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Experimental Investigation of the Effect of the Ballast/Sleeper Interventions on Railway Track Performance

by:

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DEDICATED To MY

Beloved Parents, Sister and Brothers

Who make my life meaningful...

You will always hold

A special place in my heart...

ABSTRACT

Railways are integral to the transportation system of a growing economy. There is a constant demand by numerous stakeholders of the railway industry for greater train operational speeds, tonnage, passenger comfort and safety with minimal service disruptions. This is partly achievable through the improvement of the main components of railway track. A fundamental requirement for any railway track is to be able to maintain its as-built geometry over its service life. Settlement of a ballasted railway track is mainly caused by cyclic loading due to the passage of traffic. To return the rail track to the as-built alignment and level, track maintenance needs to be carried out. Tamping is one of the most common and traditional ways of maintaining the track and is not only costly but causes interruption to the day-to-day operations and damages the long term performance of the ballasted track. It therefore remains important to policy makers, rail practitioners and researchers to identify new techniques, innovations or processes that will prolong intervals between scheduled track maintenance. Track settlement is influenced by several interrelated track variables, not least of which are the railway sleeper, ballast and ballast-sleeper interface that have potential for alteration to be optimised with respect to the track performance.

In this research, results are presented from a parametric experimental study using three different apparatus namely box test (confined), Composite Element Test (CET) (semi-confined), and Railway Test Facility (RTF) (full-scale), to investigate performance of different sleeper and ballast configurations. The box test and the CET apparatus were used ahead of the RTF (full-scale) apparatus as preliminary tests to obtain an insight into the performance and potential benefits of different sleeper types with and without the use of Under Sleeper Pads (USPs). The aforementioned apparatus were also used for comparison with the full-scale apparatus to assess the consistency of the sleeper settlement results (phase 1 tests). The purpose of using the box test and CET apparatus was also to allow a relatively large number of simplified tests to be carried out on sleeper and ballast combinations (Phase 2 tests) within a reasonable cost and time frame. Phase 2 tests were designed and conducted with consideration of the phase 1 test results. The test results from phase 1 showed that increasing the number of contact points between ballast particles and sleeper, and increasing the coefficient of friction at ballast-sleeper interface improved the settlement performance of the tarckbed. Therefore, having this in mind, further tests were designed and conducted including: finer ballast grading, two-layered ballast systems, frictional sleepers and USPs, and USP types with various stiffnesses. In addition, further tests on ballast bonding/gluing and ballast reinforcement were conducted with the aim of investigating possible ways of minimising ballast bed deterioration. Application of ballast gluing was proven beneficial (in literature) in improving ballast bed performance, however, the overall cost of super glues used for this method was high. Therefore in this study, a low cost glue was introduced and its performance was tested under conditions similar to real track. With regard to ballast reinforcement, the expanded polystyrene was added to the ballast sample with the aim of decreasing the contact forces between ballast particles and consequently minimising ballast bed deterioration and settlement.

The observations in this study sought to establish qualitative and quantitative data on the effects of different parameters on the track performance, which may lead to a longer service life with less maintenance. The results showed that the type of sleeper has an influence on the performance of a railway track. With respect to settlement performance of a railway track, sleepers with higher depth and width, coefficient friction, and bending stiffness performed better. Out of all investigated sleeper types without any additional interventions, concrete

mono-block gave the least settlement and the plastic sleeper gave the most. It was also found that inclusion of USPs (regardless of type) can be beneficial compared to the ‘no USP’ scenario in: (1) Increasing trackbed resiliency. (2) Reducing maintenance works related to settlement. (3) Reducing sleeper deflections (hogging) at the middle of sleeper. (4) Reducing the rate of deterioration of sleeper support at the rail seat (6) Reducing differences in the performance of different sleeper types. (7) Reducing tendency towards centre binding. (8) Reducing transmitted pressure on subgrade. (9) Making the trackbed stiffness more uniform. The conclusion of research was that use of concrete mono-block sleeper, USPs, finer ballast grading, a two-layered ballast system with finer ballast grading on top, frictional sleeper, frictional USP, and ballast gluing all have potential to improve the performance of ballasted railway track and reduce the maintenance requirements.

Keywords: Ballasted Railway Track, Performance Improvement, Settlement, Railway Sleeper, Ballast, Ballast-Sleeper Interface, Under Sleeper Pads (USPs).

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AUTHOR'S STATEMENT

The work presented and described in this PhD thesis was conducted at the University of Nottingham, Department of Civil Engineering, Nottingham Centre for Geomechanics (NCG), between September 2012 and September 2016. I declare that the work is my own and has not been submitted for a degree of another university.

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LIST OF ABBREVIATIONS

ADIF	Administrador De Infraestructuras Ferroviarias (Spanish Railways)
AREA	American Railway Engineering Association
AREMA	American Railway Engineering And Maintenance-Of-Way Association
BDMS	Square Bright Drawn Mild Steel
BSI	British Standards Institution
CET	Composite Element Test
CWR	Continuously Welded Rail
DB	Deutsche Bahn
DECC	Department of Energy and Climate Change
DEFRA	Department For Environment, Food And Rural Affairs
DEM	Discrete Element Modelling
DSB	Danske Statsbaner (Danish State Railways)
DSSS	Dynamic Sleeper Support Stiffness
EMGTPA	Equivalent Million Gross Tonnes Per Annum
EVA	Ethylene Vinyl Acetate
FWD	Falling Weight Deflectometer
GC	Gauge Capacity
HGVs	Heavy Goods Vehicles
kg	Kilogram
kN	Kilo Newton
kPa	Kilopascal
LU	London Underground
LVDT	Linear Variable Differential Transformer
MAV	Magyar Államvasutak (Hungarian State Railways)
mm	Millimetre
MOW	Maintenance-Of-Way
NCG	Nottingham Centre For Geomechanics
NR	Network Rail
NSB	Norges Statsbaner (Norwegian State Railways)
NTS	National Travel Surve

ÖBB	Österreichische Bundesbahnen (Austrian Federal Railways)
PBI	Pneumatic Ballast Injection
PSD	Particle Size Distribution
RENFE	Red Nacional De Los Ferrocarriles Españoles (Spanish Railways)
RSSB	Rail Safety And Standards Board
RT	Railway Track
RTF	Railway Test Facility
SBB	Schweizerische Bundesbahnen (Swiss Railways)
SNCF	Société Nationale Des Chemins De Fer (French Railways)
SP	Specification
TRK	Track
UBM	Under Ballast Mats
UIC	Union Internationale Des Chemins De Fer (International Union Of Railways)
USP	Under Sleeper Pads
WIP	Wished-In-Place

LIST OF SYMBOLS

$\widetilde{h}$	Thickness Of Ballast
q_{peak}	Maximum Deviator Stress
ε_a	Axial Strain
ε_v	Volumetric Strain
δ_{max}	Midspan Deflection Of The Sleeper
σ'_1	Major Principal Stress
σ'_3	Minor Principal Stress
ϕ_P	Peak Friction Angle
Δt	Time Shift
A_{sb}	Sleeper Bearing Area
b	Sleeper Width
C_u	Undrained Shear Strength
D	Specimen Diameter
d_{max}	Maximum Particle Size
EI	Bending Stiffness
f	Frequency
h	Depth of Ballast Below Sleeper
Mr	Resilient Modulus
p'	Mean Effective Stress
P_c	Maximum Intensity Of Pressure On Subgrade
P_m	Intensity of Pressure on Sleeper-Ballas
q	Static Rail Seat Load
q	Static Rail Seat Load
Q	Static Wheel Load For A Third Of The Sleeper
r	Radius of a Uniformly Loaded Circle Whose Area Equals The Effective Sleeper Bearing Area
V	Speed
φ°	Phase Angle
L	Sleeper Length
$\varepsilon_{1,r}$	Recoverable (Resilient) Axial Strain During Unloading

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- BAGHSORKHI, M. S., LARYEA, S., MCDOWELL, G. & THOM, N. (2015). An investigation of railway sleeper sections and under sleeper pads using a box test apparatus. Proceedings of the Institution of Mechanical Engineers, *Part F: Journal of Rail and Rapid Transit*, 0954409715613818.

1. INTRODUCTION

1.1 BACKGROUND OF STUDY

There are different modes of transport for moving passengers and goods such as road, rail, and air. Railways form an important part of the transportation system, supporting a growing economy. The approach towards using a greener, safer, and more efficient mode of transport during the last couple of decades has resulted in an increase in use of railways for both passenger and goods transportation. With regard to passenger travel, according to the National Travel Survey (NTS, 2015), the number of rail trips per person in England increased by 71% in 2014 compared to those in 1995/6 (Figure 1-1). Where rail freight is concerned, the volume of rail freight has increased by 60% since 1995 and the industry forecasts suggest that freight demand will grow by 30% over the next decade; this is the equivalent of an additional 200 freight trains per day (Network Rail, 2013).

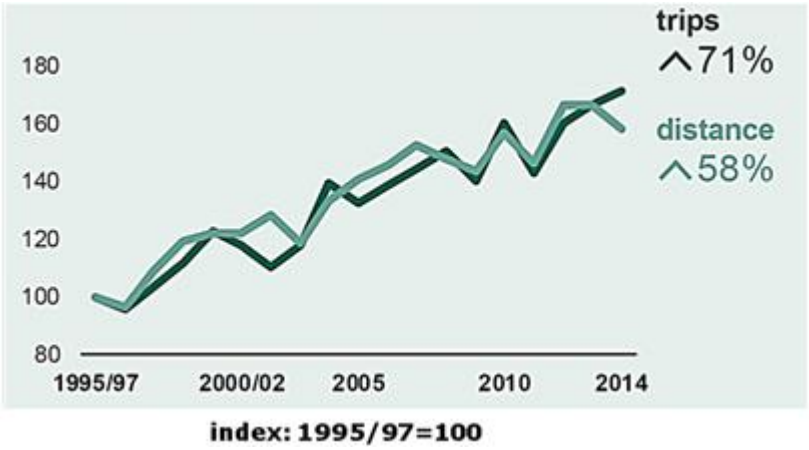


Figure 1-1 Trends in trips and distance in England from 1995/97 to 2014 (NTS, 2015)

Despite the fact that use of rail as a means of transport has increased, rail is still considered as the safest mode of transport. The statistics by the Rail Safety and Standards Board (RSSB, 2016) showed that there were no passenger or staff casualties in train accidents for the ninth consecutive year in 2015/2016.

