LABORATORY STUDY FOR DETERMINING GEOTECHNICAL ENGINEERING PROPERTIES OF CEMENT-TREATED AND -UNTREATED BACKFILL SOILS USED IN HIGH SPEED RAILWAY EMBANKMENTS

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Handan UŞUN

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We approve the thesis of **Handan UŞUN**

Assoc. Prof. Dr. İsfendiyar EGELİ
Supervisor

Asist. Prof. Dr. Selim ALTUN
Committee Member

Asist. Prof. Dr. Tahir Kemal ERDEM
Committee Member

18 December 2009

Prof. Dr. Gökmen TAYFUR
Head of the Department of Civil Engineering

Assoc. Prof. Dr. Talat YALÇIN
Dean of the Graduate School of
Engineering and Sciences

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ABSTRACT

LABORATORY STUDY FOR DETERMINING GEOTECHNICAL ENGINEERING PROPERTIES OF CEMENT-TREATED AND - UNTREATED BACKFILL SOILS USED IN HIGH SPEED RAILWAY EMBANKMENTS

Formation of high-speed train (HST) infrastructure is a rather new subject worldwide and in Turkey, as HST infrastructure is different than for normal train infrastructure (NTI). Existing NTI remains inadequate to meet high geometric and material properties required by HST. If strict criteria is not met, fatal accidents may occur.

In this study; Taiwan HST Project's design criteria and fill types were used to study replaceability of one fill strata called: Uncemented-Prepared Subgrade Layer (U-PSL) with a cemented one (C-PSL) by conducting various laboratory tests to obtain soils' index properties and parameters, which were used to find the maximum total settlements by using Plaxis V8 (2D) Programme. For this purpose; 3 groups (totally 270 no.s with 90 no.s per group) of cylindirical concrete samples were obtained with various cement contents (10, 15, 20, 25, 30 % by weight of concrete) at 3 diameters (4, 8, 10 cm) and for 2 water-cement ratios (0.4, 0.5), then 7-28 day cured and tested at IYTE-MAM to find elasticity modulus, stress, strain, force results. Using such results in the Plaxis Programme, maximum total settlements were calculated for different layers. C-PSL mixes having 20-30% cement contents met the required strict settlement criteria as with the U-PSL mix currently used in Far-Eastern HST Projects. This showed that one of such C-PSL mixes can be used in place of U-PSL with an approximately 12.5 % reduction in the layer thickness, corresponding to 1.75m, instead of 2.0m thickness of the U-PSL currently used.

Key Words: Transportation Embankments, High-Speed Train infrastructure, High-Speed Train Fill Properties.

ÖZET

HIZLI TREN DOLGULARINDA KULLANILAN ÇİMENTO KARIŞIMLI VE -KARIŞIMSIZ DOLGU ZEMİNLERİN GEOTEKNİK MÜHENDSLİĞİ ÖZELLİKLERİNİ TAYİN EDİCİ LABORATUAR ÇALIŞMALARI

Hızlı trenlerin (HT) altyapılarının oluşturulması Dünyada ve Türkiye' de oldukça yeni bir konu olup, mevcut normal trenler için olanlardan farklıdır. Bu trenlere ait mevcut demiryolları altyapısı, HT için gerekli olan yüksek geometrik ve malzeme özelliklerini sağlayamadıklarından yetersiz kalmaktadırlar. HT için gerekli altyapı oturma kriterlerinin, diğerine göre daha hassas ve emniyetli olması gerekmektedir. Gerekli kriterlere uyulmadığı taktirde, ölümcül kazaların olabileceği günümüzde bile görülmektedir.

Yaptığımız bu tez çalışmasında Tayvan'daki hızlı tren projesinde uygulanan tasarım kriterlerine ve dolgu tiplerine göre, dolguyu teşkil eden zemin tabakalarından biri olan 'hazırlanmış altyapı' tabakasının, hem çimentosuz (U-PSL) hem de çimentolu (C-PSL) halde kullanılması durumları için; çeşitli laboratuvar deneyleri gerçekleştirilmiş ve elde edilen zemin-indeks özellikleri ve parametreleri kullanılarak Plaxis-sonlu elemanlar metodu yardımıyla oturma hesapları yapılmıştır. Bunun için üç ana grup halinde ve her bir grupta değişik oranlarda (%10, %15, %20, %25 ve %30) çimento ihtivalı 90 adet silindirik beton numuneleri, 3 değişik çapta (4, 8, 10 cm), 2 su-çimento oranında (0.4, 0.5) hazırlanan toplam 270 numune 7-28 günlük kürlemeden sonra İYTE-MAM'da kırılarak; elastisite modülü, basınç mukavemeti, birim-uzama ve kuvvet gibi çeşitli sonuçlar elde edilmiştir. Bu sonuçlardan yararlanılarak Plaxis-sonlu elemanlar programı yardımıyla değişik tabakalardaki oturmalar hesaplanmıştır. Çimento muhtevası %20-%30 arasında olan çimentolu tabakaların da (C-PSL), halen Uzak-Doğu'daki HT Projelerinde kullanılan çimentosuz tabaka (U-PSL) gibi istenilen oturma kriterleri sağladıkları görülmüş ve böylece bu çimentolu tabakalardan herhangi birinin, çimentosuz tabaka ile değiştirilerek kullanılabileceği anlaşılmıştır.

Anahtar Kelimeler: Ulaştırma Dolguları, Hızlı Tren Altyapısı, Hızlı Tren Dolgu Özellikleri.

TABLE OF CONTENTS

LIST OF FIGURES

<u>Figure</u> **<u>Page</u>**

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Overview

Utilization of high-speed train transportation systems are becoming more widespread to see in the developed countries of the World, due to their offering of fast, comfortable, safe, economical and environment friendly transportation alternative.

During the early years of the Turkish Republic, railways have developed considerably, but after 1950s, emphasis on transportation has shifted to highways, due to abundant petrol supplies and low energy prices until early 1970s. This was when the first petrol ambargo by some Middle-Eastern countries occured, causing some supply shortage and resulting-in petrol price jump and leading to railways re-gaining their importance in the western countries. Another reason helping this development was rapid advancement achieved in rail technology and infrastructure in countries like France and Japan. These countries also were pioneers developing high-speed train technologies since late 1960s and early 1970s. After France and Japan, some other countries like Germany, Spain, UK, Korea, Taiwan, China started to construct their own network of high-speed train railways, usable for both people and cargo transportation. In Turkey, only for public transportation by high-speed trains(HST) were planned about a decade ago (but this may change in the furture) and so far only 1 line became partly operational. However, HST transportation will continue to gain increasing importance in the future globally, due to diminishing petrol supplies in the world, leading to increasingly unaffordable expensive petrol, consumed by land and air transportation vehicles. Hence it can be said that HST transportation with a type which depends upon renewable hydro-energy has a bright future globally, as well as in Turkey, which is a hydro-energy rich country. Thus it can be said that HST transportation is economical, fast, environment-friendly, safe and efficient in moving large cargos and people over long distances.

EU countries also give importance to develop a Europe-wide railway network connecting member countries. For this reason several agreements have been signed like; TENSRP (Trans-European North-South Railway Project), AGC (Trans-European Agreement on Main International Railway Lines), TEN (Trans-European Networks), TER (Trans-European Railway).

It is envisaged by the EU to built totally 30,000 km of railway lines by the year 2015. In the 19,000 km part of the total, the speed limit will be 250 km/hr. This long term plan, also endorses to connect a central railway line to Istanbul-Turkey, via Wien-Budapest-Belgrade-Sophia to promote better integration of transportation services among the member countries.

1.2. Objective and Scope of Study

One main objective of this research is to study geotechnical properties of one of the layers called: Uncemented-Prepared Subgrade Layer (U-PSL) used in a typical high-speed train (HST) railway embankment in the Far-East with an aim to substitute it with a cemented one (C-PSL) in laboratory conditions. To realize this objective, work consisted of the achieving following steps:

- Review of literature on high speed trains, embankments and their allowable settlements.
- Determine Turgutlu sand's index properties in laboratory using ASTM standards.
- Establish design mix for cement-sand mixtures with various cement contents, water-cement ratios to obtain various samples of C-PSL, which are 7-28 day cured and broken.
- Determine the elastisity modulus, forces, stresses, strains by uniaxial testing.
- Using obtained properties, perform settlement analyses by Plaxis V8 (2D) Programme.
- Compare settlement results with the Far-Eastern HST railway design requirements.
- Asses whether C-PSL can replace U-PSL and make recommendations.

1.3. Organization of the Thesis

This thesis consists of six chapters, as follows:

Chapter 1 consists of three subtitles. The first title is: overview, the second is: objectives and scope, the third is: organization of the thesis.

Chapter 2 gives background information about HST railway infrastructure's technical properties used in the world and in Turkey, incuding principles and advantages.

Chapter 3 consists of HST railway embankment types and properties used in Taiwan.

Chapter 4 presents laboratory tests on U-PSL and C-PSL using Type 1 Portland Cement and commercially available Turgutlu Sand, including comparisons.

Chapter 5 gives settlement analyses by using Plaxis Program with comparison of results

Chapter 6 is the final part of this thesis which presents conclusions and recommendations.

CHAPTER 2

BACKGROUND INFORMATION

2.1. History of Railways in the World and in Turkey

The history of railways is closely linked with human civilization. As necessity arises, human beings develop various methods of transporting goods from one place to another. In the past good were carried as head loads or in charts drawn by men or animals. Then efforts were made to replace animal power with mechanical power. In 1769, Nicholes Carnot, a Frenchman, carried out the pioneering work of developing system energy. This work had very limited success and it was only in the year 1804 that Richard Trevithick designed and constructed a steam locomotive. This locomotive, was used for pulling loaded wagons on roads. The credit of perfecting the design goes to George Stephenson, who in 1814 produced the first steam locomotive used for pulling rail cars in railways.

The first public railway in the world was opened to traffic on 27 September 1825 between Stockton and Darlington in the United Kingdom. Simultaneously, other countries in Europe also developed similar railway systems, including trains for carrying passenger traffic during that time. The first railway in Germany was opened from Nurenberg to Furth in 1835. The USA opened its first railway line between Mohawk and Hudson in 1833 (Chandra and Agarwal, 2008).

The first railway construction in Turkey has began between İzmir and Aydın on 23 September 1856 by an English Company. Railway length was 130 km, and it was finished in 1866.

In 1869, the first Transcontinental Railroad of North America was completed across the United States from Omaha, Nebraska to Sacramento, California. It was built by Central Pacific and Union Pacific (Wikipedia, 2009).

The first electric locomotive was demonstrated in Berlin in 1879. Electric traction was commercially applied to the first suburban and metropolitan lines, but was also quickly adopted for underground railways. One of the earliest users of electric locomotives on mainline routes was Italy, where a line was opened in 1902.

The railways proved strategically important on all fronts in the World War I. After the war, many railway companies grouped together to form national railway systems or large geographical concerns. Following the World War II, there was a period of reconstruction for railways, during which time new steam locomotives were introduced in the UK and mainland Europe and new diesels were also tested (A Dictionary of World History, 2000).

Immediately after the World War II , Japan started speedy railway reconstruction, including high speed railways. 1956 saw the first feasibility studies conducted for a new line linking Tokyo and Osaka. It was then decided to design the line for a design speed of 250 km/hr (UIC, 2006).

In 1964, the high-speed Shinkansen or bullet trains began operation, running on a specially developed track at speeds of up to 210 km/hr (A Dictionary of World History 2000). A maximum speed of 443 km/hr was recorded in a test run in 1996 (Wikipedia, 2000).

During 1981, in France, the TGV (Train Grande Vitesse) trains began operation between Paris and Lyons, with speed of 260 km/hr. A maximum speed of 515,3 km/hr was recorded on 18 May 1990 by the TGV Atlantic.

Germany launched the first ICE (Inter City Express) trains on lines between Hanover and Würzburg with a design speed of 250 km/hr, in 1991. A maximum speed of 406,9 km/hr was recorded in 1988.

Spain Railway (RENFE) has operated AVE (Alta Velocidata Espanola) trains between Madrid and Seville in 1992. AVE’s design speed was close to TGV and Shinkansen trains (250 km/hr).

Sweden decided to upgrade existing lines to a accommodate speeds of around 200 km/hr, at a reasonable cost, using tilt-body technology. Thus X2000 trains began operation between Stockholm and Gothenburg line, in 1998.

The Taiwan High Speed Railways, also known as the THSR, is Taiwan’s high-speed rail network, running approximately 335.50 kilometers from Taipei City to Kaohsiung City, which began operations on 5 January 2007. Adopting Japan’s Shinkansen technology for the core system, the THSR uses the Taiwan High Speed type: 700T train, manufactured by a consortium of Japanese companies, most notably Kawasaki Heavy Industries.

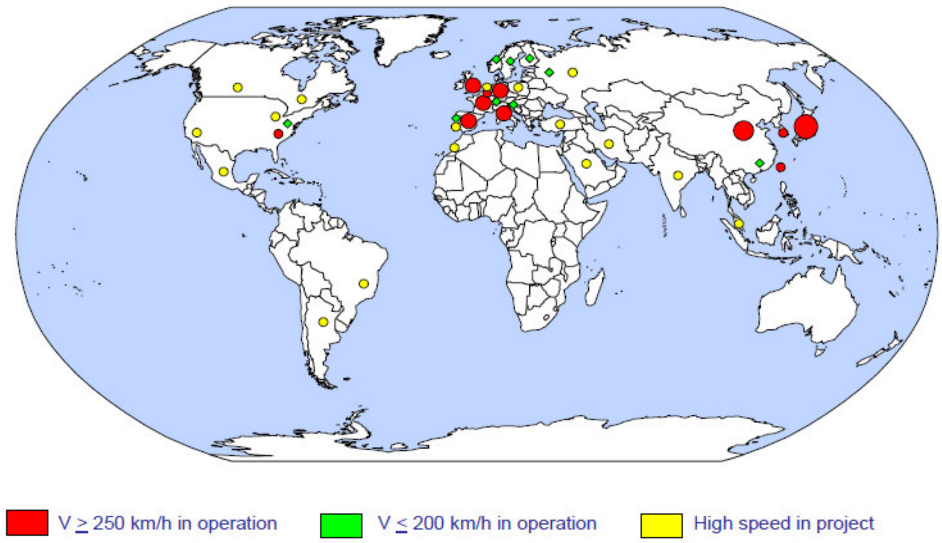


Figure 2.1. High Speed Railway Systems existing today in the World-UIC

The Turkish State Railways (TCDD) first started to build high speed railway (HSR) lines in 2003. The first line, which has a length of 533 km from Istanbul via Eskisehir to Ankara is now under construction. The Ankara-Eskisehir section of the line which has a length of 245 km and a projected travel time of 65 minutes, is already completed.

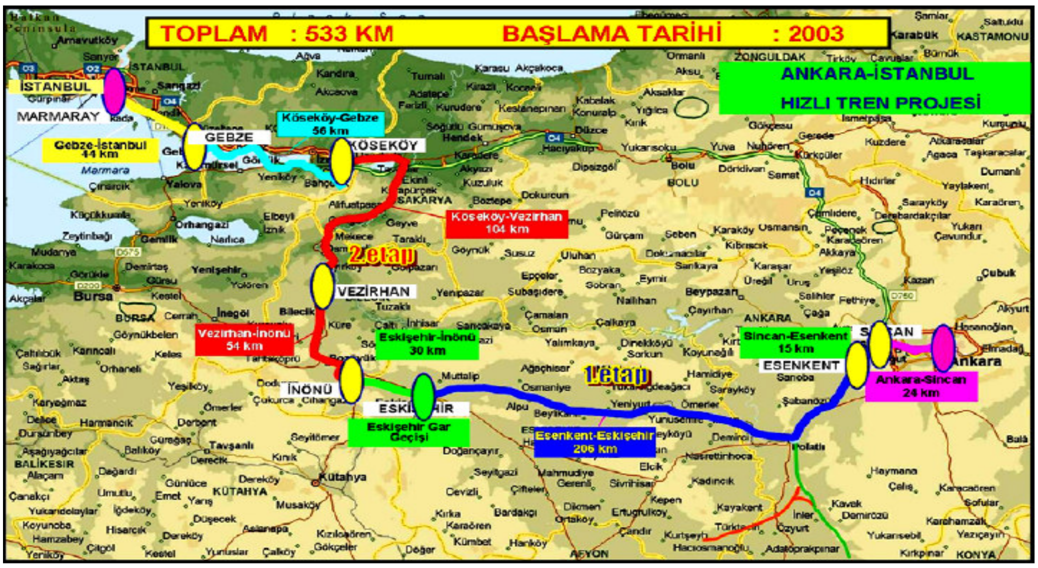


Figure 2.2. High-Speed Railway Project between Ankara-Istanbul

On the Ankara-İstanbul HSR line, trials began on 23 April 2007 and revenue earning service began on 13 March 2009. Two ETR 500 train sets have been used for testing on the completed part of the Turkish High-Speed railway network on 23 April 2007.

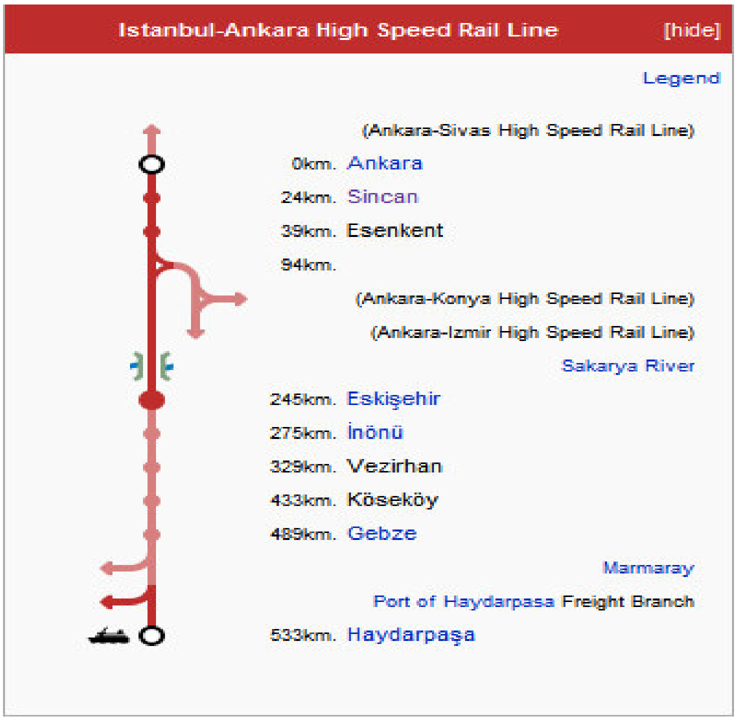


Figure 2.3. Istanbul-Ankara High Speed Railway (HSR) Line

The commercial high-speed trains can reach top speeds of 250-300 km/hr on these Turkish tracks. The first ten TCDD HT65000 high speed train sets were purchased from CAF of Spain with maximum design speeds of 260 km/hr. In addition, two ETR 500 trains sets, which can reach a top speed of 300 km/hr, have been rented from Italy and were used for testing on the first completed part of the high-speed railway network between Eskisehir and Ankara on 23 April 2007. During these tests, ETR 500 Y2 achieved the current rail speed record in Turkey, reaching 303 km/hr. For producing new train sets to be used on the Turkish high speed railway (HSR) network, a joint enterprise company (EUROTEM), between the South Korean ROTEM and Turkish TUVAVAŞ in Eskişehir has been established in 2007, which has built a factory in Adapazarı-Sakarya to build the high speed train sets types: HSR-350x and KTX-II (of Korea Train eXpress) under licence to reach design speeds of up to 324.5 km/hr. Table 2.1 gives HSR lines and their tested speeds by the country (Wikipedia, 2009)

Table 2.1. High-Speed Railway lines by the country

Country	Total Network Length (km)	Design Speeds and Scheduled Trains	Tested Speed on the test run
Austria		230 km/h, HSR	275 km/h
Belgium	326	300, 250 km/h, HSR	347 km/h
China	6003	431 km/h maglev 350, 300, 250, 200 km/h, conventional	502 km/h maglev 394 km/h conventional
Finland	60	220 km/h, HSR	255 km/h
France	1700	320, 300, 280, 210 km/h, HSR	574 km/h
Germany	1290	300, 280, 250, 230 km/h (conventional)	550 km/h maglev 406 km/h conventional
Italy	814,5	300, 260, 200 km/h, HSR	368 km/h
Japan	2459	300, 275, 260 km/h (conventional)	581 km/h maglev 443 km/h conventional
Netherland	100	300, 250, 140/160 km/h, HSR	336.2 km/h
Norway	60	210 km/h, HSR	260 km/h
Portugal	314	220 km/h, HSR	275 km/h
Russia	649.7	210 km/h, HSR	260 km/h
South Korea	240.4	300, 240 km/h, HSR	355 km/h
Spain	1272.3	300, 250 km/h, HSR	404 km/h
Sweden	0	200 km/h, HSR	303 km/h
Switzerland	79	250, 200 km/h, HSR	280 km/h
Taiwan	335,5	300, 240 km/h, HSR	315 km/h
Turkey	245	250 km/h, HSR	303 km/h
United Kingdom	1000	300 km/h, 201 km/h, HSR	335 km/h
United State	0	241 km/h, 201 km/h, HSR	264 km/h

2.2. Different Modes of Transport

There are three modes of transport. These are; land transport, air transport and water transport. Rail transport and road transport are the two components of land transport. Each mode of transport, depending upon its various characteristics, has its own strengths and weaknesses and can be best used for a particular type of traffic, as given below (Chandra and Agarwal, 2008).

2.2.1. Rail Transport

Though the initial capital expenditure on the basic infrastructure required is big, rail transport is best suited for carrying bulk and heavy commodities and a large number of passengers over long distances (Chandra and Agarwal, 2008).

2.2.2. Road Transport

Due to flexibility of its operation and the ability to provide door-to-door service, road transport is ideally suited for carrying light commodities and a small number of passengers over short distances (Chandra and Agarwal, 2008).

2.2.3. Air Transport

Despite big capital investment and heavy expenditure is required on the sophisticated equipment and the high fuel costs, air transport is still better suited for carrying passengers or goods between very distant locations in reaching their destinations over a short period of time (Chandra and Agarwal, 2008).

2.2.4. Water Transport

Due to the low cost of infrastructure and relatively slow speeds, water transport is best suited for carrying heavy and bulk goods over long distances, provided that there is no consideration of the time factor (Chandra and Agarwal, 2008).

2.2.5. Railway as a Mode of Land Transport

There are two modes of land transport, which are; railways and roads. Each has its own advantages and disadvantages, as summarized in Table 2.2.

Table 2.2. Rail Transport versus road transport

Feature	**Rail transport**	**Road transport**
Tractive resistance	The movement of steel wheels on steel rails has the basic advantage of low rolling resistance. This reduces haulage costs, because of low tractive resistance.	The tractive resistance of a pneumatic tyre on metalled road is almost five times higher, compared to that of wheels on rails.
Right of way	A railway track is defined on two rails and is within protected limits. Trains work as per a prescribed schedule and no other vehicle has the right of way, except at specified level crossing.	Roads, though having well-defined limits, can be used by any vehicular traffic and even by pedestrians.
Cost analysis	Owing to the heavy initial infrastructure cost, the initial as well as maintenance cost of the railway line is high	The cost of construction and maintenance of roads is comparatively cheaper.
Gradients and Curves	The gradients of railway tracks ar flatter (normally not more than 1 in 100) and curves are limited up to only 10° on broad gauge.	Roads are constructed normally with steeper gradients of up to 1 in 25 and with relatively much sharper curves.
Flexibility and movements	Due to the defined routes and facilities required for the reception and dispatchment of trains, railways can be used only between the fixed points.	Road transport have much more flexibility in movement and can provide door-to-door services.
Environment pollution	Railways have the minimum adverse effects on the enviroment.	Road transport creates comparatively greater pollution than the railways.
Organization and control	Railways are mostly fully or partially government undertakings, with their own organization.	Barring member state government transport, road transport is managed by the private sector.
Stability	Railways are best suited for caryying heavy goods and large numbers of passengers over long distances.	Road transport is best suited for caryying lighter goods and smaller numbers of passengers over shorter distances.

2.3. Railway Track a Gauge

Rail gauge is the distance between the inner sides of the heads of the two parallel rails that make up a single railway line (Fig.2.4).

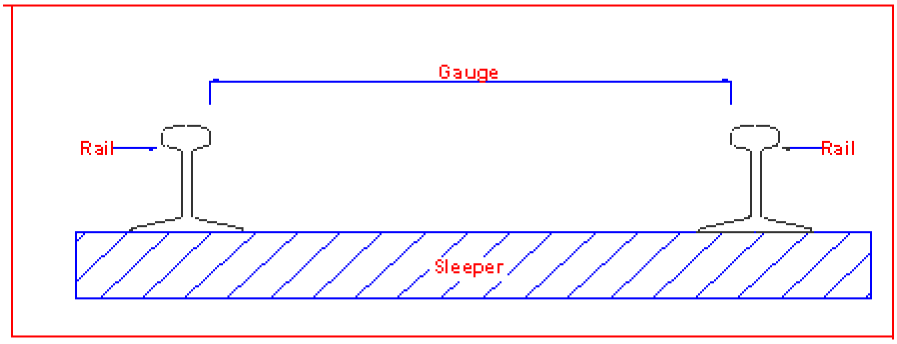


Figure 2.4. Track Gauge

2.3.1. Rail Gauges on World Railways

Various rail gauges have been adopted by different railways in the world, due to historical and other considerations. In British Railways, a rail gauge of 125 mm (5 feet) was initially adopted, but the wheel flanges at that time were at he outside of the rails. Subsequently, in order to guide the wheels better, the flanges were made inside the rails. The rail gauge then has became 1435 mm (4' 8.5"), as at that time the width of the rail at the top was 45 mm (1.75"). The 1435 mm gauge became the standard gauge in most European Railways, including Turkey. The approximate proportions of various gauges on world railways are given in Table 2.3. (Chandra and Agarwal, 2008).

Table 2.3. Various rail gauges on world railways

Type of gauge	Gauge (mm)	Gauge (feet)	% of total length	Countries
Standard gauge	1435	4' 8.5"	62	England, USA, Canada, Turkey, Persia, and China
Broad gauge	1676	5' 6"	6	India, Pakistan, Ceylon, Brazil, Argentina
Broad gauge	1524	5' 0"	9	Russia, Finland
Cape gauge	1067	3' 6"	8	Africa, Japan, Java, Australia, and New Zealand
Metre gauge	1000	3' 3.5"	9	India, France, Switzerland, and Argentina
23 various other gauge	Different Gauge	Different Gauges	6	Various countries

2.3.2. Choice of Gauge

The choice of gauge is very limited, as each country has a fixed gauge and all new railway lines are constructed to adhere to each country's standard gauge. However, the following factors mostly influence the choice of the gauge.

- **Cost considerations:** There is only a marginal increase in the cost of a railway track, if a wider gauge is adopted. In this context, the following points are important:
  a) There is a proportional increase in the cost of acquisition of land, earthwork, rails, sleepers, ballast, and other rail track items, when constructing a wider gauge.
  b) The cost of building bridges, culverts and tunnels increase only marginally due to a wider gauge.
  c) The cost of constructing station buildings, platforms, staff quarters, level crossing, signals, etc. associated with the railway network is more or less the same for all gauges.
  d) The cost of rolling stock is independent of the gauge of the track for carrying the same volume of traffic.

- **Traffic considerations:** The volume of traffic depends upon the sizes of wagons , speed and hauling capacity of the train.
    a) As a wider gauge can carry larger wagons and coaches, it can theoretically carry more traffic.
    b) A wider gauge has a greater potential of allowing higher speeds, because speed is a function of the diameter of the wheel, which in turn is limited by the width of the gauge.
    c) The type of traction and signaling equipment required are independent of the gauge.

- **Physical features of the country:** It is possible to adopt steeper gradients and sharper curves for a narrow gauge, as compared to a wider gauge.

- **Uniformity of gauge:** The existence of a uniform gauge country wide enables smooth, speedy, and efficient operation of trains. Therefore a single gauge should be adopted countrywide (Chandra and Agarwal, 2008).

2.4. Railway Track

The track or permanent way is the railway on which trains run. It consists of two parallel rails, fastened to sleepers with a specified distance between them. The sleepers are embedded in a layer of ballast of a specified thickness, which is spread over level ground, known as formation. The ballast provides a uniformly surface-levelled drainage layer and transfers the loads to a larger area of the formation. The rails are joined in series by fish plates, bolts and these are fastened to the sleepers with various types of fittings. The sleepers are spaced at a specified distance and are held in position by the ballast. Each component of the track has a specific function to perform. The rails act as girders to transmit the wheel load of trains to the sleepers. The sleepers hold the rails in their proper positions in order to provide a correct gauge with the help of fittings and fastenings, apart from transferring the total load on the tracks to their foundation and subgrade soils below.

The permanent way or track, therefore, consist of the rails, sleepers, fittings and fastenings, the ballast and formation as shown in Figure 2.5.

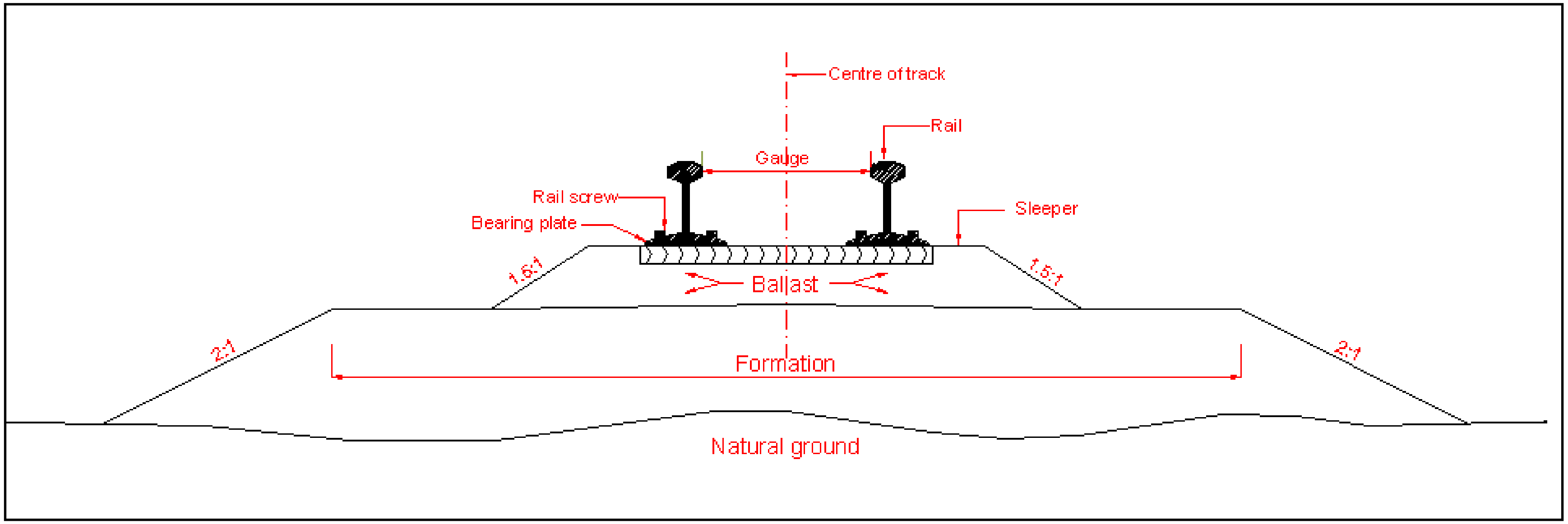


Figure 2.5. Various components of a track

In the early days, a temporary track was used to be laid for carrying earth and other building materials for the construction of a railway line, which is removed subsequently. The permanent track on which the trains move is also called the permanent way, in order to distinguish them from the temporary track constructed to carry building materials (Chandra and Agarwal, 2008).

2.4.1. Requirements of a Good Track

A permanent way or track should provide a comfortable and safe ride at the design or maximum permissible speed with minimum maintenance costs. To achieve these objectives, a sound permanent way should have the following characteristics (Chandra and Agarwal, 2008).

a) The gauge should be correct and uniform.
b) The rails should have perfect cross levels. In curves, the outer rail should have a proper superelevation to take into account the centrifugal forces.
c) The alignment should be straight and free of kinks. If a curve exists, a proper transition should be provided between the straight track and the curve.
d) The gradient should be uniform and as gentle as possible. The change of gradient should be followed by a proper vertical curve to provide a smooth ride.
e) The track should be resilient and elastic in order to absorb the shocks and vibrations of running trains.
f) The track should have a good drainage system so that the stability of the track is not affected by waterlogging.
g) The track should have good lateral strength so that it can maintain its stability, despite variations in temperature and other factors.
h) There should be provisions for easy replacement and renewal of the various track components.
i) The track should have such a structure that, not only is its initial construction cost is low, but also its maintenance cost is a minimum.

2.5. Rails

Rails are the members of the track, laid in two parallel lines to provide an unchanging, continuous and level surface for the movement of trains. In order to be able to withstand high stresses, they are made durable and of high-carbon steel (Chandra and Agarwal, 2008).

2.5.1. Functions of Rails

Rails are similar to steel girders and are provided to perform the following functions in a track (Chandra and Agarwal, 2008).

a) Rails provide continuous and level surface for the movement of trains.
b) Rails provide a pathway, which is smooth and has very little friction. The friction between the steel wheel and the steel rail is about one-fifth of the friction between the pneumatic tyre and a metal plate's surface.
c) Rails serve as a lateral guide for the wheels.
d) Rails bear the stresses developed, due to the vertical loads transmitted to them through the axles and wheels of the rolling stock, as well as, due to braking and thermal forces.
e) Rails carry out the function of transmitting the load to a large area of the formation through sleepers and the ballast layer.

2.5.2. Types of Rails

For a long time, double-headed and bull-headed rails were popular to be used in the world railway systems.

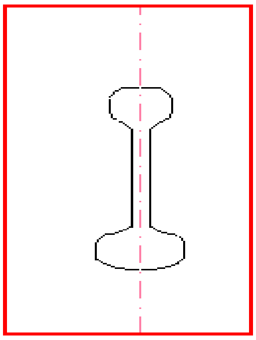


Figure 2.6. Double headed rail

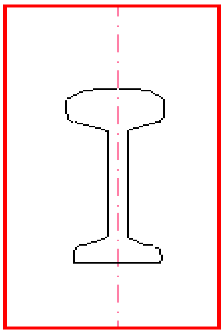


Figure 2.7. Bull-headed rail

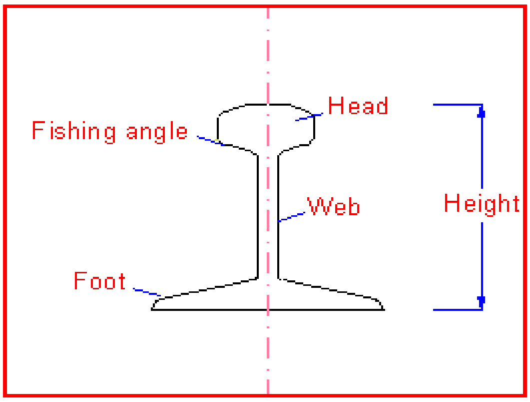


Figure 2.8. Flat-footed headed rail

As rails' wear and tear occur usually on the head, it was thought that double-headed rail could be inverted, after one side head has worn out. However, it was found that the old foot had also worn out at the sleeper supports, but this did not produce good running, after it was inverted to become the head. Bull-headed rails with the head appreciably larger than the foot were then introduced. These rails maintained better alignment, but were expensive.

