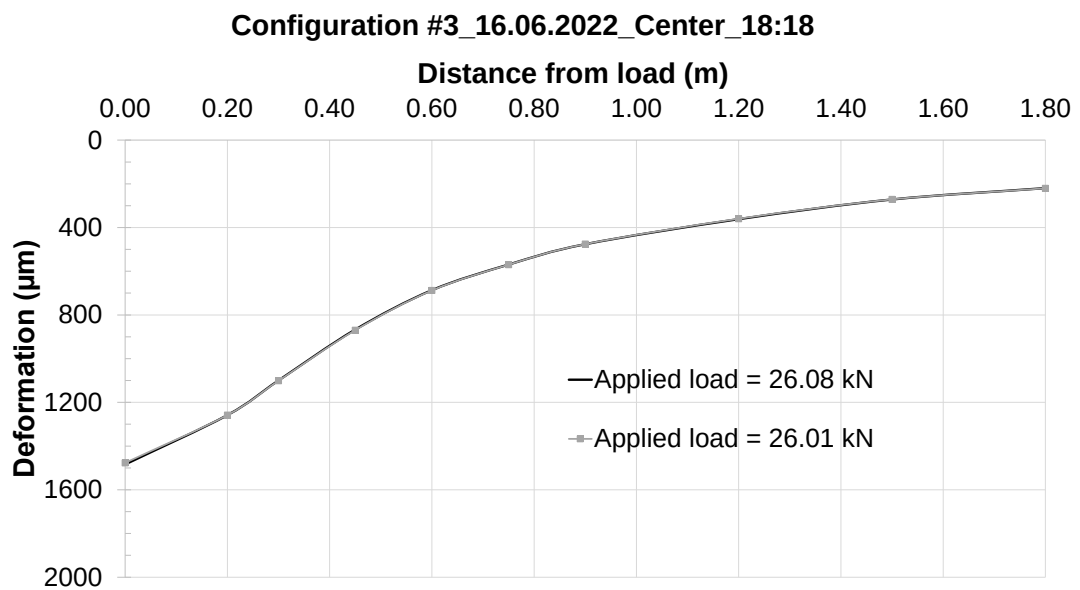


Figure B.10: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

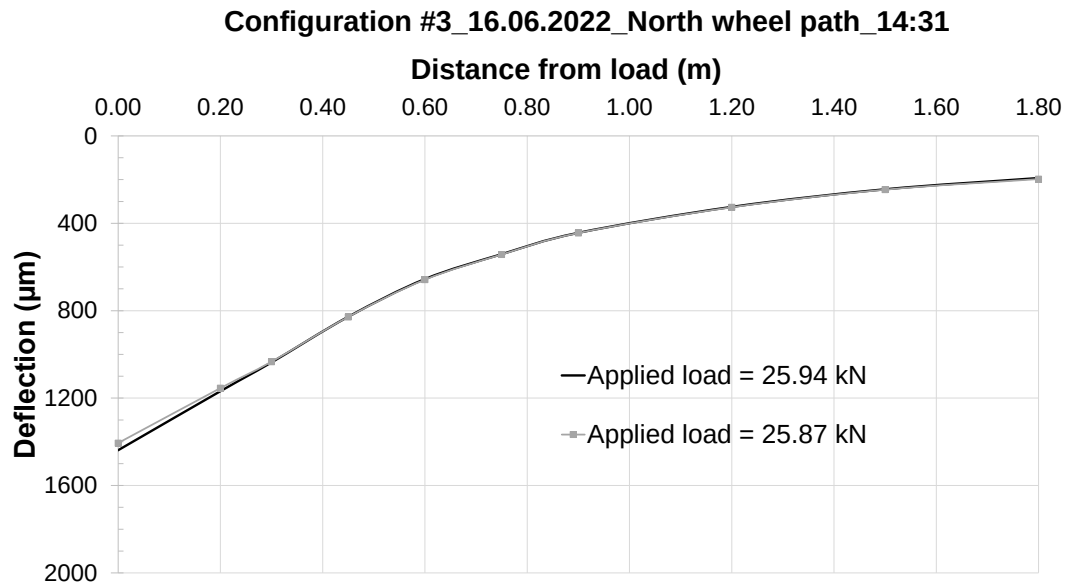


Figure B.11: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

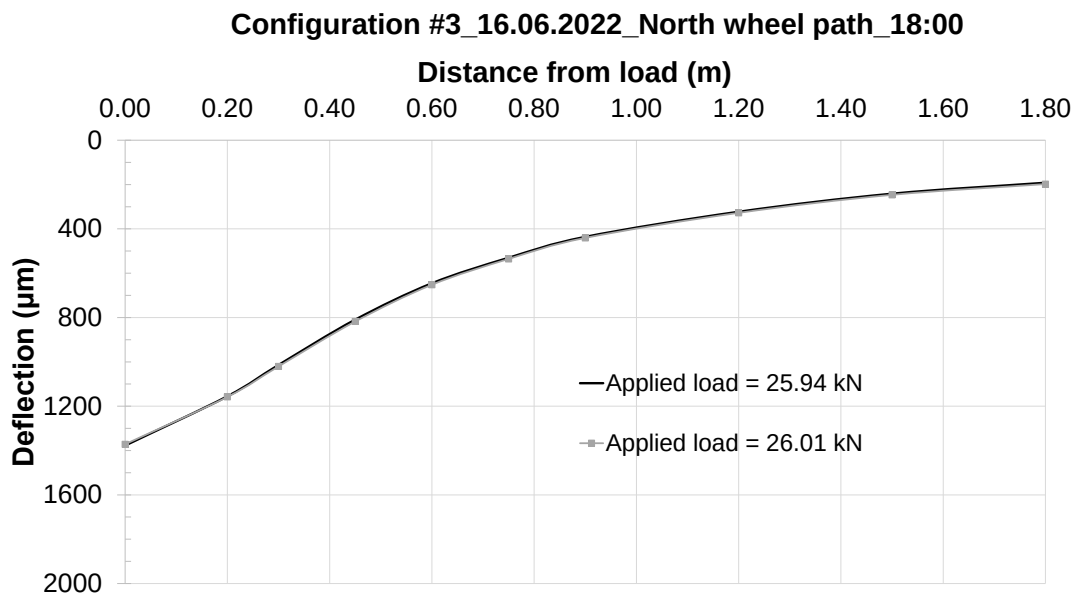


Figure B.12: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #4_16.06.2022_Center_13:50

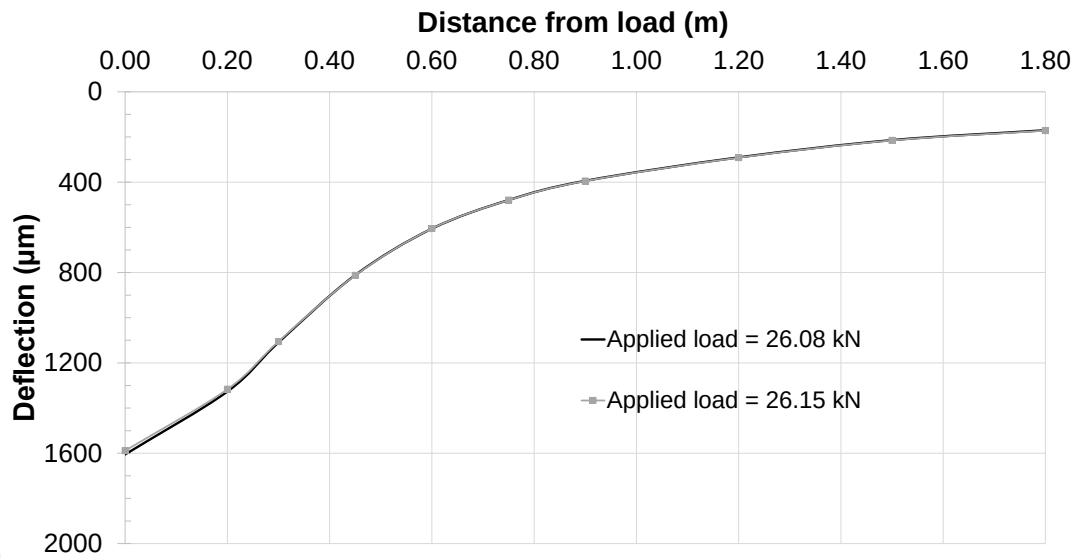


Figure B.13: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #4_16.06.2022_Center_18:16