Rail freight also makes the roads safer to drive on by reducing the number of lorries and congestion, and consequently the number of accidents on roads. According to Network Rail (2013), one freight train can remove between 43 and 77 Heavy Goods Vehicles (HGVs) from roads depending on the type of freight (Table 1-1).

Table 1-1 Potential for a fully loaded freight train to remove lorries (Network Rail, 2013)

Commodity	Fully loaded train potential (tonnes)	Equivalent number of Heavy Goods Vehicles
Coal	1,500	52
Metal and ore	1,000 to 2,500	60
Construction materials	1,500 to 3,000	77
Oil and petroleum	2,000	69
Consumer goods	600 to 1,100	43
Other traffic	1,000 to 1,500	43

Furthermore, use of rail freight is more fuel efficient compared to road freight; a gallon of fuel on average can move a tonne of goods 246 miles on the railway and 88 miles by road (Network Rail, 2013). This means use of rail freight is approximately 2.8 times more fuel efficient than road freight and therefore it is also more environmentally friendly than road freight by reducing pollution. As reported by the Department for Environment, Food and Rural Affairs and the Department of Energy and Climate Change (Defra / DECC, 2013), rail freight produces 76% less carbon dioxide than road freight. Therefore, wherever rail tracks exist, passengers and goods can be transferred by train to any location and over any distance safely, efficiently and with the least impact on the environment.

A ballasted rail track as shown in Figure 1-2 from top to bottom consists of superstructure: rails, rail fastenings, rail pads, sleepers, and substructure: ballast, sub-ballast (with or without), and subgrade. A fundamental requirement for any railway track is to be able to maintain its as-built geometry over its service life with minimal interruption of day to day operations for maintenance. Of the substructure layers, ballast has the largest contribution to rail trackbed deterioration (Selig and Waters, 1994). As shown in Figure 1-3 with the assumption of a good subgrade soil, settlement of ballast compared to subgrade and sub-ballast is more significant. The ballast settlement is caused by the passage of traffic and to return the rail track to the as-built alignment and level, track maintenance needs to be carried out. One of the most common and traditional ways of maintaining the track is through tamping. Tamping is the process of lifting the track and packing the ballast underneath the sleeper to fill the gap generated. Apart from the required line possession time and consequent interruption caused to the traffic by the tamping process, it also causes ballast particle breakage (Selig and Waters, 1994) and consequent losses of ballast performance with time. Moreover the tamping process disturbs the arrangement of ballast formed by the passage of traffic. Therefore optimisation of ballasted trackbed to obtain best performance and least maintenance is important. Best performance can be defined as least vertical settlement of trackbed while maintaining the rail track level.

Trackbed is a term defining the layer between the sleeper and subgrade and the key function of it is to provide stability and to reduce the pressure resulting from passage of a train to a bearable level for the natural ground without causing an excessive settlement or ground failure.

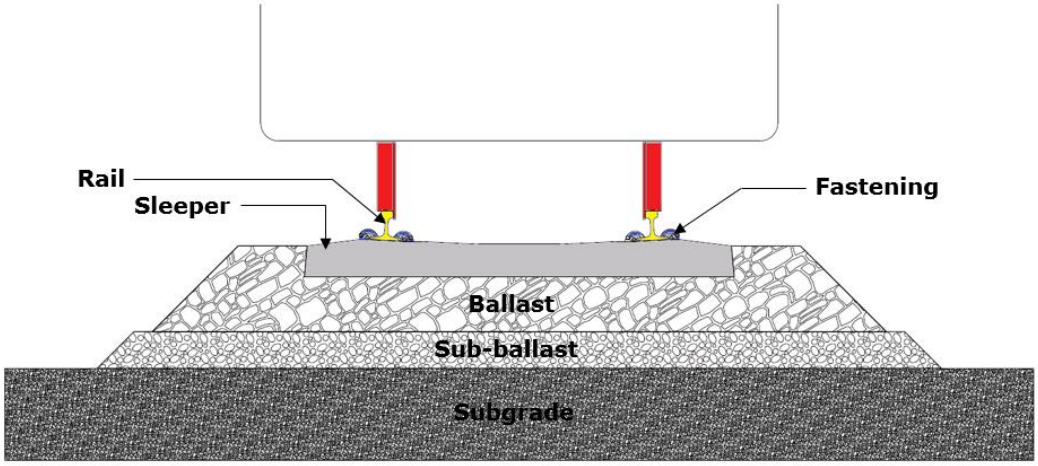


Figure 1-2 Ballasted rail track cross section

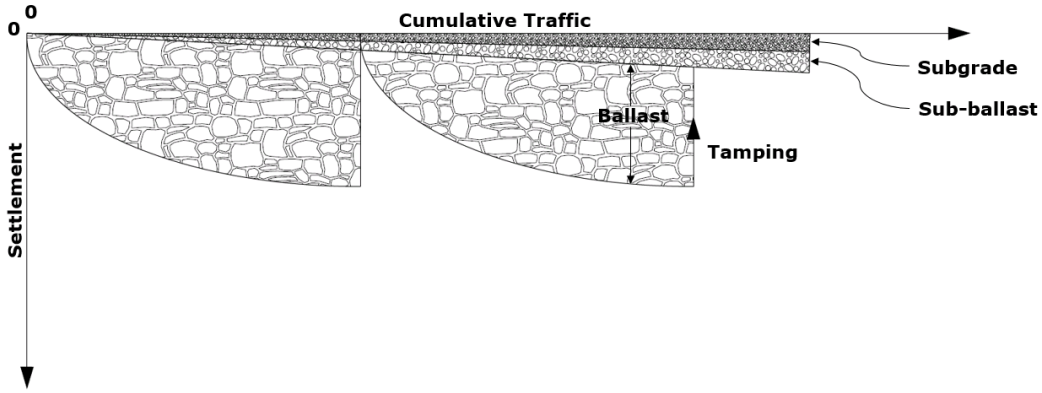


Figure 1-3 Contributions of superstructure to settlement adapted by author from (Selig and Waters, 1994)

1.2 PROBLEM STATEMENT AND KNOWLEDGE GAP

Out of ballasted track components, sleeper and ballast have the potential for alteration to be optimised in respect of the track settlement. There are different types of sleeper being used around the world; however, as a result of lack of performance data, there is no clarity for the selection of railway sleepers and over which sleeper performs optimally on track. Where there has been a comparison of the performances of different sleeper types, the studies have either had measurement shortcomings or have not been extensive (Selig and Waters, 1994). The selection of sleeper types has been guided by financial considerations (Zarembski, 1993,

Profillidis, 2000), logistical constraints (Qiao et al., 1998, Sonti et al., 1995) and experience (or rule of thumb) (Abbott, 1989) rather than research-derived evidence. Zarembski (1993) proposed strength (or survivability in a railway environment) and economic criteria as the only two basic sleeper acceptance criteria. Fundamentally, consideration to sleeper interaction with various track components has not been acknowledged. However, there has been a recent change in focus on investigating mechanical performance of railway sleepers among some studies (Remennikov and Kaewunruen, 2007, Sadeghi and Barati, 2010) which have made a valuable contribution towards sleeper selection. However, little has been observed with regard to assessment of settlement performance and optimisation of sleeper types using full-scale testing close to real track conditions.

Use of Under Sleeper Pads (USP) has relatively recently been introduced as an intervening elastic element into the track structure affixed to the underside of railway sleepers to extend the lifespan of a railway track. There are various types of USPs with respect to thickness and stiffness in response to different design requirements. The mechanical properties of USPs are obtained using standard tests. However, in real track conditions a USP may not work as optimally as it should, which is mainly as a result of ballast contact not being accurately modelled in standard tests. Therefore, it is important to identify the optimal stiffness of a USP under similar conditions representative of real track with respect to ballast contact. Moreover, research projects on USPs (Johansson et al., 2008b, Witt, 2008, Loy, 2009, Dahlberg, 2010, Lakušić et al., 2010a, Schneider et al., 2011, Insa et al., 2011, Insa et al., 2014, Alves Ribeiro et al., 2015) have been mainly numerical modelling and some experiments focused on dynamics of train-track interaction related to noise and vibration mitigation and vertical stiffness changes at transition zones and turnouts. However, little experimental study has been documented on the influence of USPs on vertical settlement performance of trackbed using different sleeper types and full-scale testing close to real track conditions.

In trackbed design, measurement of maximum stresses at the formation level of railway tracks due to transient cyclic loads is critical. Several approximation methods have been recommended for the calculation, such as equations recommended by the American Railway Engineering and Maintenance-of-Way Association (AREMA) (AREMA, 2010): the Japanese National Railway equation, the Talbot equation, the Boussinesq equation, and the Love's formula. Although the aforementioned methods are useful to approximate the maximum stresses at formation level, the effect of different sleeper types or USP inclusion on the stresses can not be measured by

them. Apart from the above mentioned calculation methods, there have also been experimental studies to measure stresses at the formation level using full-scale testing (Kwan, 2006, Brown et al., 2007a, Aursudkij, 2007). Kwan (2006) showed the effect of inclusion of geogrid reinforcement on stress level and the effect of ballast tamping on the stress level was studied by Brown et al. (2007a) and Aursudkij (2007). The sleeper used in all the aforementioned studies was concrete mono-block. Therefore, understanding of the effect of using different sleeper types and USPs on foundation maximum stress together with their effect on settlement performance of trackbed can add to previous studies.