Then about 50 years ago, flat-footed rails were introduced. The performance of these rails on the track has been found to be superior to the other two types for the same weight. Flat-footed rails have more lateral strength, the number of fastenings are few and their tendency to get loose is less. Presently, these rails are being used all over the world (Mundrey, 2000).

2.5.3. Requirements for an Ideal Rail Section

The requirements for an ideal rail section are as follows:

a) The rail should have the most economical section, consistent with sufficient strength, stiffness and durability.
b) The centre of gravity of the rail section should preferably be very close to the mid-height of the rail so that the maximum tensile and compressive stresses are equal.
c) A rail primarily consists of a head, a web, and a foot and there should be a balanced distribution of metal in its various components, so that each of them can fulfill is requirements properly. The requirements and the main considerations for the design of these rail components are as follows:

   Head: The head of the rail should have adequate depth to allow for vertical wear. The rail head should also be sufficiently wide so that not only a wider running surface is available, but also the rail has the desired lateral stiffness.

   Web: The web should be sufficiently thick so as to withstand the stresses arising due to the loads carried by it, after allowing for normal corrosion.

   Foot: The foot should be of sufficient thickness to be able to withstand vertical and horizontal forces, after allowing for loss due to corrosion. The foot should be wide enough for stability overturning. The design of the foot should be such that it can be economically and efficiently rolled.

   Fishing angles: Fishing angles must ensure proper transmission of loads from the rails to the fish plates. The fishing angles should be such that the tightening of the plate does not produce any excessive stress on the web of the rail.

   Height of the rail: The height of the rail should be adequate so hat the rail has sufficient vertical stiffness and strength as a beam.

2.6. Sleepers

In the past, sleepers for railway track consisted of slabs of stones or longitudinal timbers laid continuously under the rails. With the evolution of better rail design, it was not considered necessary to give a continuous support to the rails, which was an expensive process. Intermittent supports, with a positive means of holding the gauge, were found to be more advantageous. This led to the adoption of cross sleepers, which were first introduced in Britain in 1835 and are now employed universally.

2.6.1. Functions and Requirements of Sleepers

The main functions of sleepers are as follows.

a) Holding the rails fixed in their correct gauge and alignment ,
b) Giving a firm and even support to the rails.
c) Evenly transferring and distributing the loads coming from the rails to a wider area of the ballast underneath,
d) Acting as an elastic medium between the rails and the ballast to absorb the blows and vibrations caused by moving loads.
e) Providing longitudinal and lateral stability to the permanent way.
f) Providing the means to rectify the track geometry during their service life.

Apart from performing these functions the ideal sleeper should normally fulfill the following requirements.

a) Initial cost as well as the long term maintenance cost should be minimum.
b) Weight of the sleeper should be moderate so that it is convenient to handle.
c) Designs of sleepers and fastenings should be such that it is possible to fix and remove them and rails easily.
d) Sleeper should have sufficient bearing area so that the ballast under is not crushed.
e) Sleeper should be such that it is possible to maintain and adjust the gauge properly.
f) Material of sleeper and its design should be such that it does not break or get damaged during packing.

g) Design of sleeper should be such that it is possible to have a track circuiting for better operational safety.
h) Sleeper should be capable of resisting vibrations and shocks caused by the passage of fast moving trains.
i) Sleeper should preferably have anti-sabotage and anti-theft features.

2.6.2. Types of Sleepers

The sleepers may be classified as wooden sleepers, cast iron (CI) sleepers, steel sleepers. Table 2.4. compares important characteristics of different sleepers.

Table 2.4. Comparison of different types of sleepers

Characteristics	**Wooden**	**Steel**	**Cast Iron**	**Concrete**
Service life (years)	12-15	40-50	40-50	50-60
Weight of sleeper for BG (kg)	83	79	87	267
Handling	Manual handling; no damage to sleeper while handling	Manual handling; no damage to sleeper while handling	Manual handling; liable to break by rough handling	No manual handling; gets damaged by rough handling
Type of maintenance	Manual or mechanized	Manual or mechanized	Manual	Mechanized only
Cost of maintenance	High	Medium	Medium	Low
Gauge adjustment	Difficult	Easy	Easy	No gauge asjustment possible
Track circuiting	Best	Difficult; insulating pads are necessary	Difficult; insulating pads are necessary	Easy
Damage by white ants and corrosion	Can be damaged by white ants	No damage by white ants but corrosion is possible	Can be damaged by corrosion	No damage by white ants or corrosion
Suitability for fastening	Suitable for CF* and EF**	Suitable for CF and EF	Suitable for CF only	Suitable for EF only
Track elasticity	Good	Good	Good	Best
Creep	Excessive	Less	Less	Minimum
Scrap value	Low	Higher than wooden	High	None

*CF stands for conventional fastening. **EF stands for elastic fastening.

2.7. High Speed Rails

High-speed rail is a type of passenger rail transport that operates significantly faster than the normal speed of rail traffic. Definition of high speed usually refers to 200 km/h (125 mph) and faster design speeds. But definition of high speed varies from country to country globally, as on there is no single standard worldwide.

2.7.1. Definition of High-Speed Rail

The UIC (International Union of Railways) defines high-speed rail as services which regularly operate at or above 250 km/h on new tracks, or 200 km/h on existing tracks. A number of characteristics are common to most high-speed rail systems. Most are electrical power driven obtained via overhead lines, although this is not necessarily a defining aspect and other forms of propulsion, such as diesel locomotives may also be used, in such as; Britain's High Speed Train services. A definitive aspect of high speed railway is the use of continuous welded rails, which reduce track vibrations and discrepancies between rail segments, enough to allow trains travel smoothly (with much less vibration and noise, compared to conventional trains) at speeds in excess of 200 km/h.

2.7.2. High Speed Definition of the European Union

2.7.2.1. High Speed Railway Infrastructure

a) The infrastructure for the Trans-European High Speed Railway System (TEHRS) shall be that conforming with the Trans-European Transport Network (TETN), Article 129C of the Treaty as;
    - those built specially for high speed travel
    - those specially upgraded for high speed travel. They may include connecting lines, including junctions of new lines, upgraded for high speed with town centre stations located on them, on which speeds must take account of local conditions.

b) High speed lines shall comprise of;

- specially built high speed lines equipped for speeds generally equal to or greater than 250 km/h,
- specially upgraded high speed lines equipped for speeds of the order of 200 km/h
- specially upgraded high speed lines, which have special features as a result of topographical, relief or town-planning constraints, on which the speed must be adopted in each case.

2.7.2.2. Rolling Stock

The high speed advanced-technology trains shall be designed in such a way to guarantee a safe and uninterrupted travel as;

- at a speed of at least 250 km/h on lines, specially built for high speed, while also enabling speeds of over 300 km/h to be reached in appropriate circumstances,
- at a speed of the order of 200 km/h on existing lines, which have been or are specially upgraded,
- at the highest possible speed on other lines.

2.7.2.3. Compatibility of Infrastructure and Rolling Stock

High Speed train services assume excellent compatibility between the characteristics of the high speed railway infrastructure and those of the rolling stock. Performance levels, safety, quality of service and costs depend upon that compatibility.

- **In view of infrastructure**

As regards to infrastructure, the definition of high speed rail covers a number of notions. A line is currently described as a "high speed line", when it is a new one designed to enable trains to operate at speeds above 250 km/h throughout the whole journey or at least over a significant part of the journey.

So any line, whether it is a new one or an upgraded conventional one, suitable for carrying traffic at up to 200 km/h, may be considered for being a high-speed railway line, if it satisfies special criteria, such as; substantial reductions in journey time, the crossing of mountains or straits, the use of narrow gauge track, the "network effect", etc. From the standpoint of the infrastructure, high speed traffic will thus comprise of all traffic running on high speed lines, regardless of the type of rolling stock used.

- **In view of rolling stock**

High speed from the rolling stock standpoint refers to the high speed rolling stock, which is normally composed of fixed formation motor coach sets, sometimes coupled together to form multiple units, capable of attaining at least 250 km/h in commercial services. In certain conditions trains may run at lower speeds (200 km/h), but offering high quality services, such as tilting trains. Those may also be described as high speed trains. As far as rolling stock is concerned, high speed traffic thus means all traffic using high speed stock, irrespective of the type of line on which it operates.

Finally, the term high speed train may also be applied to certain conventional trains made up of locomotives and coaches running at 200 km/h and meeting specific conditions.

- **In view of operating standpoint**

From operational viewpoint, the "high speed railway system" is difficult to define, as each infrastructure manager or train operator (ie. state railway company) has his own interpretation of it. Until now, it is not possible to harmonise the viewpoints of the various railway parties involved. One of the most tangible consequences of this is the difficulty of compiling statistics relating to high speed rail services and drawing up high speed network maps. Currently, there are four types of high speed railway systems operationally;

a) Type 1 is the most classic and the "purest" high speed system. This constitutes a network of lines used exclusively by high speed trains which themselves do not operate on any other lines. The Japanese Shinkansen systems are such systems.

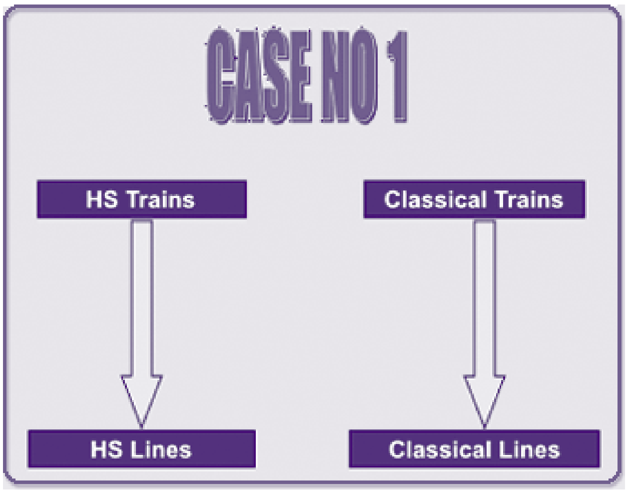


Fig 2.9. Type 1 of high speed railway system

b) Type 2 is a network of high speed lines, used exclusively by high speed trains, which also run on conventional lines. In the case of France, high speed trains also run on classic lines.

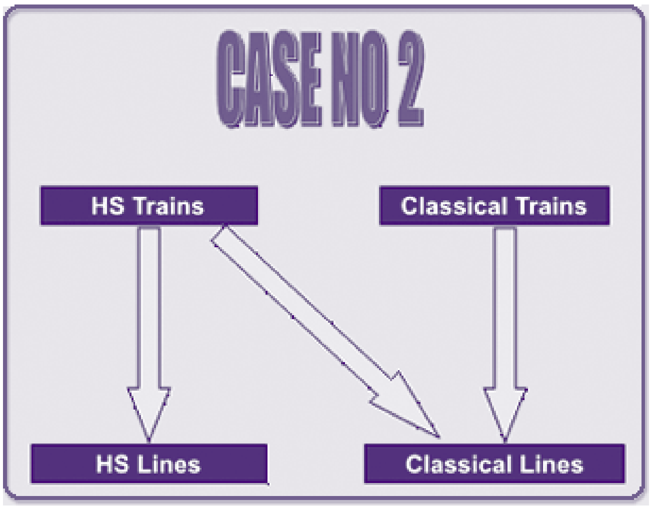


Fig. 2.10. Type 2 of high speed railway system

c) The Spanish system (AVE) constitutes type 3, that is; a system of high speed lines, which are used not only by high speed trains (> 250 km/h), but also by some conventional trains, equiped by changing gauge systems. However, at lower speeds, this invariably involves capacity reductions. On the other hand, high speed trains do not run on conventional lines.

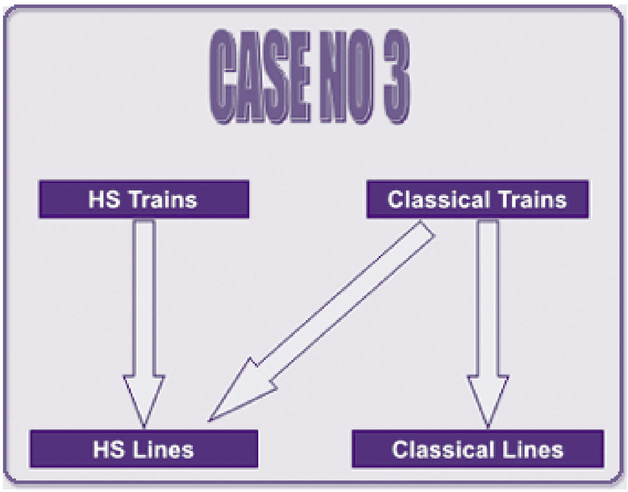


Fig. 2.11. Type 3 of high speed railway system

d) The German and Italian systems are examples of type 4 which permits all types of train run on the high speed lines and the high speed trains run on all types of lines.

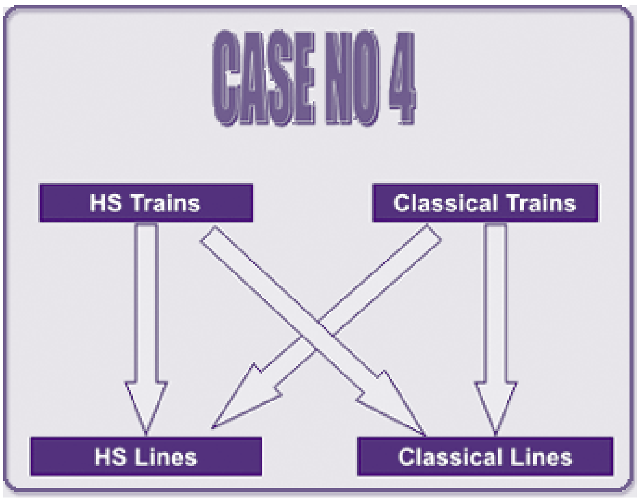


Fig. 2.12. Type 4 of high speed railway system

On the other hand, high speed railway traffic is calculated from the infrastructure standpoint. In certain cases, limits are set for using the minimum speeds, such as; 160 km/h or for the number of long-distance daytime services.

Finally, in the case of running through mountain tunnels or crossing long bridges, where the usual nominal speeds will not be more than 160 to 180 km/h, only high speed trains can cross them faster.

High speed is invariably associated with high quality of service. And it sometimes happens that a service offering a high level of comfort, frequency and accessibility, even when not accompanied by very high speed, is still labelled as high speed. This applies in particular to the case of express link services between cities and their airports, over distances of nearly 50 km., operated at about 15 minute intervals and at speeds of up to 200 km/h.

It also applies to the case, where railway services are used as a substitute for air services between airports and to services providing facilities typical of air services (for on-board staff, for passengers with embarkation cards, etc.).

Although on some networks freight trains were allowed in the past to run on the high speed lines, they are now severely restricted and at present they are allowed to operate only at nights.

An additional benefit of high speed rail service is high speed freight service, though in the European Union (EU) this is not well developed, but it is likely to be more developed in the near future. However, in the Far-East (in countries such as; Japan and Taiwan) it is well-developed. In Europe, use of conventional mail trains are common.

An essential additional feature to the "speed" based differentiation used (for example maximum trial speed, maximum operating speed, average speed, commercial speed, etc.), is the concept of the facilities provided within the high speed railway systems, not only on board the train, but throughout the whole journey from door to door. For this reason it is sometimes worthwhile adopting the term "high performance train".

- **Role of UIC**

The high-speed train is a means of transport of great capacity, resolutely with the service of the sustainable development, by its advantages in terms of safety, low fuel consumption of energy, absence of hydrocarbon pollution causing global climate change and minimal use of space. It is a formidable tool with the service of the regional planning and economic and social development, on each country scale, as on European scale. That development of high speed railway networks must be able to be based on effective modes of financing, associating all the actors who profit from this type of transport; European union, States, local authorities, companies.

UIC brings all its support to its members, wishing to develop high speed, in particular through its activities in the field of the technical harmonization and interworking, but also by facilitating the exchanges on the operational, commercial, economic and financial aspects. ref. (UIC, 2005).

The remit of the UIC High Speed group is to coordinate the activities of the different countries in the field of high speed and thereby play a part in helping create a true European high speed network and promoting high speed worldwide.

The UIC High Speed group is made up of 28 European railways from 23 countries: Austria, Belgium, Croatia, Czech Republic, Finland, France, Germany, Greece, Italy, Latvia, Luxembourg, Netherlands, Norway, Poland, Portugal, Romania, Slovaquia, Slovenia, Spain, Sweden, Switzerland, United Kingdom, Yugoslavia. Apart from the European countries, countries from other regions of the World are involved or interested in High Speed matters are: Japan, USA, Korea, China, Taiwan, Morocco, Saudi Arabia, Iran, Turkey, Russia, etc. All these countries' railways have developed high speed projects upto varying degrees.

The High Speed group is composed of a Plenary Committee bringing together all member railways (28 at the time of writing) and a Steering Committee comprising the railways making the largest contributions (CER, CD, DB AG, FS Trenitalia, PKP, RENFE and SNCF). Both Steering and Plenary committees meets twice a year.

2.7.2.4. High speed principles and advantages

Considerable advantages of high-speed trains are allowing;

- high capacity service
- environmental friendly service
- high safety

High Speed Railway components

These components are;

- Infrastructure
- Station emplacement
- Rolling Stock
- Operation rules
- Signalling systems
- Marketing
- Maintenance systems
- Financing
- Management
- Legal aspects

Benefits of high speed service for customers are :

- Commercial speed: travel with a high level of speed
- Total time of travel: benefit a short travel time from door to door
- Frequency: profit of a high level of available transports, that what signifies short total travel time (in general, the half of the frecquency is included in the total time of travel)
- Reliability: profit of a reliable system of transport, which works independent in nearly each case of weather
- Accessibility: you can enter a train spontaneous without long check in times, which supports you high level of flexibility
- Price

- Comfor: there is a higher level of comfort (in terms of space, accelerations, noise, light, etc.) than in the plane, bus, or a average car
- Safety: High speed trains are the safeties transport medium
- Freedom: during your trip, you can go every where and every time you want, else in the restaurant, to the lavatory, or only for promenade, seatbelts are nor necessaries, electronic devices aren’t limited, etc.

High Speed Advantages for the Society

- Offers high capacity of transport
- Up to 300,000 passengers per day
- Reduce traffic congestion
- Respects the environment
- Efficient use of land (1/3 motorway)
- Energy efficiency (x 9 planes / x 4 cars)
- Helps economic development
- High Speed Rail promotes logical territory structure and helps contain urban sprawl (UIC 2006).

CHAPTER 3

DESIGN PRACTICE OF HIGH SPEED RAILWAY EMBANKMENTS IN TAIWAN

3.1. General

In this section, special design practice of high-speed railway embankments in countries such as Taiwan (which was adopted from Japan) and France will be summarized, so that its parts could be modified to meet local conditions and then may be adopted for use also in Turkey.

As mentioned in the previous chapter, a rail track should be considered an engineering unit, hence a multi-layered composite system comprising all elements from natural ground up to rail level. The optimum design of a multi-layered rail track involves a gradual increase in stiffness from bottom to top layers. These are; natural ground (or natural subgrade layer), (man-made subgrade or just) subgrade layer, prepared subgrade layer, sub-ballast, ballast layer, wooden traverses and then rails connected to traverses with steel fasteners (Fig.3.1).

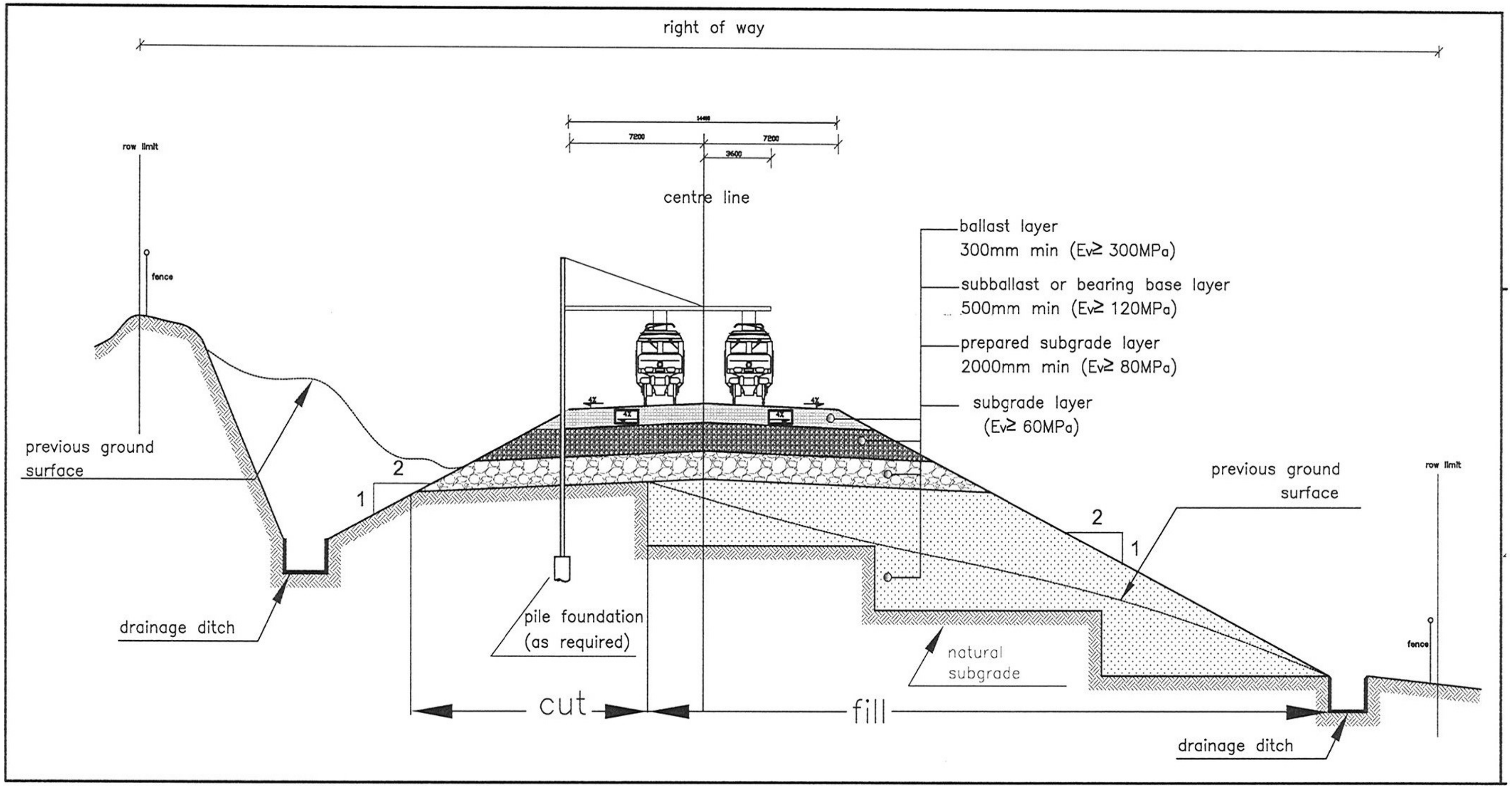


Figure 3.1. Typical cross-section in a mixed (cut-fill) portion of a high-speed train embankment usable globally

This layering type is the French (and the European) system, which is also used by Turkey. Under the traverses, French system uses ballast and sub-ballast, while Japanese and Taiwanese systems use precast concrete reinforced blocks (also called: Slab Track Layer) overlying the bearing base layer, engineering properties of which is the same as the sub-ballast layer used in France. Each system has advantages and disadvantages. The Japanese system is more expensive to construct, but easier and cheaper to maintain in the long run during operation, while the French system is cheaper to construct, but more difficult and more expensive to maintain in the long run during operation. Yet, the French system is more flexible to allow any excess unwanted rail settlements, which could easily repaired, while the Japanese system is more rigid to allow any excess rail settlements, which could hardly be repaired. So Turkey adopted the French system, which will be described here.

Achieving the required stiffnesses of the layers shown in Figure 3.1 depend not only on their material property, but also on their thickness, compaction degree and also on the stiffness of the underlying layer. Therefore the adverse effect of soft subsoil cannot be compensated by excessive compaction of the next layer. In the end, the entire system should exhibit an overall stiffness that allows minimum rail displacements on one hand, but also limits maximum rail displacement on another hand.

3.2. Ballast

The ballast layer is the select crushed granular material, placed as the top layer of the embankment containing the high speed railway substructure in which the sleepers (or traverses) are embedded.

Traditionally, angular, crushed, hard stones and rocks, uniformly graded, free of dry and dirt (and not prone to cementing action) have been considered as good ballast material . However, at present no universal agreement exists concerning the proper specifications for the ballast material’s index characteristics such as; size, shape, hardness, abrasion resistance and compaction degree that will provide the best track performance. This is a complex subject that is still being researched. Availability and economic considerations have been the prime factors considered in the selection of the ballast materials. A wide variety of materials have been used for ballast such as; crushed granite, basalt, limestone, slag and gravel.

3.2.1. Functions of Ballast

The ballast serves the following functions in a railway track.

- Resist vertical (including uplift), lateral and longitudinal forces applied to the sleepers to retain track in its required position.
- Provide some of the resiliency and energy absorption for the track.
- Provide large voids for storage of fouling material in the ballast, and movement of particles through the ballast
- Facilitate maintenance surfacing and lining operations (to adjust track geometry) to the ability to rearrange ballast particles with tamping.
- Provide immediate drainage of water falling onto the track.
- Reduce pressures from the sleeper bearing area to acceptable stress levels for the underlying material.

Note that although the average stress will be reduced by increasing the ballast layer thickness, high contact stresses from the ballast particles will required and the material in the layer supporting the ballast should be stiff (well compacted) and durable.

Other functions are:

- Alleviate frost problem by not being frost susceptible and by providing an insulating layer to protect the underlying layers.
- Inhibit vegetation growth by providing a cover layer that is not suitable for vegetation.
- Absorbs airborne noise.
- Provide adequate electrical resistance between rails.
- Enable to facilitate redesigning and reconstruction of track.

The mechanical properties of ballast layer result from a combination of the physical properties of the individual ballast material and its in-situ (i.e., in-place) physical state. Physical state can be defined by the in-place density tests, while the physical properties of the material can be described by various indices such as particle size, shape, angularity, hardness, surface texture and durability. The in-place unit weight of ballast is a result of compaction processes.

After placement and during the service life, ballast gradation changes as a result of:

- mechanical particle degradation during construction and maintenance work, and under traffic loading,
- chemical and mechanical weathering degradation from environmental changes, and migration of the fine particles from the underlying layers towards the surface layers (ie. vertically upward migration of fine particles should be prevented).

Thus the ballast layer may become fouled and may loose its open-graded characteristics so that the ability of the ballast layer to perform its important functions may decrease or may be lost completely.

3.3. Sub-Ballast (or Bearing Base) Layer

The sub-ballast layer (or bearing base layer) supports the ballast layer (in the French System) or slab track (in the Japanese System) is an intermediate layer overlying the subgrade layers. It shall be provided to ensure better distribution of loads, to protect the subgrade layer against erosion, to avoid seepage into the subgrade layer and to prevent the penetration of fine soil material into the track bed, especially to the ballast.

Sub-ballast fulfills following functions:

1. Reduce the traffic induced stress at the bottom of the ballast layer to a tolerate level for the top of the prepared subgrade.
2. Extend frost protection to the prepared subgrade and subgrade layers.

In fulfilling these functions, the sub-ballast layer reduces the otherwise required great thickness of the more expensive ballast material. However, the sub-ballast layer has some of important functions that can not be fulfilled by the ballast layer. These are:

3. Prevents interpenetration of the prepared subgrade and the ballast layers,
4. Prevents upward migration of fine material emanating from the subgrade layers,
5. Prevents subgrade attrition by ballast, which in the presence of water, leads to unwanted slump formation and hence it prevents this type of problem. This becomes more problematic, particularly if the prepared subgrade layer is hard.
6. Sheds water, i.e., intercept water, coming from the ballast and directs it away from the prepared subgrade layer into ditches at the sides of the track.

7. Permits drainage of water, that might be rising upward from the prepared subgrade and subgrade layers by capillarity action.

These are very important functions for designing high speed railway embankment in order to have a satisfactory track performance. Hence in absence of a sub-ballast layer, a high maintenance effort can be expected, unless such above mentioned functions are fulfilled in some other manner.

The most common and most suitable sub-ballast materials are broadly-graded naturally occurring or processed sand-gravel mixtures, or broadly-graded crushed natural aggregates or slags. They must have durable particles and satisfy the filter/separations requirements for ballast and subgrade.

The minimum thickness of the sub-ballast (or the bearing base) layer shall be 0.50m. The top surface of the sub-ballast or bearing base layer shall have a transverse slope of no less than 4% as shown in Figure 3.2.

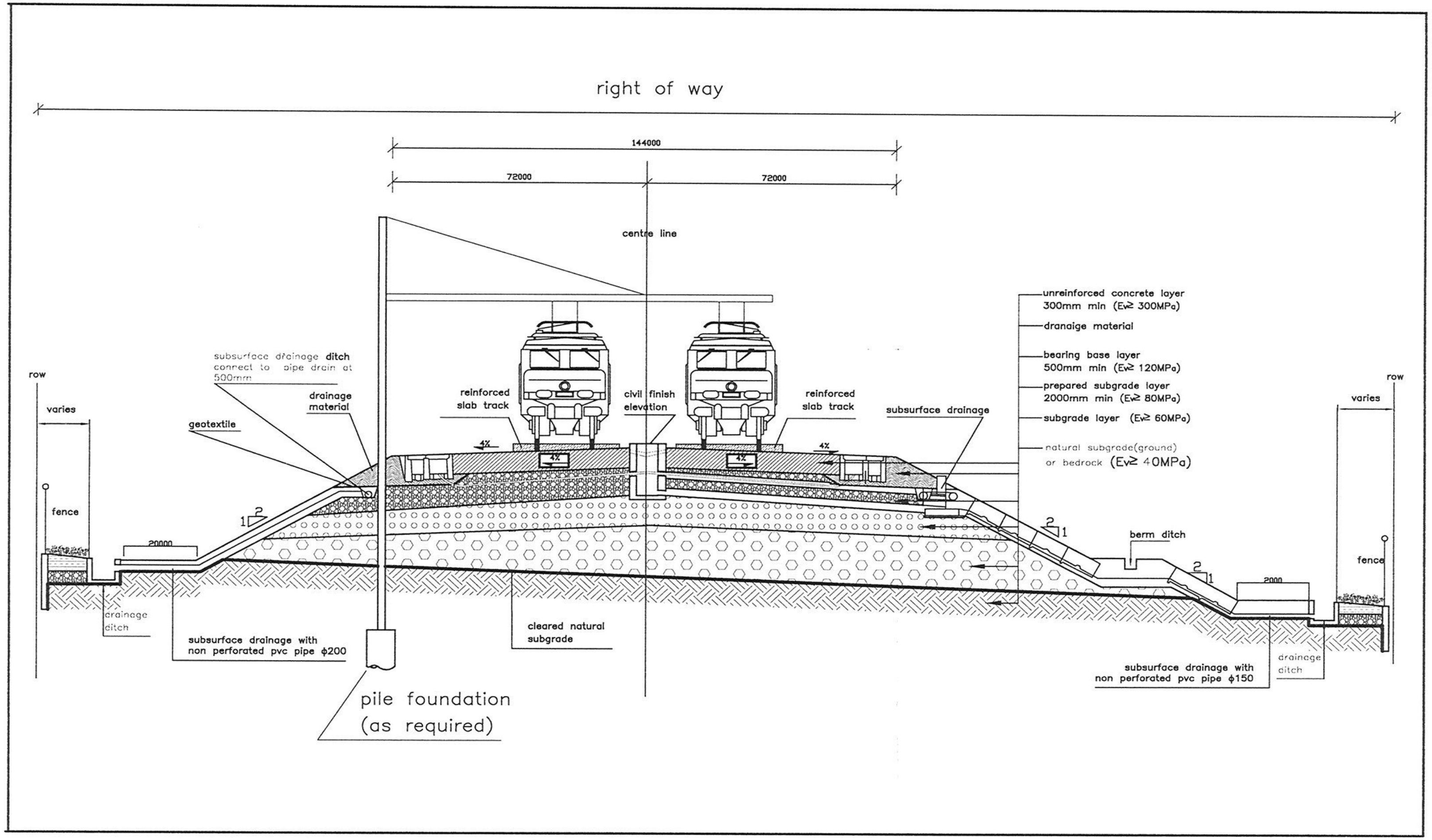


Figure 3.2. Typical fill section slab-track type high-speed train embankment usable in Japan/Taiwan

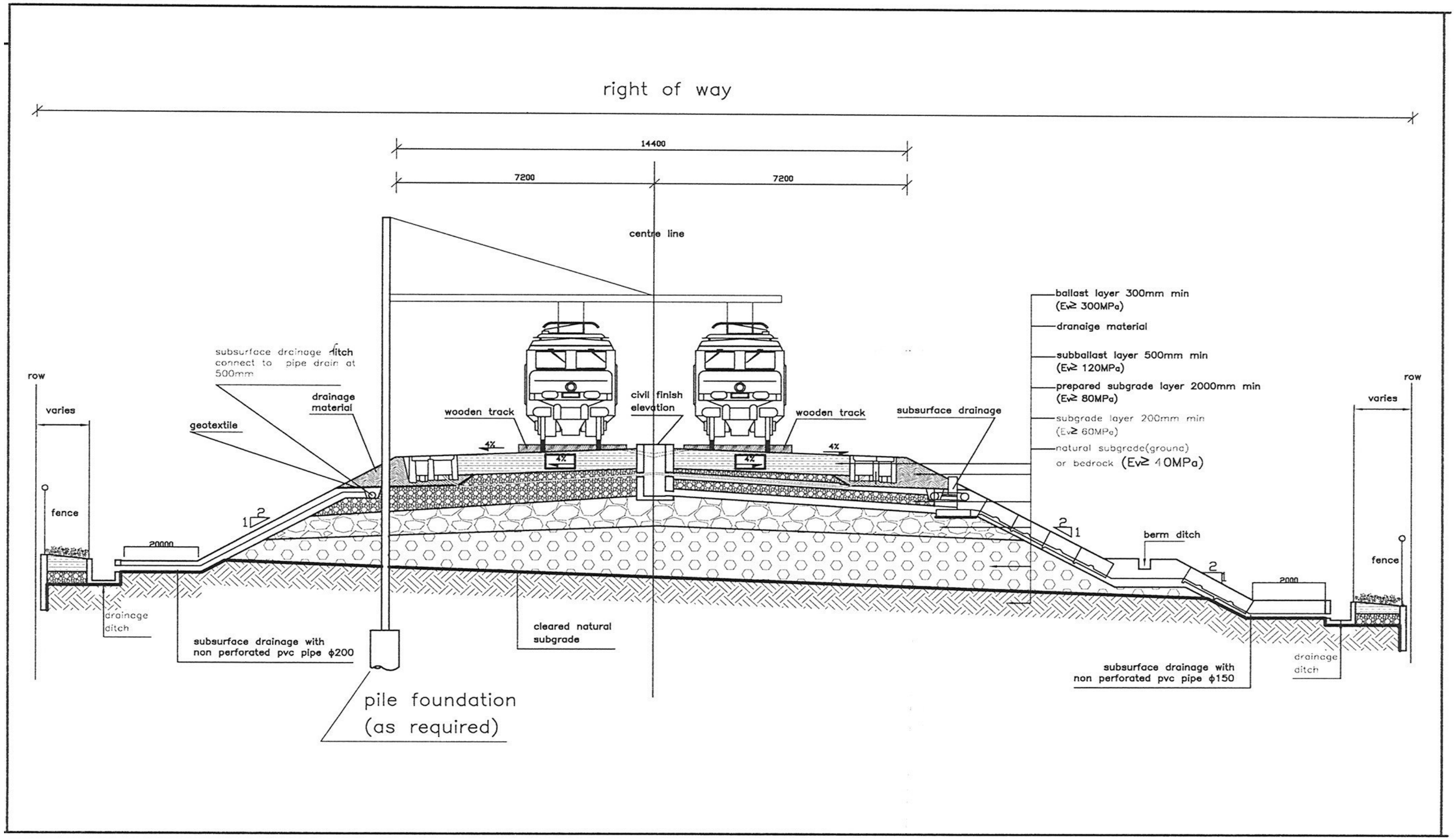


Figure 3.3. Typical fill section of a ballasted-track type high-speed train embankment usable in France/Turkey

Typical viaduct/embankment and embankment/tunnel transition in a longitudinal section is shown in Figure 3.4.