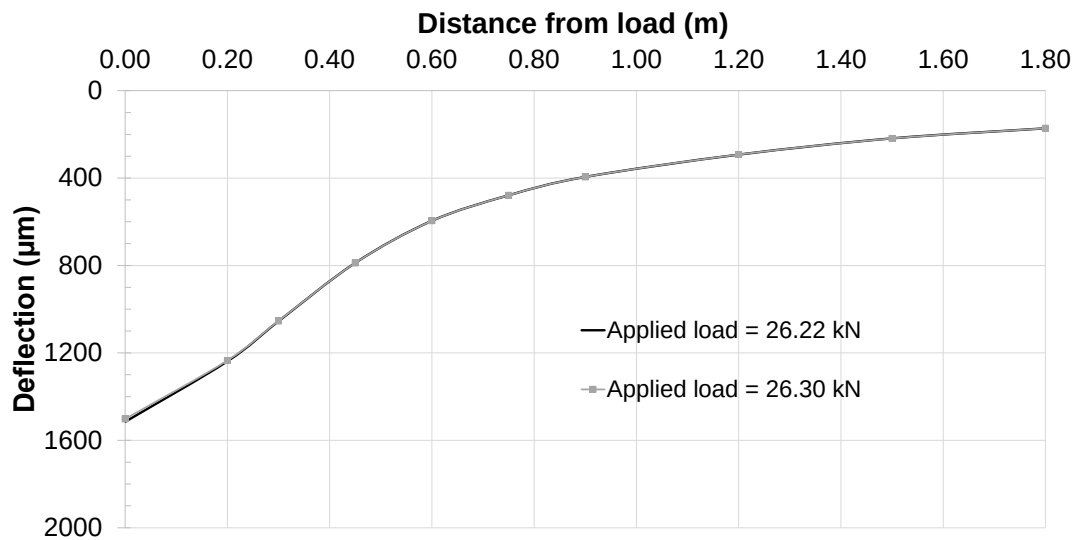


Figure B.14: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

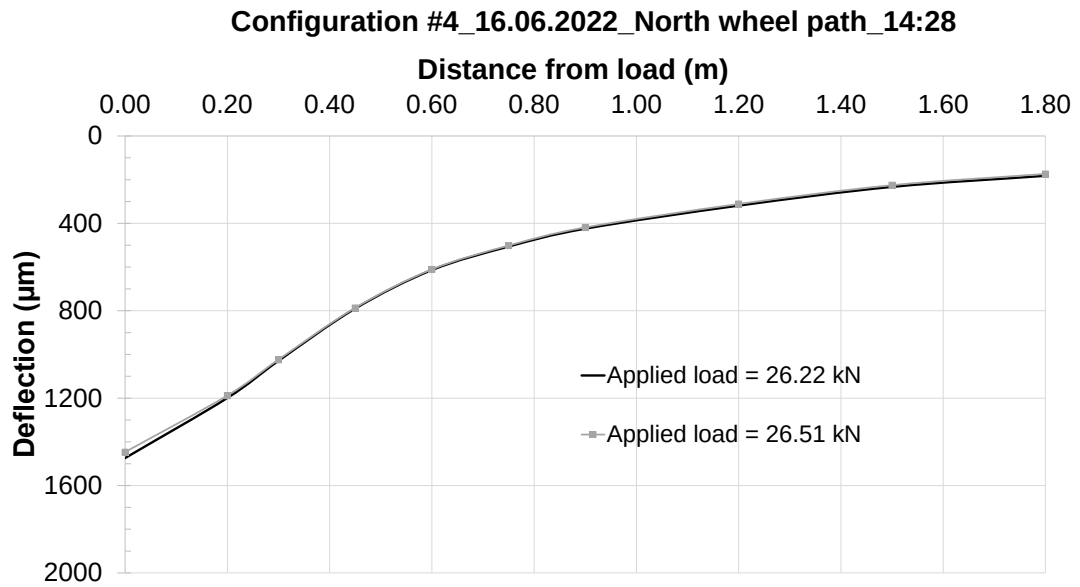


Figure B.15: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

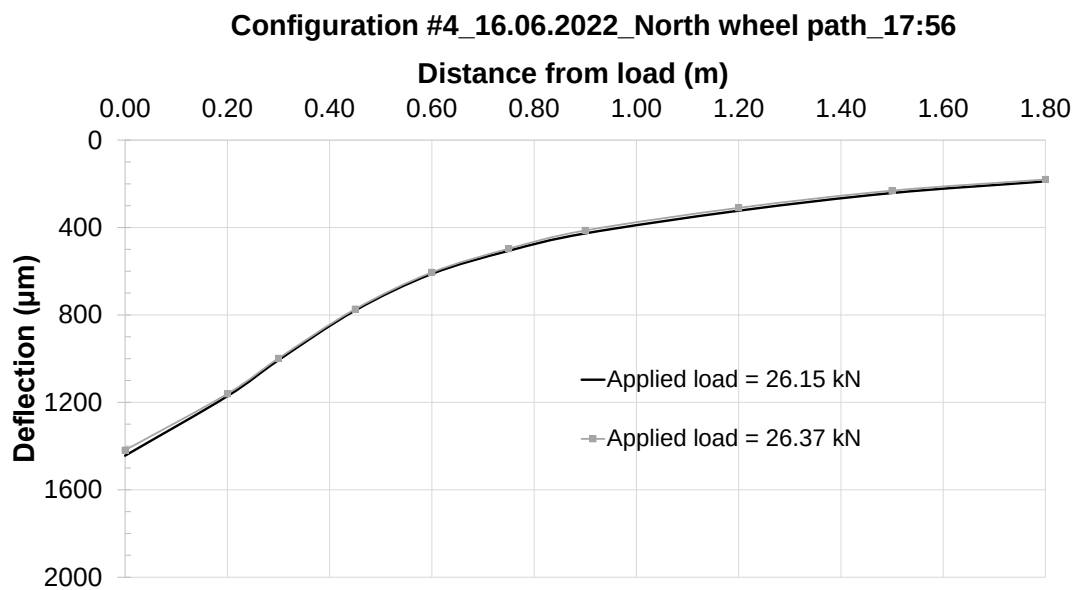


Figure B.16: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

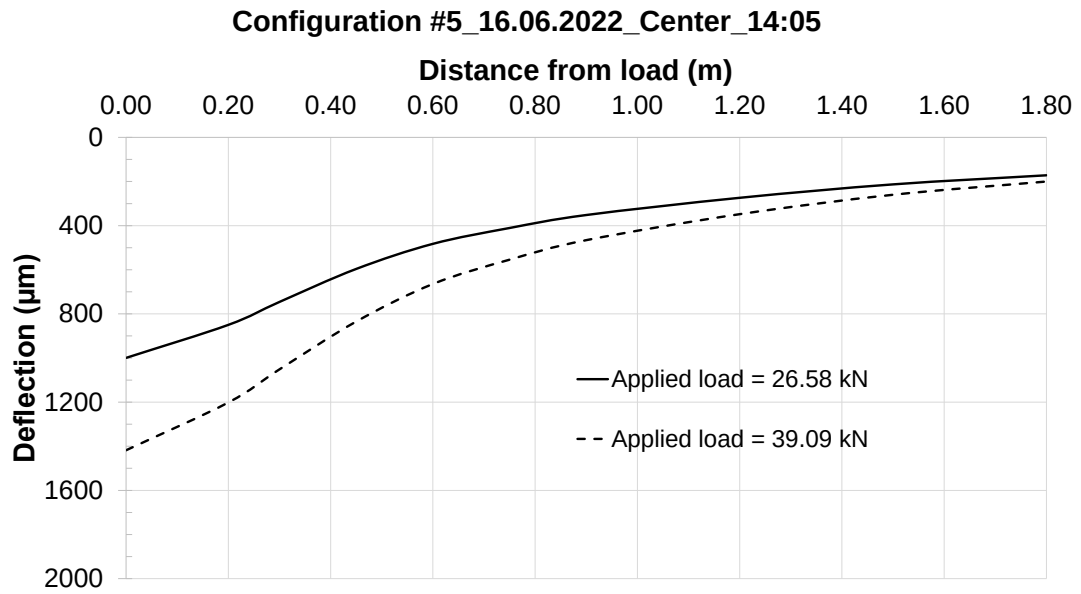


Figure B.17: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

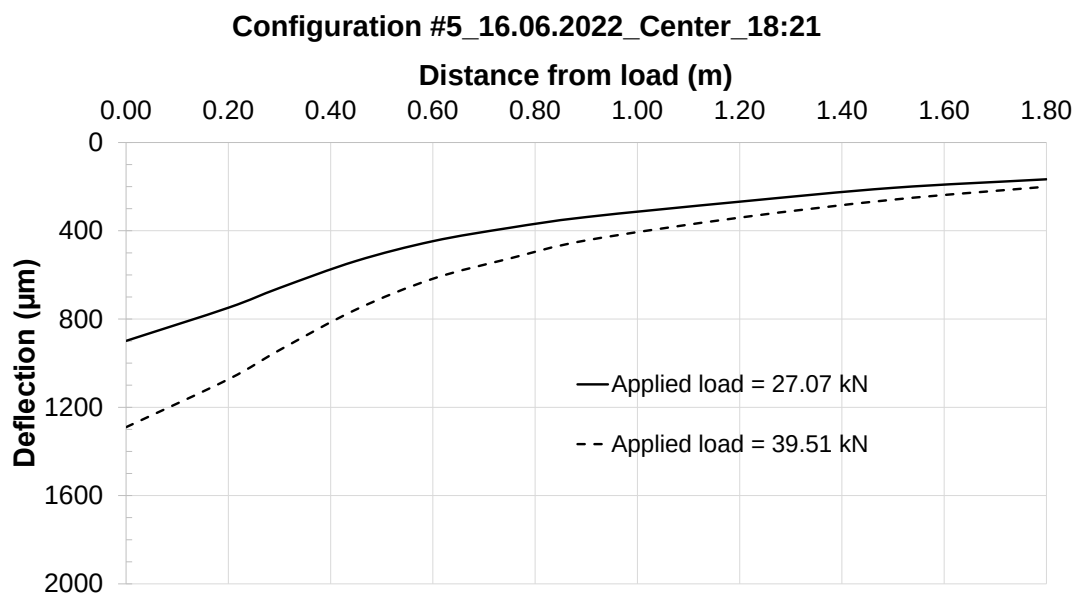


Figure B.18: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

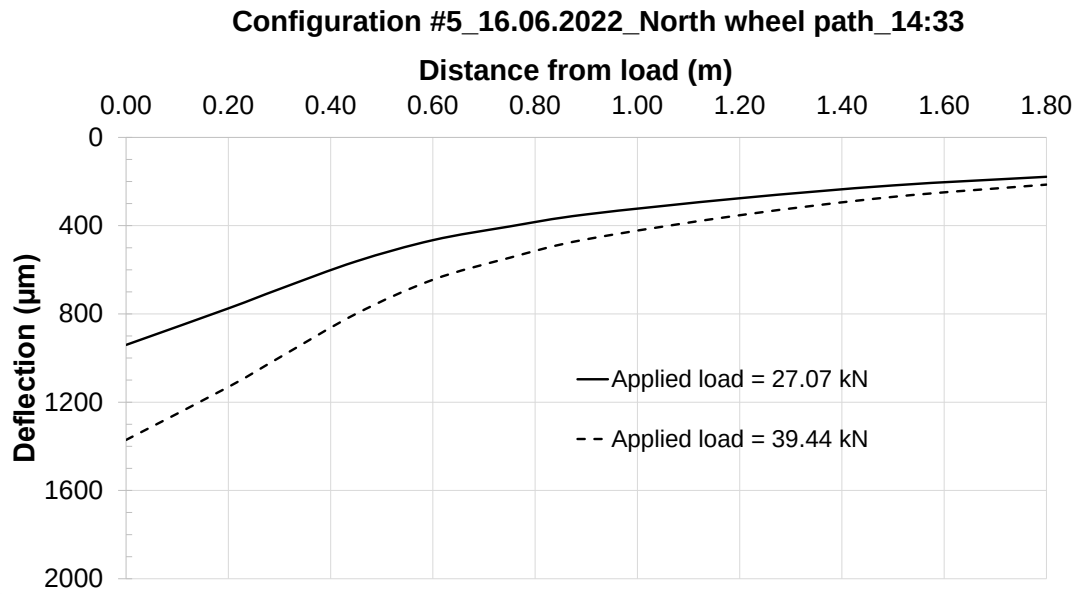


Figure B.19: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

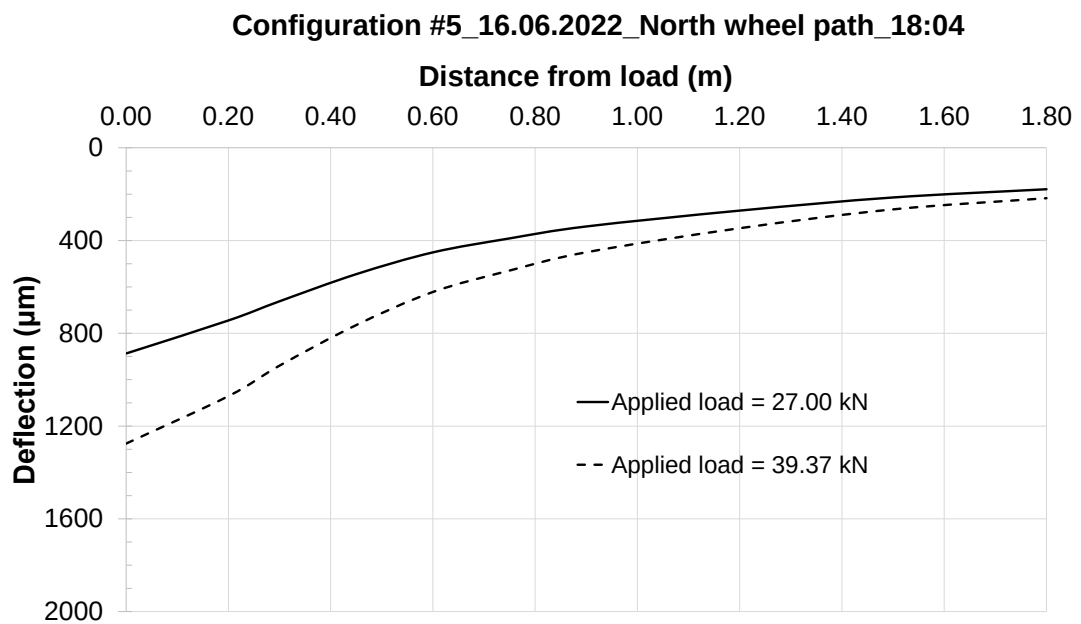


Figure B.20: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Falling weight deflectometer graphs after accelerated pavement test:

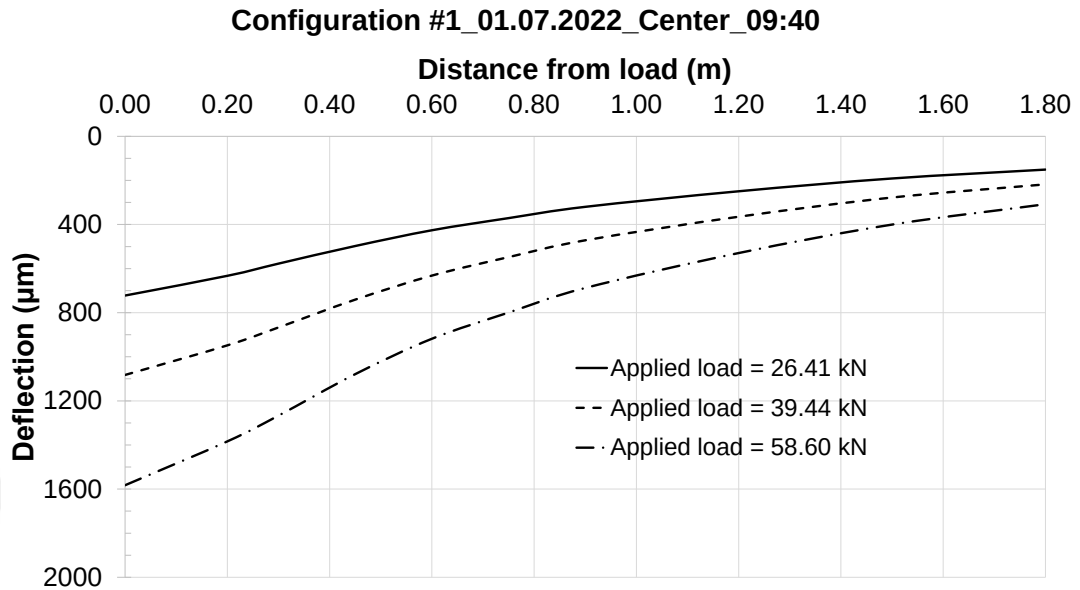


Figure B.21: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

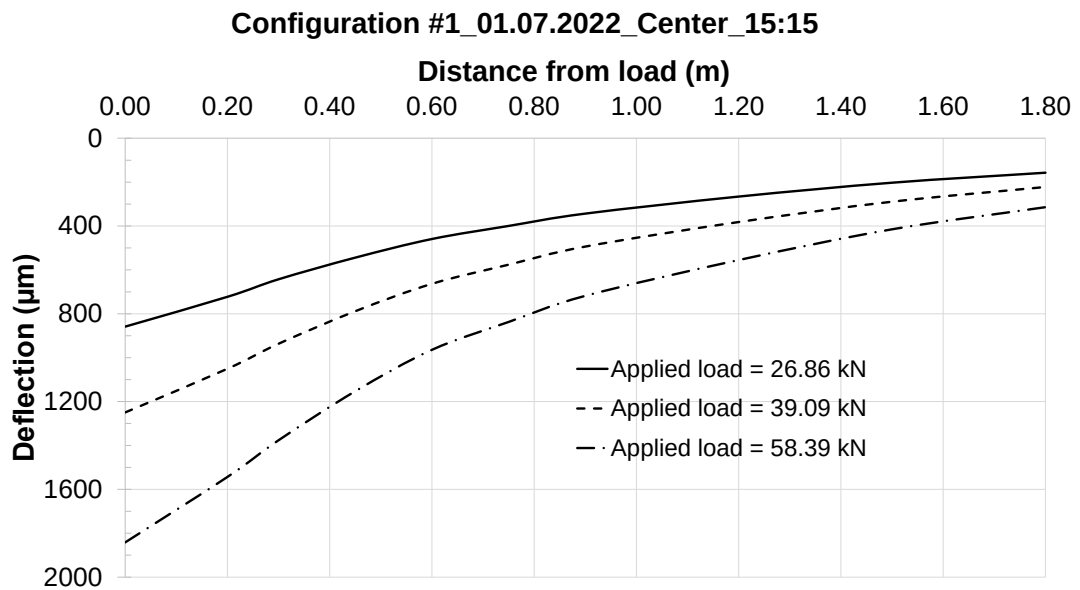


Figure B.22: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

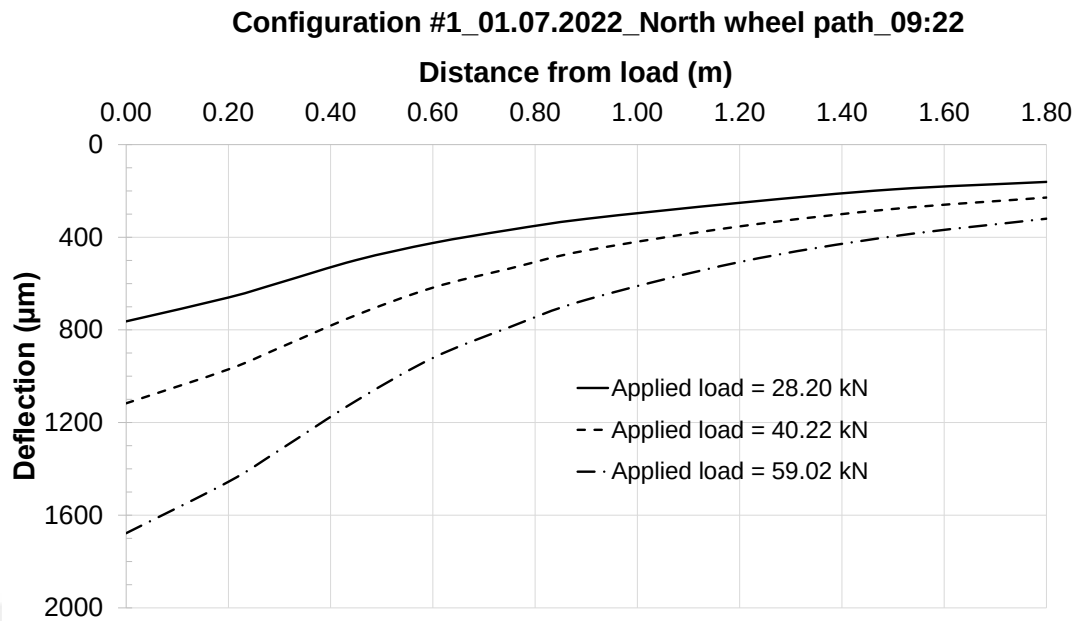


Figure B.23: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

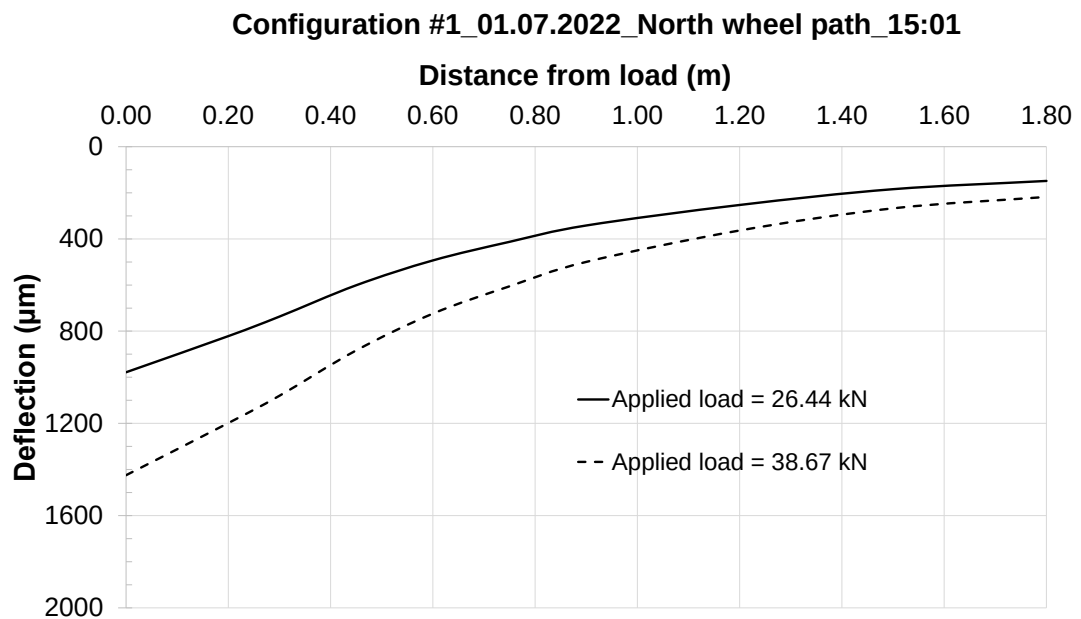


Figure B.24: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

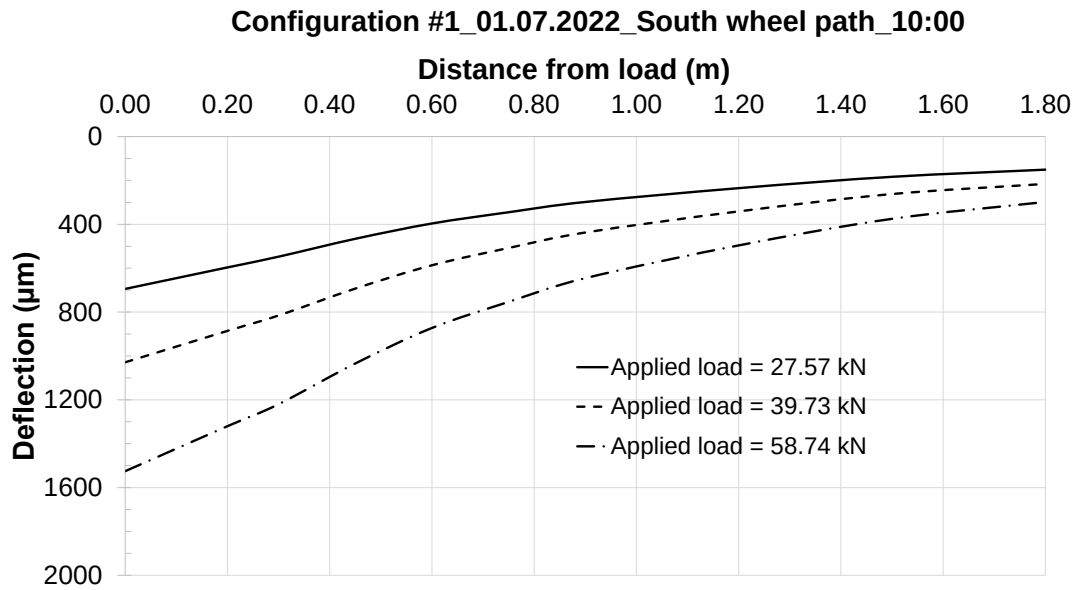


Figure B.25: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

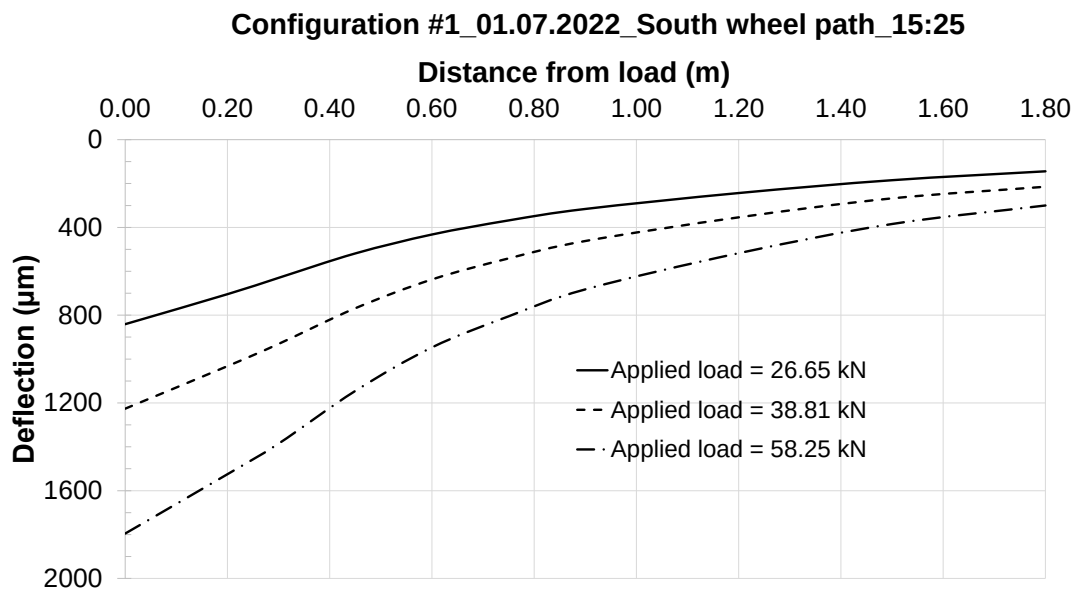


Figure B.26: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

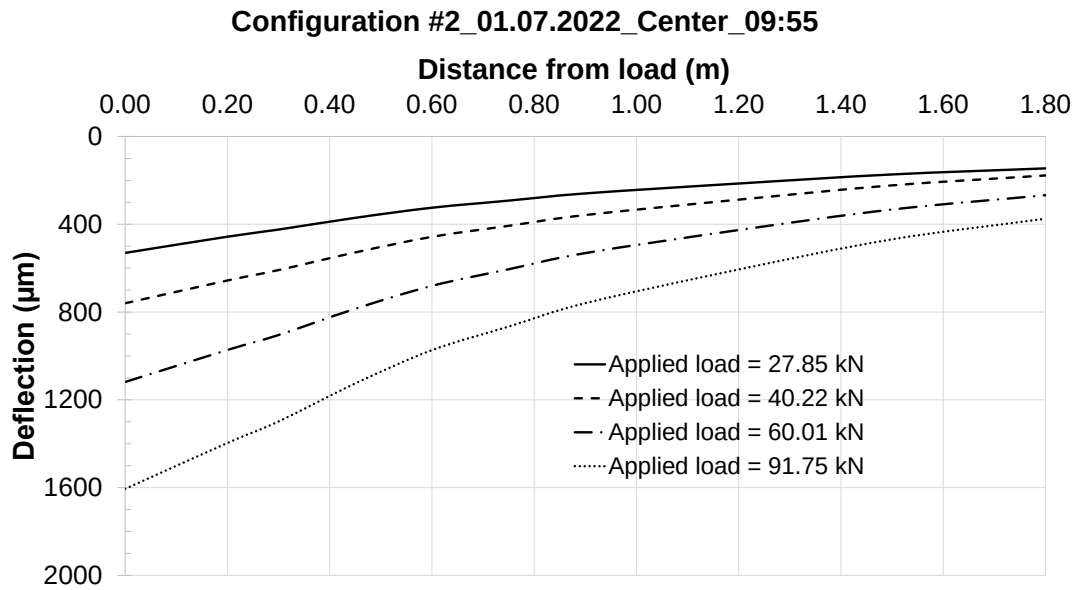