The lateral resistance of a sleeper in a ballasted track comes from contact with ballast at the base, shoulder, and crib; however, in the case of a mono-block concrete sleeper, it is predominantly from the frictional forces between the ballast particles and the sleeper underside (Zakeri, 2012, Zakeri and Bakhtiary, 2014). Several studies have been conducted on methods of evaluating and measuring the lateral resistance of sleepers in ballasted track (Koike et al., 2014, Le Pen et al., 2014, Zakeri and Bakhtiary, 2014, Zakeri, 2012). There are also studies that have recommended use of concrete lateral resistance sleepers (frictional sleepers) and compared them with standard concrete sleepers in terms of resistance to lateral loads (Zakeri et al., 2012, Zakeri and Fakhari, 2014, Montalbán Domingo et al., 2014). All aforementioned studies on frictional sleepers were focused on assessment of their lateral resistance, however, the influence of using frictional concrete sleepers on the settlement performance and trackbed stiffness of ballasted trackbed was not acknowledged; the same applies concerning use of a frictional USP.

Over the years, different ballast sizes have been used and the choice of size of ballast has been based on experience. According to a historical study by Claisse and Calla (2006) on development of ballast specifications from the early 1900s, a change to ballast of smaller size is recommended for the current ballast specifications. It is therefore valuable to test ballast grading with smaller particles and compare to the current ballast grading being used in the industry under conditions close to real track without compromising on acceptable strength and drainage of the ballast.

One of the ways of stabilising railway trackbed is ballast bonding/gluing, which was approved by Network Rail (NR) as a way of improving the lateral resistance of track and a standard was published for it by NR in 1996 (NR/SP/TRK/127). The benefits and methods for implementation of ballast gluing were identified in a study by Lakušić et al. (2010b). Although application of ballast gluing is considered beneficial for ballasted track in terms of reducing the

frequency of maintenance works, the overall cost of super glues used for this method is high. It would therefore be valuable to identify a way of producing a low cost glue and testing its performance under conditions similar to real track.

Stone-blowing maintenance is done by lifting the track and blowing smaller ballast particles below the sleeper in order to level the track. Railway ballast as a result of stone-blowing track maintenance can develop two layers. Previous studies by (Anderson and Key, 2000, Claisse et al., 2003, Claisse and Calla, 2006) investigated the effect of using smaller grade ballast as crib to act as a reservoir to fill the voids under the sleeper resulting from loading cycles. The aforementioned studies were conducted using small triaxial apparatus. This can be extended by modification of the two-layered ballast using apparatus simulating conditions closer to the real track compared to a small triaxial apparatus.

To minimise ballasted track settlement and consequently to increase its service life, several innovative solutions have been applied in recent decades to reinforce the ballast particles. Application of elastic elements between ballast particles was one example of these solutions with the purpose of improving ballast performance and durability. Studies by Keene et al. (2012) and Kennedy et al. (2013) showed that use of polymers as an elastic element can improve the elasticity and resistance to plastic deformation of the ballast layers. Application of crumb rubber made from used tyres as an elastic element, was also studied by Sol-Sánchez et al. (2015). Hence, it is useful to identify other alternative materials, especially those that do not bio-degrade for years, and test them as elastic elements to improve the ballasted track settlement.

For the purpose of this research, three different apparatus were used, namely box test (confined), Composite Element Test (CET) (semi-confined), and Railway Test Facility (RTF) (full-scale). The box test and the CET apparatus were used ahead of the RTF (full-scale) apparatus as preliminary tests to obtain an insight into the performance and potential benefits of different sleeper types with and without the use of Under Sleeper Pads (USPs). The aforementioned apparatus were also used for comparison with the full-scale apparatus to assess the consistency of the sleeper settlement results. The purpose of using the box test and CET apparatus was also to allow a relatively large number of simplified tests to be carried out with regard to sleeper and ballast alterations within a reasonable cost and time frame. The observations in this study sought to establish qualitative and quantitative data on the effects of

different parameters on the track performance, which may provide a longer service life with less maintenance.

1.3 AIM AND OBJECTIVES OF THE PROJECT

The overall aim of the study is to develop an improved understanding of the impact of sleeper, ballast and sleeper/ballast interface modifications on the performance of conventional ballasted track, particularly:

- The effect of sleeper and sleeper/ballast interface types.
- The effect of ballast grading, layering system, bonding and reinforcement.

To achieve the aim of this study, the following objectives have been set in two phases:
Phase 1, all tests in this phase were conducted using all three apparatus, first the box and CET and then the RTF to investigate the effect of:

- Sleeper Type:
  - Using box and CET, by testing concrete, timber, plastic, hollow rectangular, and U sections using the current standard NR ballast grading.
  - Using RTF, by testing concrete mono-block, concrete twin-block, timber, plastic, and steel sleepers using the current standard NR ballast grading.

- USPs:
  - Using box and CET, by testing two different types of USP on all sleeper sections except U section.
  - Using RTF, by testing two different types of USP on all sleeper types except steel sleeper.

Phase 2, the tests in this phase were carried out using one or both of the box and CET apparatus to investigate the effect of:

- Ballast grading (box & CET):
  - By testing the strength of the modified current standard NR ballast grading using smaller ballast grading (2/3 scale) by conducting monotonic triaxial tests with a large-scale triaxial apparatus.
  - By testing the 2/3 scale ballast on concrete, timber, and plastic sections with and without two different USP types.
- USP stiffness (box & CET):

- ❑ By testing USPs with a greater range of stiffness made from extra layer of USPs employing a concrete sleeper section and the current standard NR ballast grading.

➢ Two-layered ballast system (box):

- ❑ By testing two different systems; sample 1 made from the current standard NR ballast grading on top and 2/3 scale ballast at the base, Sample 2 made from 2/3 scale ballast on top and the current standard NR ballast grading at the base.

➢ Elastic elements inclusion (box)

- ❑ By testing inclusion of end-of-life protective packaging polystyrene in two different sizes separately with different inclusion percentages.

➢ Frictional Sleeper and frictional USP (box):

- ❑ By testing a frictional concrete sleeper section and frictional USP made by modifying the underside of a concrete sleeper section and USP respectively.

➢ Ballast bonding/gluing (CET)

- ❑ By testing a low cost glue made from end-of-life protective packaging polystyrene and pure d-Limonene on the current standard NR ballast grading

For all the tests conducted, the main comparison was made on the basis of:

- Settlement of sleepers
- Stiffness of trackbed,
- Coefficient of friction at ballast-sleeper interface

In addition to the aforementioned factors the following were also used as the basis of comparison for the full-scale tests:

- Pressure on trackbed subgrade (foundation)
- Deflections of sleepers
- Bending stiffness of sleepers

1.4 THESIS OUTLINE

Following the introduction presented in this chapter, the thesis consists of six more chapters as follows:

Chapter 2 presents descriptions of track components and their functions. It also contains a literature review on ballast and sleeper types and their influence on track performance as well as ballast-sleeper and ballast-subgrade interfaces. Moreover, it includes a review of literature on USPs.

Chapter 3 describes the key features of the large-scale triaxial test and the details of the experiments on two ballast samples with different gradings. The results of the test are also presented and discussed.

Chapter 4 describes the key features of the box test apparatus together with descriptions of the materials, test procedures, and the instrumentation relevant to the tests conducted. The details of preliminary tests (phase 1) and additional tests (phase 2) are also presented. In phase 1 tests, the effect of different representative sleeper section types with and without USPs was studied using the current standard NR ballast grading. Phase 2 was conducted once the consistency of the phase 1 test results were confirmed by comparing the box and CET tests (in phase 1) with those of the RTF tests. Phase 2 involved investigations into the effect of frictional sleepers and USPs, ballast reinforcement using polystyrene particles, representative sleeper section types and USPs using 2/3 scale ballast, two-layered ballast systems, and USP types with various stiffnesses. The results of all phase 1 and phase 2 tests are also presented and discussed.

Chapter 5 contains the details of key features of the Composite Element Test (CET) apparatus as an intermediate test (semi-confined) between the box test (confined) and the Railway Test Facility (full-scale). The CET test was used as an alternative way to validate and check repeatability of the box tests results in a condition with reduced degree of ballast confinement. It was also used to conduct experiments that were difficult to perform using the box test apparatus such as gluing ballast. Similar to the box tests, the CET tests were conducted in two phases. Phase 1 tests were similar to those of the box tests. Phase 2 involved use of ballast gluing, sleeper section types and USPs using 2/3 scale ballast, and use of USP types with various stiffnesses. The results of CET and box tests are also compared and discussed.

Chapter 6 describes the relevant aspects of the Railway Test Facility (RTF) together with recent modifications made to the original RTF apparatus and detailed descriptions of the materials, sample preparation, test procedures, and the instrumentation relevant to the tests undertaken. The RTF was used for full-scale experiments paralleling the tests conducted in phase 1 using the box and the CET apparatus. An analysis of the test results and a discussion of the findings are also presented in this chapter. The results from steel sleeper tests were presented separately due to the fact that it was not possible to affix a USP to the base of the sleeper and different methods of installation were used for it compared to the rest of sleepers.

Chapter 7 presents the conclusions derived from the completed test results, the suggested application of findings in practice, and the recommendations for future research.

2. LITERATURE REVIEW

2.1 INTRODUCTION

The focus of the literature review presented in this chapter is on railway ballast and sleeper behaviour as well as track improvement using Under Sleeper Pads (USPs). To achieve this, an overview of the main track system components and their functions are initially presented followed by discussions on track forces and distribution mechanism, and methods of stress measurement at sleeper-ballast and ballast-subgrade interfaces. Next, review of the literature on the following relevant topics is presented: track settlement and maintenance, ballast behaviour, railway sleeper types, and improvement of rail track performance with use of Under Sleeper Pads (USPs). Following this, track categories and design in the UK are discussed as well as Dynamic Sleeper Support stiffness as a trackbed stiffness measurement.

2.2 TRACK SYSTEM COMPONENTS AND FUNCTIONS

There are mainly two forms of railway track used worldwide, ballasted and ballastless track. Ballasted track is the predominant type and is the focus of this thesis.