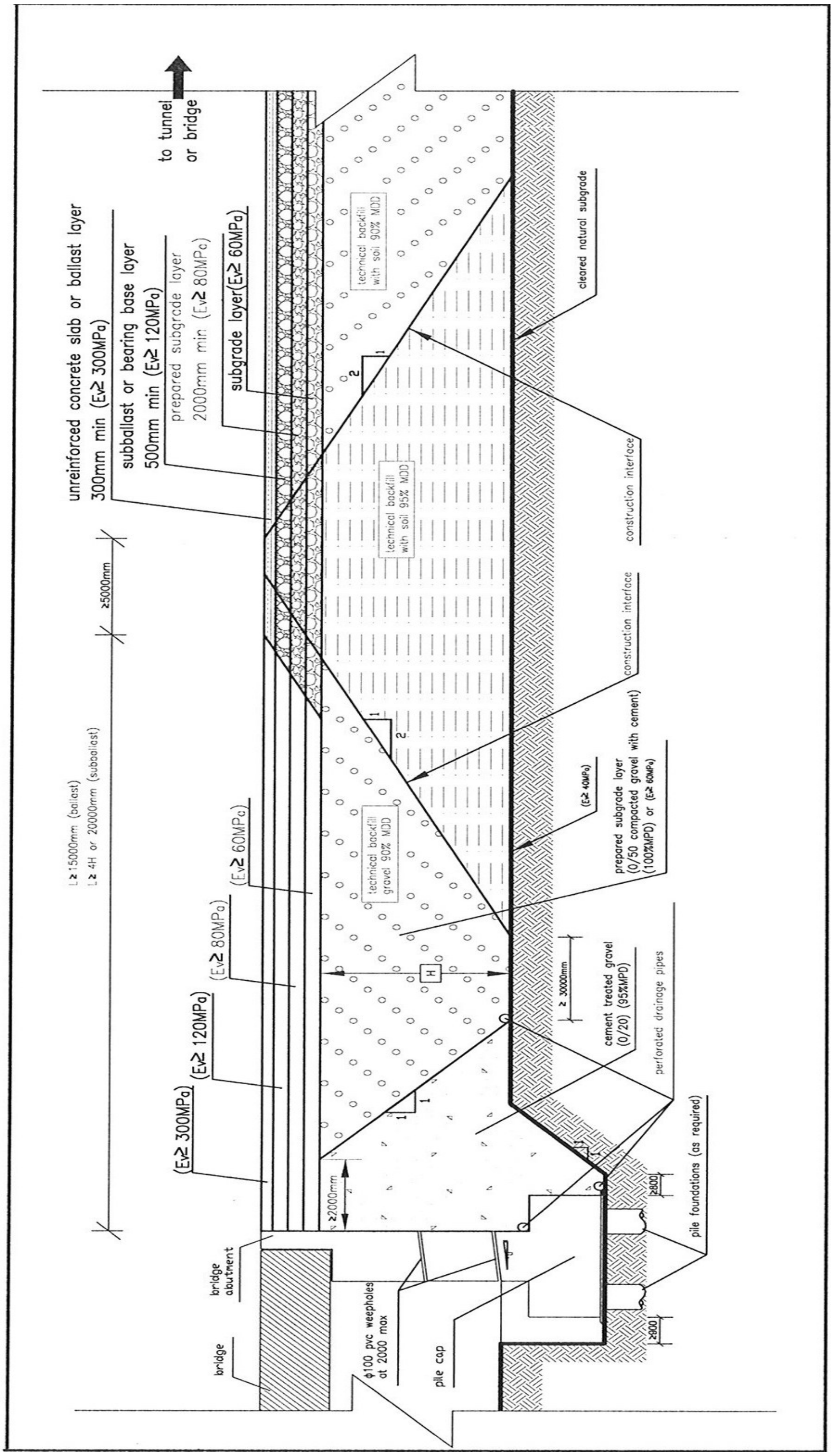


Figure 3.4. Typical longitudinal fill section of a high-speed train embankment for transi-tion zones between bridge-embankment and embankment-tunnel or bridge

The required physical material characteristics of the sub-ballast or bearing base layer are as follows (THRSC 2003) ;

$$\text{Hardness : } R = LA + MDH \leq 40 \tag{3.1}$$

where

LA= Los Angeles Test

MDH= Micro Deval Humid Test

$$\text{Flakiness : } A \leq 25 \tag{3.2}$$

$$\text{Cleanliness : } Vbg \leq 1 \tag{3.3}$$

The mechanical properties of the sub-ballast or bearing base layer shall meet the following specified values of the associated tests:

$$\text{Modified Proctor Test : } \rho_d \geq 100\%MPD \tag{3.4}$$

$$\text{Plate Bearing Test : } E_v \geq 120Mpa \tag{3.5}$$

where

ρ_d : Field dry density

MPD: Maximum dry density, as determined by the Modified Proctor Test(ASTM D698)

E_v : Deformation modulus of loading from the Plate Load Test (ASTM D 1196-93(2004))

Sub-ballast or bearing base layer should provide a very low permeability (indicative value : $\leq 10^{-6}$ m/s).

Table 3.1. Gradation for sub-ballast or bearing base material (0/31.5)

Grain Size	Percentage Passing
P(2D)	100
P(1.58D)	100-95
P(D) (D=31.5 mm)	99-85
P(D/2)	90-72
P(D/5)	80-57
P(D/10)	72-46
P(D/20)	65-37
P(D/50)	54-26
P(D/100)	45-18
P(D/200)	36-10
P(D/500)	19-0
P(D/1000)	7-0
P(D/2000)	2-0

3.4. Prepared Subgrade Layer

Depending on the quality of the top of subgrade as specified in subsection 3.5, a prepared subgrade layer may be necessary.

If required, the prepared subgrade layer shall be between the sub-ballast or bearing base layer and the subgrade. The function of this layer is to minimize the deformation of the earthwork and to further prevent water that has passed through the sub-ballast or bearing base layer from penetrating into the earthwork below (THRSC, 2003).

3.4.1. Material Properties

The material for prepared subgrade layer shall meet the quality requirements QS3 as specified in Table 3.4 and shall meet the grain size gradation requirements as shown in Table 3.2, (THRSC, 2003).

Table 3.2. Gradation for Prepared Subgrade Material

Grain Size	**Percentage Passing**
P(2D)	100
P(Dmax)	100 - 99
P(D)	99 - 85
P(D/2)	84 - 55
P(D/5)	60 - 31
P(D/10)	49 - 23
P(D/20)	40 - 17
P(D/50)	31 -11
P(D/100)	22 – 8
P(D/200)	16 – 6
P(D/500)	9 – 3
P(D/1000)	6 – 2

D = nominal grain size

$$D_{max} = 1.25D \text{ if } D \geq 50 \text{ mm}; \quad D_{max} = 1.58D \text{ if } D < 50 \text{ mm} \tag{3.6}$$

The following physical characteristics of the material shall also be required:

$$\text{Hardness: } R = LA + MDH \leq 60 \tag{3.7}$$

$$\text{Flakiness: } A \leq 25 \tag{3.8}$$

$$\text{Cleanliness: } Vbg \leq 2 \tag{3.9}$$

3.4.2. Thickness and Shape

Depending on the quality class of the top of subgrade specified in Table 3.4, the thickness of the prepared subgrade layer shall be sized according to Table 3.3, (THRSC, 2003).

Table 3.3. Thickness Requirement for Prepared Subgrade Layer

Quality Class Subgrade or Cut	Track Type	Prepared Subgrade Thickness
QS1	Ballasted Track	0.50 m added material
QS2	Ballasted Track	0.35 m added material
QS3	Ballasted Track	0.30 m added material

The top surface of prepared subgrade layer shall have a transverse slope of no less than 4%.

3.4.3. Bearing Capacity and Compactness

The mechanical properties required for prepared subgrade layer shall be defined by the following standard tests, (THRSC, 2003) :

- Modified Proctor Test (ASTM D 698)
- Plate Bearing Test (ASTM D 1196-93(2004))

Where the maximum particle size is less than 100 mm, a Dynaplate Test may be substituted for the Plate Bearing Test.

Required values for the tests mentioned above are as follows:

$$\text{Modified Proctor Test : } \rho_d \geq 95\%\text{MPD} \tag{3.10}$$

$$\text{Plate Bearing Test : } E_v \geq 80\text{Mpa} \tag{3.11}$$

$$\text{Dynaplate : R (Release coefficient)} > 50\% \tag{3.12}$$

Table 3.4. Quality class of soils (adapted from UIC 719 R)

Soil Classification (Geotechnical identification)	**Soil Quality Class**
0-1 Loose organic soils 0-2 Fine soils (incorporating more than 15% fines *), swollen, moist nd therefore not compatible (where enhancement by treating with binders is not possible for technical or economic reasons) 0-3 Thixotropic soils (quickly clay, for example) 0-4 Soluble materials (soil containing rock salt or gypsum) 0-5 Polluting materials (industrial waste, for example) 0-6 Mixed "mineral-organic"soils	QS0
1-1 Soils containing more than 40% fines* 1-2 Rocks highly susceptible to weathering for example: •chalk with $\rho_d < 1.7$ t/m^3 and highly friable •marls •weathered schists	QS1
1-3 Soils comprising 15 to 40% fines* 1-4 Rocks moderately suspectible to weathering for example: •chalk with ρd < 1.7 t/m3 and slightly friable •schists 1-5 Soft rock for example: dry Deval ≤ 6 and Los Angeles > 33	QS1**
2-1 Well graded soils containing 5 to 15% fines* 2-2 Uniform sands containing less than 5% fines* (Cu ≤ 6) 2-3 Rock or medium hardness for example: 6 < dry Deval ≤ 9 and 33 > Los Angeles > 30	QS2***
3-1 Well graded soils containing less than 5% fines* 3-2 Hard rock for example: dry Deval > 9 and Los Angeles ≤ 30	QS3

* These percentages are based on materials passing 0 mm sieve openings. The percentages given here are orders of magnitude.
**These soils can be of quality QS2 if the hydrogeological and hydrological conditions are good. (as specified in Subsection 3.5.2)
***These soils can be of quality QS3 if the hydrogeological and hydrological conditions are good. (as specified in Subsection 3.5.2)

3.5. Subgrade Layer

The subgrade layer is the platform upon which the track structure is constructed. Its main function is to provide a stable foundation for the sub-ballast and ballast layers. The influence of the traffic induced stresses extends downward as much as five meters below the bottom of the sleepers. This is considerably beyond of the ballast and sub-ballast layers. Hence the subgrade layer is a very important substructure component which has a significant influence on the track performance and maintenance. For example; subgrade layer is a major component of the superstructure support resiliency and hence contributes substantially to the elastic deflection of the rail under wheel loading. In addition, the subgrade layer's stiffness magnitude is believed to influence ballast, rail and sleeper deterioration. Subgrade layer may also be the source of rail differential settlement.

The subgrade layer may be divided into two categoris:

1. Natural subgrade layer (original ground)
2. (Man-made) subgrade layer (Placed soil/fill)

Anything other than soils existing locally is generally uneconomical to use for the subgrade layer. Existing ground will be used without disturbance, as mush as possible. However, oftenly some of the formation must be removed to construct the track at its required elevation. Placed fill is used either to replace the upper portion of unsuitable existing ground or to raise the platform to the required elevation for the rest of the track structure.

To serve as a stable platform, the following subgrade failure modes must be avoided:

1. Excessive progressive settlement from repeated traffic loading.
2. Consolidation settlement and massive shear failure under the combined weights of the train, track structure and earth.
3. Progressive shear failure (soil heave) from moisture loading.
4. Significant volume change (swelling and shrinking) from moisture change.
5. Frost heave and thaw softening.
6. Subgrade attrition.

In addition to its other functions, the subgrade layer must provide a suitable base for construction of the sub-ballast and the ballast layers.

3.5.1. Quality Class of Subgrade

The quality shall be defined according to (THRSC, 2003):

- geotechnical properties of subgrade layer, as defined in Table 3.4.
- hydrogeological conditions of the job site.
- hydrological conditions of the job site.

Four types of soils are classified as follows:

- QS0: ''Unsuitable'' Soils that need removal or stabilization.
- QS1: ''Poor'' Soils – These soils may be accepted ''as is'', however, drainage must be provided. Soil improvement may be needed.
- QS2: ''Average'' Soils
- QS3: ''Good'' Soils

3.5.2. Hydrogeological and Hydrological Conditions

Hydrogeolagical and hydrological conditions shall be classified as ''Good'', if the following conditions are satisfied, (THRSC, 2003):

- The top of the subgrade layer exists above the possible highest groundwater table (This condition exists, when the highest level of the groundwater table is more than 21.50 m below the top of the sub-ballast layer in an unfavorable climatic seasons);
- The subgrade layer is free of harmful transverse, longitudinal or vertical natural percolation;
- Rainwater is properly evacuated from the top of subgrade layer and thorough the longitudinal and transverse drainage networks.

If any of these conditions is not satisfied, the hydrogeological and hydrological conditions shall be classified ''Poor''.

All other earthwork and drainage designs shall result in ''Good'' hydrogeological and hydrological conditions.

3.5.3. Bearing Capacity and Compactness

The mechanical properties of the subgrade layer shall meet the following specified values of the associated tests, (THRSC, 2003):

- Modified Proctor Test (ASTM D698):

$$\text{For ballasted track}: \ \rho_d \geq 90\%\text{MPD} \tag{3.13}$$

$$\text{For slab track}: \ \rho_d \geq 95\%\text{MPD} \ \text{ for soils containing 5\textasciitilde 40\% fines} \tag{3.14}$$

$$\rho_d \geq 92\%\text{MPD} \ \text{ for others} \tag{3.15}$$

- Plate Bearing Test (ASTM D 1196-93(2004)):

$$E_v \geq 45\text{Mpa} \ \text{ for fine soils, and} \tag{3.16}$$

$$E_v \geq 60\text{Mpa} \ \text{ for sandy and gravel soils} \tag{3.17}$$

3.6. Possible Modification to the Prepared Subgrade Layer

Normally this is a uncemented sandy and well compacted layer, as described earlier. If the prepared subgrade layer is required to be made stiffer due to minimizing detrimental differential settlements, then cement bonding could be made use of. In the next chapter (4), properties of uncemented and cemented prepared subgrade layer will be compared, using locally obtained Turgutlu sand.

3.7. Longitudinal Section of High Speed Railway Embankments icluding the transition zones

The fill part in the previously given mixed-section of embankment (Fig.3.1) presents a section for the middle part of the embankment, which does not include ant transition zone from bridge to embankment and from tunnel to embankment (and vice versa), which should be stiffer to prevent detrimental differential settlements. This is because, bridge is usually built on piles, virtually does not settle (during operational life of the railway) and tunnel usually in rock or hard soils also does the same.

CHAPTER 4

LABORATORY TESTS ON UNCEMENTED AND CEMENTED TURGUTLU SAND

4.1. Introduction

In order to further study the properties of the 'prepared subgrade' layer used in the high speed railway embankments as described previously in section 3.4 of the thesis, similar 2 materials meeting the required criteria are obtained using locally obtained Turgutlu sand, one as is (uncemented) and the other is cemented. In this part technical characteristics and laboratory test results will be presented with the aim of replacing the uncemented natural mix with a cemented mix, which will give more rigidity to the embankment and thus will further reduce detrimental differential settlements.

For our experiments Turgutlu coarse sand which is commercially available for sale in bags was used. Turgutlu sand is provided from the Turgutlu basin. Turgutlu sand has rounded and angular particles. Sand particles are durable and do not further disintegrate. Sand particles' color varies from beige to light brown.

4.2. Laboratory Tests on Uncemented Turgutlu Sand (PART 1)

In this section, some basic index tests of Turgutlu Sand are presented. These include particle size analysis, compaction test, specific gravity, water content, Atterberg Limits, Consolidated Drained (CD) type triaxial, direct shear and permeability tests.

4.2.1. Particle Size Analysis

This test is applied to determine the variation of different grain sizes contained within a soil specimen. The mechanical or sieve analysis is performed to determine the distribution of the particle sizes. The test standard used was; ASTM D 422-63 Volume 04.08 - Standard Test Method for Particle Size Analysis of Soils.

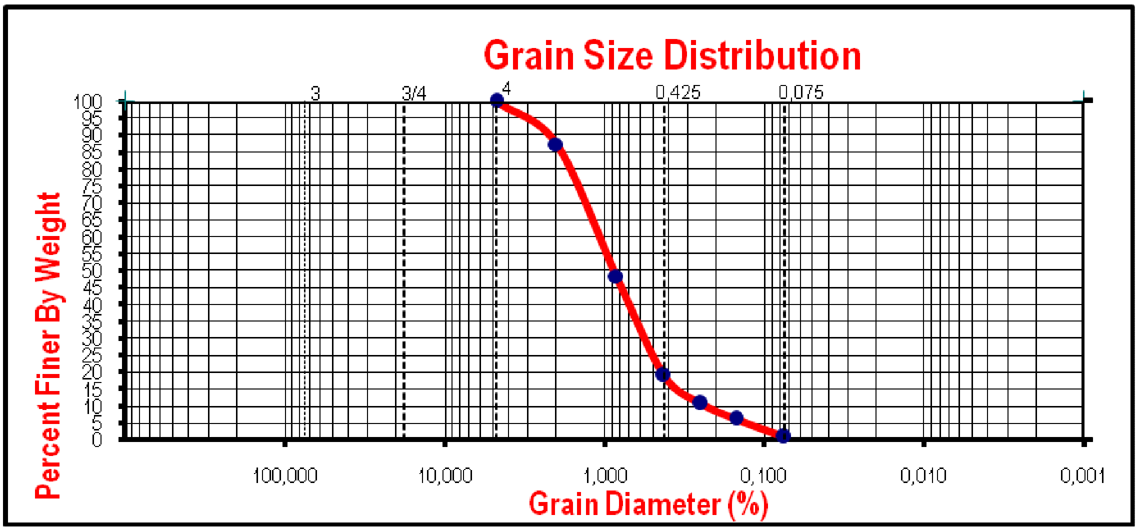


Figure 4.1. Particle size distribution (Sieve Analysis) result of Turgutlu Sand

4.2.2. The Laboratory Compaction Test

For this ASTM D 698-00 Volume 04.08: Standard Test Method for Laboratory Compaction Characteristic of Soil Using Standard Compaction Effort (600 kN – m/m^3) was used.

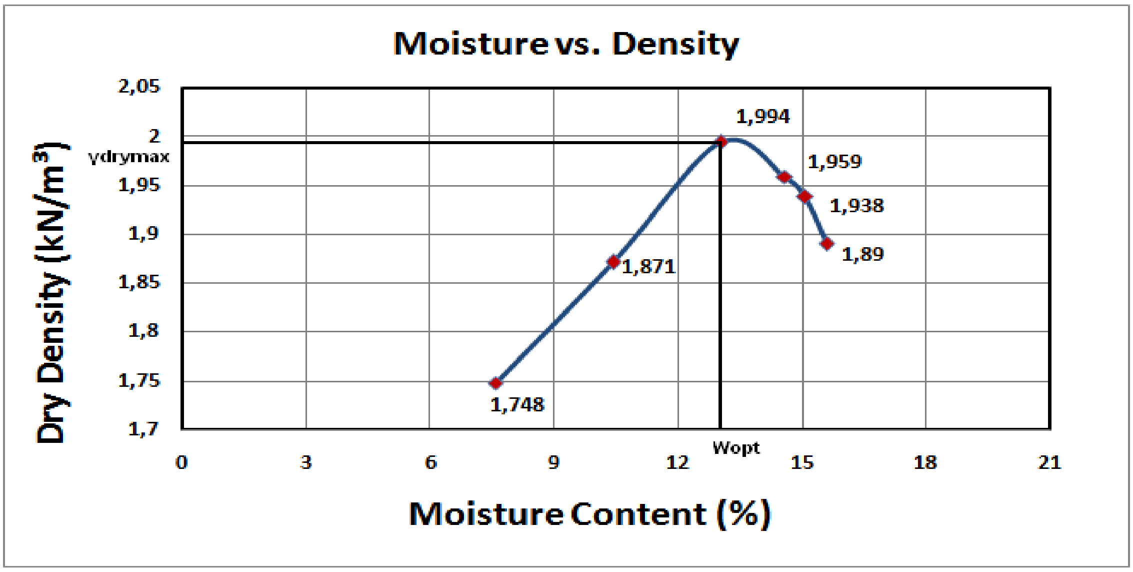


Figure 4.2. Moisture density relationship of Turgutlu sand

Optimum moisture content, (%), W_{opt} = 13.6

Maximum index unit weight , (kN/m^3), γ_{drymax} = 19.994

4.2.3. Specific Gravity of Soil Solids

Specific gravity is ratio of the mass of unit volume of a soil at a stated temperature to the mass of the same volume of gas-free distilled water at the same temperature. ASTM D 854-02 Volume 04.08 : Standard Test Method for Specific Gravity of Soil Solids by water pycnometer and AASHTO T 100 : Standard Test Method for Specific Gravity of Soils was used in this study. The specific gravity of the Turgutlu sand was found as 2.65 .

4.2.4. Determination of Water Content

The water (moisture) content is the ratio, expressed as a percentage, of the mass of “pore” or “free” water in a given mass of soil to the mass of the dry soil solids. For the test procedure; ASTM D 2216 Volume 04.08: Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock by Mass was used in this study. Natural moisture content of Turgutlu Sand was found as 3.3%, before using it for tests in the laboratory.

4.2.5. Classifiction of Turgutlu Sand using the Uniform Soil Classification System

In classifying mineral and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit and plasticity index, a widely used standard among practicing engineers is; ASTM D 2487-00 Volume 04.08: Standard Practice for Classification of Soils for Engineering Purposes: The Unified Soil Classification System-USCS) was used. Procedure involves steps, as described hereunder:

Procedure for Classification of Coarse-Grained Soils

- Class the soil as sand if 50% or more of the coarse fraction [plus No. 200 (75-µm)sieve] passes the No.4 (4.75-mm) sieve.
- If 12% or less of the test specimen passes the No.200 (75-µm) sieve, plot the cumulative particle-size distribution and then complete the coefficient of uniformity, C_u and the coefficient of curvature C_c , as given in Egs.1 and 2 below:

$$C_u = \frac{D_{60}}{D_{10}} \tag{4.1}$$

$$C_c = \frac{(D_{30})^2}{(D_{10} * D_{60})} \tag{4.2}$$

where;

D_{10}, D_{30}, and D_{60} = the particle-size diameters corresponding to 10, 30, 60% passing, respectively. Thus, $D_{10} = 0.28$ $D_{30} = 0.58$ $D_{60} = 1.2$

Table 4.1. Soil Classification Chart of Turgutlu Sand Using the Uniform Classification System (USCS)

Coarse - Granied Soil	**More than 50 % retained on No. 200 sieve**
Gravels	50 % or more of coarse
Clean Gravels	Less than 5 % fines
$C_u \geq 4$ and $1 \leq C_c \leq 6$	$C_u < 6$ and/or $6 > C_c > 1$
Group Symbol	SP
Group Name	Poorly graded sand
D_{10}	0.28
D_{30}	0.58
D_{60}	1.2
Coefficient of curvature, C_c	1.00
Coefficient of uniformity, C_u	4.28

4.2.6. Visual Classification of Soils

Based on the classification system described in ASTM D 2487-00 Volume 04.08: Standard Practice for Classification of Soils for Engineering Purposes: The Unified Soil Classification System(USCS), the identification should also include some visual examination and manual tests. Thus, visual classification of Turgutlu Sand was performed and the results are as follows;

1. Color: brownish-gray
2. Odor: none
3. Texture: coarse-grained soils
4. Major Soil Content: sand
5. Minor Soil Content: gravel, fines

Type	Approx. % by weight
sand	95
fine gravel	3
gravel	2

6. For Coarse-Granied Soil

   Gradation: Poorly graded

   Particle Shape: rounded
7. Moisture Condition: dry

Classification: brownish-gray sand, little fine gravels, poorly graded, rounded, dry.

4.2.7. Maximum and Minimum Index Density Tests

This experiment is applied to determine the relative density of cohesionless, free-draining soils using a vibrating table. The relative density of a soil is the ratio, expressed as a percentage, of the difference between the maximum index void ratio and the field void ratio of a cohesionless, free-draining soil; to the difference between its maximum and minimum index void ratios. For this purpose; ASTM D 4253-00 Volume 04.08: Standard Test Method for Maximum Index Density and Unit Weight of Soils Using a Vibrating Table-Method 2A was used, where the procedure included use of oven-drying the soil and a vertically shaking vibrating table, operated by an eccentric or cam driven electric motor. Results obtained from this test for the Turgutlu Sand is summarized in Table 4.2.

Table 4.2. Maximum, minimum unit weights of Turgutlu Sand from vibration table tests

Experiment Name	Value
The maximum index void ratio (e_{max})	0.516
The minimum index void ratio (e_{min})	0.302
The void ratio (e)	0.37
The relative density (D_r)	68.22%
The maximum index density (ρ_{max})	2.036 g/cm^3
The minimum index density (ρ_{min})	1.748 g/cm^3
The maximum unit weight (γ_{max})	19.994 kN/m^3
The minimum unit weight (γ_{min})	17.142 kN/m^3

4.2.8. Coefficient of Permeability by the Falling Head Method

For these tests, ASTM D 5084-03 Volume 04.08: Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Metarials Using a flexible (varying height) wall permeameter was used. There are six alternate methods for hydraulic systems that may be used to measure the hydraulic conductivity. We used MethodC: Falling Head, rising tailwater elevation. Finally, the coefficient of permeability was obtained as: $k = 10^{-1}$ cm/sec.

4.2.9. Direct Shear Test on Turgutlu Sand

Direct shear test is one of the most widely used laboratory tests to determine shear strength of cohesionless soils, such as; sands and silts. Using this test, effective stress parameters of soils, that are; angle of internal friction (in degrees) and cohesion (in stress terms) are obtained. In order to perform the test, the reference method used was: ASTM D 3080-04 Volume 04.08: Standard Test Method for Direct Shear Test under Drained conditions.

Figure 4.3. ELE International Direct Shear Test Machine at the Ege University

As one of the parameters to vary in the test is the degree of compaction of the prepared sample in the shear box before shearing it, this was taken to represent the stress level at the middle of a typical prepared subgrade layer of a railway embankment in fill, as shown in Fig.3.1. 4 different samples were placed into the shearbox (whose dimensions are 50x50x25mm, above the sample 5mm high porous stone top cap is placed) and then compacted to normal stresses varying between 27-218 Kpa. Shearbox has 2 parts, where the upper one is fixed and the lower one can move at a selected shearing rate, which was chosen as 2.5 mm/min. Within the maximum allowed horizontal displacement of 10mm, during the test (shearing stage) developed shear stresses, horizontal dispalcements, vertical /normal stresses and vertical displacements are transferred to a data loger and can be read from a personal computer (PC). Test results are summarized in Table 4.3 and the results obtained from the plotted graph shown in Fig.4.3 are;

(Effective) Angle of Internal Friction , Ø' (°) = 36.88
(Effective) Cohesion , C' (kPa) = 7

Table 4.3. Direct Shear Test Results of Turgutlu Sand

Reference	**A**	**B**	**C**	**D**
Normal Stress	0 kPa	27 kPa	81.5 kPa	190.5 kPa
Peak Strength	7 kPa	75.8 kPa	90.3 kPa	228.8 kPa
Corresponding Horizontal Displacement	6.428 mm	8.329 mm	2.902 mm	3.527 mm
Residual Stress	N/A	N/A	N/A	N/A
Rate of Shear Displacement	Stage 1: 1.0000mm/min	Stage 1: 1.0000mm/min	Stage 1: 1.0000mm/min	Stage 1: 1.0000mm/min
Final Height	19.76 mm	19.22 mm	19.48 mm	19.88 mm
Sample Area	3600.00 mm^2	3600.00 mm^2	3600.00 mm^2	3600.00 mm^2
Initial Wet Unit Weight	20.98 kN/m^3	20.98 kN/m^3	20.98 kN/m^3	20.98 kN/m^3
Initial Dry Unit Weight	20.98 kN/m^3	20.98 kN/m^3	20.98 kN/m^3	20.98 kN/m^3
Final Wet Unit Weight	21.23 kN/m^3	21.83 kN/m^3	21.54 kN/m^3	21.11 kN/m^3
Final Dry Unit Weight	21.23 kN/m^3	21.83 kN/m^3	21.54 kN/m^3	21.11 kN/m^3
Final Moisture Content	0.0 %	0.0 %	0.0 %	0.0 %
Particle Specific Gravity	2,65	2,65	2,65	2,65
Final Void Ratio	0.2244	0.1909	0.2067	0.2316
Final Saturation	0.00%	0.00%	0.00%	0.00%

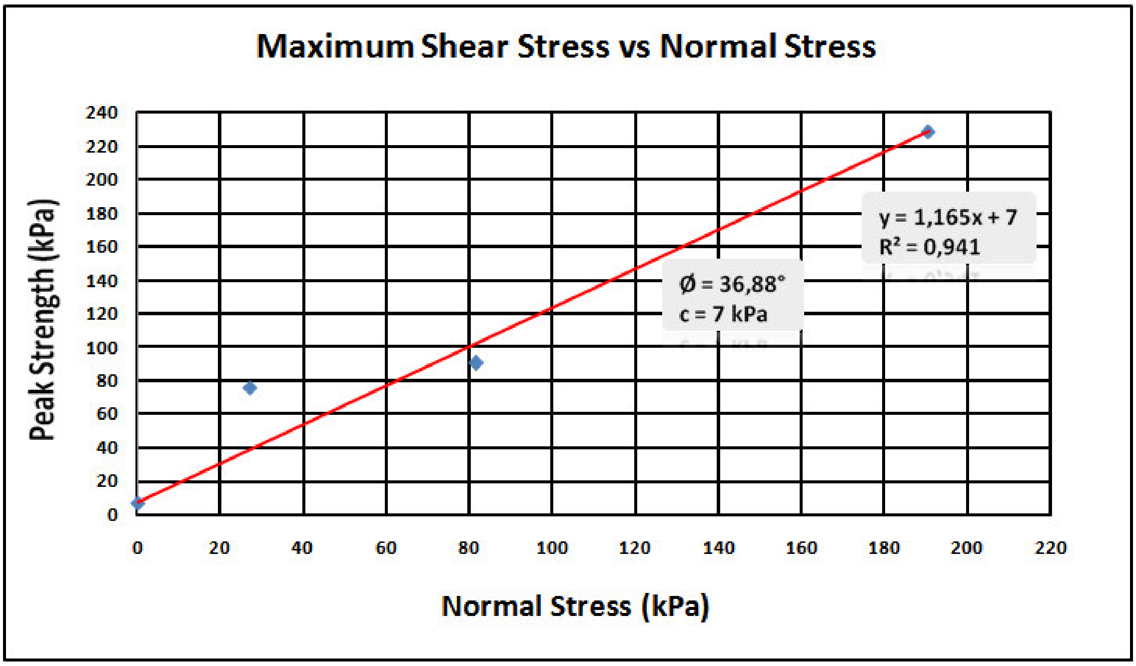


Figure 4.4. Graph showing the results of the Direct Shear Tests performed on Turgutlu sand

4.2.10. Consolidated Drained Triaxial Tests on Turgutlu Sand

Using the triaxial test aparatus owned by the Ege University (Figure 4.5) Consolidated Drained (CD) type of triaxial compression test is used to measure the shear strength of Turgutlu sand. In the conventional triaxial test, a cylindrical specimen of soil encased in a rubber membrane is placed in a triaxial compression chamber, subjected to a confining fluid pressure and then loaded axially to failure. Connections at the ends of the specimen permit controlled drainage of pore water from the specimen. The test is called "triaxial" because the three principal stresses are assumed to be known and are controlled. Prior to shear, the three principal stresses are equal to the chamber fluid pressure. During shear, the major principal stress, σ_1 is equal to the applied axial stress (P/A) plus the chamber pressure, σ_3. The applied axial stress, $\sigma_1 - \sigma_3$ is termed as the "principal stress difference" or sometimes the "deviator stress". The intermediate principal stress, σ_2 and the minor principal stress, σ_3 are identical in the test, and are equal to the confining or chamber pressure hereafter referred to as σ_3.

Types of Triaxial Tests

Triaxial tests are conducted in 2 stages, where the first stage is the consolidation stage and the second is the shearing stage. In order to describe the type of triaxial test conducted, a 2 letter designation using capital letters of C, U and D is used, where;
C means; consolidated, U means; undrained, D means; drained. There are three types of triaxial tests:

1. **Unconsolidated-undrained test (UU):** This test is performed with the drain valve closed for all phases of the test. Axial loading is commenced immediately after the chamber pressure σ_3 is stabilized.
2. **Consolidated-undranied test (CU):**This test is performed, while drainage or consolidation is allowed in the second stage to take place during the application of the confining pressure σ_3. Loading does not commence until the sample ceases to drain (or consolidate). The axial load is then applied to the specimen, with no attempt made to control the formation of the excess pore pressures. In this test, the drain valve is closed during axial loading (second stage) and excess pore pressure values should be measured.
3. **Consolidated-dranied test (CD) :** This is the performed test for the Turgutlu sand**.** In this test, the drain valve is opened and is left open for the all duration of the test, with complete sample drainage occuring during application of the vertical load. The load is applied at a very slow strain rate such that particle readjustments in the specimen do not induce any excess pore pressure development, which is not measured. Since there is no excess pore pressure, total stresses will equal to effective stresses. Also the volume change of the sample during shear should be measured.