Figure B.27: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

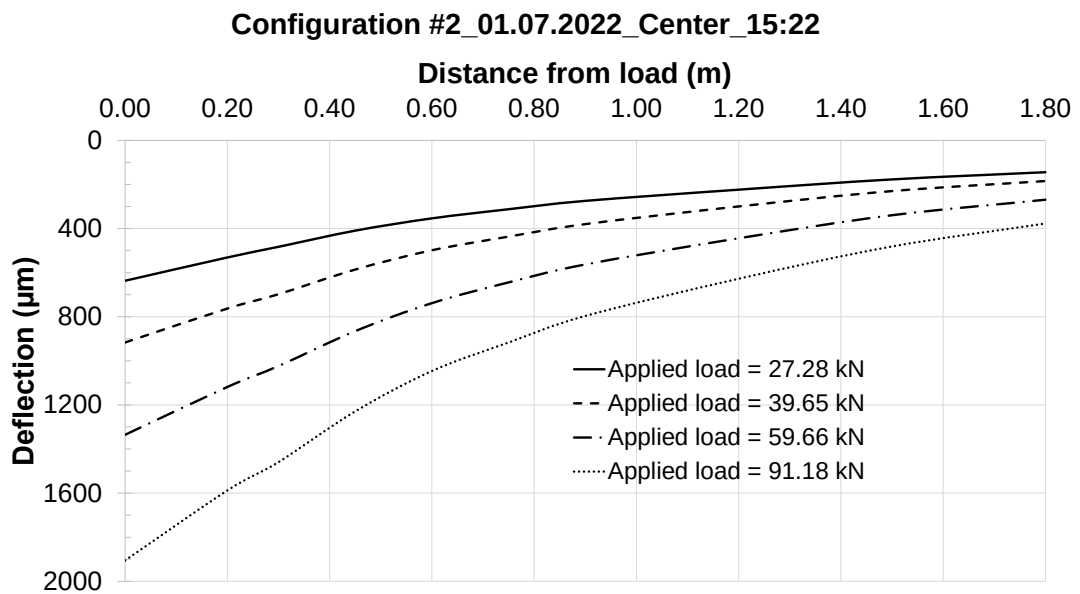


Figure B.28: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

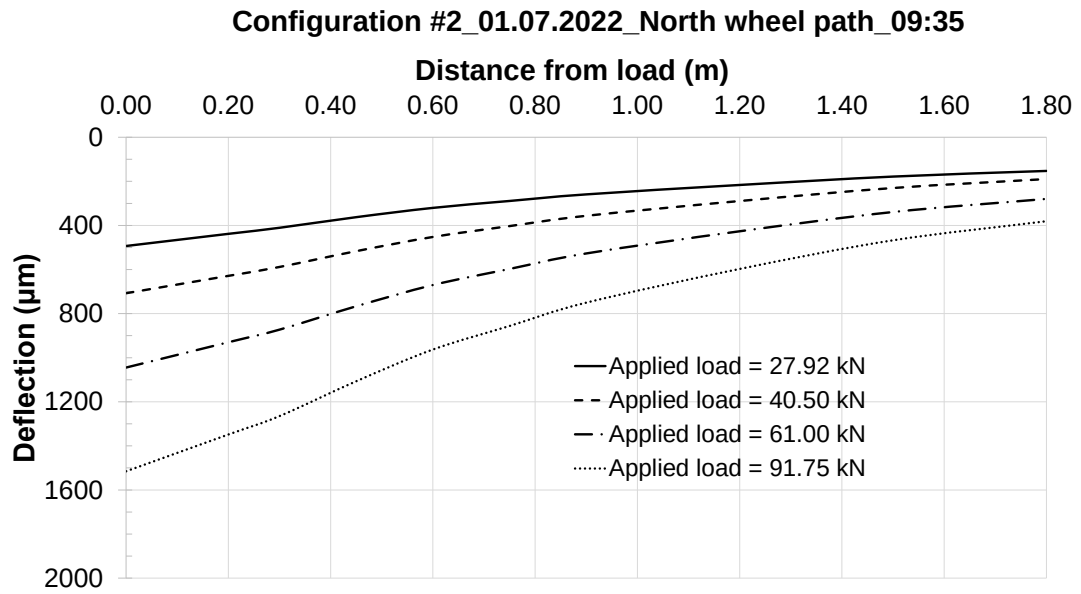


Figure B.29: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

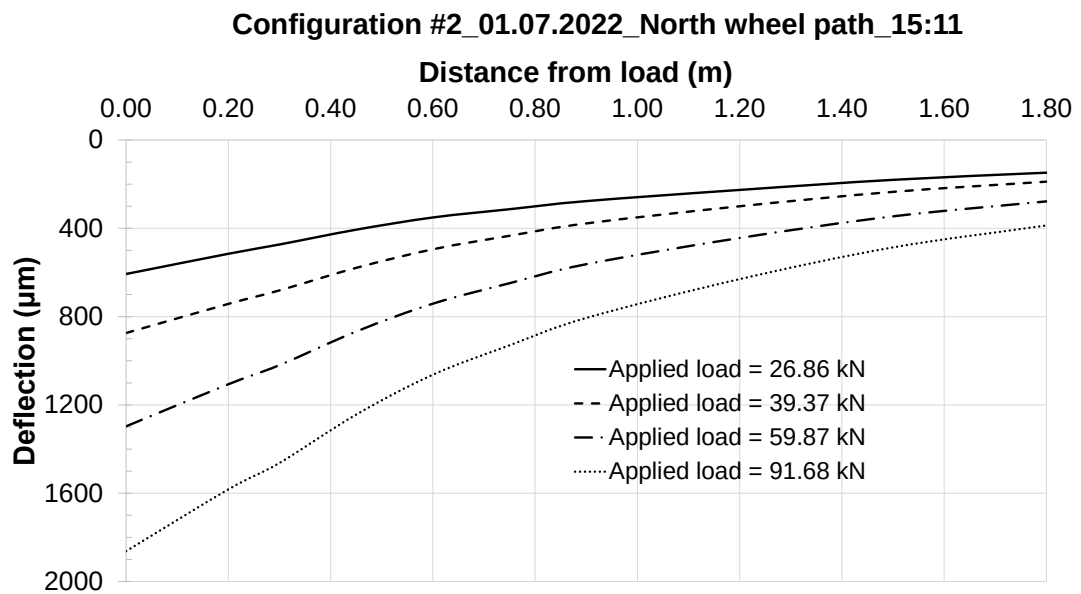


Figure B.30: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

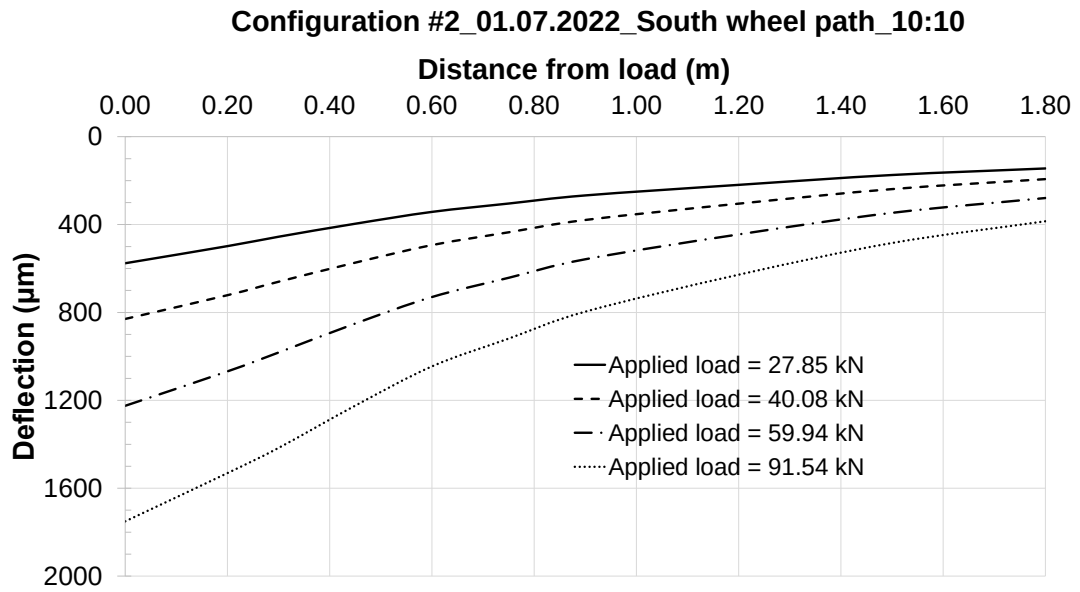


Figure B.31: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

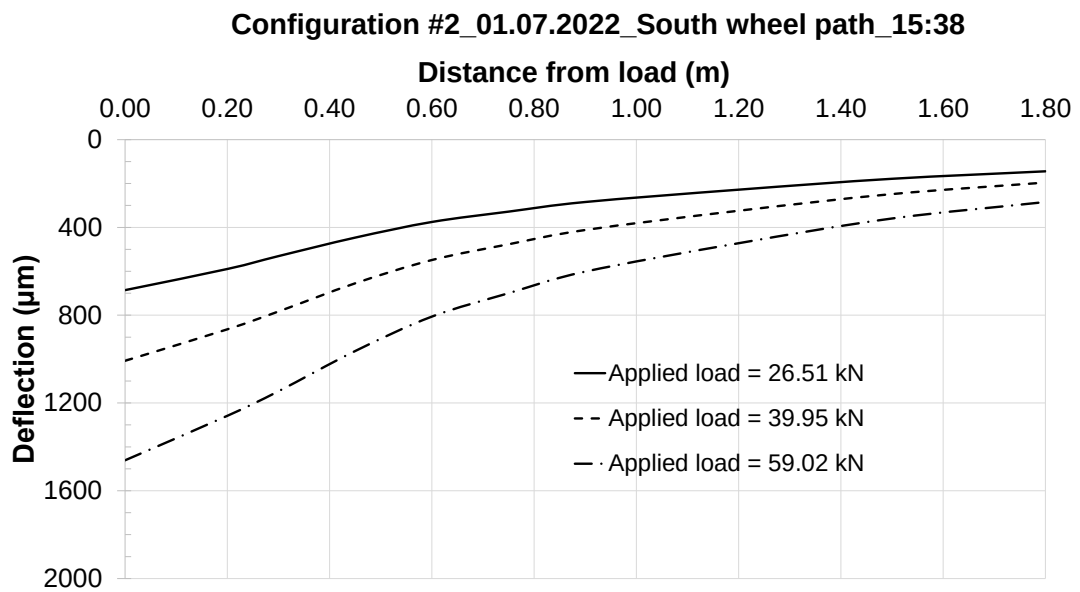


Figure B.32: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

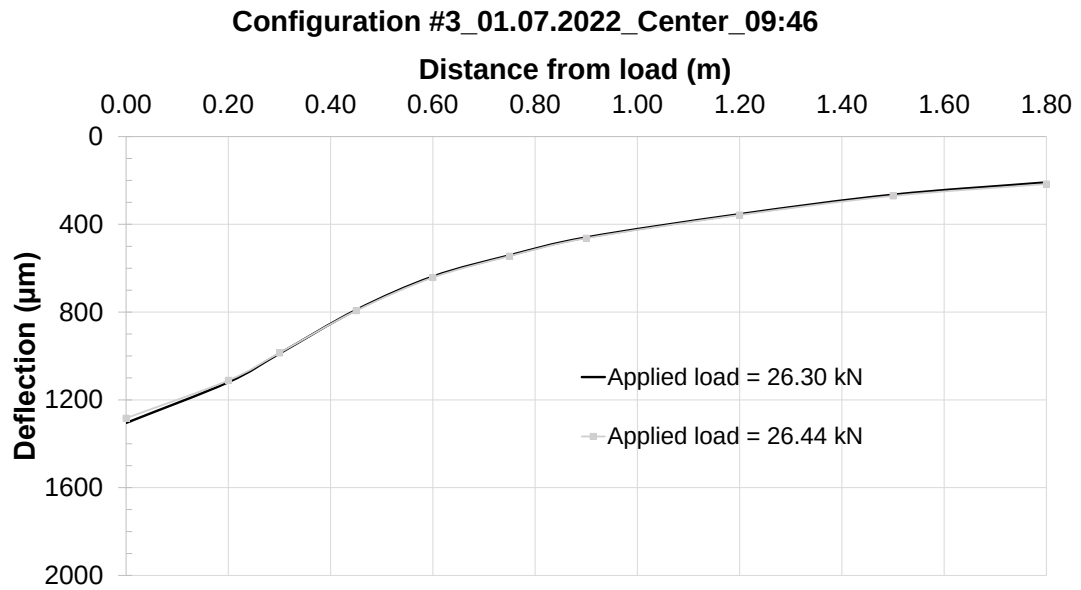