Ballasted track as a form of track has advantages and disadvantages. According to Esveld (2001) and Indraratna et al. (2011b), the main advantages of ballasted track are as follows:

- Proven technology,
- Relatively low construction costs and use of local material,
- Relatively easy design and construction,
- Simple replacement of track components,
- Relatively simple maintenance works and correction to track geometry,
- Possibility of small adjustments to track lay-out (curves),
- Good drainage properties and high hydraulic conductivity of track structure,
- Good elasticity, and
- Good damping of noise.

On the other hand, ballasted track has some disadvantages (Esveld, 2001) (Indraratna et al., 2011b):

- Degradation and fouling of ballast, and tendency of the track to "float" in both longitudinal and lateral direction after a period of time, requiring frequent track maintenance and routine checks,
- Disruption of traffic during maintenance operations,
- Limited lateral resistance,
- Reduction in drainage properties and hydraulic conductivity due to clogging of voids by crushed particles and infiltrated fines from the subgrade,
- Emission of dust from ballast resulting from passage of high speed trains, and
- Substructure becomes relatively thicker and heavier, which requires stronger bridge and viaduct construction.

The components of conventional ballasted track fall into two main categories, the railway superstructure and the substructure. The superstructure is the top part of the ballasted track and includes the rails, the fastening system, and the sleepers. The substructure is the section below the sleepers and consists of the ballast, sub-ballast and the subgrade. The superstructure and substructure connect at the sleeper-ballast interface (Selig and Waters, 1994). A schematic of a conventional ballasted track, highlighting relevant track components is shown in Figure 2-1.

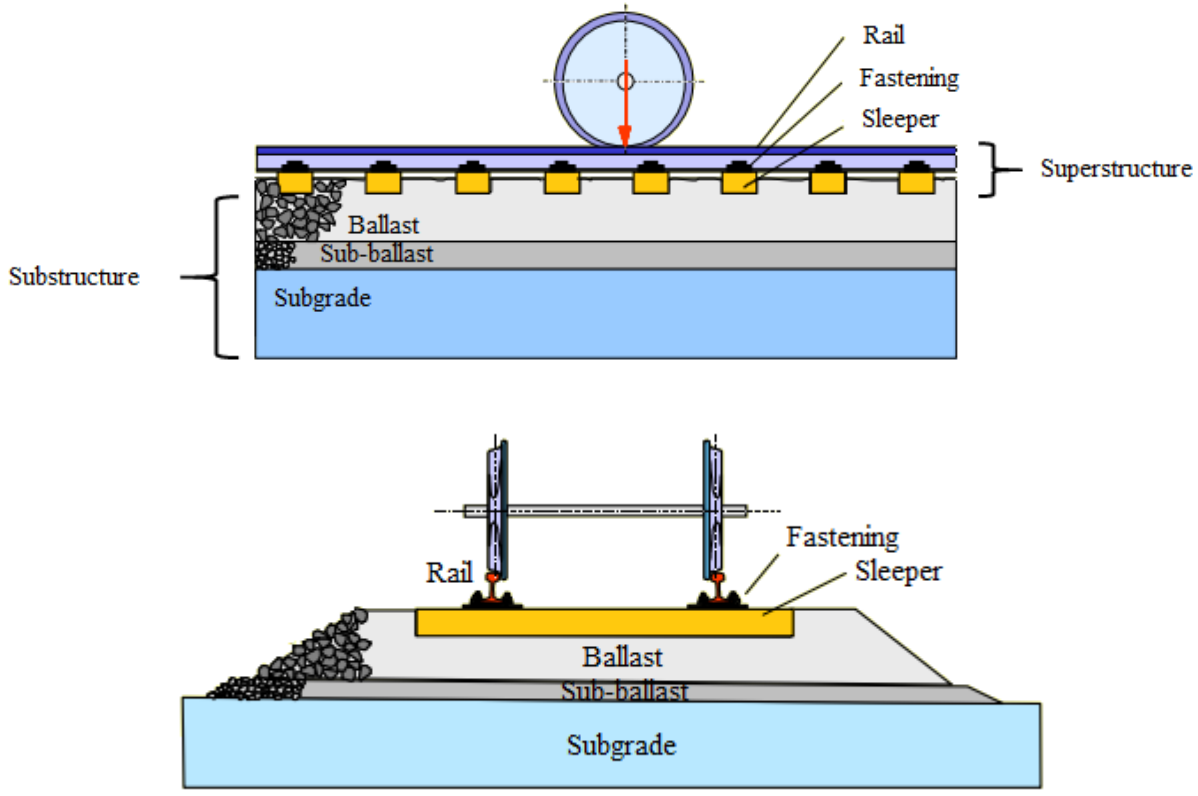


Figure 2-1 Layout of a typical ballasted track adapted by author from Esveld (2001)

It should be also noted that UIC719R (UIC, 1994) regards ballast as superstructure, because it needs regular maintenance and has to be replaced when its life expires.

Increasing consumer demands, driven largely by economic factors on current railway systems require the delivery of a safe, economical and reliable service. To achieve this goal from an engineering perspective a railway track needs to function as a stable guideway, ensuring appropriate vertical and horizontal alignment throughout the rail network. In addition, all the track system components are required to resist the mechanical and dynamic forces typically caused by the traffic loads and environmental factors. It is therefore important to understand the function of track components individually as well as their performance together as an entity. The components of conventional track structures and their functions will be described from the top down as follows.

2.2.1 RAILS

Rails can be described as the longitudinal steel members that remain in direct contact with the wheels, thereby providing even and continuous support during the passage of a train. Rails are the foremost recipients of any form of vehicular load imposed on the track structure and subsequently transferred to the underlying track elements. Deficiencies in the surface profile of rails and indeed in the train wheels in contact with them have the potential to magnify already large dynamic train loads and have implications for the integrity of the track structure. In their role within the track structure, rails need to be stiff and effectively function as beams to ensure minimum deflection between sleeper supports.

Modern rails are typically in the form of the asymmetrical I-section shown in Figure 2-2.

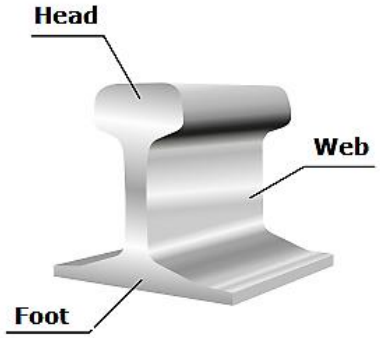


Figure 2-2 Typical modern rail cross section adapted by author from (source: https://commons.wikimedia.org)

For ease of installation and to prevent buckling (particularly in hot climates), rails are usually joined together by welds (Figure 2-3) or traditional bolted joints called “fishplate” connections (Figure 2-4). However, it is generally acknowledged that joints are potential sites for rail defects and discontinuities at which locations consequential large impact loads can be detrimental for track components below. Peak stresses at joints have been cited as affecting passenger comfort, accelerating sleeper wear, increasing ballast deterioration and even causing rail breakages leading to derailments (Selig and Waters, 1994). Therefore, Continuously Welded Rail (CWR) has become the preferred choice in the majority of modern railways wherein the rails are welded together to form a long continuous rail. CWR is robust, requires less maintenance, and delivers a smooth ride on a surface with less friction as compared to rails with fishplate connections (Esveld, 2001).



Figure 2-3 Welded joint connection



Figure 2-4 Fishplate connection adapted by author from (source: https://en.wikipedia.org)

2.2.2 FASTENING SYSTEM

A good fastening system provides consistent fixity of a rail to a sleeper and resists any vertical, lateral, longitudinal and overturning movements. Additionally, it helps to dampen traffic vibrations and diminish rail/sleeper abrasion. Fastening systems vary depending on the type of the sleeper and geometry of the rail. For wooden sleepers, steel plates are used to distribute the rail force over the wood surface. Rail pads (Figure 2-5) are used in fastening systems for concrete sleepers to provide resiliency and damping for the superstructure. In this system, the rail sits on a rail pad and together they are fastened to the sleeper using a clip. According to Fermér and Nielsen (1995), the pad stiffness has a considerable influence on the routine quality of ride, track service life and intervals between maintenance interventions.

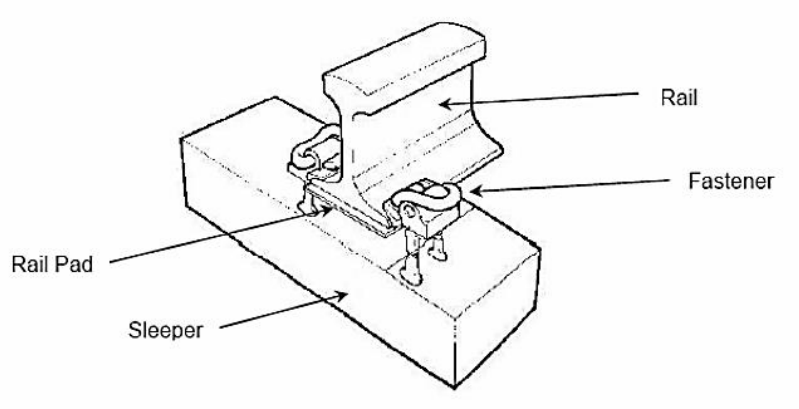


Figure 2-5 Fastening system for concrete sleepers (Steffens, 2005)

2.2.3 SLEEPER

Sleepers are the components of the track structure placed between rails and ballast. A sleeper functions mainly to disperse loads transferred through the rail to the underlying ballast and to restrain rail movement by anchoring the track superstructure to the substructure. Sleepers also support the fastening system to maintain track gauge under trafficking and fluctuating environmental conditions. They are typically manufactured from a variety of materials and come in different geometries. Choice of material type or geometry usually depends on the end use of the track. This is considered in further detail in a later section.

2.2.4 BALLAST

Railway ballast is the selected crushed granular material positioned as the top layer of the substructure; it is also placed between the sleepers (cribs), and at the ends of the sleepers (shoulders), extending down to the bottom of the ballast layer (Figure 2-6).