In this study, consolidated-undranied (CD) type traxial test, as described in the standard:ASTM WK3821:The New Test Method for Consolidated Dranied Triaxial Compression Test for Soils, was used.

This test method covers the determination of strength and stress-strain relationships of a cylindrical specimen of either an undisturbed or remolded non-cohesive soils or sands. Specimens are isotropically consolidated and sheared in compression with drainage at a constant rate of axial deformation (strain controlled).

This test method provides for the calculation of principal stresses and axial compression by measurement of axial load and axial deformation. This test method provides data useful in determining strength and deformation , such as; Mohr strength envelopes. Generally, three specimens are tested at different effective consolidation stresses to define a strength envelope. The determination of strength envelopes lead to obtaining the relationships to aid in interpreting and evaluating test results. Consolidate Darined (CD) triaxial test results of Turutlu Sand is summarized in Table 4.4.

Figure 4.5. General view of the used Triaxial Test Set-up owned by the Ege University

Table 4.4. Summary of Consolidated Drained (CD)Triaxial Test Results on Turgutlu Sand

Test Details		
Standard	BS1377: part 8: 1990, Clauses 4, 5, 6, 7	
Specimen Details		
Specimen Reference	**Effective Minor Principal Stress (σ_3')**	**Effective Major Principal Stress (σ_1')**
A	218.2kPa	985.0kPa
B	230.4kPa	1050.4kPa
C	305.0kPa	1364.2kPa

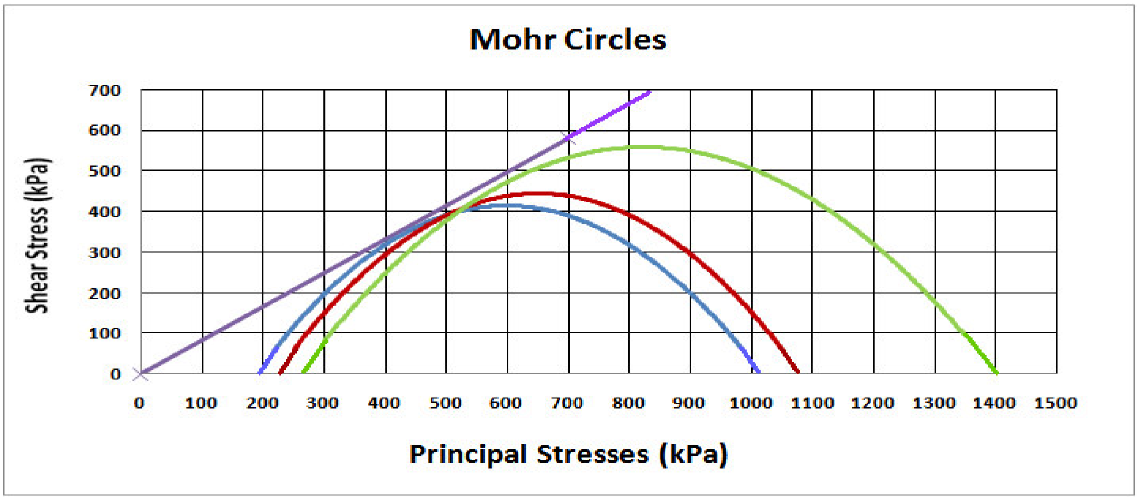


Figure 4.6. Plotted graph of Consolidated Drained Triaxial Test results on Turgutlu Sand

Table 4.5. Summary of Laboratory Tests Results on Uncemented Turgutlu Sand

No	Experiment Name	Used Method	ASTM-D	Value	Unit
1	Particle Size Analysis	Wet Sieve analysis	ASTM-D 422-63	Look at Figure 4.1	(%)
2	The laboratory Compaction Test	Standard Proctor Method	ASTM-D 698-00	$W_{opt} = 13.6$ $\gamma_{drymax} = 19.994$	(%) kN/m^3
3	Specific Gravity of Soil Solids	Pycnometer Method	ASTM-D 854-02	2.65	-
4	Determination of Water Content	Oven Dried Method	ASTM-D 2216	(3.3%) in laboratory conditions	(%)
5	Classification of Soil for Engineering Purposes	USCS	ASTM-D 2487-00	SP	-
6	Coefficient of Permeability	Falling Head Method	ASTM-D 5084-03	10^{-1}	cm/sec
7	Maximum Index Density	Vibration Table Method	ASTM-D 4253-00	Look at Table 4.2	-
8	Triaxial Compression Test	Consolidated-Drained (CD)	ASTM WK3821	c = 7 Ø = 37	kPa (°)
9	Direct Shear Test	Consolidated-Drained (CD)	ASTM-D 3080-04	c = 7 Ø = 36.88	kPa (°)

4.3. Laboratory Tests on cemented Turgutlu Sand (PART 2)

4.3.1. Introduction

In our tests; type 1 Portland cement and Turgulu sand was used, after designing for the right mix proportions at 2 water-cement ratios of 0.4 and 0.5, 3 different sized samples obtained from the same mix were tested after 7 and 28 days for their compressive strengths. But before presenting the results, some basic concepts related to the process will be given below.

Concrete is a composite material that consists of a cement paste within which various sizes of fine and coarse aggregates are embedded. It contains some amount of entrapped air and may contain purposely-entrained air by the use of air-entraining admixtures. Various types of chemical admixtures and/or finely divided mineral admixtures may be used in the production of concrete to improve or alter its properties or to obtain a more economical concrete. Since the cement paste is a plastic material when the cement and water are first mixed, the mixture of the concrete-making materials is also plastic, when first mixed. Since the cement paste gains rigidity and hardness with time (due to the chemical reactions taking place between the cement and the water), the plastic concrete mixture also gains rigidity and hardness in time. Therefore, by placing the plastic concrete mixture into a mold having the desired shape and dimensions, a rock-like material having the desired shape and dimensions is obtained, when the concrete hardens.

The plastic state of the concrete starting from the time that the concrete making materials mixed, until concrete gains rigidity is called ‘‘fresh concrete’’, while the state of the concrete that has already gained its rigidity is called ‘‘hardened concrete’’.

4.3.2. Compressive Strength of Concrete

The ‘‘compressive strength’’ of a concrete is the measured maximum resistance of the concrete to axial compressive loading. Compressive strength is expressed as force per unit of cross-sectional area.

The compressive strength of concrete is one of its most important and useful properties. The reasons for this can be listed as follows:

- As one of the properties of the hardened concrete, compressive strength is the most easily determined one.
- In structural applications, concrete is employed primarily to resist compressive stresses.
- Calculations for the design of structures are usually based on the compressive strength of the concrete.
- By means of correlations with other more complicated tests, the results of compressive strength tests can be used as a qualitative indication of other important properties of hardened concrete, such as; the shear strength, the tensile strength, abrasion resistance, and permeability.

The compressive strength of concrete is usually determined by a ‘‘standard test method’’. Other than the standard method, there are some methods where concrete core specimens are cut from the hardened concrete and tested by using a non-destructive test method, where the compressive strength is determined by measuring the surface hardness of the concrete or by measuring the velocity of an ultrasonic wave travelling the concrete (Erdoğan, T.Y, 2003).

The most widely used test methods for determining the compressive strength of concrete are:

1. Determination of the compressive strength by conducting compressive strength tests on ‘‘standard test specimens’’ (the standard test method),
2. Drilling cores from the hardened concrete and determining the compressive strength by testing the core specimens in compression and,
3. Determination of the compressive strength by measuring the rebound hardness of the concrete’s surface.

4.3.2.1. The ‘‘Standard Test Method’’ for Determining the Compressive Strength of Concrete

The main standard for this purpose is ASTM C 39 which is what was used. However , our sample sizes were different than the original standard used samples, though the same height to diameter ratio of 2 were kept. In the original ASTM standard, the compressive strength of concrete is found by conducing compressive strength tests on 7 or 28 day cured ‘standard’ concrete cylinder specimens of 15 cm in diameter and

30 cm in length. Sometimes in Turkey, 20 cm concrete cube specimens are also used (Erdoğan, 2003) .

4.3.2.1.1 Preparation of the Standard Cylinder Test Specimens

The freshly mixed concrete is placed in the mold in three layers, which are approximately equal. Each layer is compacted by 25 strokes of a 16 mm diameter steel rod and the top surface is finished by troweling. After storing the concrete containing mold at between 16 and 27 °C for the first 24 hours, the specimen is removed from the mold and stored in a moist room or in saturated lime water at 23 ± 1.7 °C, until the testing day comes.

The top and the bottom ends of the cylinder specimen should be perfectly smooth. Therefore, the end surfaces are ‘‘capped’’ with a thin layer (~ 5 mm thick) of a capping material. Usually a mortar or a stiff portland cement paste or sulfur are used as capping material. (No capping is necessary when cube specimens are used).

4.3.2.1.2. Testing the Specimens and Determination of the Compressive Strength

The concrete specimen is tested in a suitable compressive strength testing machine equipped with two steel bearing blocks, one of which is a spherically seated block that will bear on the upper surface of the specimen and the other, a solid block on which the specimen will rest. When the machine is on, the movable top block moves vertically downward and thus an axial load is applied to the specimen. The constant rate (1.4 – 3.5 kgf/cm^2) until the specimen breaks.

The compressive strength of the specimen is calculated as follows:

$$\sigma_c = P_{max} / A \tag{4.3}$$

where:

σ_c = compressive strength

P_{max} = magnitude of the load that causes breaking

A = cross-sectional area of the specimen

The compressive strength of concrete is usually determined at an age of 28 days of the specimen. The 28-day compressive strength is the strength value used in concrete designs. Sometimes, the compressive strength at 7 days is also determined. The 7-day compressive strength is the strength is approximately 65-70% of its 28-day strength ($\sigma_7 = 0.65\text{-}0.70\ \sigma_{28}$).

At least three specimens should be tested; the average of their compressive strength is found for determining the compressive strength of a concrete sample on a particular testing day.The compressive strength values obtained for cylinder specimens and cube specimens prepared from the same concrete sample are not the same: $\sigma_{cylinder} = 0.85\ \sigma_{cube}$.

4.3.3. Procedure for Selection of the right Mix Proportions

The relative quantities of the materials to be used in a particular concrete are determined in the following sequence:

1. Choice of slump,
2. Choice of maximum size of aggregate,
3. Estimation of mixing water and air content,
4. Selection of water/cement ratio (or ''water/cementitious materials'' ratio),
5. Calculation of cement content,
6. Estimation of coarse aggregate content,
7. Estimation of fine aggregate content,
8. Adjustment for aggregate moisture, and
9. Trial batch.

Step 1. Choice of Slump: The slump value that the concrete particular quality should have may be dictated by the job specifications. If slump is not specified, an appropriate slump value can be selected from Table 4.6.

Table 4.6. Recommended Slump values for Various Types of Construction

Type of Construction	Maximum Slump (mm)	Minimum Slump (mm)
Reinforced foundation walls	75	25
Reinforced footing	75	25
Plain footing	75	25
Substructure walls	75	25
Pavement and slabs	75	25
Mass concrete	50	25
Building columns	100	25
Beams	100	25
Reinforced walls	100	25

Step 2. Choice of Maximum Size of Aggregate: The maximum size of aggregate in an aggregate sample is determined by the sieve analysis; it is equal to the size of the minimum sieve through which all the particles can pass.

Concrete with well-graded larger-sized aggregated requires less mortar per unit volume of concrete. Generally, the maximum size of aggregate should be the largest that is economically available and consistent with dimensions of the structure.

The maximum size of aggregate to be used in concrete should not exceed one-fifth of the narrowest dimension between the sides of the forms, one-third the depth of the slabs, and three-fourths of the minimum clear spacing between the individual reinforcing bars.

Step 3. Estimation of Mixing Water and Air Content Amounts: The quality of water per unit volume of concrete required to produce a given slump is dependent on the maximum size of aggregate, particle shape, gradation, concrete temperature, amount of entrained air, and use of chemical admixtures. The quantity of water required and the air content per cubic meter of non-air entrained concrete can be estimated by using the values given in Table 4.7.

Table 4.7. Approximate Amounts of Mixing Water and Air Content Requirements for Non-Air Entrained Concrete

Slump, mm	**Water Content kg/m^3**						
	(Maximum Aggregate Size, mm)						
	9.5	12.5	19	25	37.5	50	75
25-50	207	199	190	179	166	154	130
75-100	228	216	205	193	181	169	145
150-175	243	228	216	202	190	178	160
Entrapped Air(%)	3	2.5	2	1.5	1	0.5	0.3

The quantity of water required and air content per cubic meter of air-entrained concrete can be estimated by using the values given in Table 4.8.

Table 4.8. Approximate Amounts of Mixing Water and Air Content Requirements for Air- Entrained Concrete

Slump, mm	**Water Content kg/m^3**						
	(Maximum Aggregate Size, mm)						
	9.5	12.5	19	25	37.5	50	75
25-50	181	175	168	160	150	142	122
75-100	202	193	184	175	165	157	133
150-175	216	205	197	184	174	166	154
Recommended total air content for level of exposure (%)							
Mix exposure	4.5	4	3.5	3	2.5	2	1.5
Moderate exposure	6	5.5	5	4.5	4.5	4	3.5
Severe exposure	7.5	7	6	6	5.5	5	4.5

As can be seen from Table 4.8., different amounts of water and air are given for the different exposure conditions. These exposure conditions are defined as follows:

Mild exposure: This exposure includes indoor or outdoor services, where concrete will not be subjected to freezing or to de-icing agents. Air entrainment is desired for beneficial effects other than durability, such as; to improve workability or in low cement factor concrete to improve strength.

Moderate exposure: Service in a climate, where some freezing is expected, but where the concrete will not be continually exposed to moisture for long periods before freezing. Slaps that are not in direct contact with wet soil, slabs that will not receive direct application of de-icing salts, exterior beams, columns and walls can be given.

Severe exposure: Concrete that is exposed that is exposed to high saturation, prior to freezing or de-icing agents.

Step 4. Selection of Water/Cement Ratio: The water/cement ratio affects the strength, the porosity, and the durability of the hardened concrete.

When the relationship between the strength and the water/cement ratio has not been found by conducting tests with the available materials, the values given in Table 4.9. can be used for selecting the proper water/cement ratio that will lead to the required minimum strength of the concrete.

Table 4.9. Relationship between the ''Water/Cement'' Ratio and Compressive Strength of Concrete

Compressive Strength at 28 days, MPa	''Water/Cement'' Ratio, by weight	
	Non-air-entrained concrete	Air-entrained Concrete
40	0.42	
35	0.47	0.39
30	0.54	0.45
25	0.61	0.52
20	0.69	0.6
15	0.79	0.7

For severe conditions of exposure, the water/cement ratio can be selected from the values given in Table 4.10.

Table 4.10. Maximum Permissible ‘‘Water/Cement’’ Ratios for Concretes in Severe Exposure

Type of Structure	Structures that will be wet continuously and exposed to freezing and thawing	Structures exposed to seawater or sulfates
Thin sections and sections with less than 25 mm cover over steel	0.45	0.4
All other structures	0.5	0.45

Step 5. Calculation of Cement Content: Since the amount of water per cubic meter of concrete is already determined from Table 4.8 and the water/cement ratio is already determined from Table 4.9, the amount of cement per cubic meter of concrete is found by calculation as;

$$\textbf{Cement ratio} = \textbf{(water content / (w / c))} \tag{4.4}$$

Step 6. Estimation of Coarse aggregate content: The volume of dry coarse aggregate per cubic meter of concrete can be estimated by using the values given in Table 4.11.

The volume of coarse aggregate found per unit volume of concrete should be converted to dry weight of coarse aggregate, by multiplying it by the oven-dry-rodded unit weight of the coarse aggregate. The dry weight of coarse aggregate can be converted to saturated-surface dry weight by multiplying it with ‘‘1 + % absorption’’.

Table 4.11. Finding the volume of coarse aggregate per unit volume of concrete

Maximum size of aggregate, mm	Volume of dry-rodded coarse aggregate per unit volume of concrete			
	Fineness Moduli of Fine Aggregate			
	2.4	2.6	2.8	3
9,5	0.5	0.48	0.46	0.44
12,5	0.59	0.57	0.55	0.53
19	0.66	0.64	0.62	0.6
25	0.71	0.69	0.67	0.65
37,7	0.75	0.73	0.71	0.69
50	0.78	0.76	0.74	0.72
75	0.82	0.8	0.78	0.76

Step 7. Estimation of Fine aggregate content: The volume of fine aggregate in one cubic meter of concrete is equal to ''1-(volume of cement + volume of water + volume of coarse aggregate + volume of air)''.

$$\text{Volume of cement} = \text{weight of cement / specific gravity of cement} \tag{4.5}$$

$$\text{Volume of water} = \text{weight of water / specific gravity of water} \tag{4.6}$$

The calculated volume of fine aggregate per unit volume of concrete can be converted to dry weight of fine aggregate by multiplying it by the dry bulk specific gravity of fine aggregate. Similarly, the calculated volume of fine aggregate can be converted to saturated surface dry weight of fine aggregate by multiplying it by the saturated surface dry bulk specific gravity of the fine aggregate (Table 4.11).

Step 8. Adjustment for the Aggregate Moisture: The weights of fine and coarse aggregates determined by the above mentioned calculations can be their weights either for a dry state or for a saturated surface dry state. However, the aggregates actually to be used in concrete may be neither in an oven-dry state nor in a saturated surface dry state.

If the aggregate weights are found for their oven-dry state and if the actual aggregates contain some water in them, their use will lead to the inclusion of extra water in the concrete mix. The inclusion of extra water will then increase the slump of the fresh concrete and decrease the strength of the hardened concrete.

If the aggregate weights are found for their saturated surface dry state and if the actual aggregates are in an air-dry or oven-dry state, some of the mixing water will be absorbed by the aggregates. This will lead to a decrease in workability. Therefore, adjustment need to be made in the calculations to find the right proportions of the actual materials to be used in the concrete.

Step 9. Trial Mix: A small amount of concrete should be prepared by using the calculated mixture proportions of the materials and tests should be conducted to see whether the desired slump, the desired air content and the desired strength can be obtained with such a concrete mix. A 0.02 m^3 of concrete is sufficient for such a trial batch. Therefore, the weights of the materials calculated per cubic meter of concrete should be divided by 50, and a trial batch should be prepared with these smaller quantities of materials.

When the trial batch is prepared, its slump and air content should be determined by tests. The slump and the air content values found by these tests should be compared with the slump and air content values used in the calculations.

If the slump of the trial batch does not match the slump value that is used in the mix design, re-estimation of the water content should be made by increasing or decreasing the water content by 4 kg for each 1 cm difference in slump.

If the air content of the trial batch does not match the air percentage assumed in making the mix design, re-estimation of the water content should be made by increasing or decreasing the water content by 2.5 kg for each 1% difference in the air content. Thus, the whole sequence of the calculation could be repeated using the new values of the materials that will be used in making the concrete. The strength of the trial batch concrete can be found by preparing specimens and conducting compressive strength tests. The strength obtained with these specimens should be compared with the desired strength (Erdoğan, 2003).

4.3.4. Laboratory Experiments

The first thing to get an expected quality concrete is to choose materials which will form the concrete, to identify characteristics of the concrete and to calculate material's usage rate.

Major factors are explained above. Now we will explain experiments depending on these factors.

4.3.4.1. Materials of Concrete Mixture

These metarials have been used for our experiments:

- Cement of Portland Type (I)
- Turgutlu coarse sand, and
- Water

Also there is such an air pore inside every type of concrete that this pore occurs when the concrete are mixed with its metarials or when the concrete are placed.

4.3.4.2. Calculation of Materials Proportion to Create Concrete

For calculating the material proportions for the mix-design of the Cemented-Prepared Subgrade Layer(C-PSL) to use in stead of the Uncemented-Prepared Subgrade Layer(U-PSL) seen in the High Speed Train embankment section (Fig.3.3) procedure in the Turkish Standards TS 802 was used. This was a standard adopted from the American Concrete Institute (ACI). Used materials were: Turgutlu Coarse Sand (commercially available in sacks to be used in construction projects), Portland Cement-Type 1 and (IZSU)-Tap water. For briefness and providing clarity to the reader, used mix-design portions (weighs) will be given in Tables in the Appendix E. Here only the procedure used will be mentioned. But before explaining the mix-design procedure, first assumptions made needed to be given, as shown in Table 4.12.

Table 4.12. Summary of accepted values for create concrete

No	Name	Value	Unit
1	**Slump**	70	mm
2	**Max Size of Aggregate**	50	mm
3	**Mixing Water and Air Content**	depended on w/c ratio	-
4	**Water/Cement Ratio (w/c)**	0.4 – 0.5	-
5	**Cement Content**	10,15,20,25,30	(%) by weight of concrete
6	**Coarse Aggregate Content**	depended on w/c ratio	-

4.3.4.3. Experimental Studies

Firstly; Turgutlu coarse sand, which is commercially available in sacks, are left to oven-dry for 1 day in a large pan and from there is sub-sampled for the calculated (see Appendix Tables) exact amount in a smaller metal container (Figures 4.7.a-b).

a) 

b) 

Figure 4.7 (a-b). Weighing of Turgutlu Sand after Owen-Drying

The same procedure is also applied to the Portland Cement-Type 1 (PC-1) (Figures 4.8a-b).

a) 

b) 

Figure 4.8(a-b). Weighing of Portland Cement (Type 1)

After adding the calculated exact amount of water to the sand-cement mixture (given in the Appendices), the mixture is thoroughly mixed by hand (Figures 4.10-4.13)

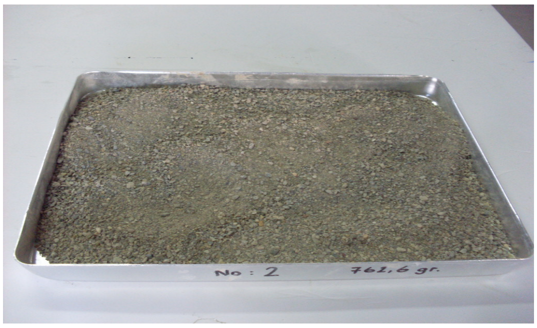


Figure 4.9. Placing Turgutlu Sand on the Pan

Figure 4.10. Adding PC-1 on Turgutlu Sand

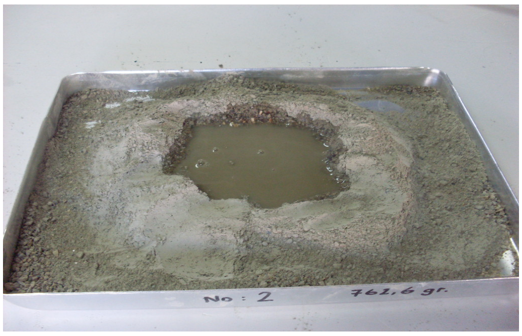


Figure 4.11. Mixing Sand with Shovel

Figure 4.12. Adding water to Turgutlu Sand

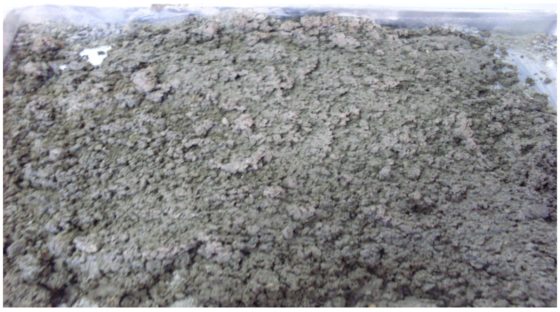


Figure 4.13. Thoroughly Mixing the Sand-Cement Mixture

Proper mixing of the sand-cement-water mixture is important to provide homogeneous mixture at the right consistency and to avoid segregation of the materials used. Then the mixture is carefully poured from a small height into stainless steel molds of 3 different sizes, as shown in Figures 4.14(a-b)-4.16(a-b) and also explained under section 4.3.3.

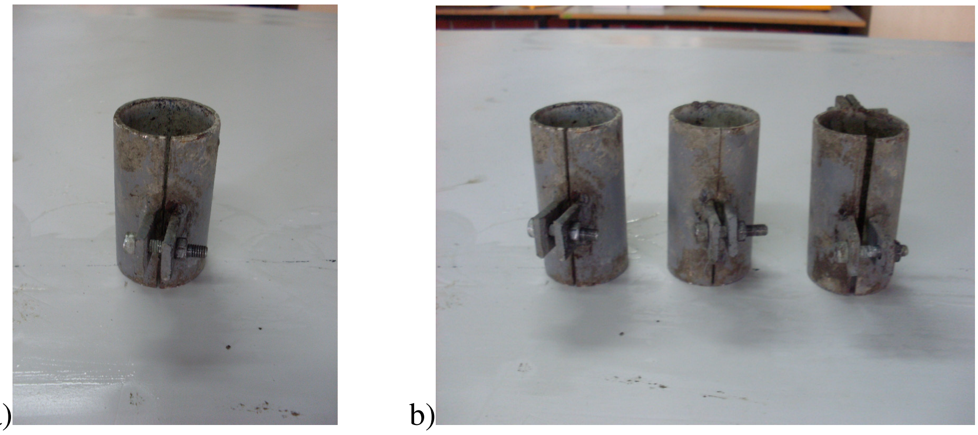
It is noted that; there were 3 numbers of specimens for the (small) sample **A** size (with Diameter, D=4 cm and Length, L=8 cm), as shown in Figure 4.16(a-b). There were also 3 numbers of specimens for the (medium) Sample **B** size (with Diameter, D=8 cm, Length, L=16 cm), as shown in Figure 4.15 (a-b). Furthermore, there were also 3 numbers of specimens for the (Large) Sample **C** sizes (with Diameter, D=10 cm, Length, L=20 cm), as shown in Figure 4.14 (a-b). So, 3 sets of 3 specimens in each set, where the first one was tested, after 7-days of water curing and the other 2 were tested, after 28-days of water curing.

a)  b) 

Figure 4.14(a-b). (Large) Sample **C** Sizes (D=10 cm, L=20 cm)

a)  b) 

Figure 4.15(a-b). (Medium) Sample **B** Sizes (D=8cm, L=16 cm)



a) b)

Figure 4.16(a-b). (Small) Sample **A** Sizes (D=4 cm, L=8 cm)

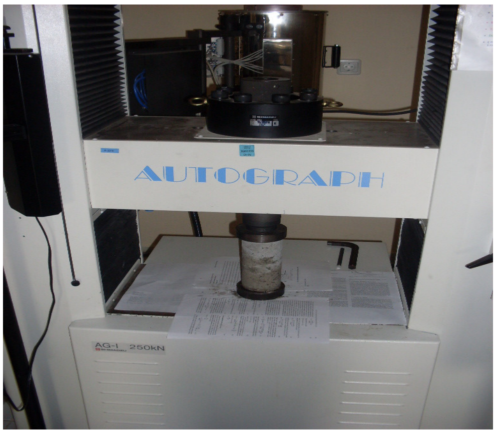
Figure 4.17. Test Specimens

Altogether, 270 specimens were prepared in 3 main groups. In each group, there were 5 different cement contents (10%, 15%, 20%, 25%, 30%) and 2 different water-to-cement ratios (w/c=0.4 ve w/c = 0....5). Of these, there were 30 numbers of 7-day cured then tested in compression of 3 different specimen sizes and there were 60 numbers of 28-day cured then tested in compression of 3 different specimen sizes. For the Unconfined Compression Tests, Universal Machine at the IYTE-MAM was used. Area correction was considered and the largest compressive force applied during compression was taken as the break force and used in the analyses (Figures : 4.18(a-b), 4.19(a-b) and 4.20).

a) 

b) 

Figure 4.18(a-b). Universal Test Machine used (at IYTE-MAM) for the Unconfined Compression Tests conducted on the prepared C-PSL Test Specimens-Shearing Failure

a) 

b) 

Figure 4.19(a-b). Universal Test Machine used (at IYTE-MAM) for the Unconfined Compression Tests conducted on the prepared C-PSL Test Specimens-Splitting Failure

Figure 4.20. Universal Test Machine used (at IYTE-MAM) for the Unconfined Compression Tests conducted on the prepared C-PSL Test Specimens-Shearing Failure, Close-up View

4.4. Evaluation of Results of Laboratory Tests Conducted on Turgutlu Sand

Evaluation of the results of the Unconfined Compression tests conducted on the Cemented-Prepared Subgrade Layer (C-PSL) specimens will be studied from 2 different viewpoints, as sub-titled below:

4.4.1. Evaluation of Results of Elasticity Modulus

a) Laboratory tests on 7 day cured specimens results show that;

a1) For (small) A–size specimens (Ø4 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5, elsticity modulus (E) increases faster with increasing cement content (Figure 4.21, See Appendix F).

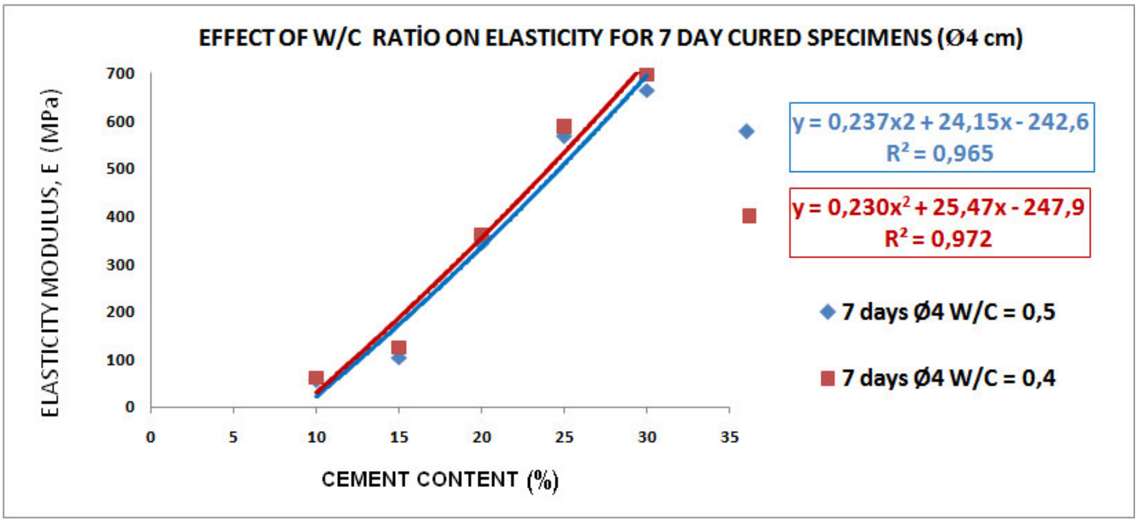


Figure 4.21. Effect of w/c ratio on elasticity for 7-day cured specimens (Ø4 cm)

a2) For (medium) B-size specimens (Ø8 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5 , elasticity modulus (E) increases at the same rate with increasing cement content (c) up to about 20%, after which E increase slowly for w/c = 0.4 , while E continues to increase fastly for w/c=0.5 (Figure 4.22).

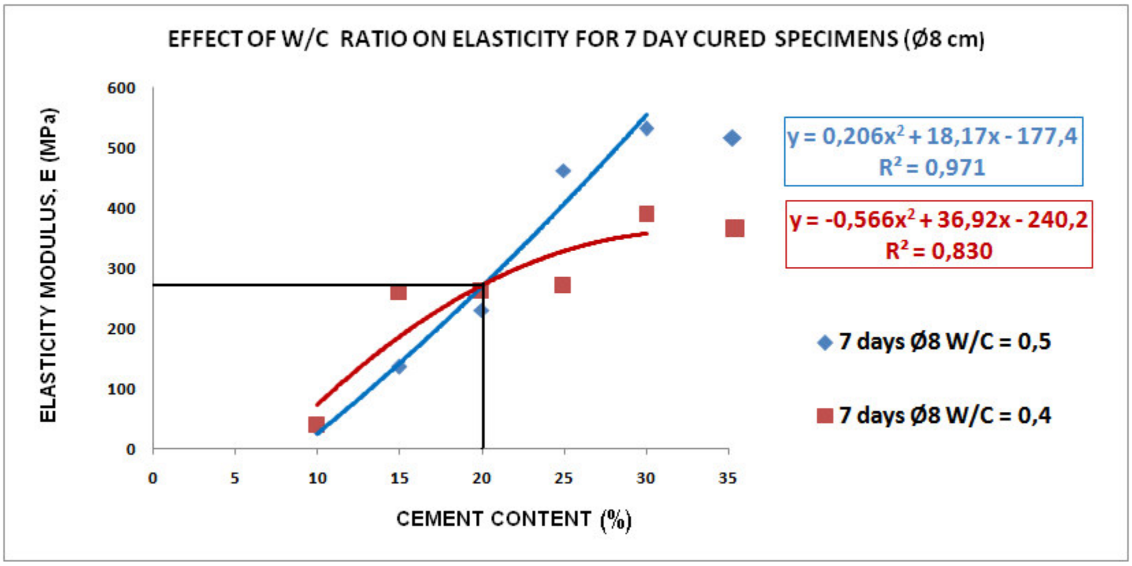


Figure 4.22. Effect of w/c ratio on elasticity for 7-day cured specimens (Ø8 cm)

a3) For (large) C-size specimens (Ø10 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5 , elasticity modulus (E) increases at the same rate with increasing cement content (c) up to about 20%, after which E for the higher w/c increases faster than the lower w/c (Figure 4.23).

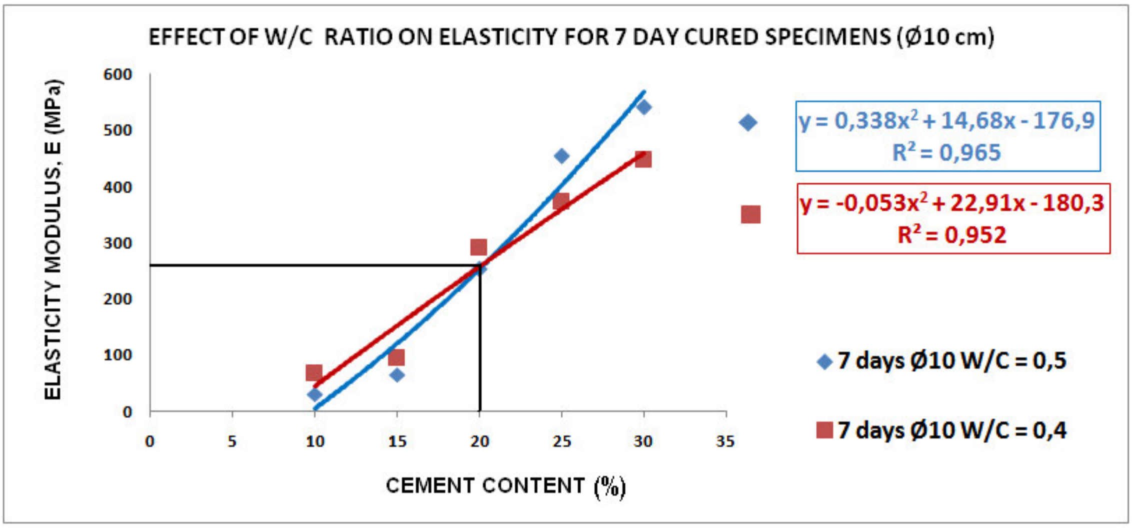


Figure 4.23. Effect of w/c ratio on elasticity for 7-day cured specimens (Ø10 cm)

a4) With increasing cement content, elasticity modulus (E) increases almost at the same rate for any sample size (Figure 4.24).