Figure B.33: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

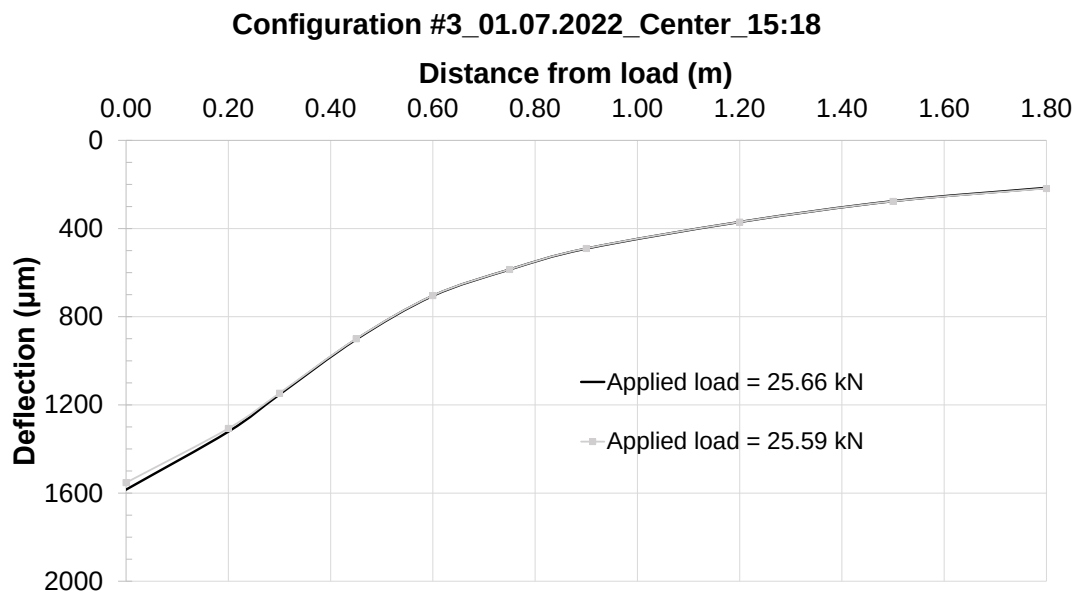


Figure B.34: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

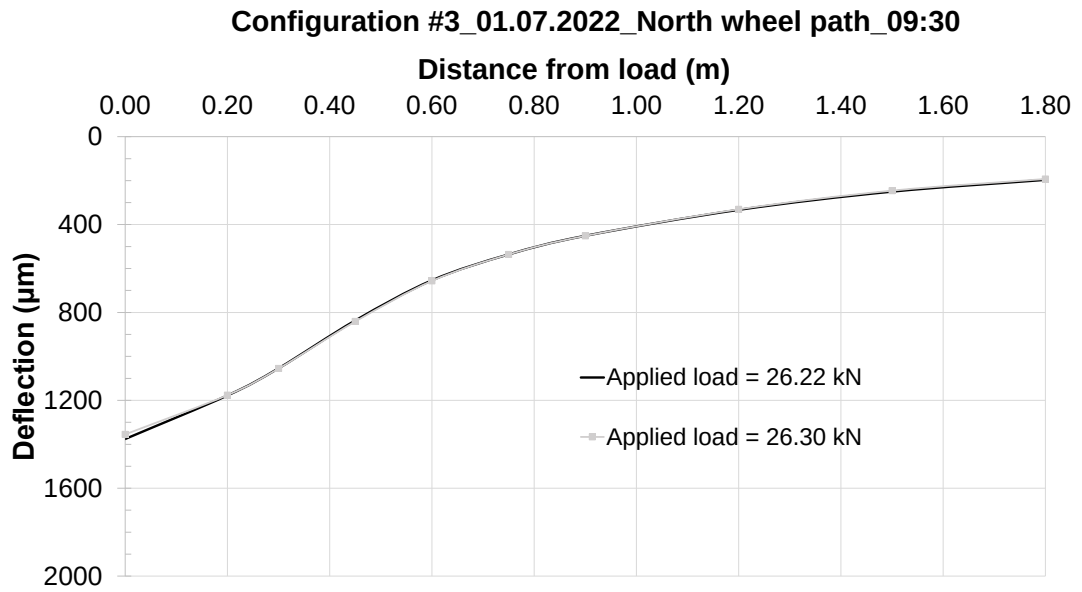


Figure B.35: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

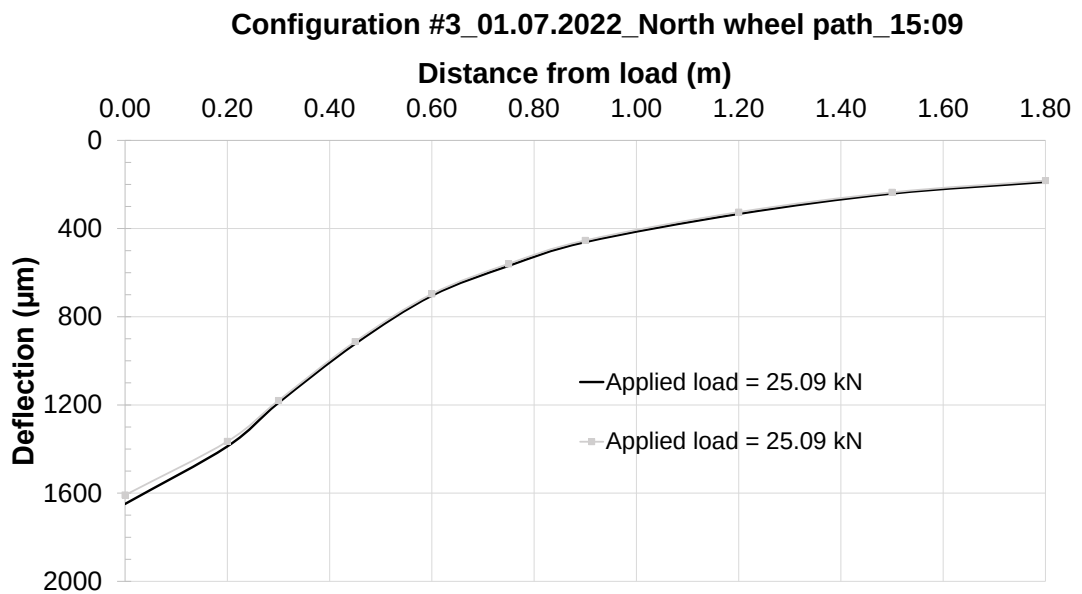


Figure B.36: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #3_01.07.2022_South wheel path_10:06

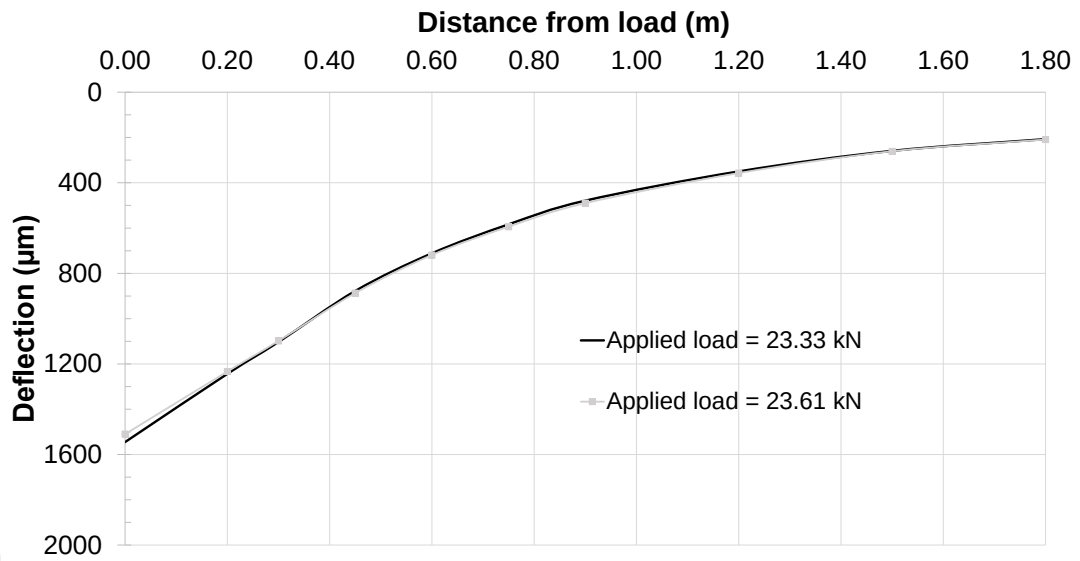


Figure B.37: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #3_01.07.2022_South wheel path_15:30

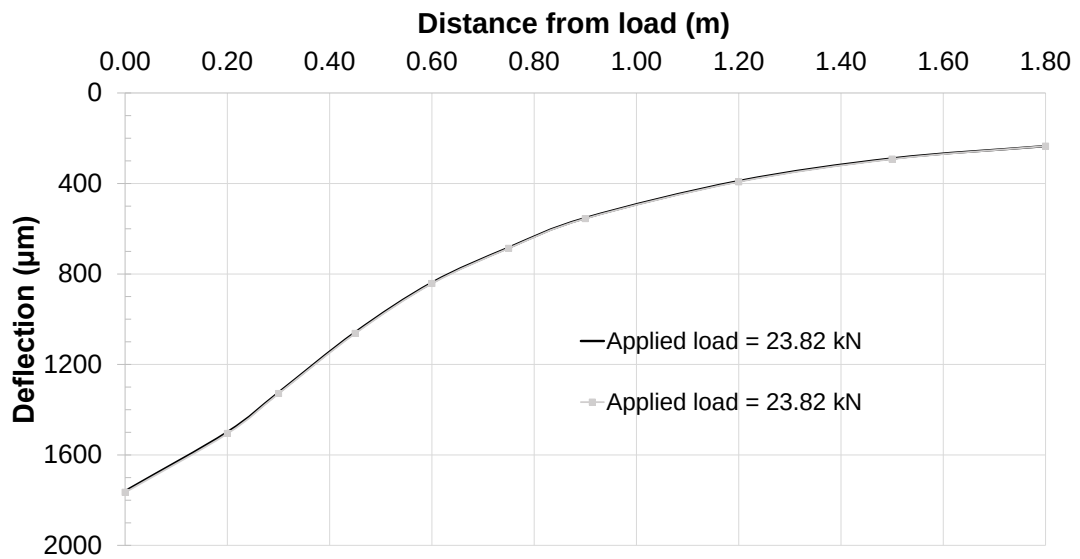


Figure B.38: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #4_01.07.2022_Center_09:44