Good ballast material traditionally consists of angular, crushed, hard stones and rocks, uniformly graded, free of dust and dirt, not prone to cementing action. However, there is no universal agreement with regard to the best specification for proper ballast material. As a result, specifications vary geographically, and availability, experience, and economic considerations continue to be the main drivers in the selection of track ballast. Literature records the use of a wide range of ballast materials such as granite, basalt, limestone, slag and gravel. However, igneous or metamorphic rocks due to their angular shape and relatively uniform grading and strength are reported as the preferred choice (Boucher and Selig, 1987, Klassen et al., 1987, Raymond, 1985, Watters et al., 1987). In spite of geographic disparities, angular stones are generally preferable to naturally rounded ones as they interlock better with each other thereby, restraining track movement.

In the UK railway ballast is required to be well graded with a predominant particle size ranging between 31.5-mm and 50-mm in diameter as shown in Table 2-1 (Network Rail, 2009).

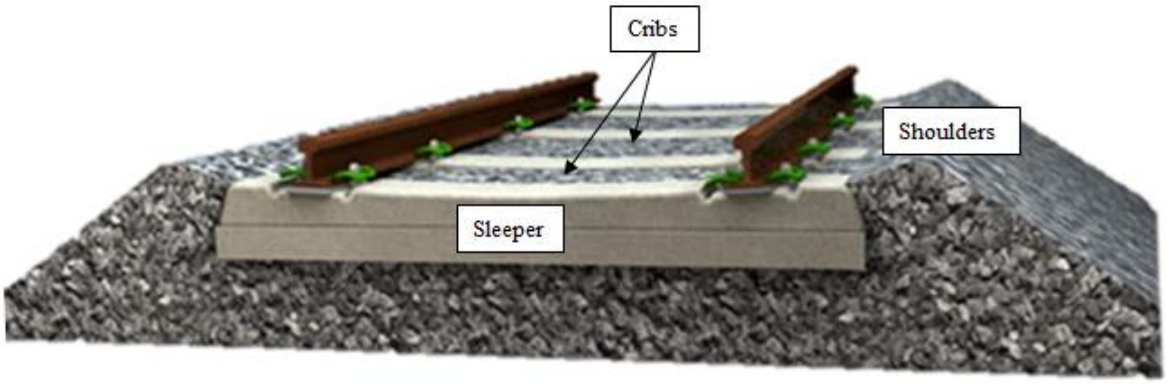


Figure 2-6 Ballast layer – cribs and shoulders adapted by author from (source: http://www.railsystem.net/)

Table 2-1 Ballast Particle Size Distribution (Network Rail, 2009)

Sieve size (mm)	Percentage passing by mass
80mm	100%
63mm	100%
50mm	70 to 99%
40mm	30 to 65%
31.5mm	1to 25%
22.4mm	0 - 3%
31.5mm to 50mm	≥ 50%

The functions of railway ballast are widely documented in the literature (e.g. (Hay, 1953, Doyle, 1980, Jeffs and Marich, 1987, Jeffs, 1989, Jeffs and Tew, 1991a, Selig and Waters, 1994, Profillidis, 1995b, Indraratna et al., 1998, Ionescu et al., 1998, Schmutz, 2000)). It is generally

agreed that the main function of ballast is to reduce/spread high stresses occurring at the sleeper-ballast interface via load transfer to a level that can be resisted by the subgrade on a long term basis. It also transfers the load evenly into subgrade and allows good drainage of water.

The depth of ballast required for load spread is usually determined by the expected level of stress/traffic or the strength of the underlying subgrade. For example, weaker subgrades or significantly larger stresses would warrant a commensurately thicker ballast layer.

Railway ballast, when described, is often categorised by its location in the track structure. The role of crib ballast is to provide pressure on the ballast below and at contact with the sleepers to retain track position by resisting vertical, lateral and longitudinal forces applied to the sleepers. The purpose of the shoulder ballast is to protect the track from buckling due to temperature induced rail loads, with or without trains present. Large voids are also required in the ballast for storage of fouling materials and drainage of water falling onto the track. Furthermore, ballast needs to have the capability to rearrange during level correction and re-alignment operations.

More details on ballast fouling, maintenance, and mechanical behaviour of ballast material under repeated loading will be discussed in a later section.

2.2.5 SUB-BALLAST

Ballasted track can be built with or without sub-ballast and it is positioned between the ballast and the subgrade materials (Figure 2-1). The sub-ballast generally consists of a well-specified material that does not degrade when it is placed directly below a ballast layer and it consists of normally finer materials compared to ballast. The most suitable and common materials for sub-ballast are broadly-graded naturally occurring or processed sand-gravel mixtures, crushed natural aggregates or slag (Selig and Waters, 1994).

Sub-ballast reduces the stress on the subgrade and prevents penetration of the overlying ballast particles into the subgrade. The sub-ballast layer is typically 100-mm thick and offers a cheaper option to the otherwise thicker ballast (Lim, 2004). The grading of sub-ballast should theoretically create a filter zone; however, this has been neglected historically due to lack of attention being placed on the grading of sub-ballast (AREMA, 2003).

2.2.6 SUBGRADE

The subgrade is essentially the base and foundation for the track structure above. It tends to be prepared from existing natural ground, placed material or a combination of the two. As the track is designed to dissipate overburden stresses from its own self-weight and the passage of trains, the subgrade is required to have sufficient bearing capacity to prevent excessive and differential settlement. Additionally, a subgrade should also provide adequate drainage in order to maintain track serviceability.

Characteristics of a subgrade can have a significant effect on overall track stiffness (Selig and Waters, 1994). Raymond (1978) reported that the stiffer a subgrade, the lower the settlement and impact load would be. According to Zicha (1989), speed restrictions and/or load limitations need to be applied for subgrades with a low bearing capacity or poor drainage.

2.3 TRACK FORCES AND LOAD DISTRIBUTION MECHANISM

2.3.1 TRACK FORCES

The track structure must accommodate the imposed repeated forces in the vertical, lateral and longitudinal directions arising mainly from traffic and temperature variations. Understanding of the aforementioned forces is basic to substructure track design.

Vertical forces

Vertical forces can be either downward or upward. The upward response (uplift force) to a downward force (vertical wheel force) of the rail is shown in Figure 2-7. If the downward force is not offset by the weight of rail and sleeper together with any ballast frictional forces, the sleeper will lift momentarily. As the wheel moves ahead the sleeper is then forced down. This movement causes a pumping action, which can deteriorate the components of the track structure.

The downward force in a railway track consists of static and dynamic loads. The static load on the track includes the dead weight of the train and the weight of the superstructure while dynamic loads depend on track conditions, train characteristics, operating conditions, train speed, and environmental conditions (Selig and Waters, 1994). The dead wheel load can be taken as the vehicle weight divided by the number of wheels.

According to the field measurements carried out by Broadley et al. (1981) and Frederick and Round (1985), the dynamic forces could increase the wheel load by a factor of three, depending on the different types of rail track system.

Figure 2-8 (a) and (b) illustrate static and dynamic wheel loads plotted as cumulative frequency distribution curves for the Colorado test track and the mainline track between New York and Washington respectively (Selig and Waters, 1994). The static wheel load distribution was calculated by dividing known individual gross car weights by the corresponding number of wheels, and the dynamic wheel load distribution was determined by strain gauges attached to the rail. The vertical axes present the percentage of the total number of wheel loads out of 20,000 axles which exceed the load given on the horizontal axis. As can be seen in the figures, the dynamic increment is greater for high vertical wheel loads and is more substantial for the mainline track between New York and Washington as compared to the Colorado test track. This is because of the relatively perfect condition of the Colorado track. It should also be noted that the high dynamic load for the mainline track between New York and Washington occurred at high speeds (Selig and Waters, 1994).According to Selig and Waters (1994), the magnitude of the dynamic load can be up to 2.4 times the static load. Since the dynamic load can be much larger than the static load, the maximum stresses and strains in the track system that occur under dynamic loading are the main causes of adverse effects to the track (Selig and Waters, 1994).

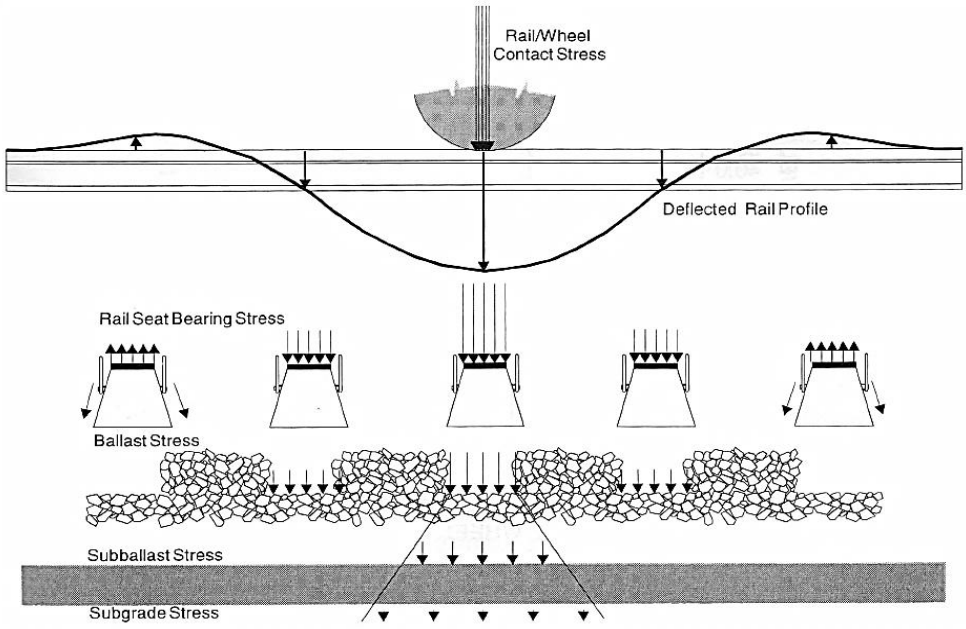


Figure 2-7 Typical wheel load distribution into the track structure (Selig and Waters, 1994)