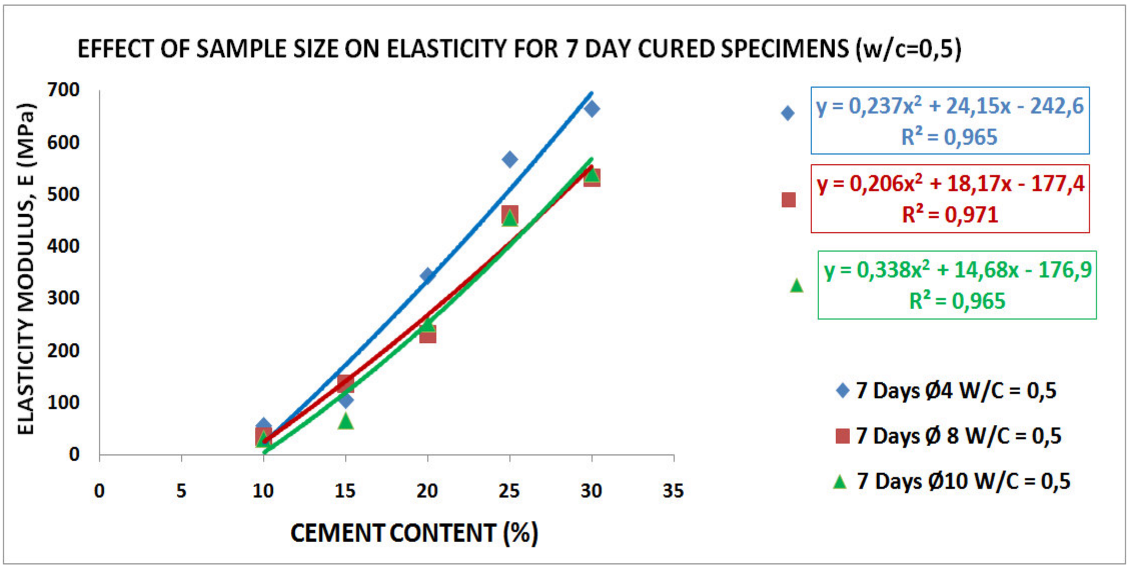


Figure 4.24. Effect of sample size on elasticity for 7-day cured specimens (w/c=0.5)

a5) For (small and large) A and C-sizes specimens, elasticity modulus (E) increases at the same rate with increasing cement content (c). But if the sample size decreases to (medium) B-size specimens size, elasticity modulus (E) increases at a slower rate with increasing cement content compared to sizes A and C (Figure 4.25).

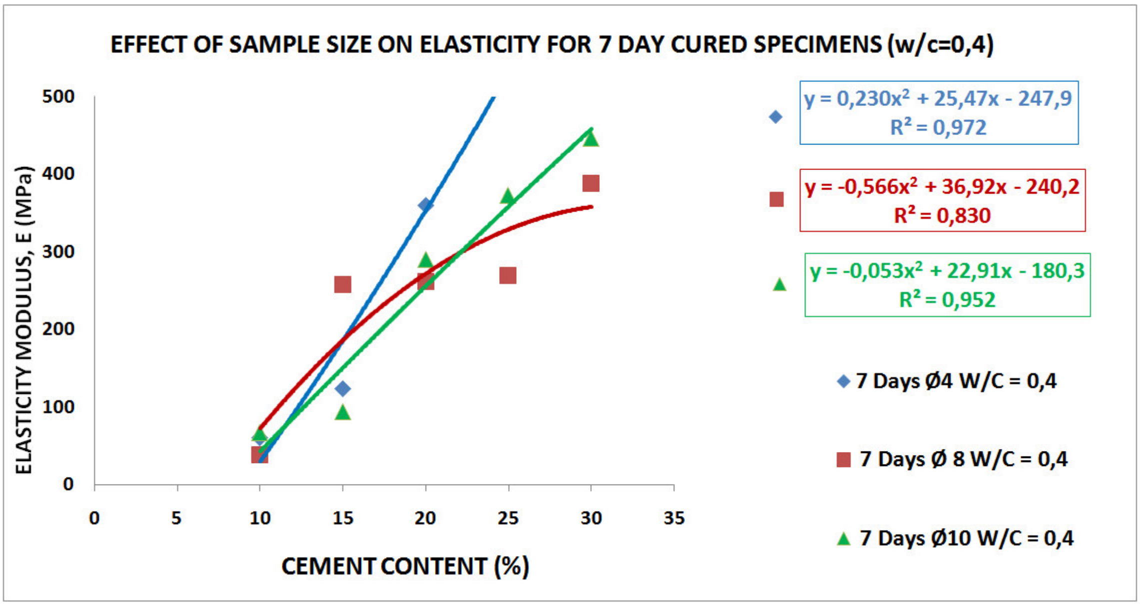


Figure 4.25. Effect of sample size on elasticity for 7-day cured specimens (w/c=0.4)

b) Laboratory tests on 28 day cured specimens results show that;

b1) For (small) A–size specimens (Ø4 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5, elsticity modulus (E) increases almost at the same rate up to about 30% (Figure 4.26).

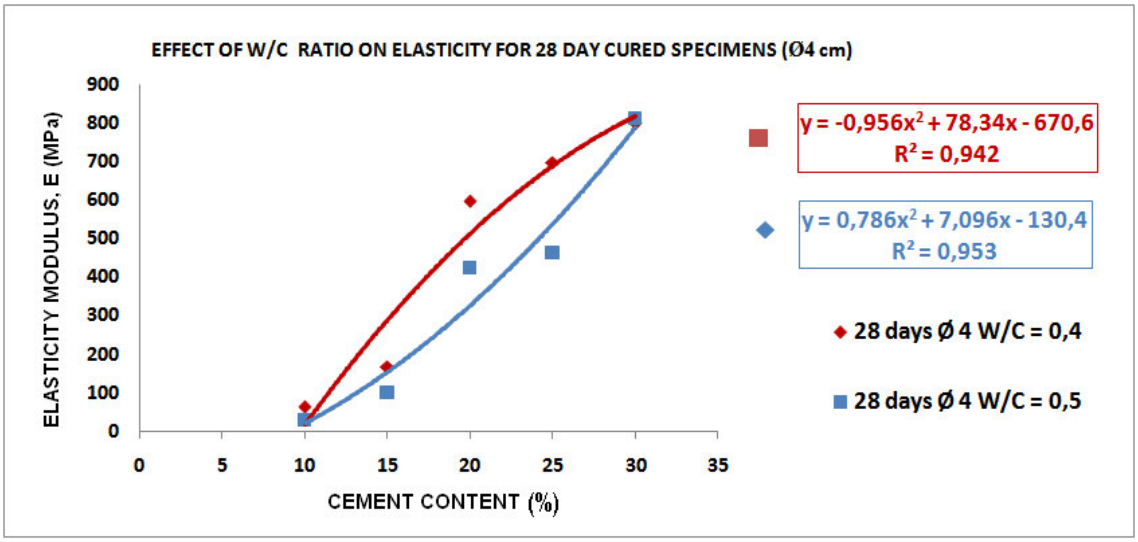


Figure 4.26. Effect of w/c ratio on elasticity for 28-day cured specimens (Ø4 cm)

b2) For (medium) B-size specimens (Ø8 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5 , elasticity modulus (E) for w/c=0. 5 increases faster (compared to E for w/c=0.4) increasing cement content (c) up to about 30% (Figure 4.27).

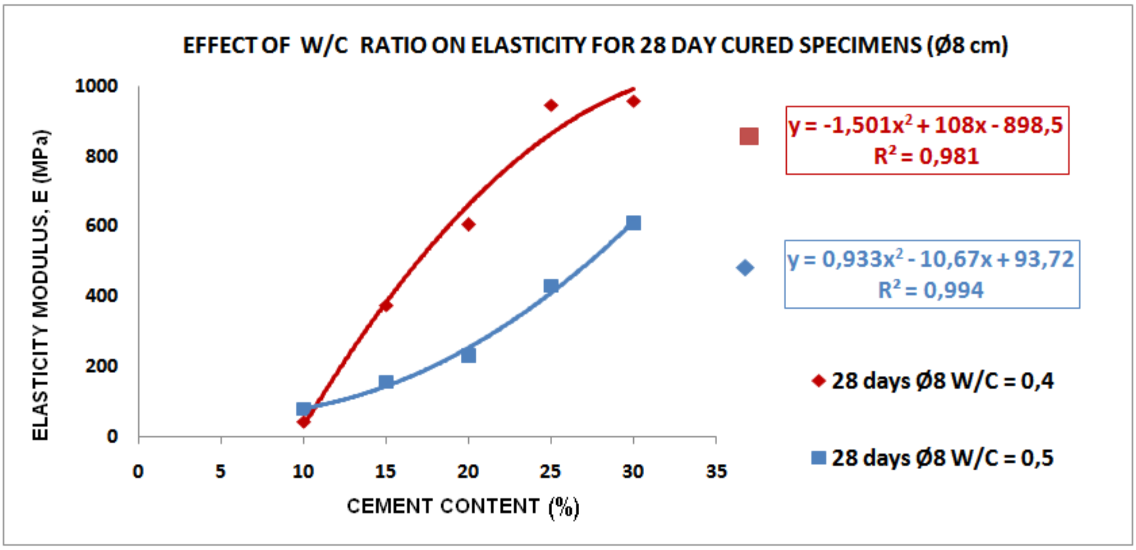


Figure 4.27. Effect of w/c ratio on elasticity for 28-day cured specimens (Ø8 cm)

b3) For (large) C-size specimens (Ø10 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5 , elasticity modulus (E) increases at the same rate with increasing cement content (C) up to about 20%, after which E decreases faster for w/c = 0.4, while E continues to slowly increase for w/c=0.5 (Figure 4.28).

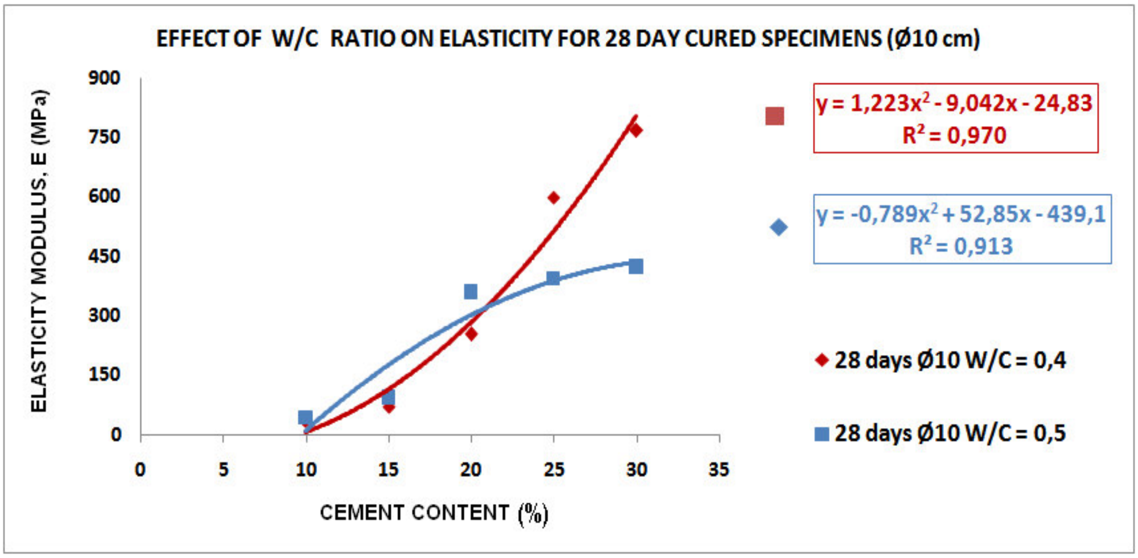


Figure 4.28. Effect of w/c ratio on elasticity for 28-day cured specimens (Ø10 cm)

b1) For (small and medium) A and B-sizes specimens, elasticity modulus (E) increases almost at the same rate with increasing cement content (c). But if the sample size increases to (large) C-size specimens, elasticity modulus (E) decreases faster with increasing cement content (Figure 4.29).

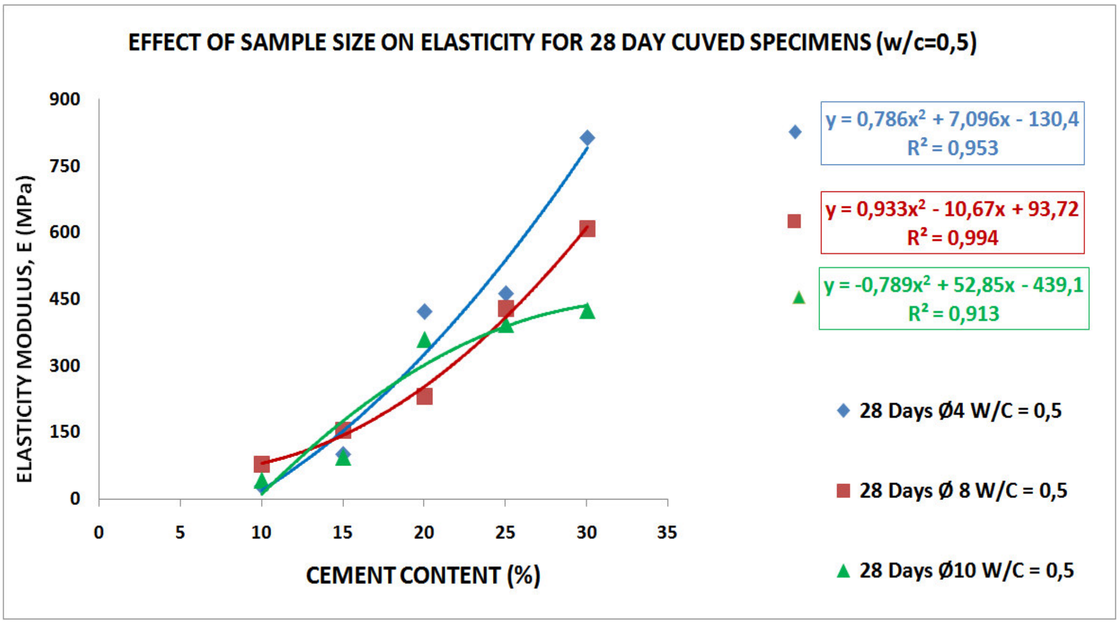


Figure 4.29. Effect of sample size on Elasticity for 28-day cured specimens (w/c=0.5)

b1) Thus with increasing cement content, elasticity modulus (E) increases almost at the same rate with increasing cement content (c) irrespective of sample size up to about 30%, (small and medium) A and B-sizes specimens elasticity modulus (E) decreases at the same rate with increasing cement content (c). But (large) C-size specimens elasticity modulus (E) increases faster with increasing cement content (Figure 4.30).

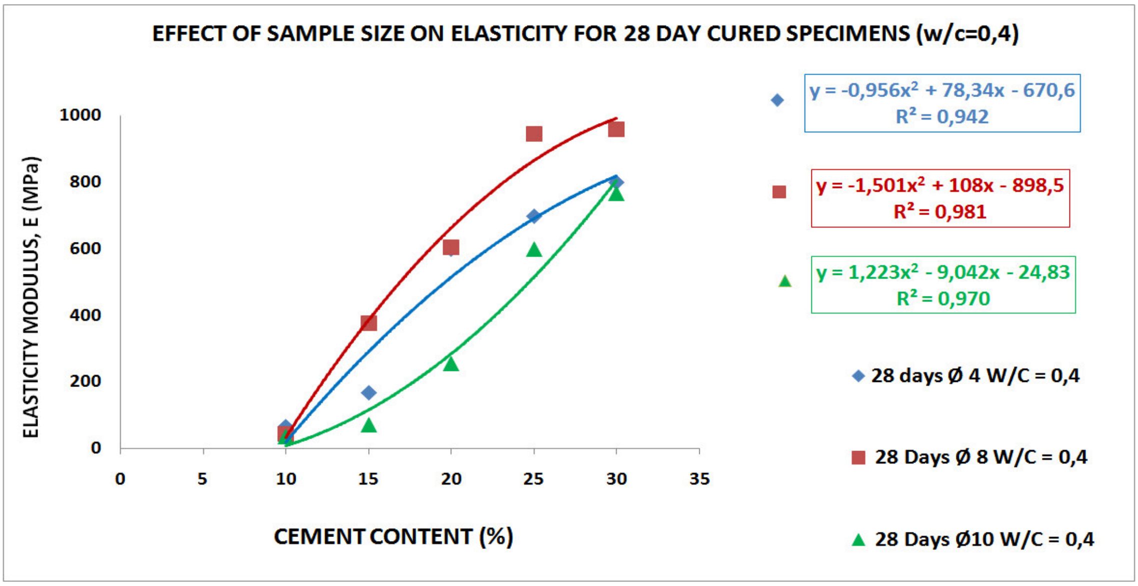


Figure 4.30. Effect of sample size on Elasticity for 28-day cured specimens (w/c=0.4)

4.4.2. Evaluation of Results of Stresses

a) Laboratory tests on 7 day cured specimens results show that;

a1) For (small) A–size specimens (Ø4 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5, stress increases faster for w/c=0.4 with increasing cement content (c) (compared to stress for w/c=0.5) up to about 30% (Figure 4.31, See Appendix G).

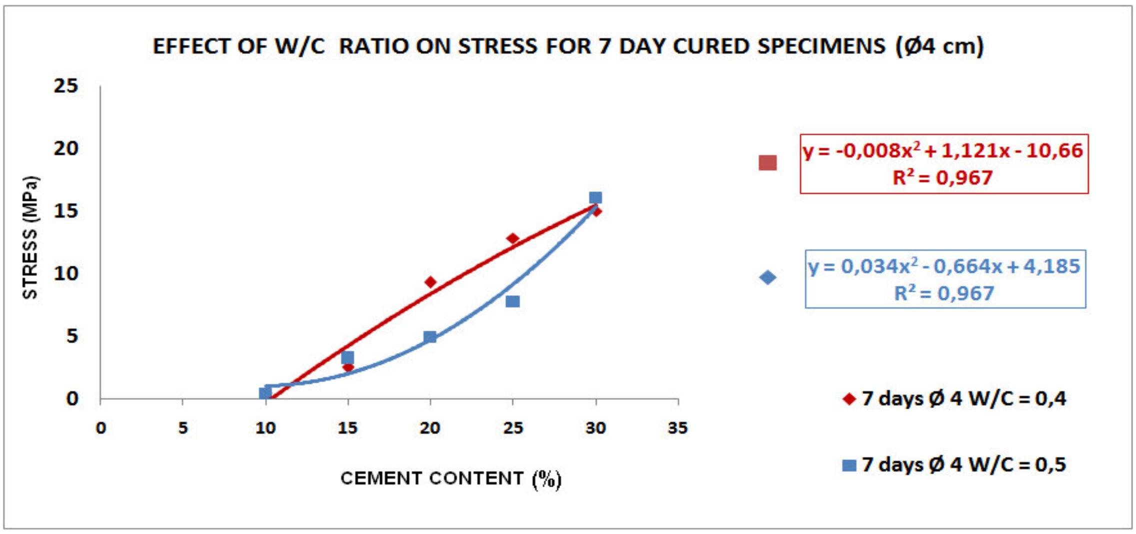


Figure 4.31. Effect of w/c ratio on stress for 7-day cured specimens (Ø4 cm)

a2) For (medium) B-size specimens (Ø8 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5 , stress increases at the same rate with increasing cement content (c) up to about30%, after which stress increases faster for w/c = 0.5 , while stress continues to slowly increase for w/c=0.4 (Figure 4.32).

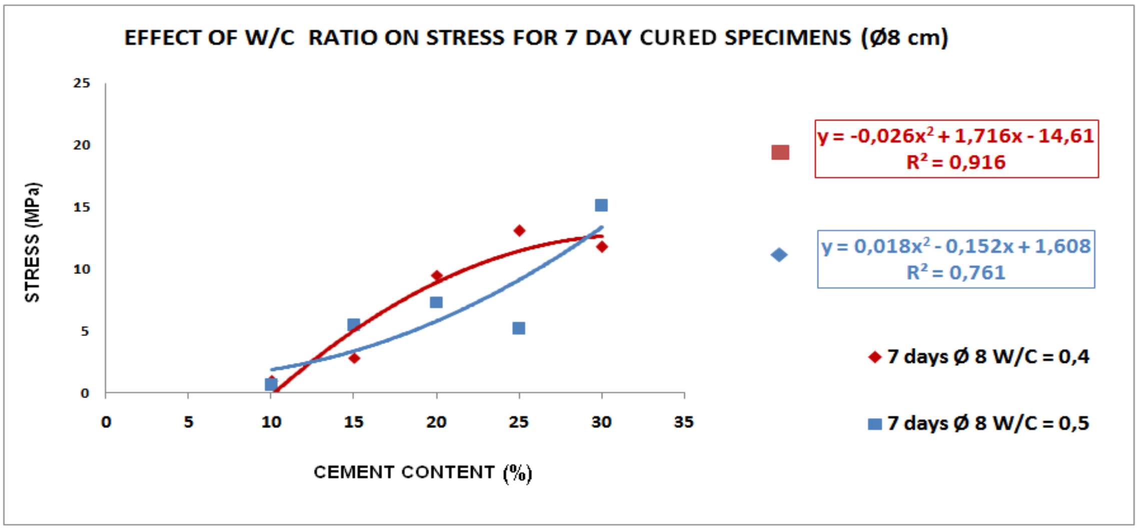


Figure 4.32. Effect of w/c ratio on stress for 7-day cured specimens (Ø8 cm)

a3) For (large) C–size specimens (Ø10 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5, stress increases almost at the same rate up to about 30%, after which stress increases faster for w/c = 0.5 , while stress continues to slowly increase for w/c=0.4 (Figure 4.33).

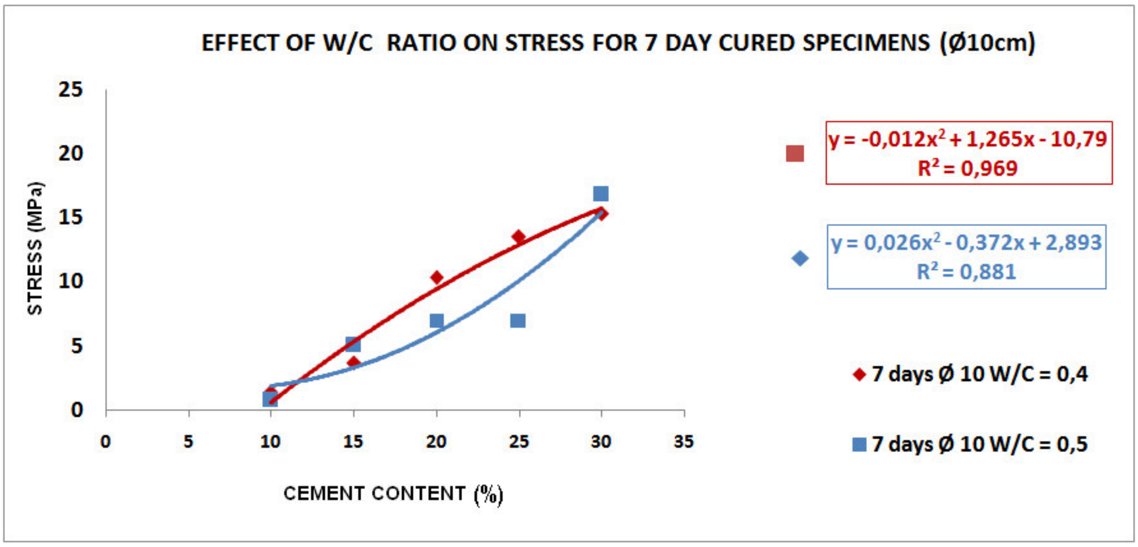


Figure 4.33. Effect of w/c ratio on stress for 7-day cured specimens (Ø10 cm)

a1) With increasing cement content, stress increases almost at the same rate for any sample size (Figure 4.34).

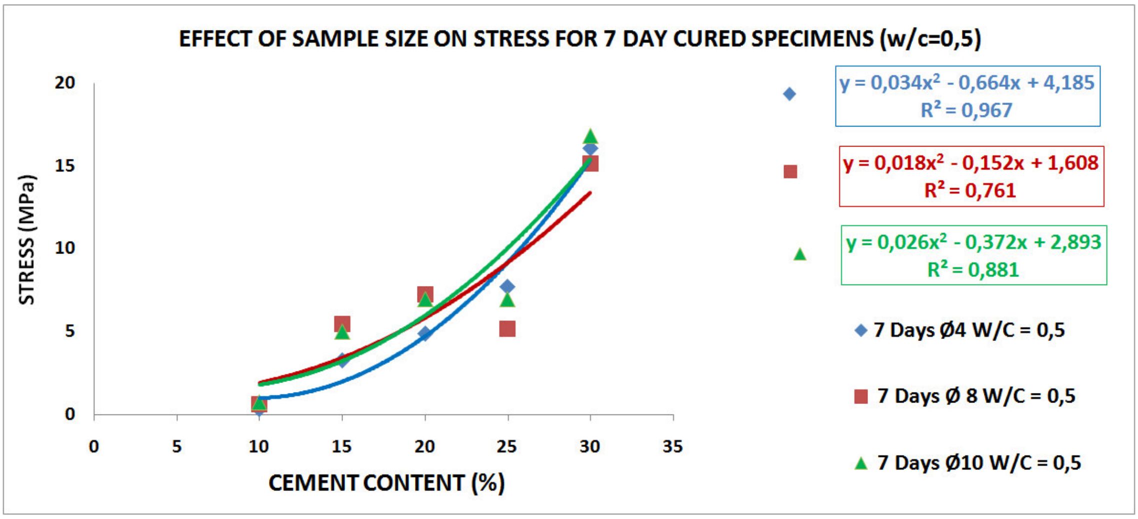


Figure 4.34. Effect of sample size on stress for 7-day cured specimens (w/c=0,5)

a1) Thus with increasing cement content, stress increases almost at the same rate with increasing cement content (c) irrespective of sample size up to about 20%, for (small and large) A and C-sizes specimens, stress increases at the same rate with increasing cement content (c). But (medium) B-size specimens stress increases slowly with increasing cement content (Figure 4.35).

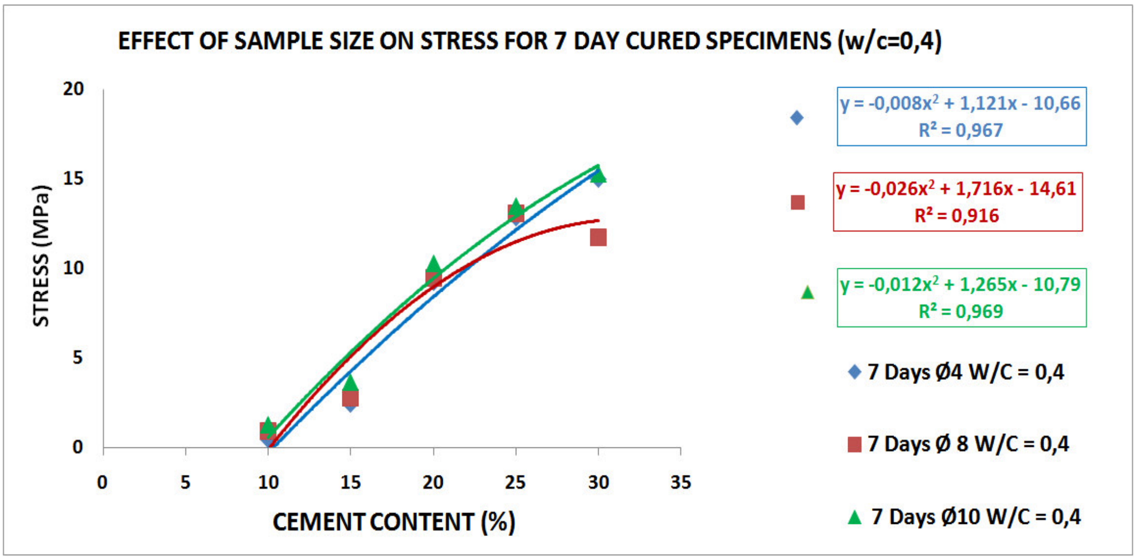


Figure 4.35. Effect of sample size on stress for 7-day cured specimens (w/c=0,4)

b) Laboratory tests on 28 day curved specimens results show that;

b1) For (small) A–size specimens (Ø4 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5, stress increases almost at the same rate (Figure 4.36).

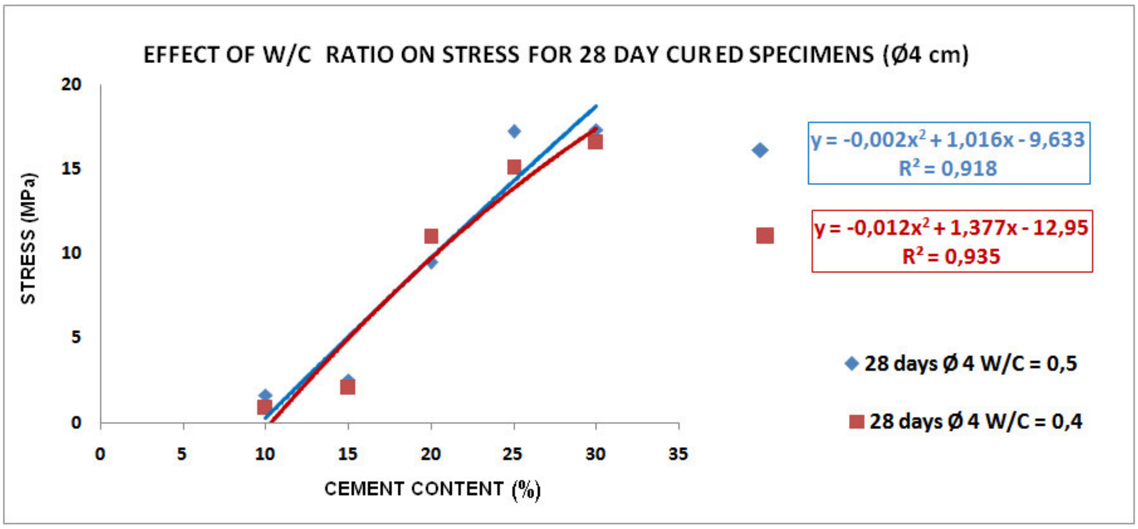


Figure 4.36. Effect of w/c ratio on stress for 28-day cured specimens (Ø4 cm)

b2) For (medium) B-size specimens (Ø8 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5 , stress increases at the same rate with increasing cement content (c) up to about 30%, after which stress increases slowly for w/c = 0.4, while stress continues to increases fastly for w/c=0.5 (Figure 4.37).

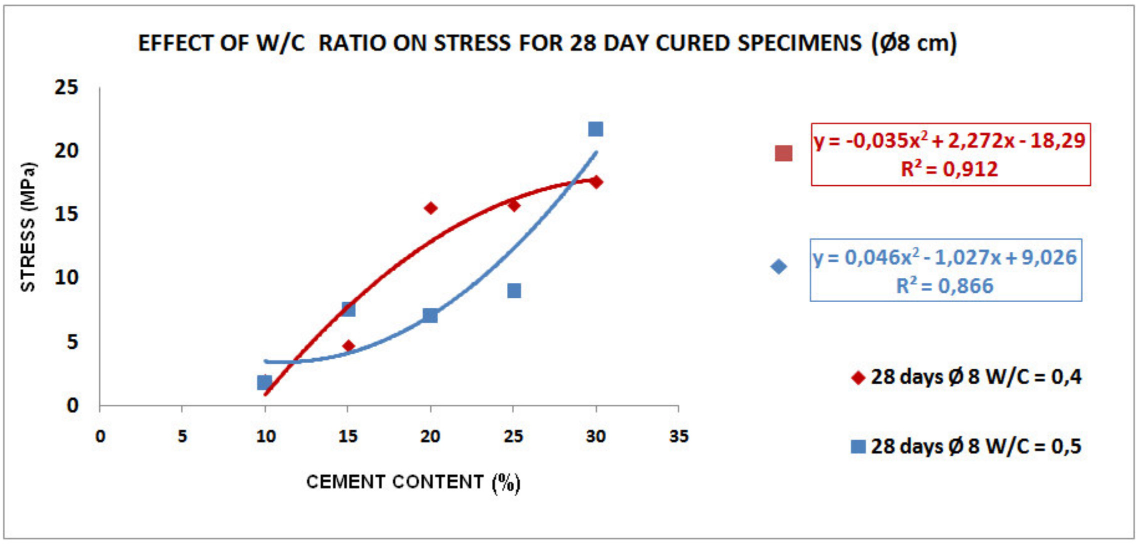


Figure 4.37. Effect of w/c ratio on stress for 28-day cured specimens (Ø8 cm)

b3) For (large) C-size specimens (Ø10 cm), if the water to cement ratio (w/c) increases from 0.4 to 0.5, stress increases at the same rate with increasing cement content (c) up to about 25%, after which stress increases slowly for w/c = 0.5 , while stress continues to increases fastly for w/c=0.4 (Figure 4.38).

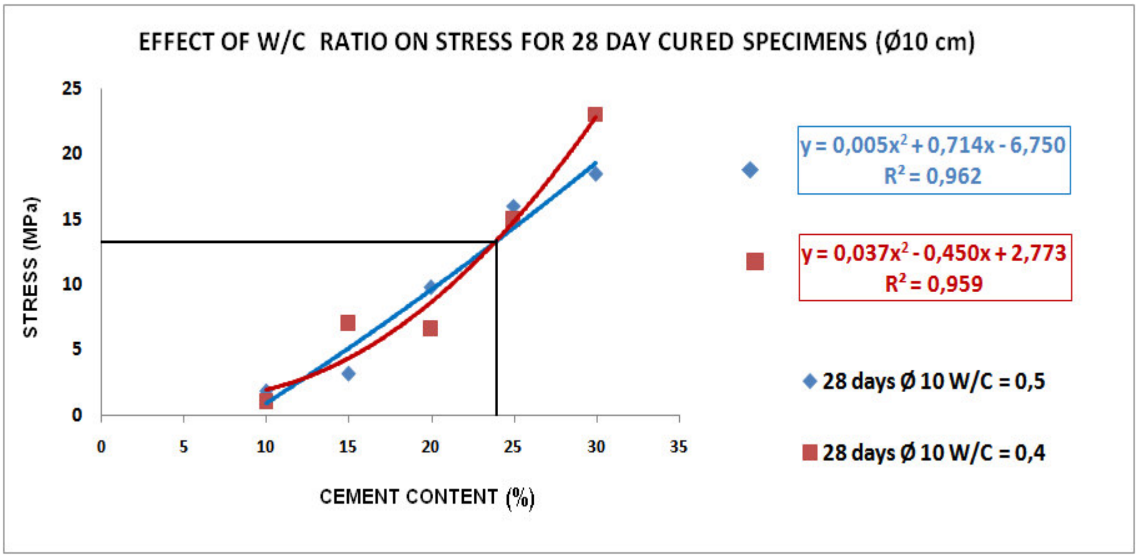


Figure 4.38. Effect of w/c ratio on stress for 28-day cured specimens (Ø10 cm)

b4) With increasing cement content, stress increases almost at the same rate with increasing cement content (c) for any sample size up to about 30% (Figure 4.39).