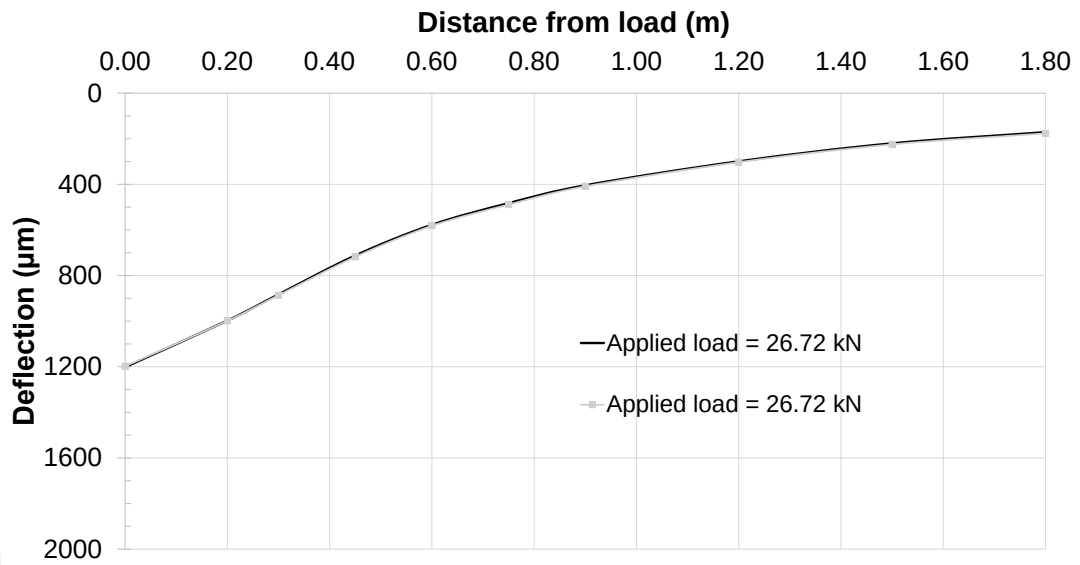


Figure B.39: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #4_01.07.2022_Center_15:17

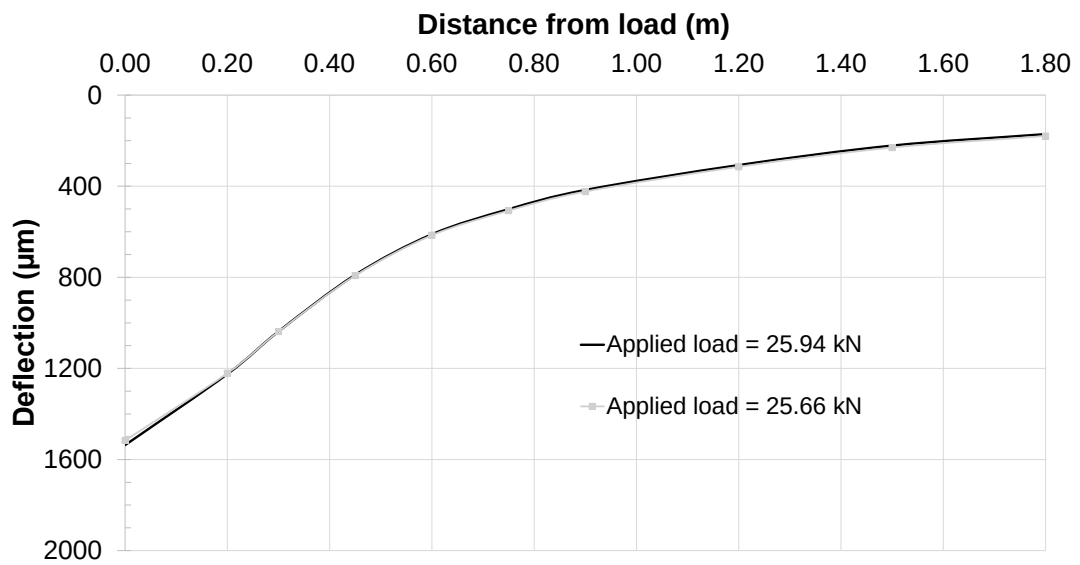


Figure B.40: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #4_01.07.2022_North wheel path_09:26

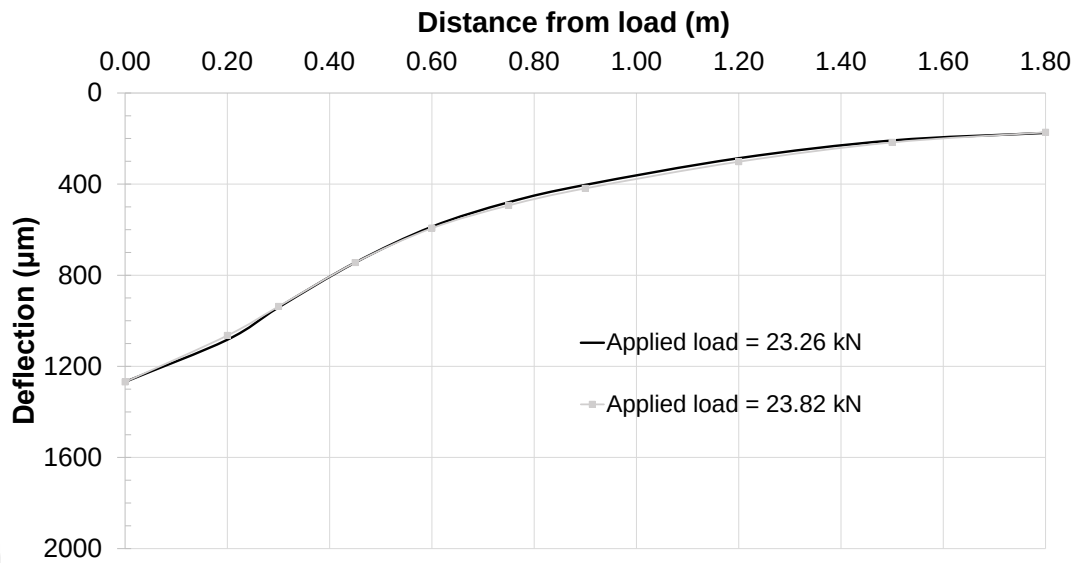


Figure B.41: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #4_01.07.2022_North wheel path_15:07

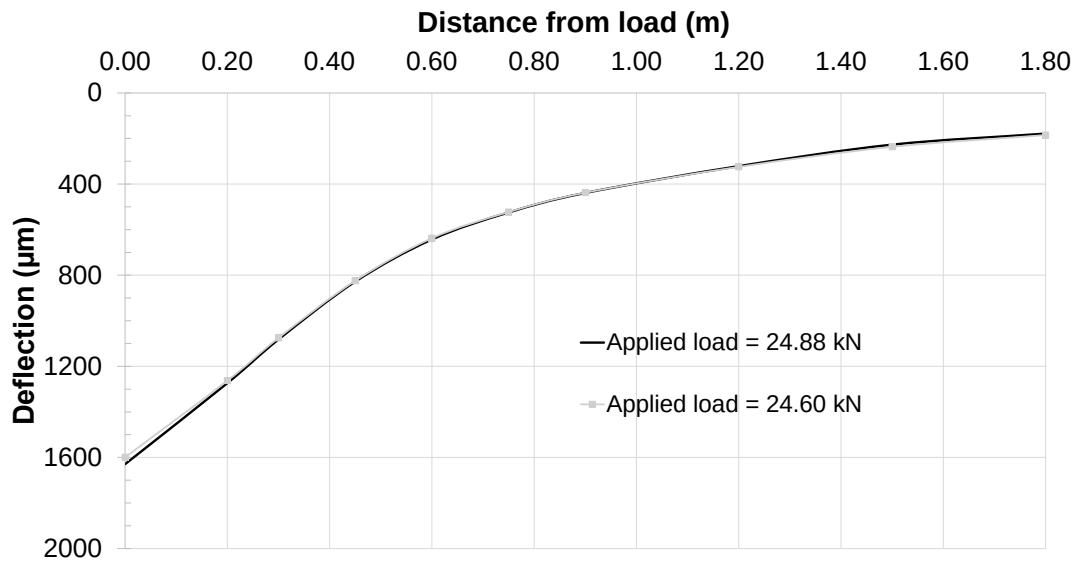


Figure B.42: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #4_01.07.2022_South wheel path_10:03

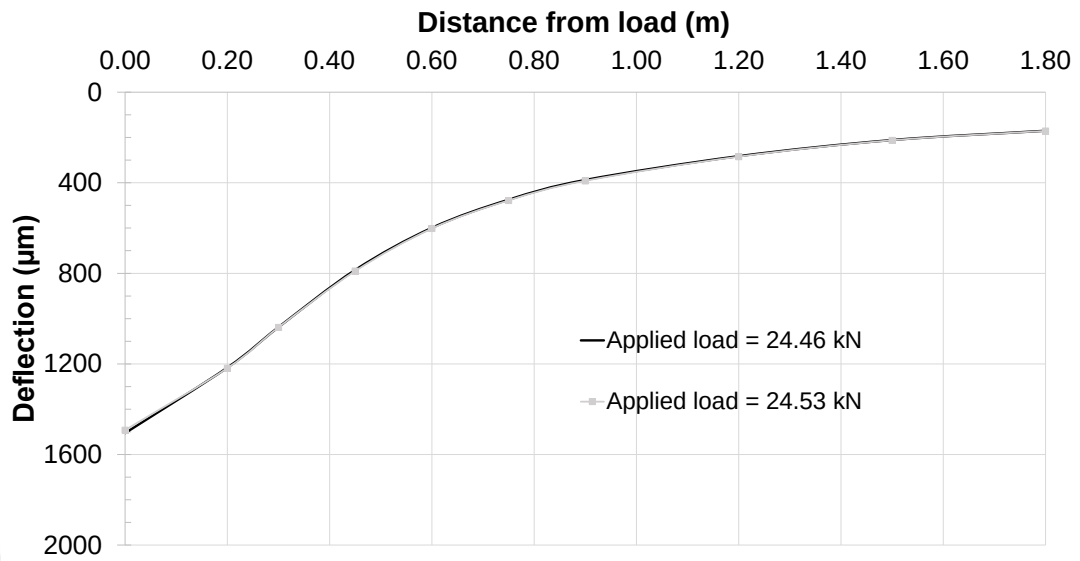


Figure B.43: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #4_01.07.2022_South wheel path_15:30