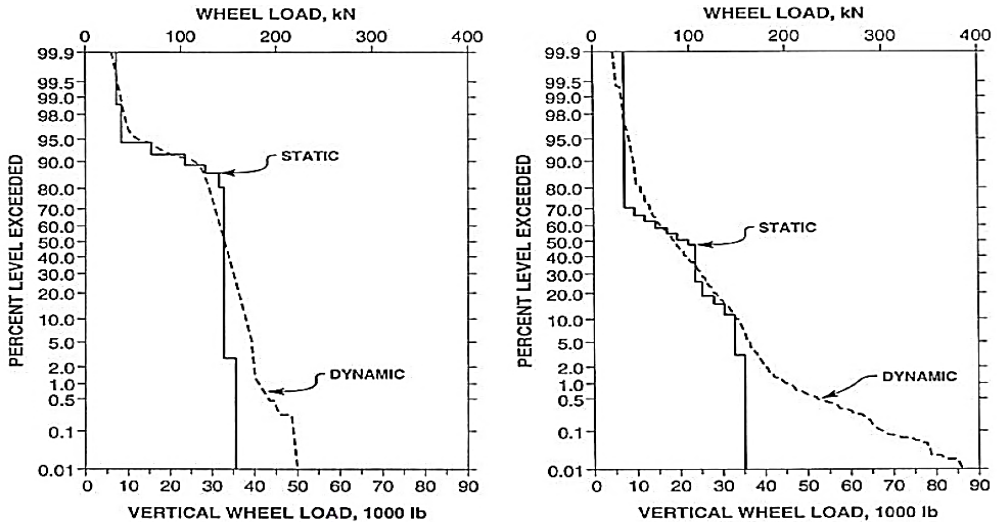


Figure 2-8 Static and dynamic wheel loads for (a) Colorado test track and (b) mainline track between New York and Washington (Selig and Waters, 1994).

Lateral forces

Lateral forces are those forces that occur parallel to the longitudinal axes of the sleepers. As described by Selig and Waters (1994), these forces are mainly from lateral wheel forces, and buckling reaction forces. Lateral wheel forces result from the lateral component of friction force between the wheel and rail, and from lateral forces applied by the flange of the wheel against the rail. Therefore these forces can be high around track bends/curves where high lateral friction forces are generated due to contact between the rail and wheel. The main reason for the lateral wheel forces is the reaction of train to deviations of track geometry. Buckling lateral forces can be also generated as a result of compressive longitudinal stress in the rail due to increase in temperature of the rail (Selig and Waters, 1994).

Longitudinal forces

Longitudinal forces occur parallel to the rail. These forces are induced from the acceleration and braking of trains as well as the thermal expansion or contraction of rails (Selig and Waters, 1994).

2.3.2 VERTICAL LOAD DISTRIBUTION MECHANISM

The vertical load of train wheels is transferred to the rails and then from the rails to the underlying components of the track. Load transfer in the track works on the principle of stress reduction. This means that the highest level of stress occurs at the wheel-rail interface and layer by layer it reduces, with the lowest stress on the foundation. The approximate magnitude of

pressure on each track component resulting from application of train vertical wheel load is presented in Figure 2-9.

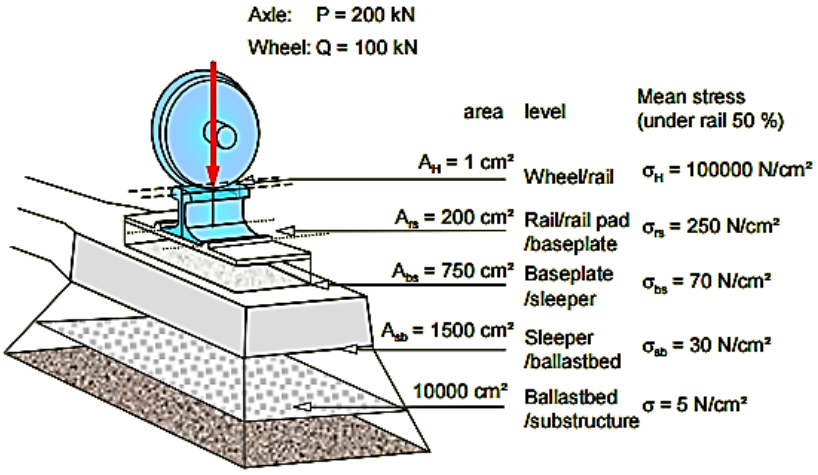


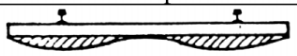
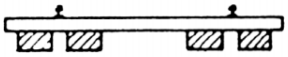
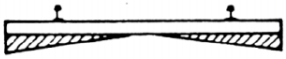
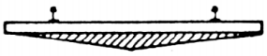
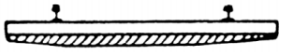
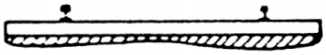
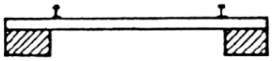
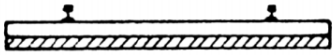
Figure 2-9 Load transfer principle adapted by author from Esveld (2001)

The measurement of stresses distributed at the sleeper-ballast and ballast-subgrade interfaces is discussed in this section.

Stresses at sleeper-ballast interface

The measurement of actual sleeper-ballast contact stress is very difficult and varies from study to study (Table 2-2). There are several factors influencing the magnitude of sleeper-ballast contact stress including particle size, development of 'structure' within the ballast (compaction and density), tamping quality, and the geometrical integrity of the track (Le Pen, 2008). The larger the number of contacts is between ballast particles and the underside of the sleeper, the better is the load distribution at the sleeper-ballast interface. According to Indraratna et al. (2011b), an extended support area at the sleeper-ballast interface decreases the contact pressure, and consequently it minimises the ballast breakage and track settlement. In a study by Shenton (1975) was found that after a sleeper had been installed for a while, the number ballast particles involved directly in supporting it was relatively small and may only consist of 100–200 contact points. This was based on a typical ballast size in the 25-50-mm range and a typical sleeper width of 250-mm.

Table 2-2 Proposed pressure distribution at sleeper-ballast interface (Sadeghi, 2008)

Item No.	Distribution of pressure	Developers	Remarks
1		ORE (1987), Talbot (1933)	Laboratory test
2		ORE (1987), Talbot (1933), Bartlett (1960), Clark (1957)	Tamped either side of rail
3		ORE (1987), Talbot (1933)	Principal bearing on rails
4		ORE (1987), Talbot (1933)	Maximum intensity at ends
5		Talbot (1933)	Maximum intensity at center
6		Talbot (1933)	Center bound
7		Talbot (1933)	Flexure of sleeper produces variations in form
8		ORE (1987), Talbot (1933), Kerr (2003), Schramm (1961)	Well-tamped sides
9		ORE (1987), Talbot (1933)	Stabilized rail seat and sides
10		AREMA (2006), Raymond (1977), Talbot (1933)	Uniform pressure

Regarding real (in service) track measurements on the distribution of sleeper-ballast contact pressure, an attempt by British Railways, in which pressure plates were installed in the base of a sleeper, provided a good indication of maximum pressure at the sleeper-ballast interface. These measurements are presented in Figure 2-10.

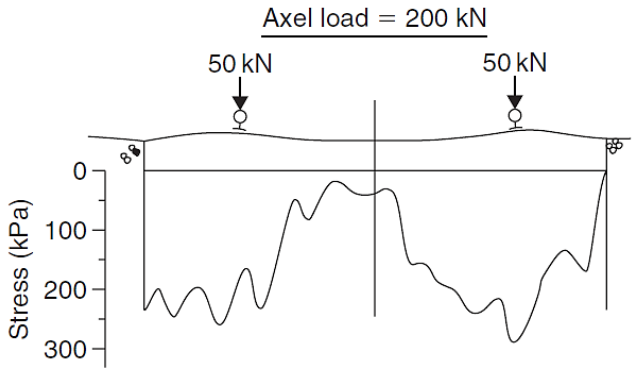


Figure 2-10 Measurement of pressure at sleeper-ballast interface (Shenton, 1975)

For design purposes, however, pressure at the sleeper-ballast interface is usually assumed to be uniform and simplified by Equation 2-1 (Jeffs and Tew, 1991b). In this equation, it is assumed that only an effective length of sleeper is involved in sleeper load support.

$$P_a = \left(\frac{q_r}{BL}\right) F_2 \qquad \textbf{Equation 2-1}$$

where,
P_a = average contact pressure,
q_r = maximum rail seat load,
B= width of sleeper,
L= effective length of sleeper supporting the load q_r, and
F_2 = a factor depending on the sleeper type and track maintenance.

The effective length of sleeper is shown in Figure 2-11and it is assumed to be one-third of the total sleeper length underneath each rail (Jeffs and Tew, 1991b). Therefore Equation 2-1 becomes:

$$P_a = \left(\frac{3q_r}{BL}\right) F_2 \qquad \textbf{Equation 2-2}$$

where, *L* = total length of sleeper.

The simplified sleeper-ballast contact pressure from Equation 2-2 is shown in Figure 2-11.

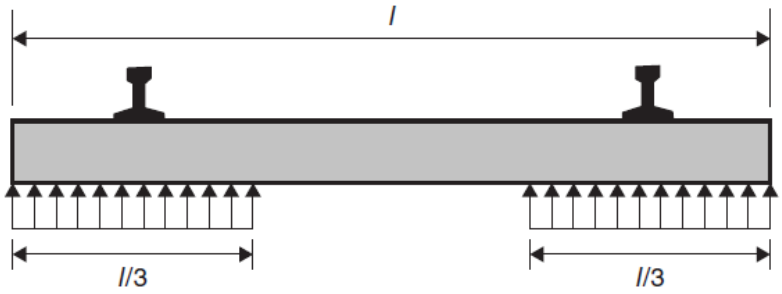


Figure 2-11 Simplifed sleeper-ballast contact pressure adapted by author from Jeffs and Tew (1991b)

This pressure can, however, vary after a number of loading cycles. For instance in the case where ballast is not well supported, it can lead to development of a gap underneath the ends of a concrete mono-block sleeper. This condition according to Raymond (1978) is known as centre

binding. In the case of centre binding, the sleeper is mainly supported at the centre and the rail loads are mainly transferred to the ballast in contact with the central section of the sleeper.