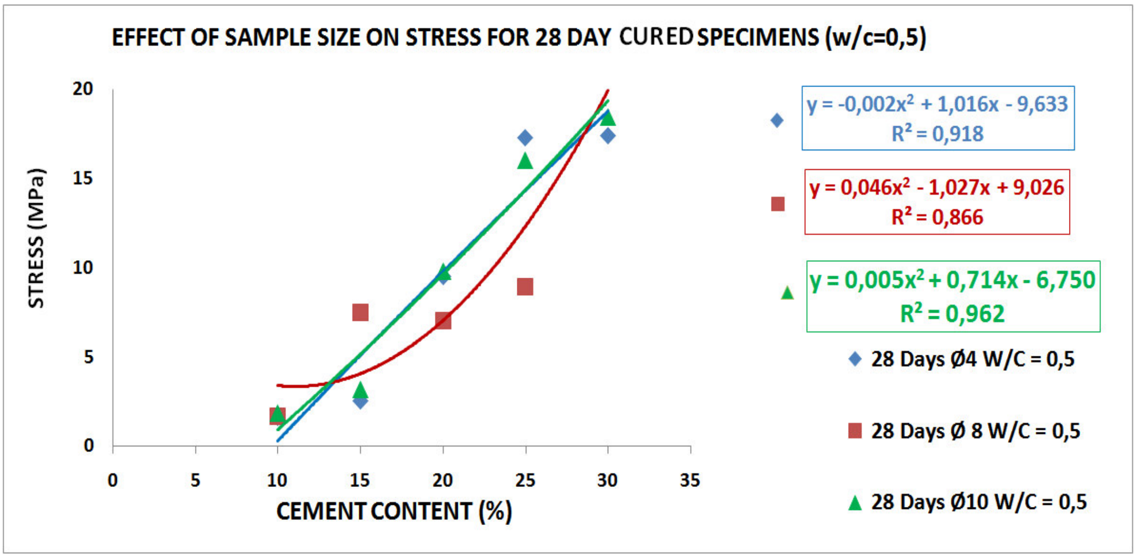


Figure 4.39. Effect of sample size on stress for 28-day cured specimens (w/c=0,5)

b5) For (large and medium) C and B-sizes specimens, stress increases at the same rate with increasing cement content (c) up to about 30%. But if the sample size decreases to small (A) size, stress increases slowly with increasing cement content (Figure 4.40).

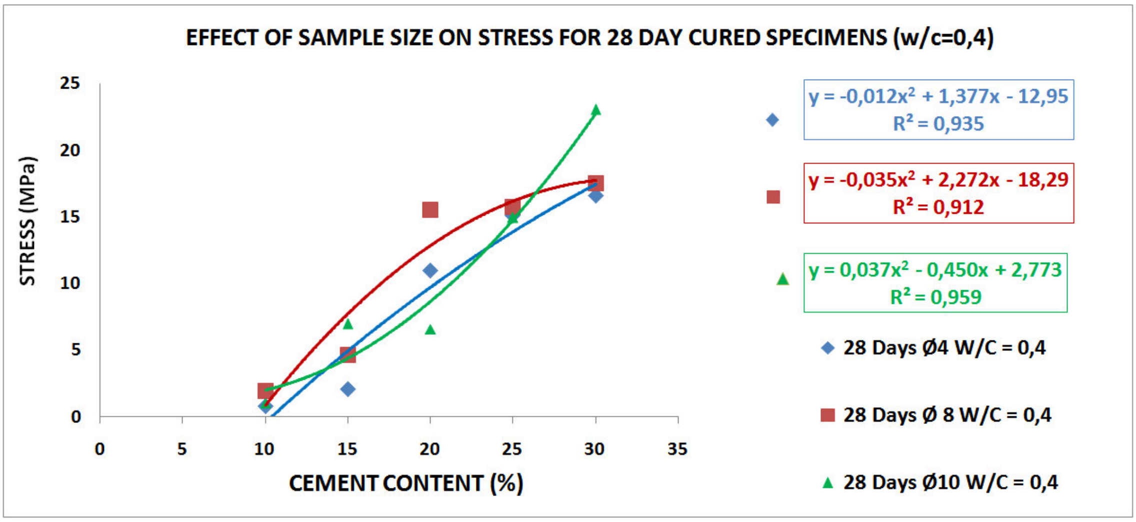


Figure 4.40. Effect of sample size on stress for 28-day cured specimens (w/c=0.4)

CHAPTER 5

ANALYSIS OF HIGH-SPEED RAIL SETTLEMENT

5.1. Introduction

Limiting the values of top of rail (total and differential) settlements to tolerable values for the short term (during construction) and for the long term (generally, 30 years after construction) are crucially important safety issues for the high-speed railway trains traveling at speeds between:200-400 km/hr. In some Far-Eastern countries (Taiwan, Japan etc.), design speed on average is 400 km/hr, minimum radius of curvature is 90 km and the maximum tolerable total and differential (long term) settlement at the top of rail in any transverse section of the embankment is 2 mm, while the maximum tolerable total and differential settlement at the top of rail for any (20 m long) longitudinal section of the embankment is 10 mm (THSRC-Design Manual, Volume 9, 2003). Since one of the objectives of this study is to prove that Cemented–Prepared Subgrade Layer (C-PSL) can replace the uncemented Prepared Subgrade Layer (U-PSL) currently in use, as shown in Figure 3.3, top of rail settlements should be calculated precisely for both cases and compared with the tolerable values. For this purpose Plaxis V8 (2D) software program using the finite element method (FEM) was used.

- **Background Information about the Plaxis Program**: Development of PLAXIS began in 1987 at the Technical University of Delft, as an initiative of the Dutch Department of Public Works and Water Management. The initial goal was to develop an easy-to-use 2D finite element code for the analysis of river embankments on the soft soils of the lowlands of Holland. In subsequent years, PLAXIS was extended to cover most other areas of geotechnical engineering. Because of continuously growing activities, a company named PLAXIS b.v. was formed in 1993. In 1998, the first PLAXIS version for Windows was released. In the mean time a calculation kernel for 3D calculations was being developed. After several years of development the PLAXIS 3D Tunnel program was released in 2001. PLAXIS is intended to provide a tool for practical analysis to be used by the geotechnical engineers, who are not necessarily numerical specialists. Quite often practising engineers consider non-linear finite element computations cumbersome and time-consuming. The PLAXIS research and development team has addressed this issue by designing robust and theoretically sound computational procedures, which are encapsulated in a logical and easy-to-use shell. As a result, many geotechnical engineers world-wide have adopted the product and are using it for engineering purposes.

- **CUR consortium:** Research and development is supported by the Center for Civil Engineering Research and Codes (CUR). A consortium of more than 30 European companies contribute financially to these developments and a CUR committee checks the efficiency and quality of the resulting software. The CUR consortium also provides a valuable link with the engineering practice. Future developments are discussed within the CUR consortium and feedback is provided after new releases of the code.

5.2. Rail Displacements and Embankment Settlements

The elastic deformation of the track bed is an essential characteristic of the conventional rail-track structure. It facilitates the load distribution from the wheel via rail to a number of sleepers. Consequently, if the track beds with its underlying different embankment layers are too stiff, this situation may cause higher load concentrations, hence an increased grain attribution/abrasion in the ballast layer. This, in turn, gradually creates locally different stiffness, hence may yield to differential rail deformations under the traffic loads. These differential rail deformations can cause an adverse rearrangement of dynamic wheel forces, which, in the end, may progressively worsen the rail geometry, thus accelerating wheel/rail and rail/sleeper wear and tear. That is why routine weekly and monthly maintenance periods, condition of rails (such as existence of tiny cracks) and their connections should be checked, including the top of rail settlements. Some new equipment using ultrasound and laser technology and mountable on the front or back side of a maintenance locomotive allows this checks to be done fastly, while the train moves at 100 km/hr speed. Thus, it is important to provide and maintain an optimum rail-track structure (considered as a multi-layered composite system, ranging from natural ground to rail level) to avoid any excessive rail displacements, as each layer settlements are cumulative until to the top of rail to add-up to each other. Hence, it is important that top of rail settlements are within the tolerable limits. Otherwise excessive settlements may affect the stability and safety of the high speed trains and their passengers, apart from causing fast condition deterioration of the train and the railway infrastructure. Experience in the Far-East has shown that the maximum tolerable elastic top of rail deflection (Δz) under a passing wheel load of about 200 kN should be in the range of;

$$\Delta z \leq 2.0 \text{ mm} \tag{5.1}$$

for the high speed trains traveling at 200 – 400 km/hr velocities.

In the case of very stiff ground, a track bed (e.g. paved track on rock and on bridges) the elastic displacements at the top of the rail may result only from the loose rail-sleeper connections itself. On the other hand, in situ measurements have disclosed that the allowable elastic top of rail displacements may decrease with increasing speed. Therefore, the allowable minimum values of Δz should be few times smaller for the high speed trains (HST) than for the conventional trains traveling at speeds<160 km/hr.

European (France, Germany, Spain) practice of using ballast paved rail-tracks are more difficult and costly to maintain in the long term, but it is easier to bring them to tolerable values, because of frequent re-leveling done easily during any routine monthly or seasonal maintenances. On the other hand; Far-Eastern (Taiwan, Japan) practice of using reinforced concrete slab-track paved rail-tracks are easier and cheaper to maintain in the long term, but it is difficult and costly to fix them, if they have undergone excessive (total-differential) settlements. However the maximum tolerable values for any 20m long longitudinal section of the embankment (in the long term of 30 years after construction) are almost the same for both cases (Taiwan High Speed Railway Project/THSRP-Design Manual Volume 9, 2003) as;

$$s \leq 3 \text{ cm} \quad \text{for normal train speeds, } v \leq 160 \text{ km/hr} \tag{5.2}$$

$$s \leq 10 \text{ mm for the HST trains, traveling between: } 200 – 400 \text{ km/hr} \tag{5.3}$$

On the other hand, maximum total and differential settlements in the lateral (transverse) section of the embankment and/or between the two rails, should also be limited for the high speed trains to;

$$\Delta s_{transverse} \leq 2 \text{ mm} \tag{5.4}$$

5.3. Analysis of Settlements for High Speed Train Embankments

In this section, high speed train embankment modeling and analyzed with Plaxis V8 (2D) programme and they are summarized as follows:

1. First of all high speed train embankment was modeled (Figure 5.1),

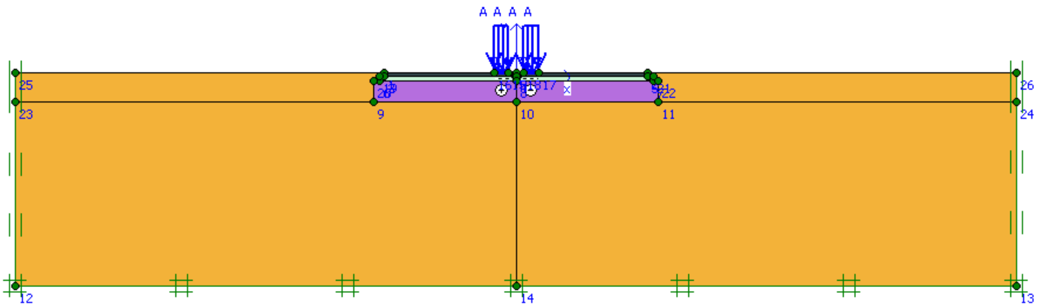


Figure 5.1. Embankment model

2. As there are four different high speed embankment layers above the natural subgrade layer (Figure 3.3) called as; ballast layer, sub-ballast layer, (uncemented)-prepared subgrade layer and subgrade layer. All of these layers have different index properties. So their individual material properties and test conditions should be separately assumed and introduced into the Plaxis Program, as given in Table 5.1.

Table 5.1. Material properties for embankment

Parameter	Name	Unit	Ballast Layer	Sub-ballast Layer	Prepared Subgrade Layer	Subgrade Layer
Material Model	Model	-	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Type of material behavior	Type	-	Drained	Drained	Drained	Drained
Soil unit weight above phreatic level	γ_{unsat}	kN/m^3	19	18	17	16
Soil unit weight below phreatic level	γ_{sat}	kN/m^3	22	21	20	19
Permeability in horizontal direction	k_x	m/sec	0.2	0.03	0.01	0.00001
Permeability in vertical direction	k_y	m/sec	0.2	0.03	0.01	0.00001
Young's modulus	E_{ref}	kN/m^2	300000	120000	80000*	60000
Poisson's ratio	υ	-	0.45	0.41	0.35	0.2
Cohesion	c_{ref}	kPa	1	1,5	7	5
Friction angle	Ø	°	40	38	37	35
Dilatancy angle	ψ	°	-	-	2	-

* This value is for uncemented prepared subgrade layer .

3. Next step is to generate the finite element mesh for the high speed train (HST) embankment (Figure 5.2.). Then, initial conditions are determined and entered into the program. It is noted that the Ground Water Table level is not considered to exist in this analyses.

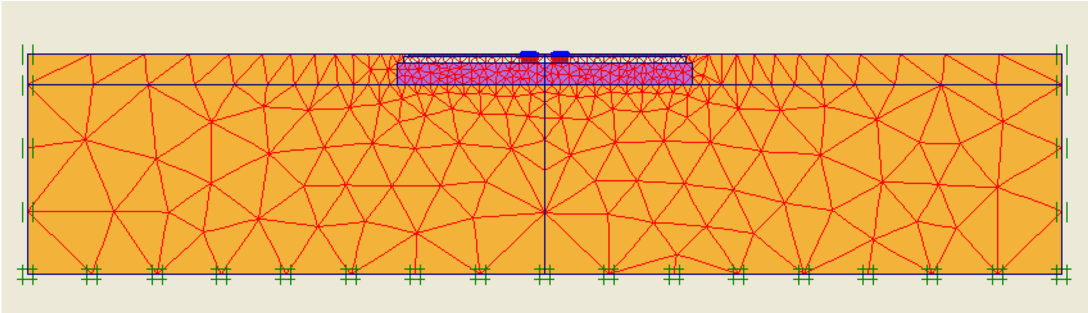


Figure 5.2. Model of meshed embankment

4. Analyses were made for the seven different elasticity modulus values (belonging to one value per layer) for the high-speed train embankment. One of them is for the U-PSL's elasticity modulus value (of 80 Mpa) and the other 6 values are the laboratory obtained C-PSL's elasticity modulus values. The other 2 variables used were; 2 water-to cement ratios (w/c=0.4, 0.5) and 3 cement contents (c=20, 25, 30%). Tests done for the lower cement contents of 10% and 15% were not reported, as they did not meet the criteria of equaling or exceeding 80 Mpa elasticity modulus value. Laboratory obtained results are given in Table 5.2.

Table 5.2. Compare elasticity modulus with 80 Mpa

w/c	**Elasticity Modulus, E (Mpa)**		
	CementContent, C (%)		
	20	**25**	**30**
0.5	337.80	428.56	614.89
0.4	485.817	747.63	842.03

5. General rules are given for the calculation of the associated static effects by TSE EN 1991-2. Rail traffic actions are defined by means of load models. Load Model 71 was used as loading in Plaxis programme. Load Model 71 (and Load Model SW/0 for continuous bridges) to represent normal rail traffic on mainline railways. Load Model 71 represents the static effect of vertical loading gue to normal rail traffic. The load arrangement and the characteristic values for vertical loads shall be taken as shown in Figure 5.3.

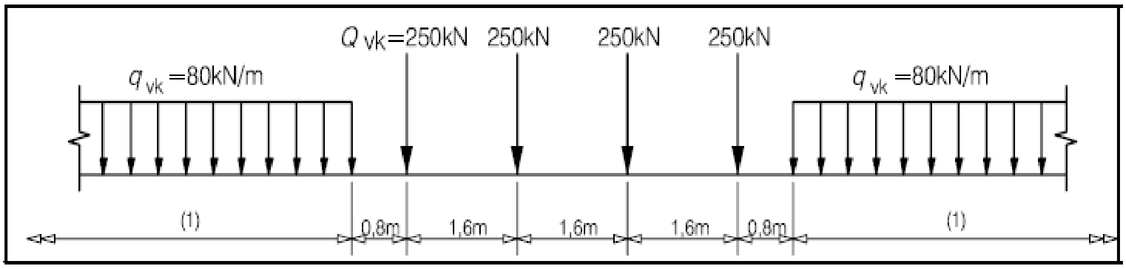


Figure 5.3. Load Model 71 and Characteristic values for vertical loads.

Load Model SW/0 represents the static effect of vertical loading due to normal rail traffic. The load arrangement and the characteristic values for vertical loads shall be taken as shown in Figure 5.4.

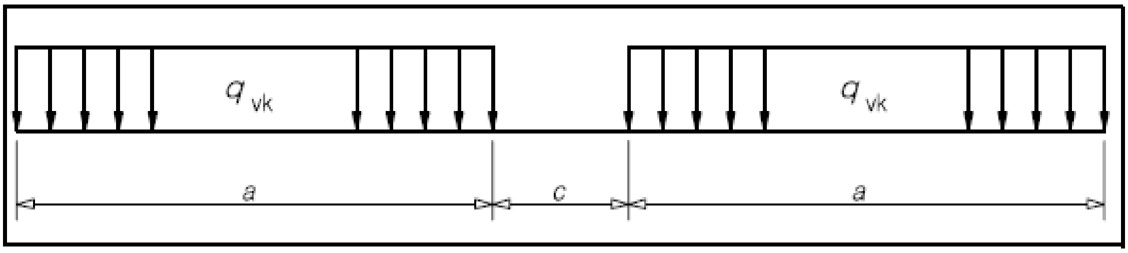


Figure 5.4. Load Model SW/0

Table 5.3. Characteristic values for vertical loads for Load Model SW/0

Load Model	q_{vk} [kN/m]	a [m]	c [m]
SW/0	133	15.0	5.3

6. Finally, total settlements were calculated using the Plaxis Program with 6 different laboratory obtained elasticity modulus values of the C-PSL (Table 5.2) to be compared with that of the 80 Mpa value of the U-PSL, which is currently in use in the high speed railway embankments (Figure 3.3). All settlement values should be within the limiting (maximum tolerable) value given in the equation 5.4, so that any C-PSL could be used as a substitute of U-PSL, which is currently in use Worldwide.

5.4 Evaluation of Test Results

5.4.1. Total Settlement Results of the U-PSL

Settlement analyses with the Plaxis V8 (2D) Programme that were conducted on the 2 m thick uncemented-prepared subgrade layer (U-PSL), for which the elasticity modulus value is taken as E_1 = 80 Mpa, are shown in Figures 5.5- 5.8.

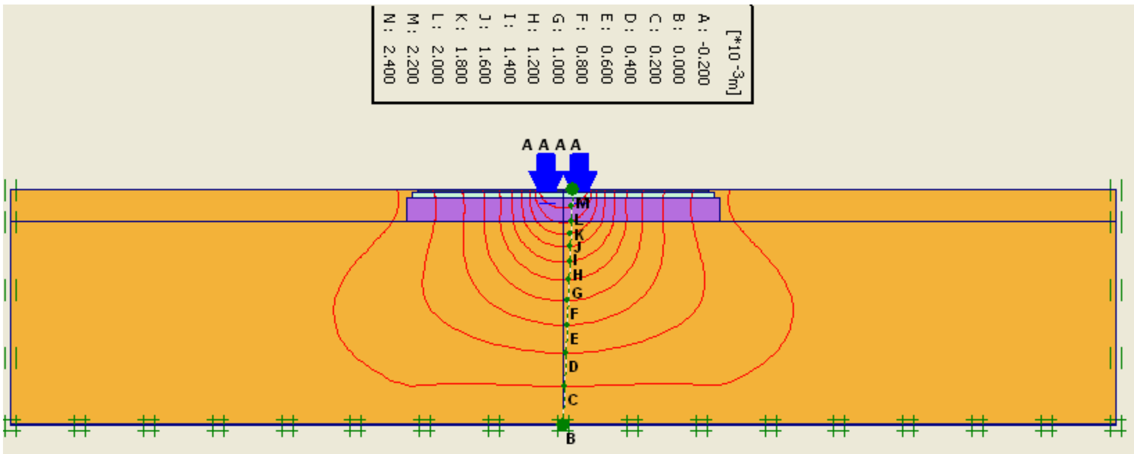


Figure 5.5. Total settlement contour lines for the U-PSL with E_1=80 Mpa

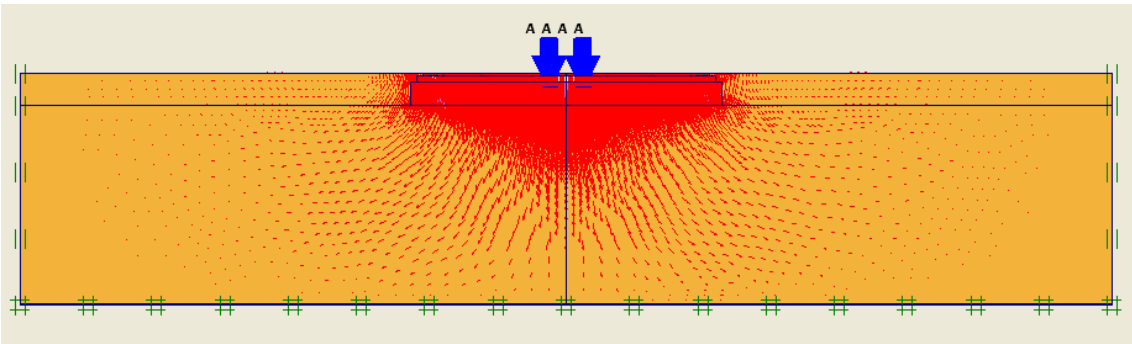


Figure 5.6. Total settlements shown with arrows for the U-PSL with E_1=80 MPa

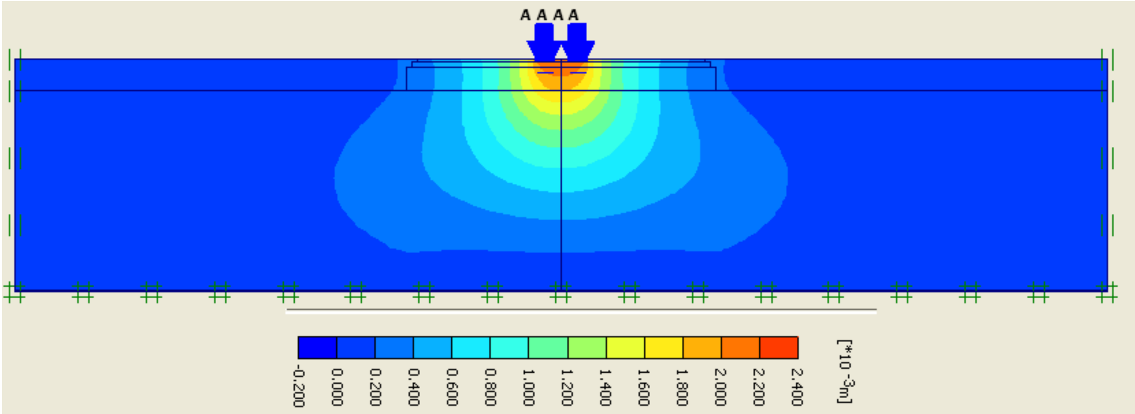


Figure 5.7. Total settlements shown by shading for the U-PSL with E_1=80 MPa

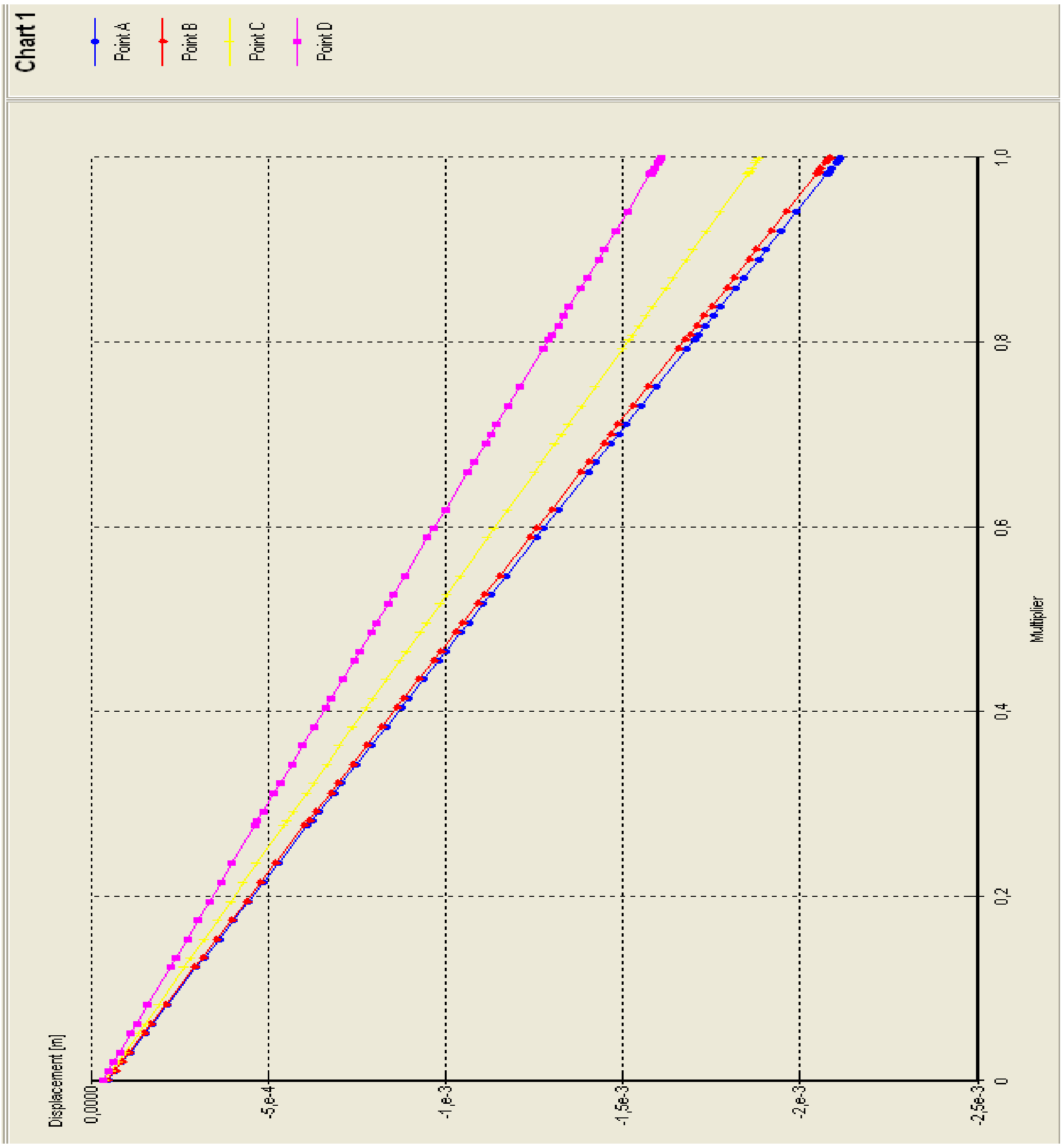


Figure 5.8. Total settlement results for the U-PSL with E_1=80 Mpa

Analyses results of total settlements for the U-PSL are summarized in Table 5.4.

Table 5.4. Analyses results of total settlements for the U-PSL with E_1=80 Mpa

Layers	**U-PSL**	**ASV ***	**Notes**
	ΔS_1 (mm)	**ΔS (mm)**	
Ballast (A)	1.993	< 2	OK
Sub-ballast (B)	1.986	< 2	OK
Prepared Subgrade (C)	1.875	< 2	OK
Subgrade (D)	1.598	< 2	OK

* ASV = Allowable Settlement Values

As a result; while the 2m thick U-PSL's total settlement itself was 1.875 mm, the total settlement at the top of rail was 1.993 mm. This is acceptable as it is less than the allowable value of 2 mm.

5.4.2. Settlement Results of the Cemented–Prepared Subgrade Layer

Settlement analyses with the Plaxis V8 (2D) Programme that were conducted on the also 2m thick cemented-prepared subgrade layer (C-PSL), for which the Laboratory obtained elasticity modulus value (E_2) values from Table 5.2 are used. Results are shown in Figures 5.9 – 5.12 for the values of;

A) Water-cement ratio, w/c = 0.4; cement content, C= 20% and E_2 = 485.817 Mpa

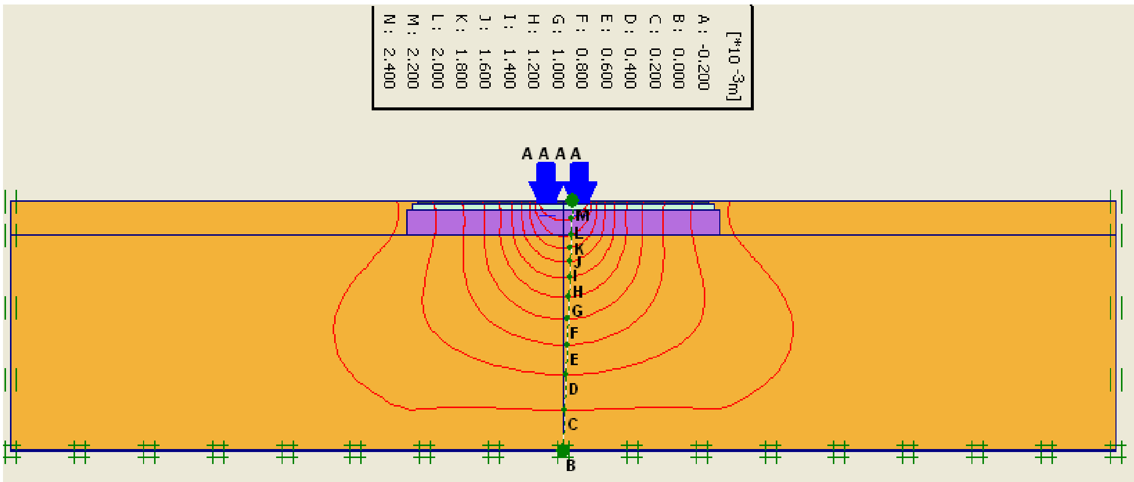


Figure 5.9. Total settlement contour lines for the C-PSL with E_2 = 485.817 Mpa

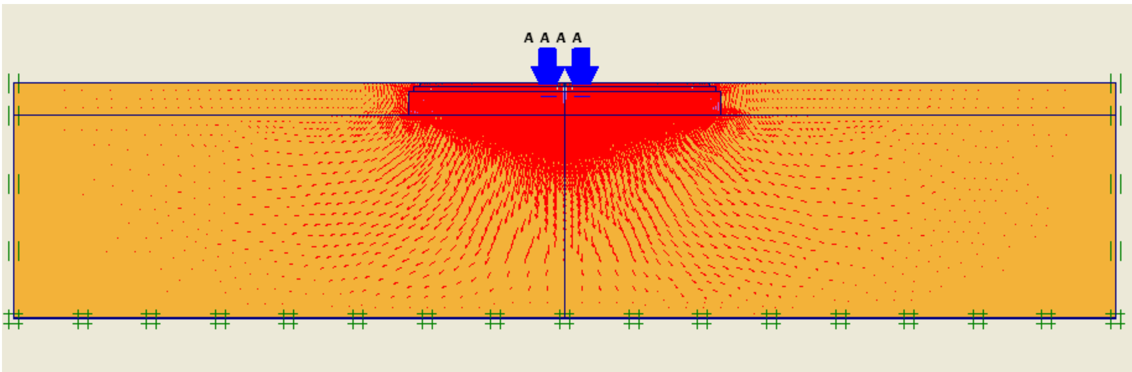


Figure 5.10. Total Settlements shown with arrows for the C-PSL with E_2 = 485.817 Mpa

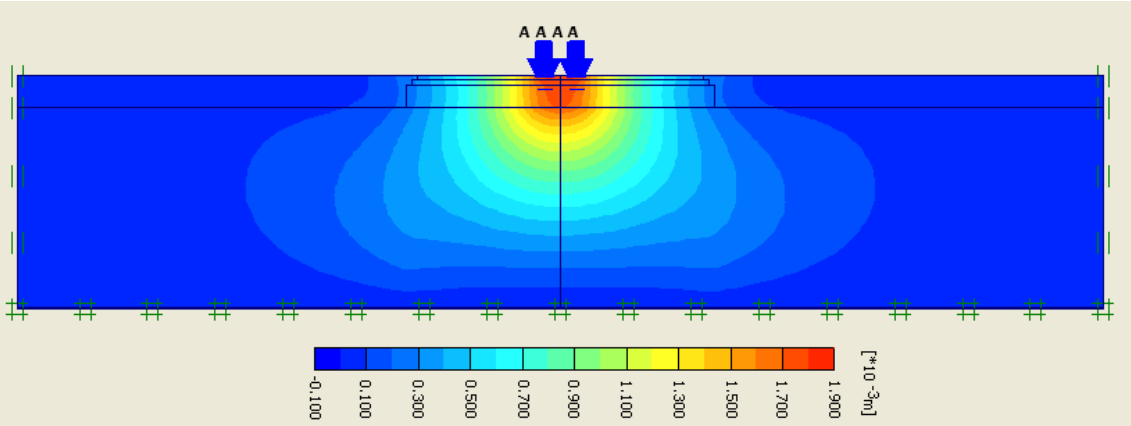


Figure 5.11. Total settlements shown by shading for the C-PSL with E_2 = 485.817 Mpa

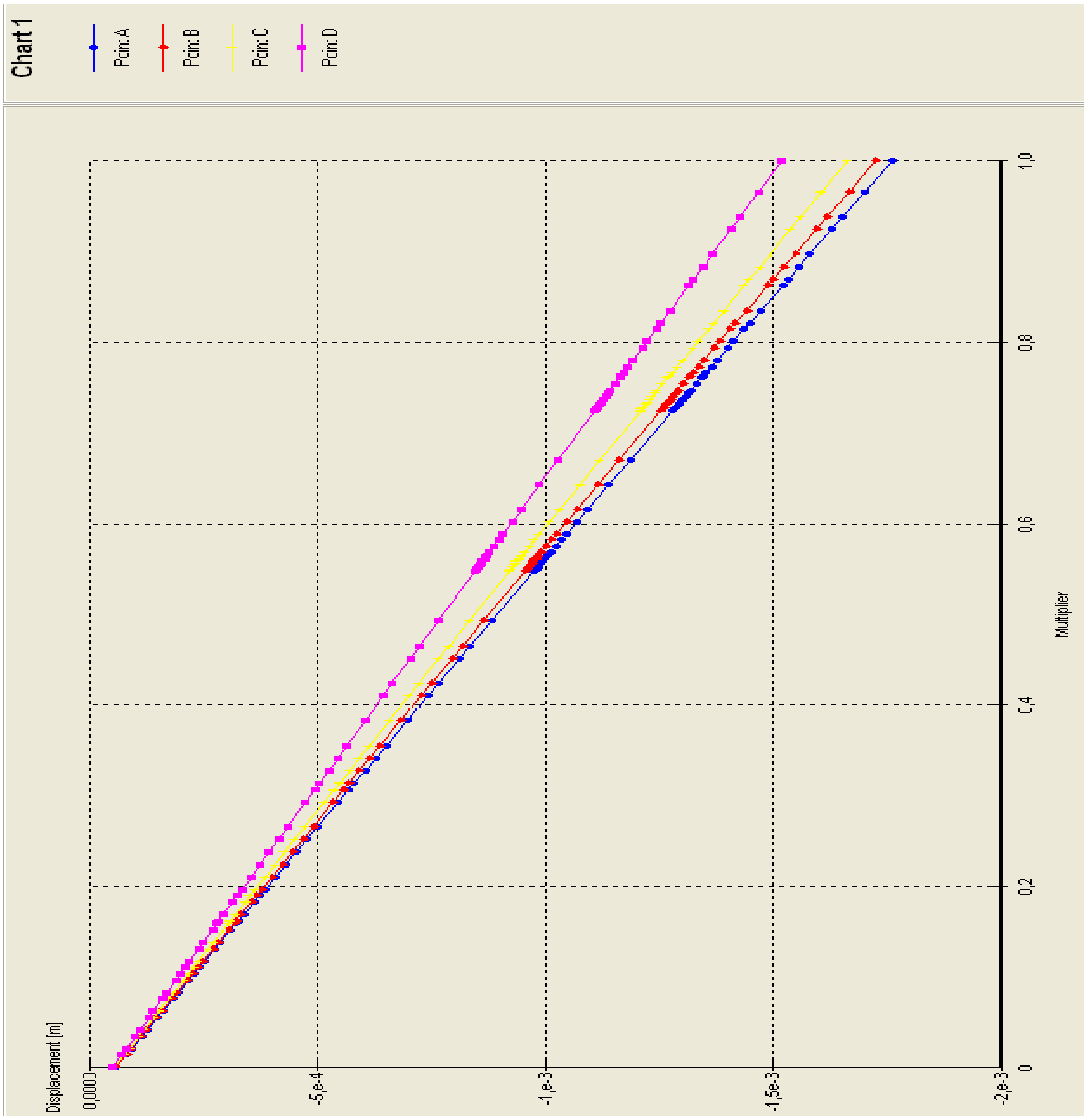


Figure 5.12. Total settlement results for the C-PSL with E_2 = 485.817 Mpa

Analyses results of total settlements for the C-PSL with E_2 = 485.817 Mpa are summarized in Table 5.5.