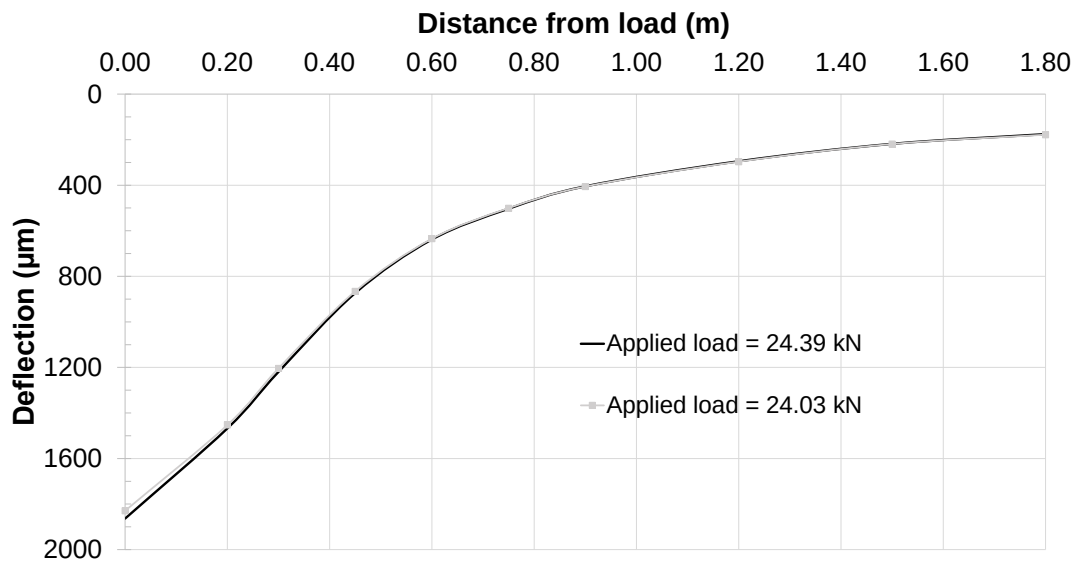


Figure B.44: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

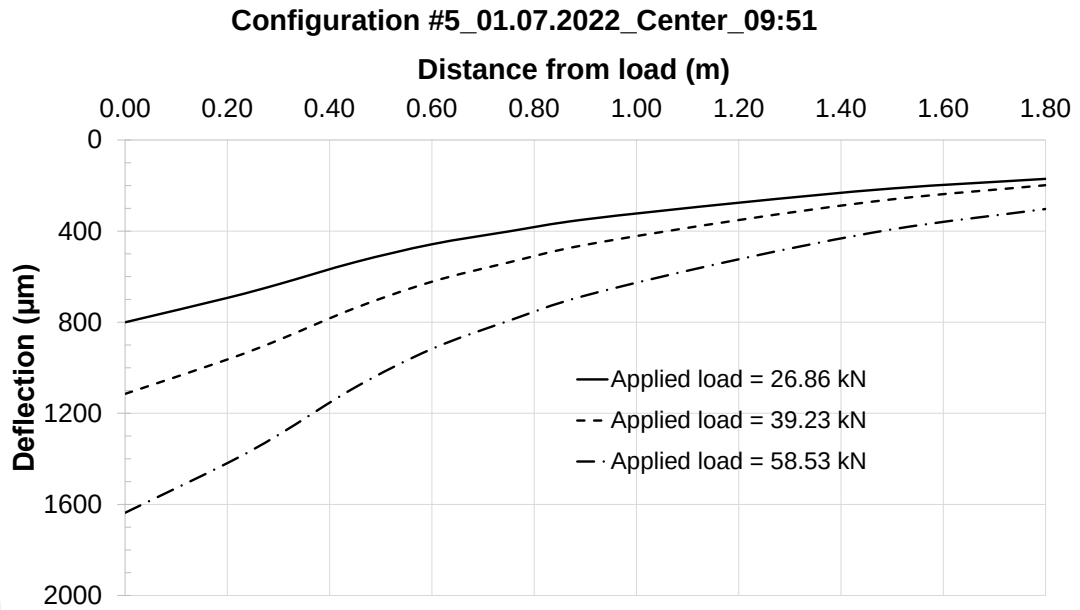


Figure B.45: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

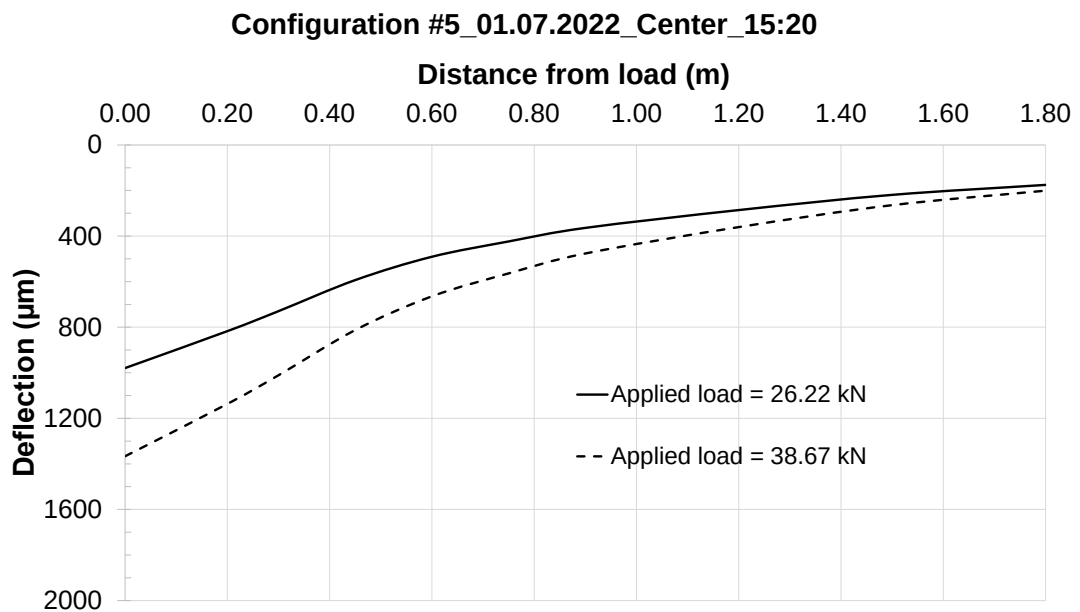


Figure B.46: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #5_01.07.2022_North wheel path_09:31