Stresses at ballast-subgrade interface

In trackbed design, measurement of maximum stresses at the formation level of railway tracks due to transient cyclic loads is critica, although, the current industry thinking regards trackbed performance to be strain dependent, rather than stress dependent. Several methods have been recommended for the calculation, including both simplified and more complex analysis in addition to empirically or semi-empirically derived analytical formulations. The recommended calculation methods by the American Railway Engineering and Maintenance-of-Way Association (AREMA, 2010) in its Manual for Railway Engineering, are as follows; it should be noted that the effect of sub-ballast is not considered in the AREMA recommended equations: In these equations:

$\boldsymbol{P_c}$ = Maximum intensity of pressure on subgrade (psi)
$\boldsymbol{P_m}$ = Intensity of pressure on sleeper-ballast (psi) = $\frac{\boldsymbol{2q}}{\boldsymbol{A_{sb}}}$
$\boldsymbol{h}$ = Depth of ballast below sleeper (inches)
$\boldsymbol{q}$ = Static rail seat load (lb)
$\boldsymbol{r}$ = Radius of a uniformly loaded circle whose area equals the effective sleeper bearing area ($\boldsymbol{A_{sb}}$) under one rail seat (inches).

a) Talbot Equation:

$$\boldsymbol{P_c = \frac{16.8 P_m}{h^{1.25}}} \qquad \textbf{Equation 2-3}$$

The Talbot equation is an empirical formula and it was developed at the University of Illinois based on field tests conducted during the 1910s (AREA, 1918). In those tests, a number of different ballast types were tested, including sand, slag, crushed stone, and gravel, and the stresses from applied static loads were measured at different depths and locations under several sleepers.

b) Japanese National Railways Equation:

$$\boldsymbol{P_c = \frac{50 P_m}{10 + h^{1.35}}} \qquad \textbf{Equation 2-4}$$

The Japanese national railways equation was developed for narrow gauge tracks (Indraratna et al., 2011a). The value of *h* (depth of ballast below sleeper) in this equation is in centimetres.

c) Boussinesq Equation:

$$P_c = \frac{6q}{2\pi h^2} \qquad \textbf{Equation 2-5}$$

The basis of the Boussinesq equation is the theory of elasticity and it assumed that the rail seat load (wheel load) is a point load on the substructure surface that forms a semi-infinite, elastic, and homogeneous (Poulos and Davis, 1974).

d) Love's Formula:

$$P_c = P_m \left[1 - \left(\frac{1}{1+\frac{r^2}{h^2}} \right)^{3/2} \right] \qquad \textbf{Equation 2-6}$$

The Love formula is also based on elasticity theory and an extension of Boussinesq results. In the formula the load applied by the sleeper to the ballast is assumed as a uniform pressure over a circular area equal to the sleeper bearing area (Indraratna et al., 2011a).

It should be noted that the static rail seat load is not equal to the static wheel load, since part of the wheel load is transmitted to the adjacent sleepers and only 40% to 60% of the wheel load is directly applied to each rail seat. According to Indraratna et al. (2011a) an assumption of 50% for the static rail seat load is considered to be reasonable. This assumption is consistent with NR/L2/TRK/4239 (NetworkRail, 2015), where at section 10.4.2 concerning the Falling Weight Deflectometer (FWD) it is stated that "*The FWD applies a pre-determined impulse load close to the rail seat areas of a sleeper which has been disconnected from the rail. A pulse of load, having a peak value of 12.5 tonnes (typically the maximum load seen by a sleeper under a passing 25 tonne axle load), is applied three times*."

According to AREMA (2010), the generally accepted railway practice of limiting subgrade pressure to 25 psi (approximately 172kPa) is recommended. The above equations, therefore, can be used to determine the required depth of ballast to achieve the recommended pressure on subgrade.

The limitations of the methods described above according to Li and Selig (1998) are as follows:

- The situation for tracks is oversimplified under heavier axle loads and higher train speeds,
- Subgrade soil conditions, effects of repeated dynamic loads, granular layer quality, etc. are not reflected,

- With the assumption of a homogeneous half-space (semi-infinite) to represent ballast, subballast, and subgrade, properties of individual layers are not considered, and
- The effect of the number of repeated stress applications on subgrade soil performance is not included as a design parameter.

The 2:1 trapezoidal assumption is also an approximate method to estimate vertical stresses at any given horizontal plane within the loaded area below the sleeper. The method assumes that stress disperses with depth in the form of a trapezoid with a 2:1(vertical:horizontal) ratio as shown in Figure 2-12. The vertical dissipated stresses reduce with increasing depth due to the increase in the area over which the load is spread. According to Jeffs and Tew (1991b) the load spread method provides an average value of the vertical stress. The maximum vertical pressure at a given depth below a rectangular sleeper can be estimated as follows:

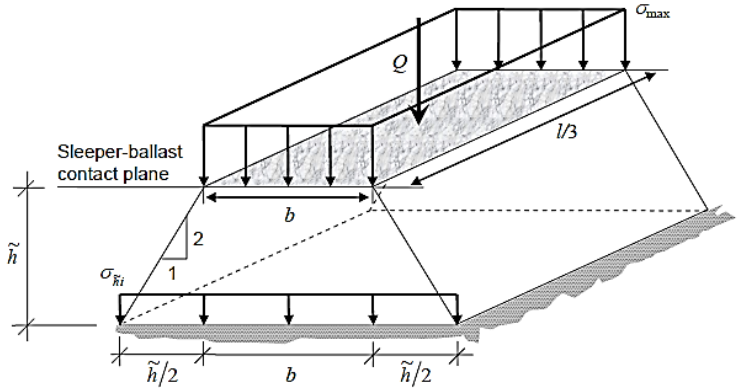


Figure 2-12 Approximation by 2:1 method for the calculation of the induced vertical stress at depth due to an applied load Q (Indraratna et al., 2011a)

$$\sigma_{z\,max} = \frac{Q}{(b+\tilde{h})\left(\frac{l}{3}+\tilde{h}\right)} \qquad \textbf{Equation 2-7}$$

$$\sigma_{max} = \frac{Q}{A_{sb}} \qquad \textbf{Equation 2-8}$$

where in the above equations:

Q = static wheel load for a third of the sleeper
$\tilde{h}$ = thickness of ballast
$\sigma_{z\,max}$ = maximum stress on top of rubber mat
σ_{max} = maximum sleeper-ballast contact stress
l = sleeper length
b = sleeper width
A_{sb} = contact area between the sleeper and ballast for a third of the sleeper

Apart from all the aforementioned calculation methods, there have also been experimental studies to measure stresses at the formation level using full-scale testing (Kwan, 2006, Brown et al., 2007a, Aursudkij, 2007). Kwan showed the effect of inclusion of geogrid reinforcement on stress level and the effect of ballast tamping on the stress level was studied by Brown et al. and Aursudkij. In all the aforementioned studies, the sleeper used was concrete mono-block and effect of using different sleeper types and ballast-sleeper interventions on foundation maximum stress was not included; similarly in the calculation methods introduced previously.

2.4 TRACK SETTLEMENT AND MAINTENANCE

2.4.1 TRACK SETTLEMENT

A railway track settles under repeated traffic loading due to permanent deformation in the ballast and the underlying subgrade. Therefore, the amount of settlement can be said to be governed directly by the quality and behaviour of the ballast and subgrade in addition to the loading level (Selig and Waters, 1994).

Track settlement can be divided into two stages (Dahlberg, 2001):

- The first stage immediately follows construction, installation or maintenance of the railway track at which point comparatively higher settlement occurs until the ballast is consolidated and the gaps between ballast particles are reduced,
- The second stage starts after the first and tends to be influenced by both the ballast and associated underlying layers. At this stage settlement tends to be slow and linear with regard to load and time.

As indicated by Dahlberg (2001), during the second stage several processes can occur to contribute to track settlement:

a) A continuation of particle rearrangement under repeated traffic leads to further volumetric reduction of the ballast;
b) The penetration of the sub-ballast and subgrade into the ballast as they are displaced under traffic;
c) Particle breakage due to train loads or environmental factors e.g. breakdown resulting in the particle being crushed into two or more pieces;
d) Particle volume abrasion at inter-particle contacts e.g. cornered stones become rounded, consequently occupying less space.
e) Accumulation of plastic stain due to micro-slip between ballast particles during loading

f) Reduction in sleeper support from the underlying layers resulting in sleepers sinking into the ballast
g) Displacement of ballast due to reduction in lateral and longitudinal sleeper confinement causing sleepers to sink further into the ballast

All aforementioned points are applied to both ballast and subgrade; the first four aforementioned processes are related to densification of ballast and subgrade, and the three last processes are linked to inelastic behaviour of the ballast and subgrade materials.

2.4.2 TRACK MAINTENANCE

The accumulation of track settlement carries on until it deteriorates to the condition that supporting the required level of service can no longer be achieved. Track maintenance is then needed to return the track to the as-built or acceptable level, to maintain the riding quality and fulfil the safety requirement. Ballast tamping, stoneblowing, and cleaning or renewal are the most common methods of track maintenance.

Ballast Tamping

The tamping operation consists of lifting the track and squeezing the ballast laterally under the sleeper to fill the gap generated by the lifting process. Although ballast tamping is one of the effective ways of rectifying track geometry defects, some ballast damage occurs during the process. As part of the operation, the tamping tools are inserted into the ballast and then squeezed together beneath sleepers, as illustrated in Figure 2-13. Both aforementioned actions are done by vibration, which damages ballast and results in some ballast particle breakage (Selig and Waters, 1994).