Table 5.5. Analyses results of total settlements for the C-PSL with E_2 = 485.817 Mpa

Layers	**C-PSL**	**ASV ***	**Notes**
	ΔS_2 (mm)	**ΔS (mm)**	
Ballast (A)	1.761	< 2	OK
Sub-ballast (B)	1.723	< 2	OK
Prepared Subgrade (C)	1.664	< 2	OK
Subgrade (D)	1.52	< 2	OK

As a result; while the 2m thick U-PSL's total settlement itself was 1.664 mm, the total settlement at the top of rail was 1.761 mm. This is acceptable, as it is less than the allowable value of 2 mm.

B) For w/c = 0.4; Cement Content, C=25% and E_3= 747.63 Mpa

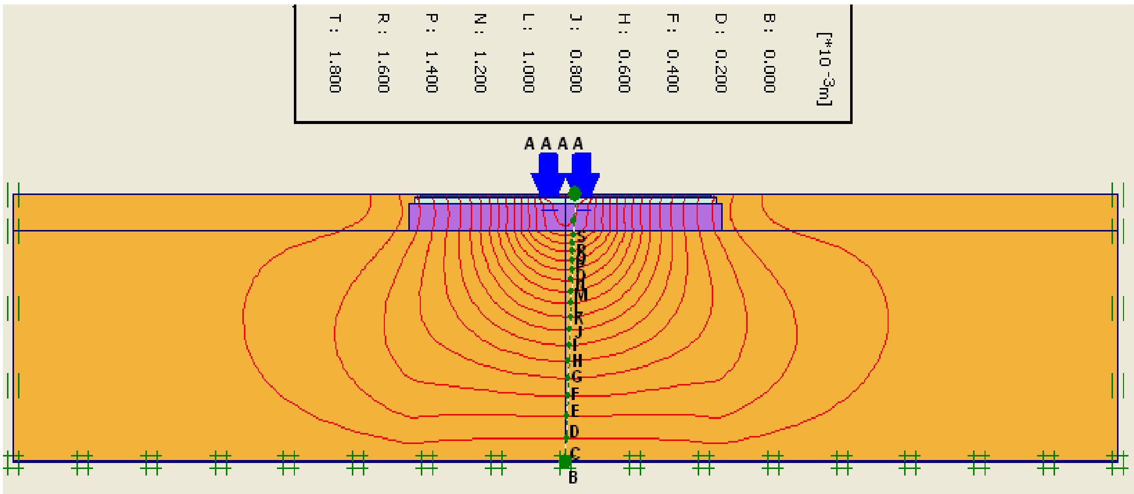


Figure 5.13. Total settlement contour lines for the C-PSL with E_3 = 747.63 Mpa.

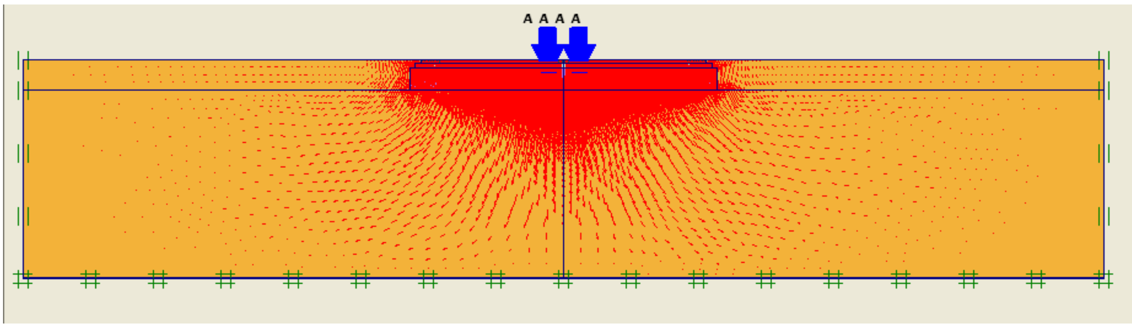


Figure 5.14. Total displacement with arrows for cemented embankment for E_3 = 747.63 Mpa

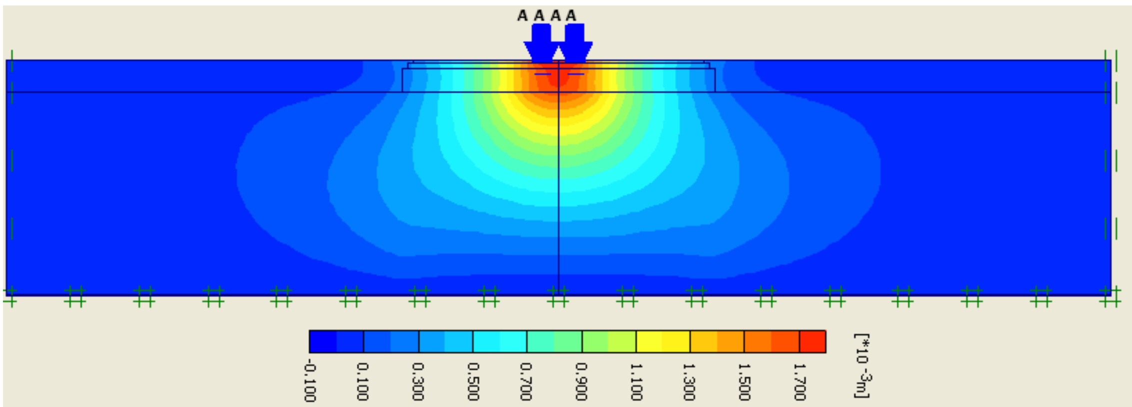


Figure 5.15. Total settlements shown by shading for the C-PSL with E_3 = 747.63 Mpa

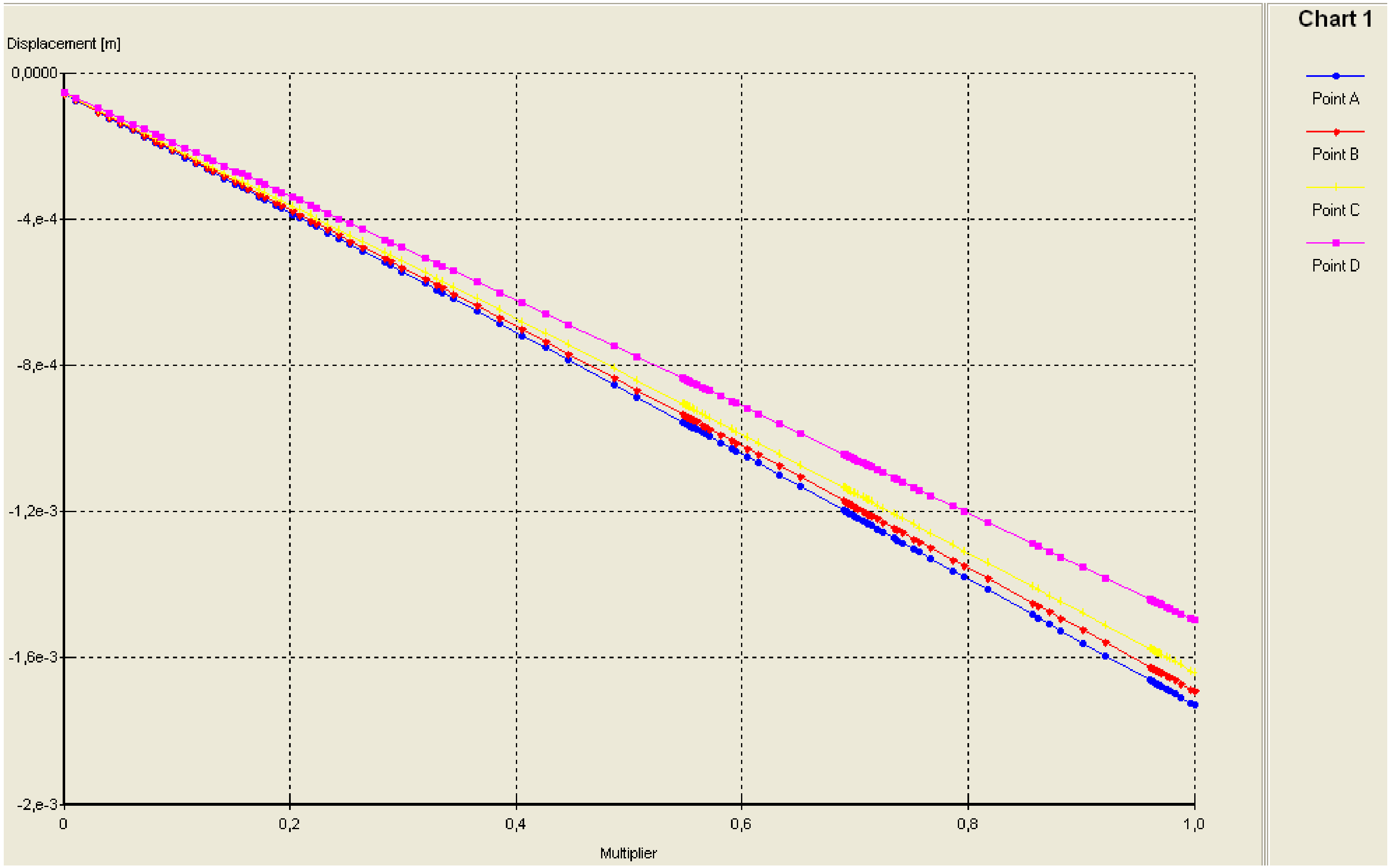


Figure 5.16. Total settlement results for the C-PSL with E3 = 747,63 Mpa

Analyses results of total settlements for the C-PSL with E_3 = 747.63 Mpa are summarized in Table 5.6.

Table 5.6. Analyses results of total settlements for the C-PSL with E_3 = 747.63 Mpa

Layers	**C-PSL**	**ASV ***	**Notes**
	ΔS_3 (mm)	**ΔS (mm)**	
Ballast (A)	1.724	< 2	OK
Sub-ballast (B)	1.686	< 2	OK
Prepared Subgrade (C)	1.635	< 2	OK
Subgrade (D)	1.494	< 2	OK

As a result; while the 2m thick U-PSL's total settlement itself was 1.635 mm, the total settlement at the top of rail was 1.724 mm. This is accaptable as it is less than the allowable value of 2 mm.

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C) For w/c = 0.4; Cement Content, C=30% and E_4 = 842.03 Mpa

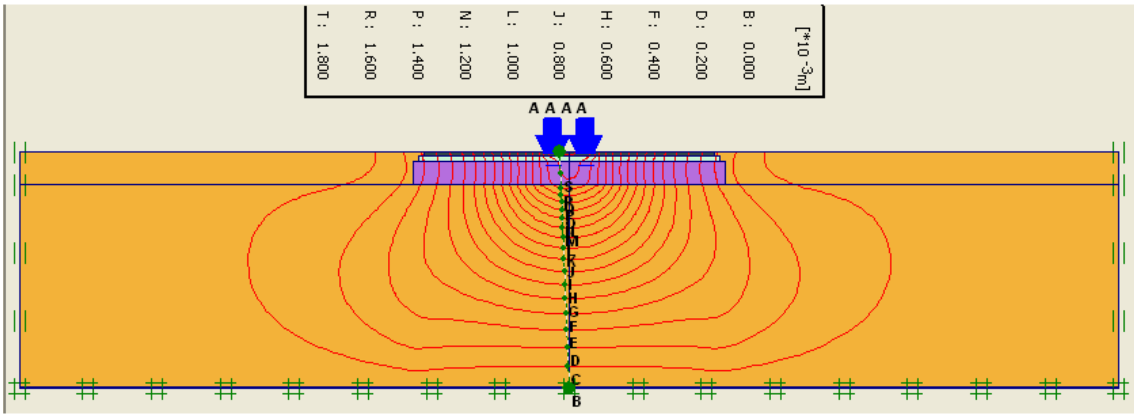

Figure 5.17. Total settlement contour lines for the C-PSL with E_4 = 842.03 Mpa.

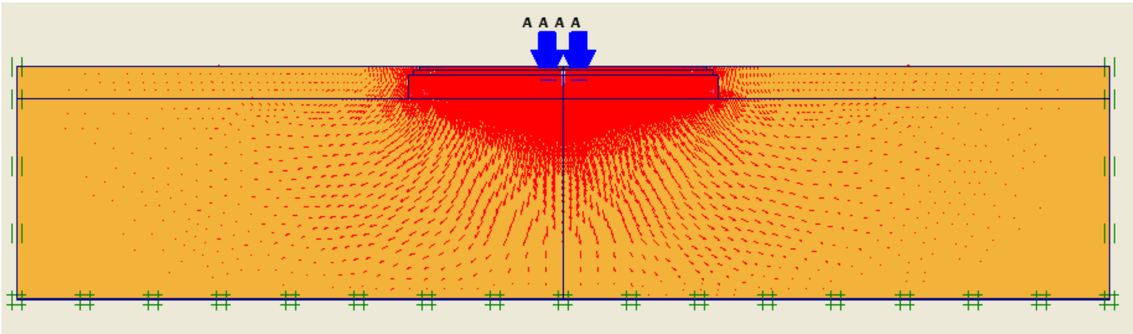

Figure 5.18. Total settlements shown with arrows for the C-PSL with E_4 = 842.03 Mpa

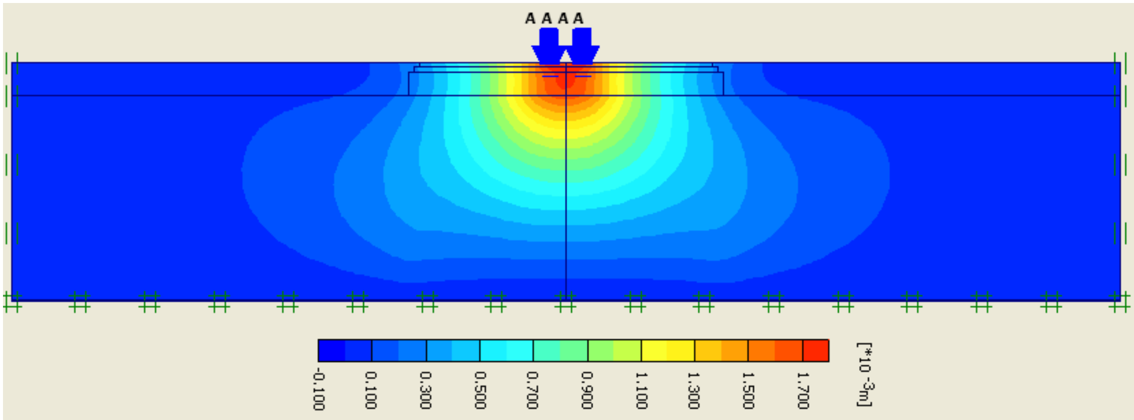

Figure 5.19. Total settlements shown by shading for the C-PSL with E_4 = 842.03 Mpa

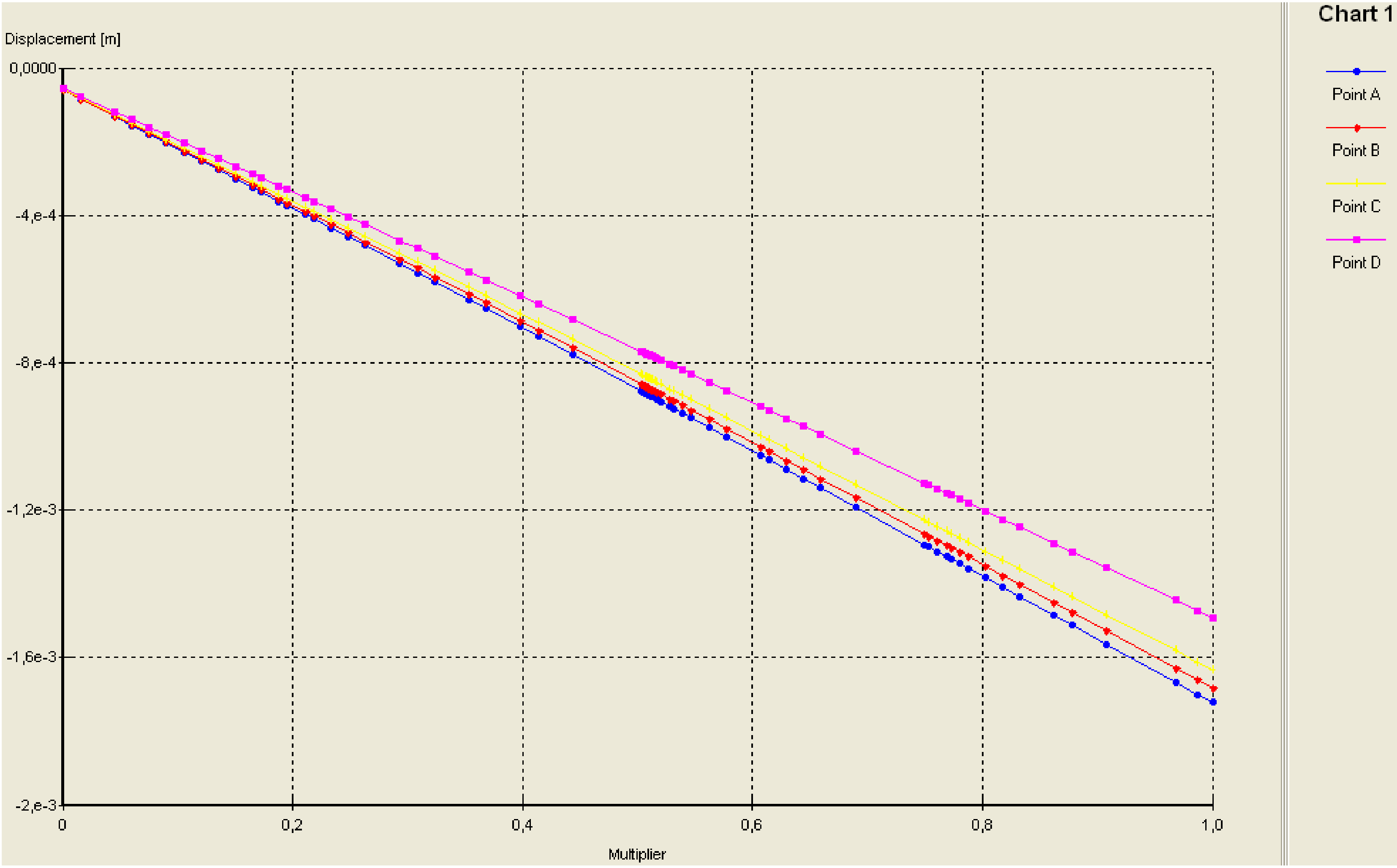

Figure 5.20. Total settlement results for the C-PSL with E4 = 842,03 Mpa

Analyses results of total settlements for the C-PSL with E_4 = 842.03 Mpa are summarized in Table 5.7.

Table 5.7. Analyses results of total settlements for the C-PSL with E_4 = 842.03 Mpa

Layers	**C-PSL**	**ASV ***	**Notes**
	ΔS_4 (mm)	**ΔS (mm)**	
Ballast (A)	1.722	< 2	OK
Sub-ballast (B)	1.683	< 2	OK
Prepared Subgrade (C)	1.634	< 2	OK
Subgrade (D)	1.473	< 2	OK

As a result; while the 2m thick C-PSL's total settlement itself was 1.634 mm, the total settlement at the top of rail was 1.722 mm. This is accaptable as it is less than the allowable value of 2 mm.

D) For w/c = 0.5; Cement Content, C=20% and E_5 = 337.80 Mpa

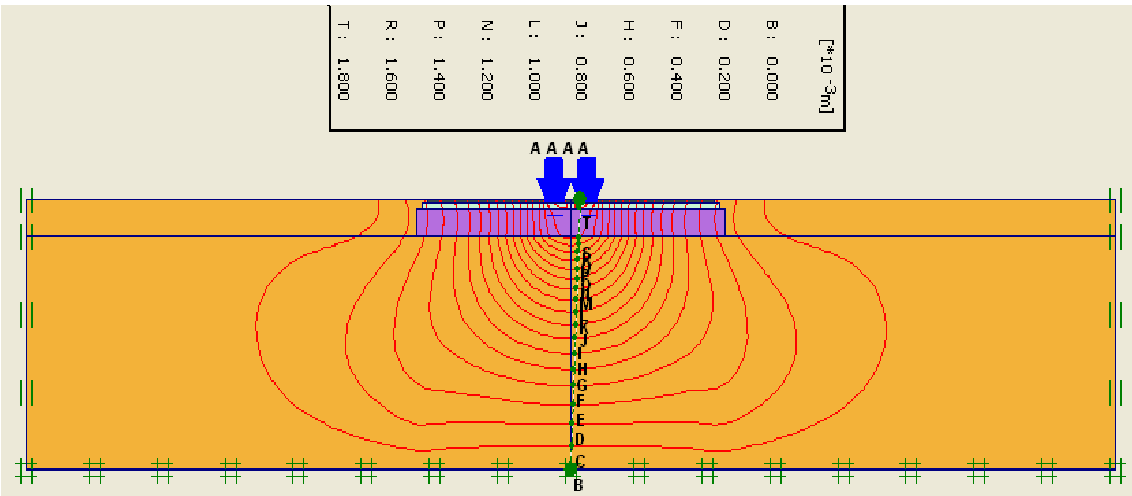

Figure 5.21. Total settlement contour lines for the C-PSL with E_5 = 337.80 Mpa.

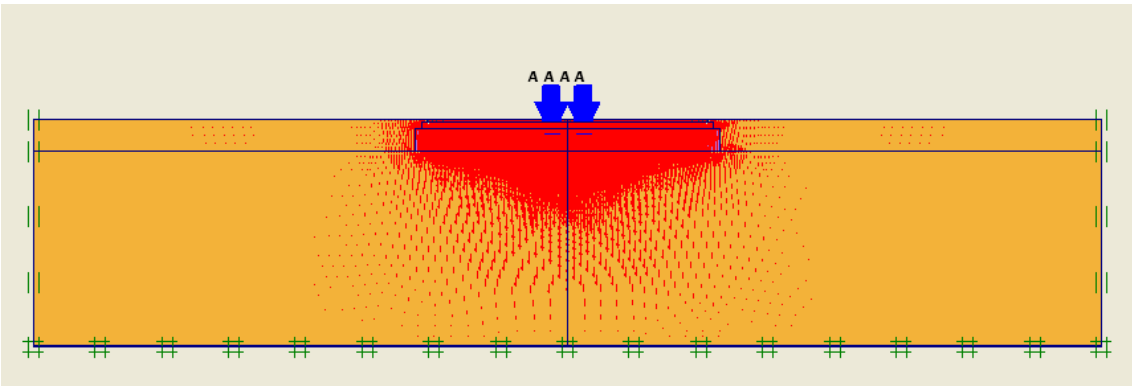

Figure 5.22. Total settlements shown with arrows for the C-PSL with E_5 = 337.80 Mpa

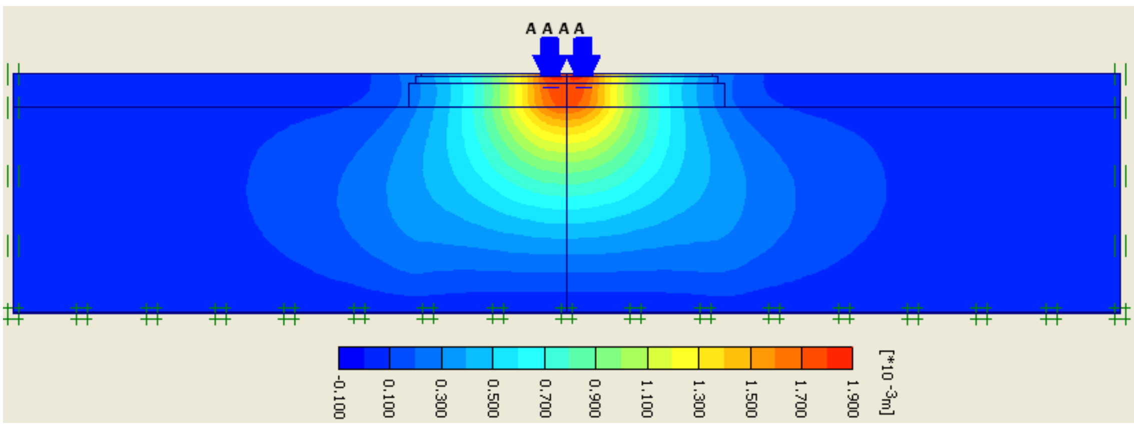

Figure 5.23. Total settlements shown by shading for the C-PSL with E_5 = 337.80 Mpa

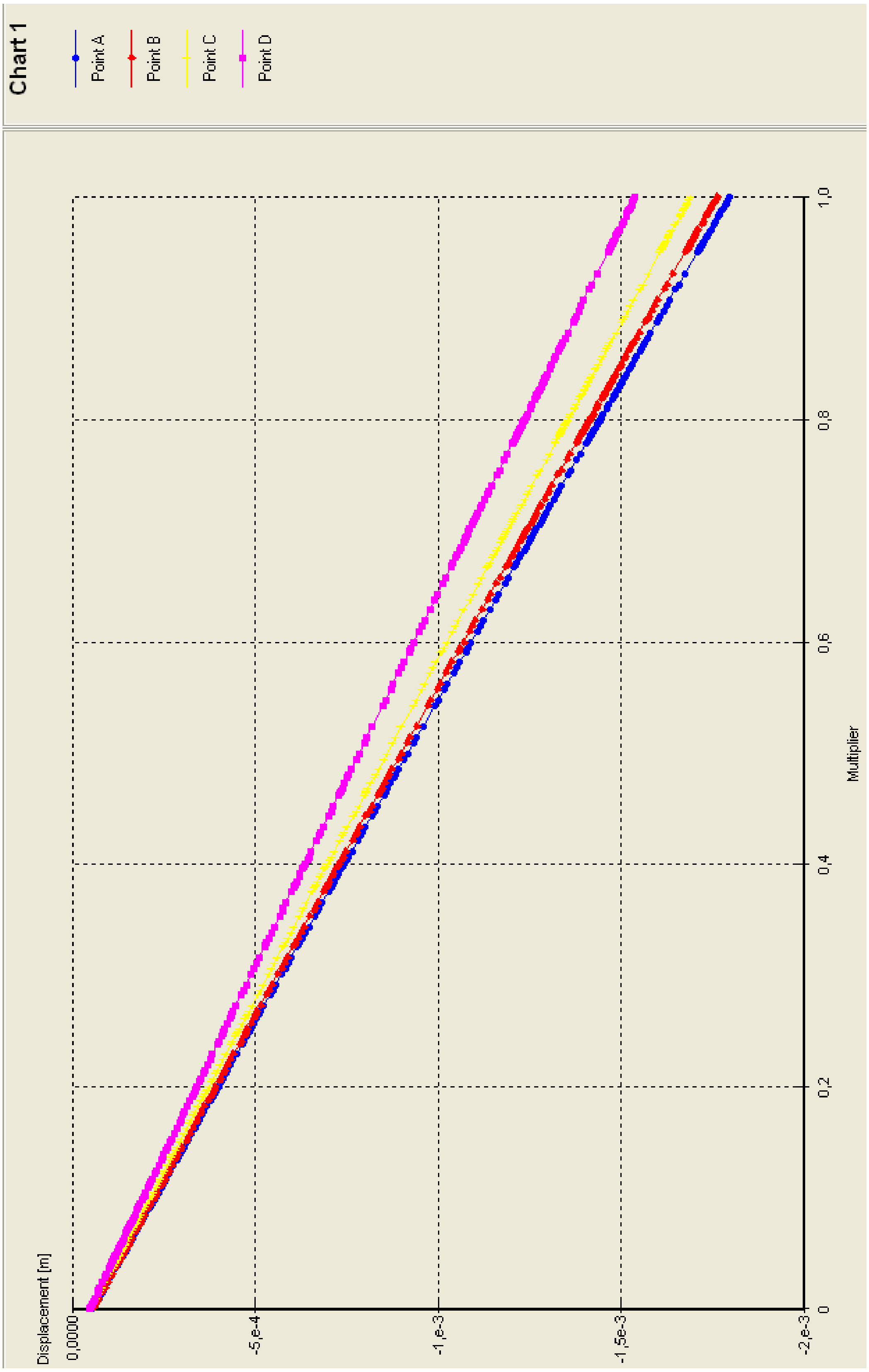

Figure 5.24. Total settlement results for the C-PSL with E5 = 337,80 Mpa

Analyses results of total settlements for the C-PSL with E_5 = 337.80 Mpa are summarized in Table 5.8.

Table 5.8. Analyses results of total settlements for the C-PSL with E_5 = 337.80 Mpa

Layers	**C-PSL**	**ASV ***	**Notes**
	ΔS_5 (mm)	**ΔS (mm)**	
Ballast (A)	1.785	< 2	OK
Sub-ballast (B)	1.749	< 2	OK
Prepared Subgrade (C)	1.675	< 2	OK
Subgrade (D)	1.536	< 2	OK

As a result; while the total settlement of the 2m thick C-PSL itself is 1.675 mm, the total settlement at the top of rail is 1.785 mm. This is acceptable, as it is less than the allowable value of 2 mm.