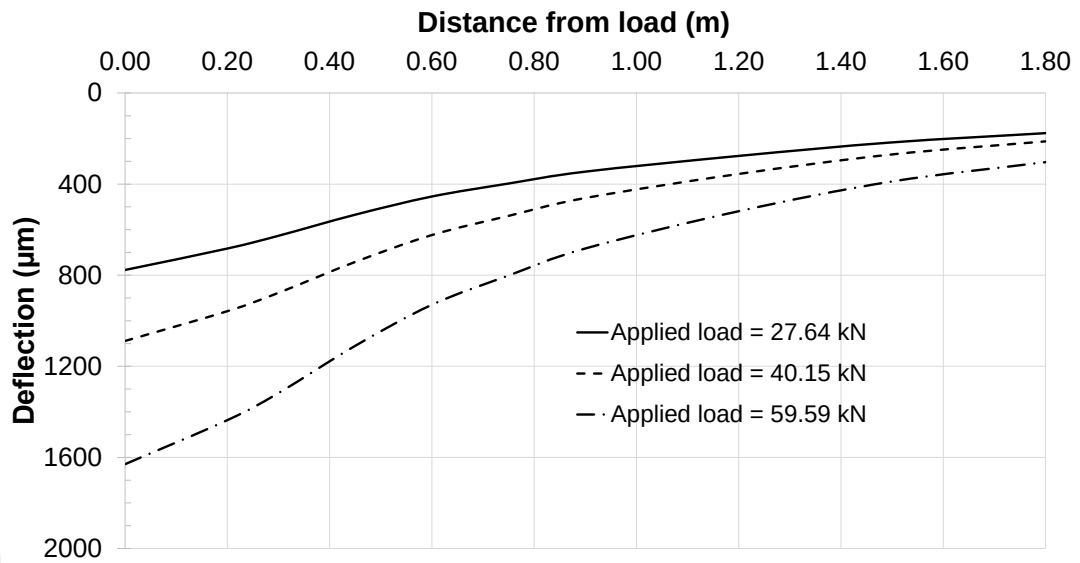


Figure B.47: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

Configuration #5_01.07.2022_North wheel path_15:10

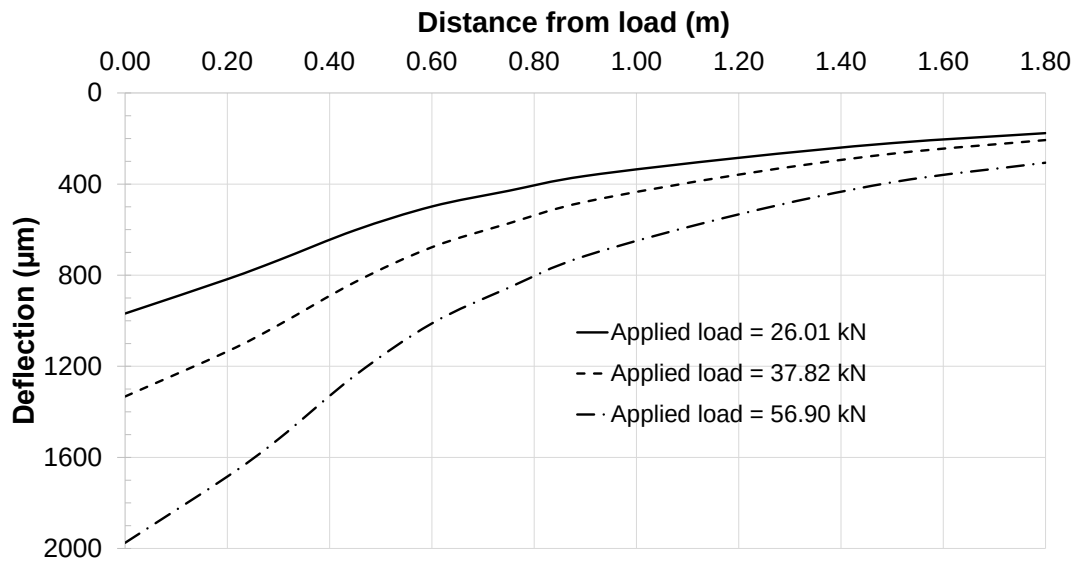


Figure B.48: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

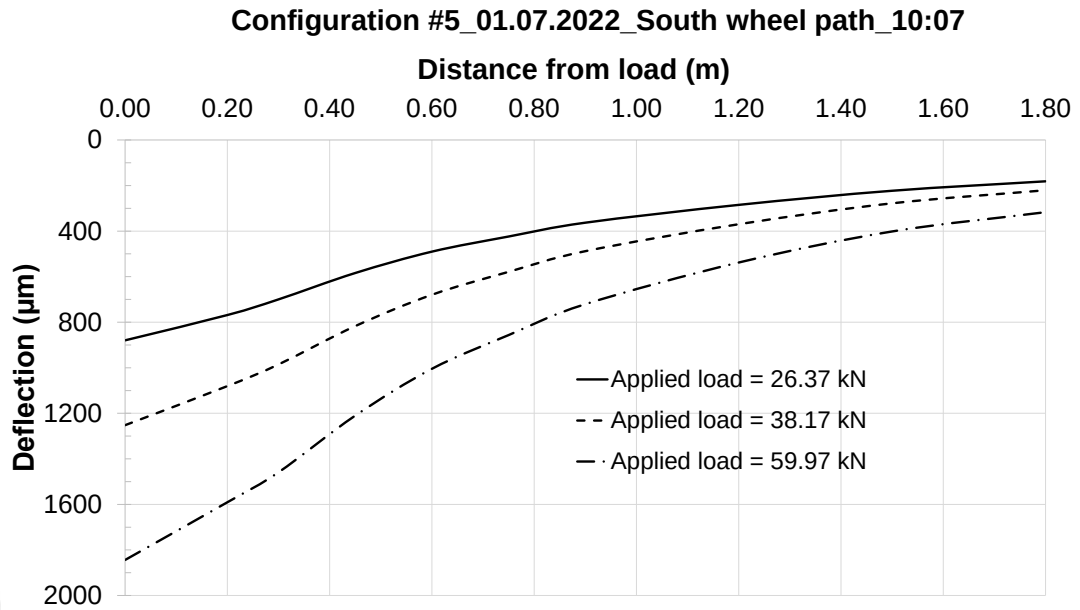


Figure B.49: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

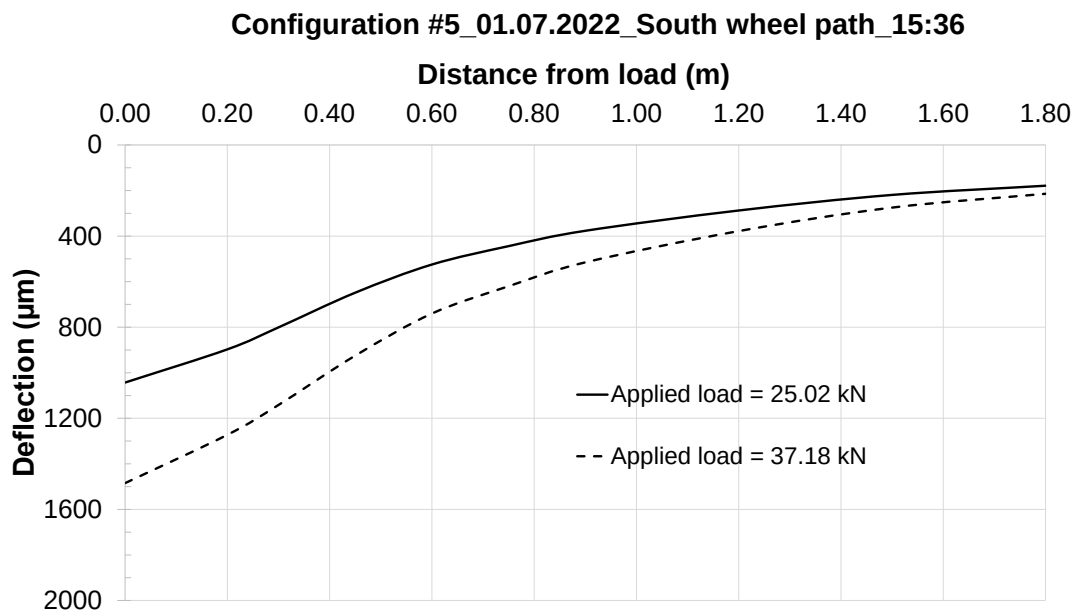


Figure B.50: Load - Deflection graph obtained from the Falling Weight Deflectometer test.

APPENDIX-C: FWD TEST MEASUREMENT LISTS

Falling weight deflectometer measurement lists prior accelerated pavement test:

Table C.1: FWD tests prior APT, applied load levels, measured maximum deflections, pavement and ambient temperatures.

Config. no	Date	Time	Location	Applied load level (kN)	Max. Deflection D ₀ (μm)	Temperatures (Degree Celsius)		
						Pavement	Pavement surface	Ambient
1	16.06 2022	13:30	Center	26.51	822.50	41.10	34.95	31.58
				26.65	820.60			
				38.88	1229.50			
				38.67	1228.70			
				57.96	1839.60			
				57.89	1849.00			
1	16.06 2022	14:24	North Wheel Path	27.28	758.40	39.40	40.56	32.18
				27.50	755.10			
				39.73	1167.10			
				39.58	1168.80			
				58.67	1810.40			
				58.46	1823.10			
1	16.06 2022	18:14	Center	27.57	812.60	38.00	37.08	26.11
				27.57	805.40			
				40.22	1219.00			
				40.08	1217.90			
				59.73	1843.50			
				59.52	1851.50			
1	16.06 2022	17:53	North Wheel Path	27.43	824.70	38.00	37.36	26.13
				27.64	817.10			
				39.80	1220.70			
				39.65	1223.90			
				58.88	1835.40			
				58.53	1848.50			
4	16.06 2022	13:50	Center	26.08	1605.50	41.10	42.54	31.60
				26.15	1588.80			

Table C.1: Continued.

4	16.06 2022	14:28	North Wheel Path	26.22 26.51	1474.40 1447.10	39.40	45.21	31.89
4	16.06 2022	18:16	Center	26.22 26.30	1515.40 1502.20	38.00	37.81	26.04
4	16.06 2022	17:56	North Wheel Path	26.15 26.37	1443.80 1419.00	38.00	38.74	26.19
3	16.06 2022	14:00	Center	25.45 26.37 37.18 37.18	1461.40 1483.10 2143.80 2161.80	41.10	46.57	31.68
3	16.06 2022	14:31	North Wheel Path	25.94 25.87	1438.50 1405.70	39.40	44.40	32.10
3	16.06 2022	18:18	Center	26.08 26.01	1484.30 1476.90	38.00	38.22	26.10
3	16.06 2022	18:00	North Wheel Path	25.94 26.01	1377.50 1371.60	38.00	38.31	26.23
5	16.06 2022	14:05	Center	26.58 26.65 39.09 38.95	999.10 993.40 1418.20 1417.00	41.10	46.52	32.11
5	16.06 2022	14:33	North Wheel Path	27.07 27.08 39.44 39.09	940.30 919.60 1370.90 1348.80	39.40	46.54	32.37
5	16.06 2022	18:21	Center	27.07 27.28 39.51 39.58	899.10 895.30 1289.80 1296.20	38.00	38.10	26.05
5	16.06 2022	18:04	North Wheel Path	27.00 27.28 39.37 39.30	886.80 894.00 1275.30 1276.20	38.00	38.32	26.08

Table C.1: Continued.

2	16.06 2022	14:10	Center	27.50	586.60	41.10	43.45	32.22
				27.28	575.10			
				40.15	849.30			
				40.36	849.80			
				60.51	1273.30			
				60.44	1286.60			
2	16.06 2022	14:36	North Wheel Path	27.36	555.80	39.40	44.58	32.51
				27.28	546.10			
				40.22	836.10			
				39.94	820.30			
				60.15	1307.00			
				60.15	1301.00			
2	16.06 2022	18:23	Center	27.57	560.30	38.00	36.41	26.04
				27.50	547.80			
				40.57	826.40			
				40.50	817.10			
				60.86	1239.70			
				60.65	1237.00			
2	16.06 2022	18:07	North Wheel Path	27.85	564.00	38.00	36.85	26.06
				27.85	559.10			
				40.72	822.20			
				40.50	815.40			
				60.58	1233.40			
				60.37	1237.00			

Falling weight deflectometer measurement lists after accelerated pavement test:

Table C.2: FWD tests after APT, applied load levels, measured maximum deflections, pavement and ambient temperatures.

Config. no	Date	Time	Location	Applied load level (kN)	Max. Deflection D ₀ (μm)	Temperatures (Degree Celsius)		
						Pavement	Pavement surface	Ambient
1	01.07 2022	09:40	Center	26.51	721.90	30.30	37.15	28.42
				27.14	728.00			
				39.44	1082.00			
				39.44	1082.80			
				58.60	1582.20			
				58.53	1584.00			
1	01.07 2022	09:22	North Wheel Path	28.20	763.20	30.30	32.18	25.29
				28.42	757.90			
				40.22	1117.20			
				40.29	1127.70			
				59.02	1677.80			
				58.88	1693.90			
1	01.0 2022	10:00	South Wheel Path	27.57	694.20	31.50	36.23	29.46
				27.28	681.90			
				39.73	1028.70			
				39.65	1031.80			
				58.74	1525.10			
				58.74	1535.30			
1	01.07 2022	15:15	Center	26.86	858.10	41.30	46.63	31.04
				26.93	845.90			
				39.09	1249.30			
				39.02	1242.80			
				58.39	1841.70			
				58.17	1831.10			
1	01.07 2022	15:01	North Wheel Path	26.44	978.40	42.50	43.15	31.63
				26.58	977.60			
				38.67	1424.90			
				38.45	1425.20			

Table C.2: Continued.

1	01.07 2022	15:25	South Wheel Path	26.65	840.70	40.30	44.43	31.41
				26.65	830.70			
				38.81	1226.30			
				38.81	1233.70			
				58.25	1795.40			
4	01.07 2022	09:44	Center	58.17	1814.70	30.30	34.41	28.57
				26.72	1201.70			
4	01.07 2022	09:26	North Wheel Path	26.72	1198.00	30.30	31.78	26.10
				23.26	1267.90			
4	01.07 2022	10:03	South Wheel Path	23.82	1268.50	31.50	35.43	29.50
				24.46	1505.40			
4	01.07 2022	15:17	Center	24.53	1492.20	41.30	47.28	30.98
				25.94	1536.30			
4	01.07 2022	15:07	North Wheel Path	25.66	1516.20	42.50	48.97	31.56
				24.88	1629.10			
4	01.07 2022	15:30	South Wheel Path	24.60	1599.50	40.30	46.24	31.37
				24.39	1863.20			
3	01.07 2022	09:46	Center	24.03	1829.40	30.30	35.72	28.75
				26.30	1304.80			
3	01.07 2022	09:30	North Wheel Path	26.44	1283.40	30.30	34.32	26.54
				26.22	1375.90			
3	01.07 2022	10:06	South Wheel Path	26.30	1354.70	31.50	35.53	29.46
				23.33	1545.10			
3	01.07 2022	15:18	Center	23.61	1509.80	41.30	46.72	31.04
				25.66	1583.90			
3	01.07 2022	15:09	North Wheel Path	25.59	1552.40	42.50	49.03	31.60
				25.09	1648.60			
3	01.07 2022	15:30	South Wheel Path	25.09	1608.50	40.30	42.30	31.41
				23.82	1760.10			
3	01.07 2022	15:30	South Wheel Path	23.82	1765.20			
				23.82	1765.20			

Table C.2: Continued.

5	01.07 2022	09:51	Center	26.86	800.40	30.30	35.93	28.79
				27.00	794.30			
				39.23	1114.70			
				39.37	1116.30			
				58.53	1636.40			
				58.46	1645.30			
5	01.07 2022	09:31	North Wheel Path	27.64	776.70	30.30	34.46	27.12
				27.64	771.00			
				40.15	1088.80			
				40.08	1089.80			
				59.59	1629.50			
				59.38	1630.60			
5	01.07 2022	10:07	South Wheel Path	26.37	879.80	31.50	34.47	29.27
				26.22	861.40			
				38.17	1251.90			
				38.17	1242.60			
				56.97	1844.00			
				57.04	1847.80			
5	01.07 2022	15:20	Center	26.22	979.60	41.30	43.43	31.03
				26.44	973.30			
				38.67	1366.40			
				38.59	1367.90			
5	01.07 2022	15:10	North Wheel Path	26.01	968.00	42.50	46.29	31.35
				25.87	937.60			
				37.82	1332.60			
				37.96	1329.50			
				56.90	1975.10			
				56.83	1972.80			
5	01.07 2022	15:36	South Wheel Path	25.02	1043.00	40.30	43.34	31.35
				25.23	1047.70			
				37.18	1484.30			
				37.11	1485.80			

Table C.2: Continued.

2	01.07 2022	09:55	Center	27.85	530.50	30.30	35.10	29.38
				27.64	518.80			
				40.22	759.50			
				40.08	756.80			
				60.01	1118.80			
				59.94	1118.80			
				91.75	1605.40			
				91.68	1611.30			
2	01.07 2022	09:35	North Wheel Path	27.92	493.30	30.30	34.09	27.61
				27.78	487.70			
				40.50	707.20			
				40.57	706.30			
				61.00	1044.90			
				60.79	1044.20			
				91.75	1514.90			
				91.89	1525.30			
2	01.07 2022	10:10	South Wheel Path	27.85	576.30	31.50	35.05	29.11
				27.85	563.20			
				40.08	829.60			
				39.94	821.20			
				59.94	1224.90			
				60.01	1221.90			
				91.54	1750.70			
				91.61	1749.20			
2	01.07 2022	15:22	Center	27.28	636.50	41.30	44.34	31.19
				27.07	624.60			
				39.65	916.70			
				39.65	901.10			
				59.66	1335.10			
				59.66	1325.00			
				91.18	1905.20			
				90.09	1912.40			

Table C.2: Continued.

2	01.07 2022	15:11	North Wheel Path	26.86	606.80	42.5	46.2	31.14
				26.93	597.30			
				39.37	873.90			
				39.58	868.40			
				59.87	1297.10			
				59.31	1282.10			
				91.68	1862.40			
				91.47	1866.6			
2	01.07 2022	15:38	South Wheel Path	26.51	685.70	40.30	43.29	31.58
				26.58	691.00			
				38.95	1008.00			
				39.02	1011.90			
				59.02	1460.80			
				58.95	1462.30			