E) For w/c = 0.5; Cement Content, C=25% and E_6 = 428.56 Mpa

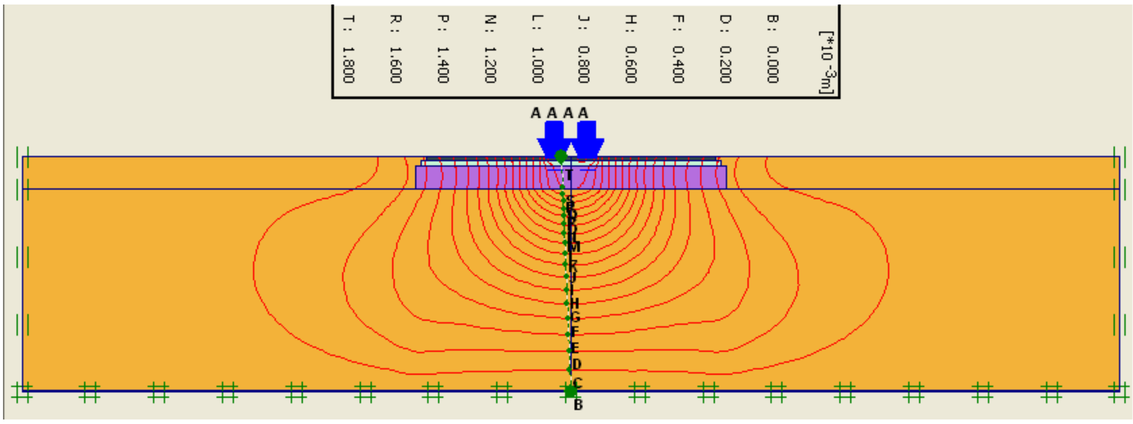

Figure 5.25. Total settlement contour lines for the C-PSL with E_6 = 428.56 Mpa

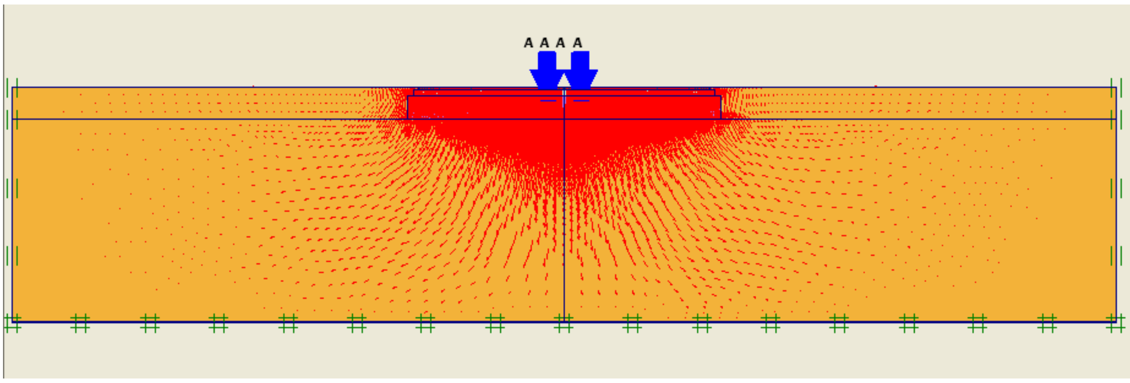

Figure 5.26. Total settlements shown with arrows for the C-PSL with E_6 = 428.56 Mpa

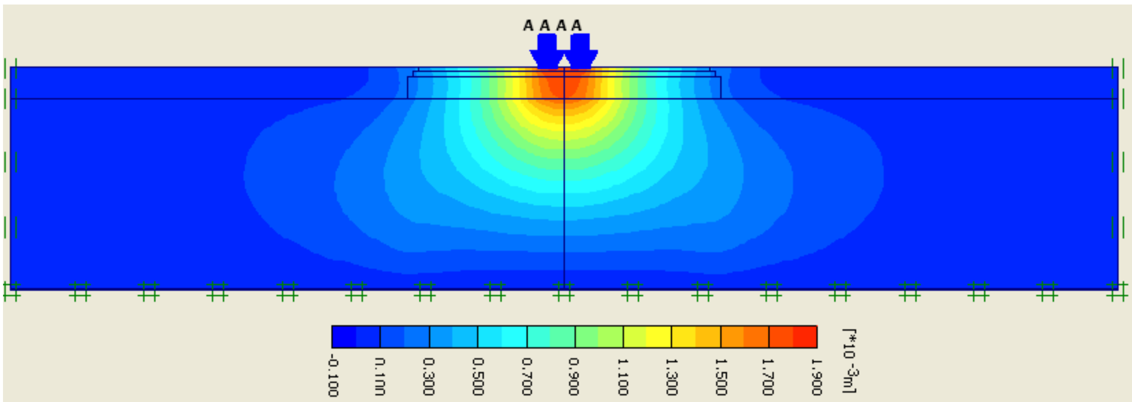

Figure 5.27. Total settlements shown by shading for the C-PSL with E_6 = 428.56 Mpa

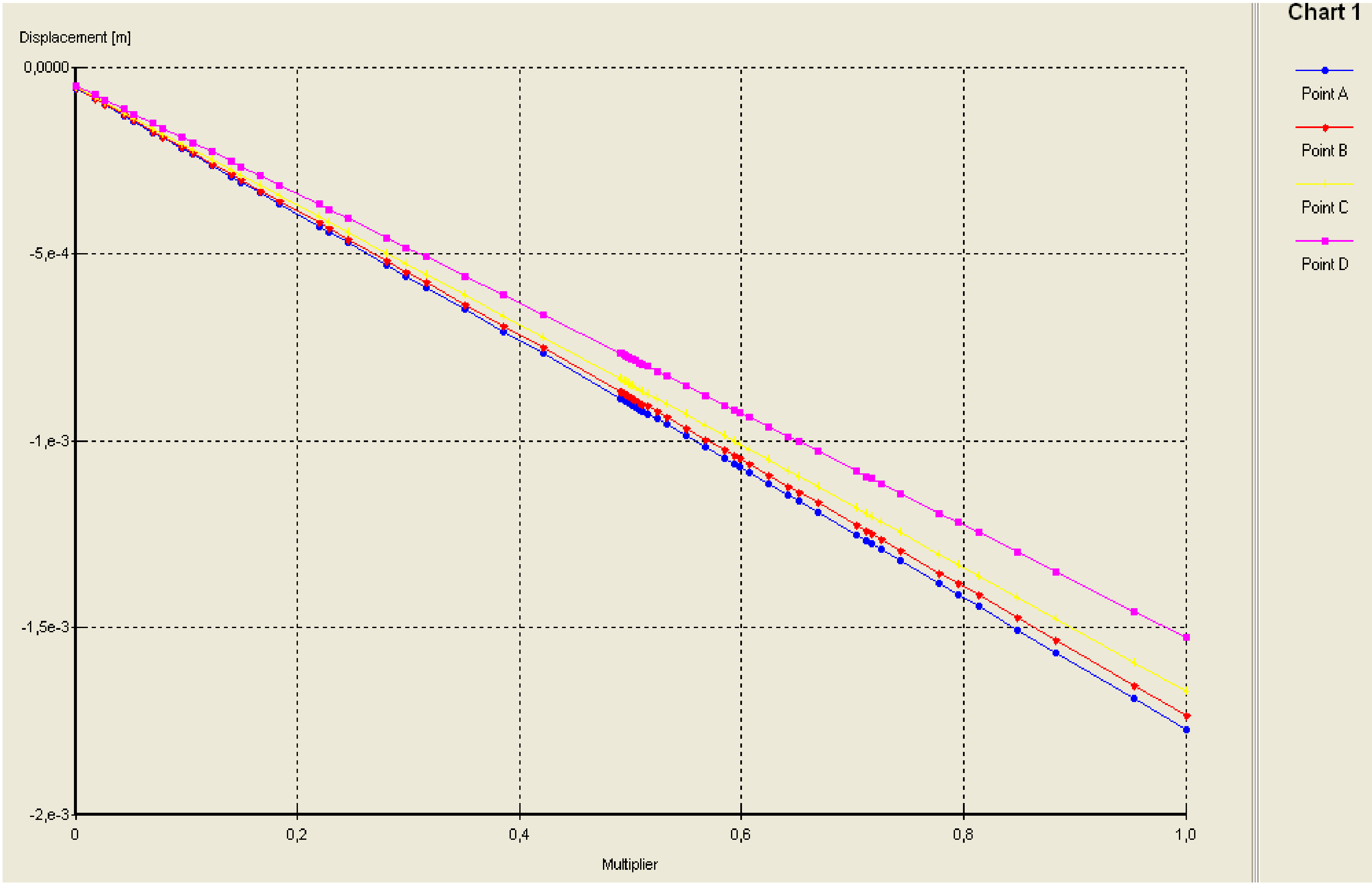

Figure 5.28. Total settlement results for the C-PSL with E6 = 428,56 Mpa

Analyses results of total settlements for the C-PSL with E_6 = 428.56 Mpa are summarized in Table 5.9.

Table 5.9. Analyses results of total settlements for the C-PSL with E_6 = 428.56 Mpa

Layers	**C-PSL**	**ASV ***	**Notes**
	ΔS_6 (mm)	**ΔS (mm)**	
Ballast (A)	1.772	< 2	OK
Sub-ballast (B)	1.735	< 2	OK
Prepared Subgrade (C)	1.673	< 2	OK
Subgrade (D)	1.527	< 2	OK

As a result; while the total settlement of the 2m thick C-PSL itself is 1.673 mm, the total settlement at the top of rail is 1.772 mm. This is acceptable as it is less than the allowable value of 2 mm.

F) For w/c = 0.5; Cement Content, C=30% and E_7 = 614.89 Mpa

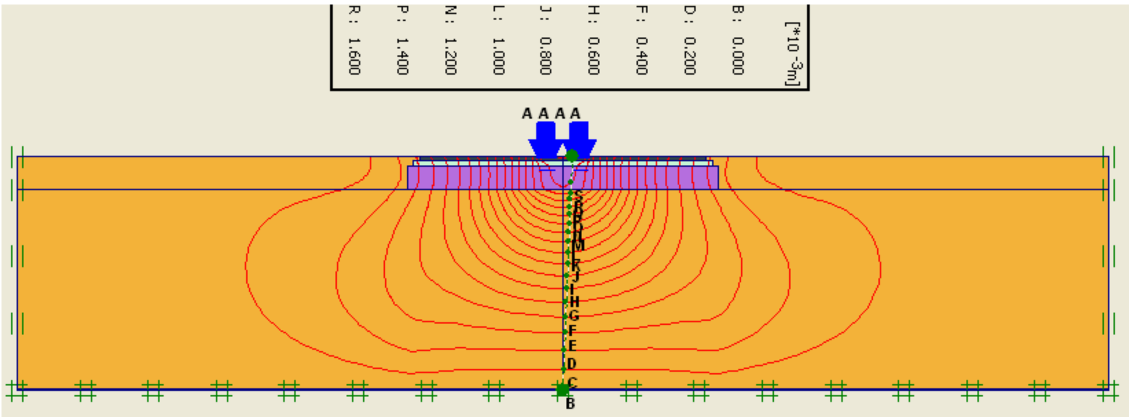

Figure 5.29. Total settlement contour lines for the C-PSL with E_7 = 614.89 Mpa.

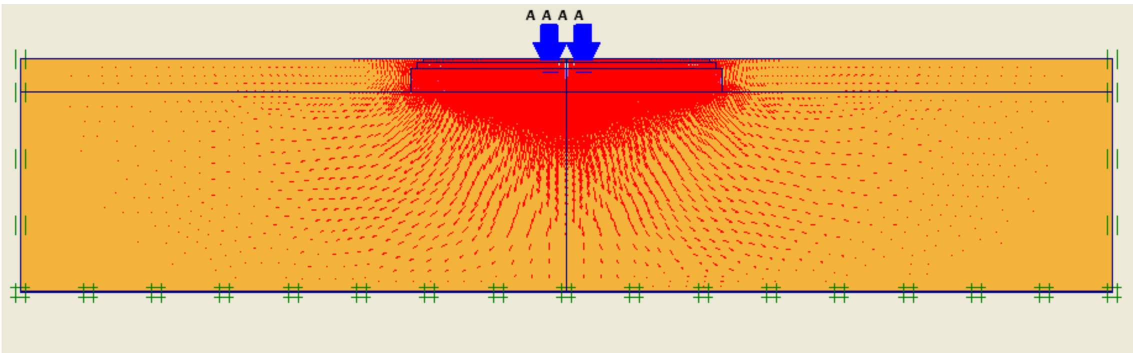

Figure 5.30. Total settlements shown with arrows for the C-PSL with E_7 = 614.89 Mpa

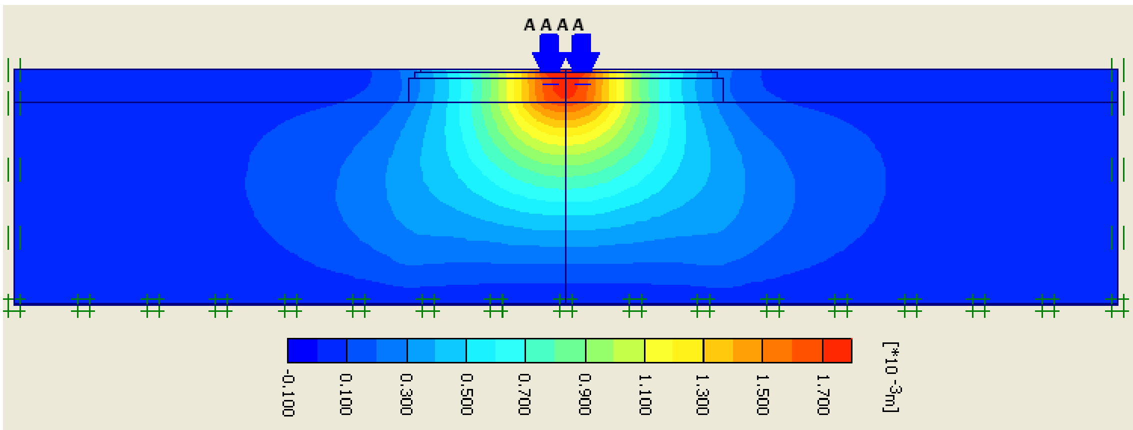

Figure 5.31. Total settlements shown by shading for the C-PSL with E_7 = 614.89 Mpa

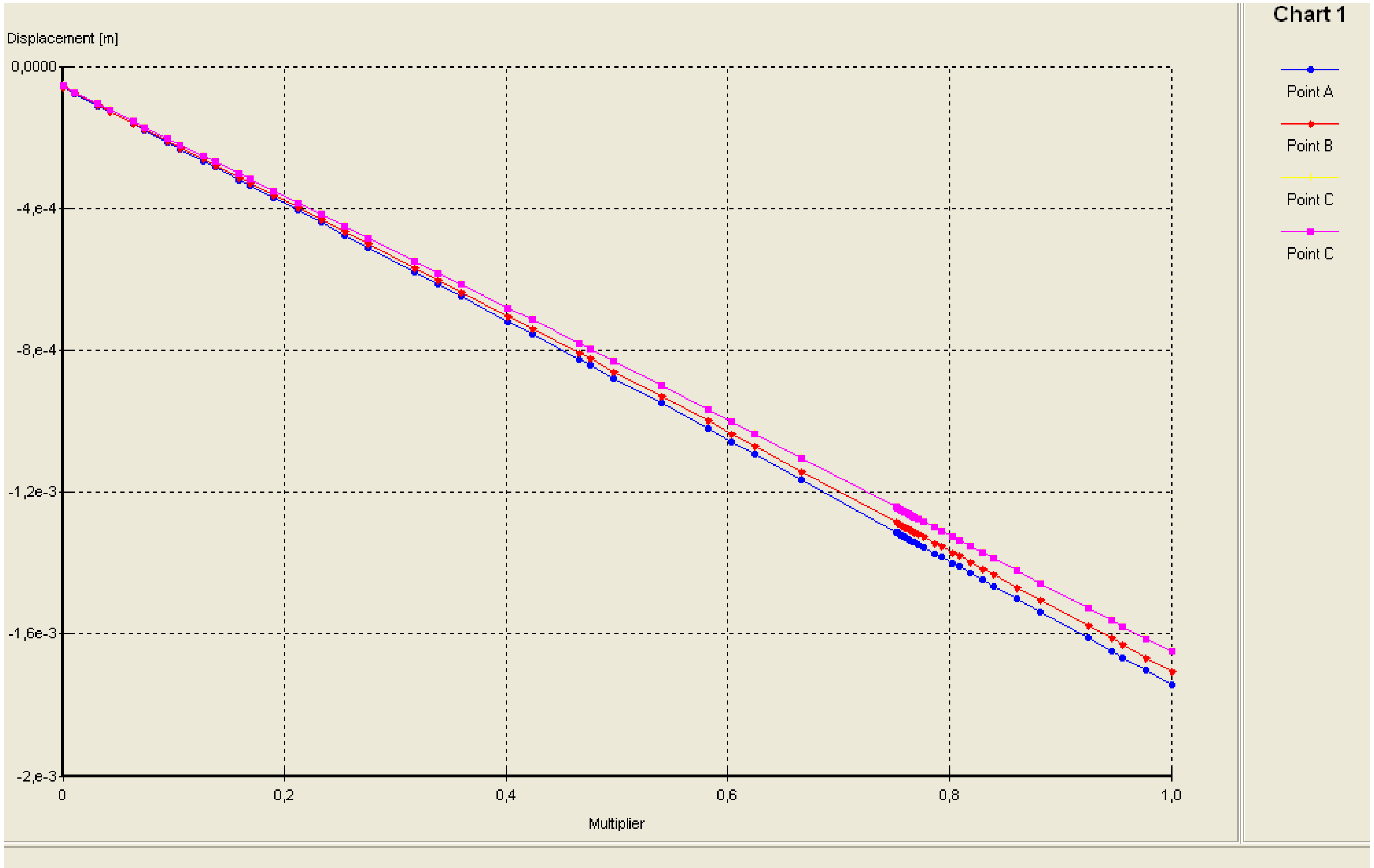

Figure 5.32. Total Settlement Results for the C-PSL with E7 = 614,89 Mpa

Analyses results of total settlements for the C-PSL with E_7 = 614.89 Mpa are summarized in Table 5.10.

Table 5.10. Analyses results of total settlements for the C-PSL with E_7 = 614.89 Mpa

Layers	C-PSL	ASV *	Notes
	ΔS_7 (mm)	ΔS (mm)	
Ballast (A)	1.742	< 2	OK
Sub-ballast (B)	1.704	< 2	OK
Prepared Subgrade (C)	1.65	< 2	OK
Subgrade (D)	1.508	< 2	OK

As a result; while the total settlement of the 2m thick C-PSL itself is 1.65 mm, the total settlement at the top of rail is 1.742 mm.. This is acceptable as it is less than the allowable value of 2 mm.

Table 5.11. Analyses Results of Total Settlements for the 2 m thick C-PSL, if used in HST Embankments in Stead of U-PSL

Layers	U-PSL	C-PSL						ASV *	Notes
	ΔS_1 (mm)	ΔS_2 (mm)	ΔS_3 (mm)	ΔS_4 (mm)	ΔS_5 (mm)	ΔS_6 (mm)	ΔS_7 (mm)	Δs (mm)	
Ballast	1,993	1.761	1.724	1,722	1.785	1.772	1.742	< 2	OK
Sub-ballast	1,986	1.723	1.686	1.683	1.749	1.735	1.704	< 2	OK
Prepared Subgrade	1.875	1.664	1.635	1.634	1.675	1.673	1.65	< 2	OK
Subgrade	1.598	1.52	1.494	1.473	1.536	1.527	1.508	< 2	OK

*ASV = Allowable Settlement Values

Results in Table 5.11 show that; if the U–PSL in the HST embankments is replaced with the C–PSL, allowable settlement values (ASV) are not exceeded. This means that; the C-PSL can be used instead of the U-PSL with a reduced thickness of 1.75 m (Figure 3.3).

CHAPTER 6

CONCLUSION AND RECOMMENDATION

6.1. Conclusion

Formation of high-speed train (HST) infrastructure is a rather new subject worldwide and in Turkey, as HST infrastructure is different than for normal train infrastructure (NTI). Existing NTI remains inadequate to meet high geometric and material properties required by HST. If strict criteria is not met, fatal accidents may occur.

In this study; Taiwan HST Project's design criteria and fill types were used to study replaceability of one fill strata called: Uncemented-Prepared Subgrade Layer (U-PSL) with a cemented one (C-PSL) by conducting various laboratory tests to obtain soils' index properties and parameters, which were used to find the maximum total settlements by using Plaxis V8 (2D) Programme. For this purpose; 3 groups (totally 270 no.s with 90 no.s per group) of cylindirical concrete samples were obtained with various cement contents (10, 15, 20, 25, 30 %) at 3 diameters (4, 8, 10 cm) and for 2 water-cement ratios (0.4, 0.5), then 7-28 day cured and tested at IYTE-MAM to find elasticity modulus, stress, strain, force results. Using such results in the Plaxis Programme, maximum total settlements were calculated for different layers. C-PSL mixes having 20-30% cement contents met the required strict settlement criteria as with the U-PSL mix currently used in Far-Eastern HST Projects. This showed that one of such C-PSL mixes can be used in place of U-PSL with an approximately 12.5 % reduction in the layer thickness, corresponding to 1.75m, instead of 2.0m thickness of the U-PSL currently used.

6.2. Recommendations for Future Studies

Using a similar laboratory and design procedures explanied in this thesis, sub-ballast and subgrade layer can also be designed, analyzed and if the results meet top of rail settlement criteria for the high-speed train embankment, such layers can be replaced with their cemented equivalents. This may bring construction cost reductions due to reduced layer thicknesses.

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APPENDIX A

RESULTS OF ELASTICITY FOR 7 DAY

Table A.1. 7 days results of elasticity modulus for W/C = 0.5 and C = 10%

	SAMPLE 1 - W/C=0.5 C=10% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	113.82	28.814	24.16	55.598
B	60.52	18.883	22.09	33.831
C	56.35	16.4	19.116	30.622

Table A.2. 7 days results of elasticity modulus for W/C = 0.5 and C = 15%

	SAMPLE 2 - W/C=0.5 C=15% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	82.361	104.02	126.23	104.2036667
B	135.02	122.87	151.45	136.4466667
C	18.67	110.82	66.312	65.26733333

Table A.3. 7 days results of elasticity modulus for W/C = 0.5 and C = 20%

	SAMPLE 3 - W/C=0.5 C=20% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	209.23	675.28	146,4	343.6366667
B	318.72	73.961	299.22	230.6336667
C	255.73	399.98	102.91	252.8733333

Table A.4. 28 days results of elasticity modulus for W/C = 0.5 and C = 25%

	SAMPLE 4 - W/C=0.5 C=25% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	628.46	659.39	413.38	567.0766667
B	587.03	435.41	362.68	461.7066667
C	952.02	183.33	228.23	454.5266667

Table A.5. 7 days results of elasticity modulus for W/C = 0.5 and C = 30%

	SAMPLE 5 - W/C=0.5 C=30% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	592.77	348.9	1053.9	665.19
B	643.08	462	490.92	532
C	695.56	339.91	588.57	541.3466667

Table A.6. 7 days results of elasticity modulus for W/C = 0.4 and C = 10%

	SAMPLE 6 - W/C=0,4 C=10% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	12.56	13.912	7.2402	11.2374
B	56.49	44.907	12.352	37.91633333
C	174.94	12.751	12.61	66.767

Table A.7. 7 days results of elasticity modulus for W/C = 0.4 and C = 15%

	SAMPLE 7 - W/C=0.4 C=15% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	156.52	38.101	332.41	123.5036667
B	189,42	46,037	539,76	258.4056667
C	110.625	77,453	91,831	93,303

Table A.8. 7 days results of elasticity modulus for W/C = 0.4 and C = 20%

	SAMPLE 8 - W/C=0.4 C=20% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	195.621	182.89	137.96	172.157
B	287,74	225.03	272.44	261.7366667
C	446,76	327.45	96.009	290.073

Table A.9. 7 days results of elasticity modulus for W/C = 0.4 and C = 25%

	SAMPLE 9 - W/C=0.4 C=25% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	492.9	431.35	43.492	322.5806667
B	19.911	636.8	152.52	269.7436667
C	383.91	667.5	64.763	372.0576667

Table A.10. 7 days results of elasticity modulus for W/C = 0.4 and C = 30%

	SAMPLE 10 - W/C=0.4 C=30% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	154.31	665.45	361.74	393.83
B	122.85	582.64	460.73	388.74
C	174.29	567.24	598.3	446.61

APPENDIX B

RESULTS OF ELASTICITY FOR 28 DAY

Table B.1. 28 days results of elasticity modulus for W/C = 0.4 and C = 10%

	SAMPLE 1 - W/C=0,4 C=10 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	135.64	156.81	19.627	23.719	20.74	28.61	64.191
B	72.691	93.701	29.02	26.674	9.5369	34.163	44.29765
C	63.373	38.261	21.696	34.363	28.976	21.225	34.649

Table B.2. 28 days results of elasticity modulus for W/C = 0.4 and C = 15%

	SAMPLE 2 - W/C=0.4 C=15 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	96.659	177.29	331	181.69	93.854	118.2	166.4488
B	144.96	170.36	415.08	1187.7	257.18	84.732	376.6687
C	20.252	6.582	157.19	138.93	53.573	41.164	69.61517

Table B.3. 28 days results of elasticity modulus for W/C = 0.4 and C = 20%

	SAMPLE 3 - W/C=0.4 C=20 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	424.93	777.64	601.11	993.12	410.24	386.27	598.885
B	454.83	774	1240.3	530.14	256.62	373.07	604.8267
C	228.23	340.84	520.04	111.82	163.64	157.86	253.7383

Table B.4. 28 days results of elasticity modulus for W/C = 0.4 and C = 25%

	SAMPLE 4 - W/C=0.4 C=25 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	645	737.83	804.39	941.58	502.32	561.66	698.7967
B	1294.3	912.46	530.13	276.51	640.83	2017.7	945.3217
C	1385	652.28	212.78	243.45	398.71	700.44	598.7767

Table B.5. 28 days results of elasticity modulus for W/C = 0.4 and C = 30%

	SAMPLE 5 - W/C=0.4 C=30 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	579.81	902.87	604.48	864.07	890.33	957.86	799.9033
B	819.74	1073.69	467.91	1011.4	423.68	1956.5	958.82
C	1455.6	757.59	374.13	892.28	459.64	665.02	767.3767

Table B.6. 28 days results of elasticity modulus for W/C = 0.5 and C = 10%

	SAMPLE 6 - W/C=0.5 C=10 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	24.408	30.71	37.68	32.69	22.83	27.537	29.30917
B	156.94	187.62	59.973	21.555	18.526	24.93	78.25733
C	11.449	92.439	15.457	91.339	19.387	21.623	41.949

Table B.7. 28 days results of elasticity modulus for W/C = 0.5 and C = 15%

	SAMPLE 7 - W/C=0.5 C=15 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	192.67	245.16	46.176	24.275	61.151	34.019	100.5752
B	220.09	123.49	108.97	127.03	204.83	149.58	155.665
C	113.93	86.489	52.139	70.888	80.743	158.91	93.84983

Table B.8. 28 days results of elasticity modulus for W/C = 0.5 and C = 20%

	SAMPLE 8 - W/C=0.5 C=20 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	145.26	261.95	362.37	238.94	635	894.93	423.075
B	404.99	208.65	160.2	344.91	76.591	188.2	230.5902
C	528.32	260.38	802.34	436.15	73.521	57.718	359.7382

Table B.9. 28 days results of elasticity modulus for W/C = 0.5 and C = 25%

	SAMPLE 9 - W/C=0.5 C=25 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	729.15	456.39	435.47	755.82	157.94	238.12	462.1483
B	887.65	380.05	290.58	492.15	419.6	107.06	429.515
C	1366.4	172.75	282.35	248.03	189.66	104.91	394.0167

Table B.10. 28 days results of elasticity modulus for W/C = 0.5 and C = 30%

	SAMPLE 10 - W/C=0.5 C=30 % 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	2273.9	530.65	614.55	628.12	421.19	408.6	812.835
B	425.39	784.43	1020.1	822.71	177.87	419.64	608.3567
C	483.48	568.95	332.55	380.86	258.46	516.72	423.5033

APPENDIX C

RESULTS OF STRESSES FOR 7 DAY

Table C.1. 7 days results of stress for W/C = 0.4 and C = 10%

	SAMPLE 1 - W/C=0.4 C=10% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	0.512843091	0.428564211	0.47219832	0.471201874
B	0.87892154	0.882812267	0.97295859	0.911564132
C	1.428150832	1.242920326	1.235271585	1.302114248

Table C.2. 7 days results of stress for W/C = 0.4 and C = 15%

	SAMPLE 2 - W/C=0.4 C=15% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	1.629143021	1.77064687	4.009331332	2.469707074
B	2.186302165	2.493012106	3.696000239	2.791771503
C	3.279143024	2.939984923	4.690590703	3.636572883

Table C.3. 7 days results of stress for W/C = 0.4and C = 20%

	SAMPLE 3 - W/C=0.4 C=20% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	5.420153591	15.86251299	6.513048562	9.265238382
B	7.429164024	10.62172464	10.34351394	9.464800869
C	10.52815403	12.96366053	7.352352385	10.28138898

Table C.4. 7 days results of stress for W/C = 0.4 and C = 25%

	SAMPLE 4 - W/C=0.4 C=25% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	17.53730223	12.12949677	8.836319148	12.83437271
B	19.6612249	9.199588187	10.30931951	13.05671087
C	17.05478211	11.30961857	12.06588809	13.47676292

Table C.5. 7 days results of stress for W/C = 0.4 and C = 30%

	SAMPLE 5 - W/C=0.4 C=30% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	15.81176377	14.00728288	15.14636505	14.98847057
B	12.4044444	12.7292835	10.17099203	11.76823998
C	11.11075131	15.84819025	18.97078548	15.30990901

Table C.6. 7 days results of stress for W/C = 0.5 and C = 10%

	SAMPLE 6 - W/C=0.5 C=10% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	0.623194302	0.298867147	0.124058061	0.348706503
B	0.529150351	0.95741612	0.407212701	0.631259724
C	1.080384591	0.588953016	0.60138228	0.756906629

Table C.7. 7 days results of stress for W/C = 0.5 and C = 15%

	SAMPLE 7 - W/C=0.5 C=15% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	1.623902318	0.806377757	7.285591581	3.238623886
B	3.730173461	1.826240264	10.83310223	5.463171984
C	2.815394023	1.909316533	10.27130266	4.998671072

Table C.8. 7 days results of stress for W/C = 0.5 and C = 20%

	SAMPLE 8 - W/C=0.5 C=20% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	6.749886679	4.217974073	3.699185818	4.889015523
B	10.43676876	6.672381653	4.664294596	7.257815002
C	5.152382491	8.856285674	6.859964412	6.956210859

Table C.9. 7 days results of stress for W/C = 0.5 and C = 25%

	SAMPLE 9 - W/C=0.5 C=25% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	12.47911133	9.896451669	0.772543741	7.71603558
B	3.587201958	9.686066984	2.239669356	5.170979433
C	5.939246972	13.24857305	1.728615345	6.972145123

Table C.10. 28 days results of stress for W/C = 0.5 and C = 30%

	SAMPLE 10 - W/C=0.5 C=30% 7 DAYS RESULTS			
	1. GROUP	2.GROUP	3.GROUP	AVERAGE
A	17.24971128	22.02594844	8.926543192	16.06740097
B	17.37181765	16.21545792	11.77497488	15.12075015
C	11.91769343	28.29460011	10.16613187	16.79280847

APPENDIX D

RESULTS OF STRESSES FOR 28 DAY

Table D.1. 28 days results of stress for W/C = 0.4 and C = 10%

	SAMPLE 1 - W/C=0.4 C=10% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	0.434203	0.927184	0.637207	1.024194	0.451943	1.59241	0.844524
B	3.462862	3.938463	1.39416	1.20765	0.829968	0.937211	1.961719
C	0.762962	1.457085	0.639626	0.956093	1.066043	1.422954	1.050794

Table D.2. 28 days results of stress for W/C = 0.4 and C = 15%

	SAMPLE 2 - W/C=0.4 C=15% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	6.067567	2.503717	0.468037	1.060133	1.353361	0.913518	2.061056
B	4.131189	3.818784	2.11533	1.459438	11.95993	4.595908	4.680097
C	4.801497	3.062365	1.996321	1.839522	13.77538	16.37214	6.974537

Table D.3. 28 days results of stress for W/C = 0.4 and C = 20%

	SAMPLE 3 - W/C=0.4 C=20% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	6.073206	10.4096	13.01482	10.31373	8.909624	8.300612	9.5036
B	17.35317	19.99849	16.63821	15.31866	10.61862	13.41004	15.5562
C	7.491941	10.40324	11.30866	6.708901	12.08023	10.79237	9.797558

Table D.4. 28 days results of stress for W/C = 0.4 and C = 25%

	SAMPLE 4 - W/C=0.4 C=25% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	23.1481	21.83986	11.23289	15.26478	14.32307	17.88128	17.28166
B	23.5484	19.65501	11.18591	13.96646	26.43463	21.85272	15.73432
C	16.7842	14.34617	12.74759	14.59475	16.71824	20.95373	16.02411

Table D.5. 28 days results of stress for W/C = 0.4 and C = 30%

	SAMPLE 5 - W/C=0.4 C=30% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	20.79664	8.018666	18.94141	16.29672	19.33614	20.86995	17.37659
B	20.62331	15.8242	11.84336	19.95653	16.86669	20.21609	17.55503
C	21.28644	16.61976	13.71419	16.17518	21.44133	21.4901	18.4545

Table D.6. 28 days results of stress for W/C = 0.5 and C = 10%

	SAMPLE 6 - W/C=0.5 C=10% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	0.434203	0.927184	0.637207	1.024194	0.451943	1.59241	0.844524
B	4.103212	1.429143	1.195216	0.823751	0.894282	1.634108	1.679952
C	0.762962	1.457085	0.639626	0.956093	1.066043	1.422954	1.050794

Table D.7. 28 days results of stress for W/C = 0.5 and C = 15%

	SAMPLE 7 - W/C=0.5 C=15% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	6.067567	2.503717	0.468037	1.060133	1.353361	0.913518	2.061056
B	13.69758	5.298427	3.102277	3.846762	15.48807	3.461932	7.482508
C	4.801497	3.062365	1.996321	1.839522	13.77538	16.37214	6.974537

Table D.8. 28 days results of stress for W/C = 0.5 and C = 20%

	SAMPLE 8 - W/C=0.5 C=20% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	3.806327	5.661559	8.723539	5.932231	20.35116	21.50716	10.997
B	11.64286	5.641917	7.827188	9.165394	4.081453	3.696	7.009136
C	7.608585	7.673599	12.40244	6.014779	2.249685	3.682869	6.605326

Table D.9. 28 days results of stress for W/C = 0.5 and C = 25%

	SAMPLE 9 - W/C=0.5 C=25% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	13.27422	45.62915	9.383301	11.90394	5.018712	5.362692	15.09534
B	6.936605	5.23315	14.58505	16.09734	2.977937	7.721499	8.925263
C	28.06897	9.476789	16.97542	18.57497	9.261669	7.436489	14.96572

Table D.10. 28 days results of stress for W/C = 0.5 and C = 30%

	SAMPLE 10 - W/C=0.5 C=30% 28 DAYS RESULTS						
	1. GROUP		2.GROUP		3.GROUP		AVERAGE
A	25.01462	16.52411	0.011278	21.82294	20.55416	15.69335	16.60341
B	29.27736	22.40758	24.16699	17.76504	15.24561	21.0212	21.6473
C	26.30785	30.40757	24.46832	20.62387	14.40449	22.04176	23.04231

APPENDIX E

MIX DESIGN PROPORTIONS IN 1 m^3

C (%)	V_C (m^3)	V_W (m^3)	V_{AG} (m^3)	V_{AIR} (m^3)	V_{total} (m^3)*	W_C (kg)	W_W (kg)	W_{AG} (kg)	W_{total} (kg)**	w/c
10	0,07978	0,09895	0,8027	0,018	1	247,32	98,95	2127,16	2473,43	0,4
15	0,117	0,145	0,720	0,017	1	362,7	145	1908	2415,7	
20	0,152	0,189	0,641	0,018	1	471,2	189	1698,65	2358,85	
25	0,1861	0,2307	0,5659	0,018	1	576,91	230,7	1499,63	2307,24	
30	0,2183	0,2708	0,4939	0,017	1	676,73	270,8	1308,84	2256,37	
10	0,0786	0,122	0,7814	0,018	1	243,7	122	2070,7	2436,40	0,5
15	0,1145	0,1771	0,691	0,017	1	353	177	1832	2362	
20	0,148	0,229	0,606	0,017	1	458,8	229	1605,9	2293,7	
25	0,180	0,278	0,526	0,017	1	558	278	1394	2230	
30	0,209	0,325	0,449	0,017	1	647,9	325	1189,85	2162,75	

*$V_{total} = V_C + V_W + V_{AG} + V_{AIR}$

**$W_{total} = W_C + W_W + W_{AG}$

V_C = Volume of cement

V_W = Volume of water

V_{AG} = Volume of aggregate

V_{AIR} = Volume of air

W_C = Weight of cement

W_W = Weight of water

W_{AG} = Weight of aggregate

APPENDIX F

INFORMATION ABOUT HOW STRESS (σ) VALUES WERE FOUND FROM THE UNCOMPRESSION TEST RESULTS OF THE TESTED SPECIMENS

The maximum load (MN) recorded by the universal testing machine was divided by the initial area of the specimen (m^2) to obtain stress values in (MN/m^2 = Mpa). For the sample area, area correction was applied, if applicable. That is if the sample has shown some bulge at the mid-height during breaking, which is when an average area was taken.

APPENDIX G

INFORMATION ABOUT HOW ELASTICITY MODULUS (E) VALUES WERE FOUND FROM THE UNCONFINED COMPRESSION TEST RESULTS OF THE TESTED SPECIMENS

After finding stresses (see Appendix F), Elasticity Modulus values were found as follows;

1. $E = \frac{\sigma}{\varepsilon}$

where;

$\varepsilon = \frac{\Delta L}{L}$ and

ε = strain,

ΔL = displacement amount (mm),

L = original height of specimens (mm)

2. From the plot of σ v.s. Є is obtained as an excel graph, which is represented by an equation in the form of y = ax+b, where the coefficient (a) donates the Elasticity Modulus value in MPa.