The lifting process during tamping disturbs the ballast layer under the sleeper up to a depth of 150 to 200-mm, which loosens the ballast layer and causes a higher rate settlement after tamping. According to Stewart and Selig (1984), the relative compaction of ballast under the sleeper reduces by 10-12% when large lifts of 38 to 50-mm are applied.

The process of tamping also rearranges the ballast particles. This creates new contact points between particles which increases the breakages of ballast under additional traffic loading. According to a study by Selig and Waters (1994), the tamping process created two times as many breakages of ballast particles compared to when tamping was avoided; Figure 2-14. Their study was done using a box test and applying 500k loading cycles including one test with no

maintenance and another test in which the ballast was rearranged every 100k loading cycles to simulate the tamping process.

A study by Wright (1983) in the track laboratory at British Railways Research on effecting tamping showed that approximately 2 to 4-kg of fines less than 14-mm was generated per tamp for a single sleeper.

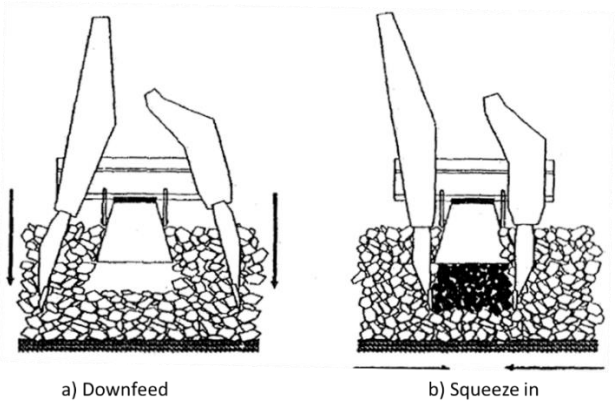


Figure 2-13 Schematic of tamping process (Selig and Waters, 1994)

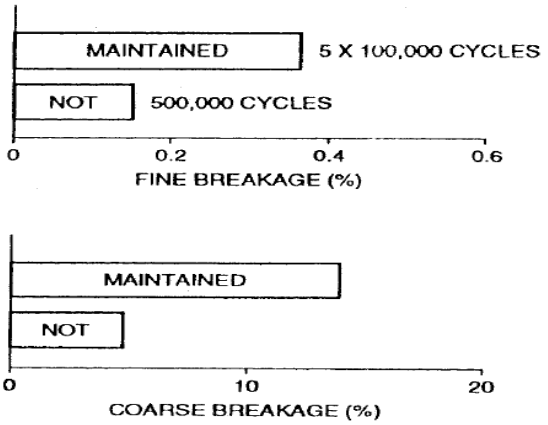


Figure 2-14 Effect of particle rearrangement on particle breakage (Selig and Waters, 1994)

Stoneblowing

Prior to the introduction of mechanised tamping, track re-levelling had been done by 'hand shovel packing'. In this method, the sleeper was lifted and the gap under the sleeper was filled by shovelling small size stones (5 to 12-mm) into it (Claisse and Calla, 2006). This process caused minimum disturbance to the existing well-compacted ballast. The mechanised version of this operation is known as Pneumatic Ballast Injection (PBI) or stoneblowing (Anderson et al., 2001). A schematic illustration of stoneblowing process is shown in Figure 2-15.

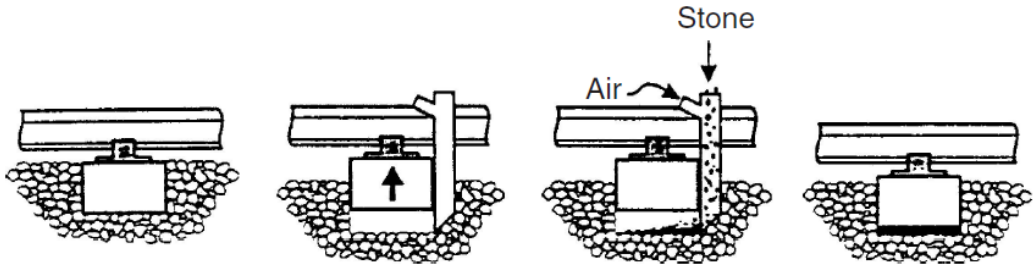


Figure 2-15 Schematic of stoneblowing process (Anderson et al., 2001)

To avoid any drainage problem and to conform to the Network Rail ballast Particle Size Distribution (PSD) presented in Table 2-1 (Network Rail, 2009), the stone size used for stone blowing is 14 to 20-mm which is larger than for hand shovel packing.

As a result of there being minimum disturbance to the existing well-packed ballast during a stoneblowing operation, the rate of track settlement is not expected to increase under subsequent traffic loading cycles after the process. The damage to the ballast during stoneblowing is also expected to be less compared to that of tamping.

Cleaning and renewal

Ballast cleaning or renewal is needed when there are more than 30% of fines less than 22-mm size or there is more than 40% pollution in the ballast (Esveld, 2001). The process is done by using a ballast cleaning machine which digs out the ballast under the sleepers to a minimum of 250-mm. The dug out ballast then is transported by a chain upward to a system of vibrating sieves where aggregate smaller than 35-mm is filtered out. Then clean coarse aggregates are returned to the track. Ballast may need to be completely replaced with fresh ballast instead of being cleaned on-track if it becomes extremely dirty. Since the process of ballast cleaning and renewal is costly, time consuming, and disturbing to the traffic flow, it is not frequently performed (Indraratna et al., 2011a).

2.5 MECHANICAL BEHAVIOUR OF BALLAST MATERIAL UNDER REPEATED LOADING

The behaviour of granular materials such as ballast under repeated loading is said to be non-linear and stress dependent (Selig and Waters, 1994). Loads from railway traffic can cause deformation of ballast and the materials underneath it. The deformation response of ballast under repeated loading is generally categorised into permanent (or residual) deformation and

resilient (or recoverable) deformation. The stress-strain behaviour of a granular material over a single loading cycle is shown in Figure 2-16.

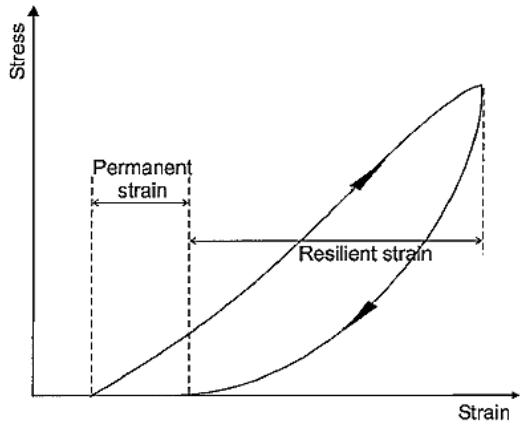


Figure 2-16 Strains in granular materials during one cycle of load application (Lekarp et al., 2000a)

When the ballast is loaded for the first time, there is a rapid and significant strain response which recovers only partially upon unloading. This sudden increase can be associated with the re-arrangement and corner breakage of the ballast particles. Application of additional load cycles has been shown to contribute to further increasing the permanent strain (Figure 2-17) (Selig and Waters, 1994). The plastic strain increment generally decreases after several load applications as less re-arrangement and corner breakage of the ballast particles occurs.

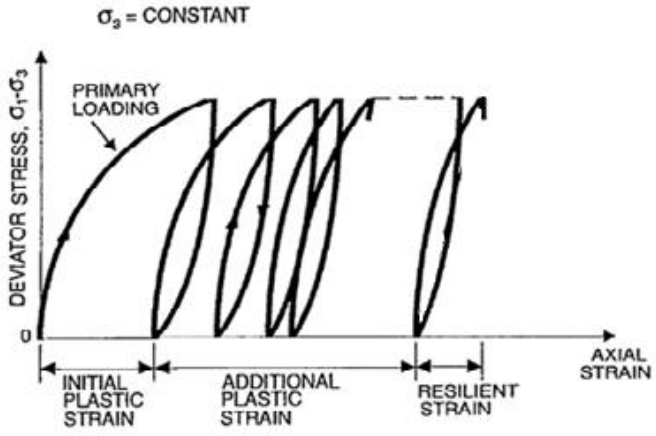


Figure 2-17 Stress-strain curve from a triaxial test under repeated load (Selig and Waters, 1994)

2.5.1 RESILIENT STRAIN BEHAVIOUR OF GRANULAR MATERIAL

The difference between the maximum strain generated at peak load and the permanent strain when the ballast is unloaded after each cycle is known as the resilient or recoverable strain. For a triaxial test, Seed et al. (1962) calculated the resilient modulus of materials as the repeated deviator stress divided by the recoverable axial strain when unloaded. This is shown in Equation 2-9:

$$\boldsymbol{Mr = \frac{q}{\varepsilon_{1,r}}} \qquad \textbf{(Equation 2-9)}$$

Where:

Mr= Resilient modulus,
q= Deviator stress= σ_1-σ_3, and
$\varepsilon_{1,r}$= Recoverable (resilient) axial strain during unloading.

The resilient behaviour of granular material is affected by several factors (Lekarp et al., 2000a) such as stress level, density, maximum grain size, grading, fines content, moisture content, stress history, number of load cycles, aggregate type, particle shape, load duration, loading frequency and load sequence. However, the most significant influence on granular material resilient behaviour is from stress, i.e. stress level, stress history and stress sequence. Lekarp et al. (2000a) showed that the resilient modulus increased significantly with an increase in confining pressure and sum of principal stresses. This is supported by several studies (Hicks and Monismith, 1971, Uzan, 1985, Thom and Brown, 1989, Sweere, 1990).

As alluded to earlier, the magnitude of the resilient strain generally decreases with number of cycles since the material stiffens. As a result, resilient modulus increases gradually with number of cycles, becoming almost constant after a certain number (Figure 2-18). Hicks (1970) reported that a constant resilient response could be achieved after 100 loading cycles of the same stress amplitude. However, observations of Hicks were challenged by Boyce (1976) who claimed that it will take approximately 2000 loading cycles before the resilient strain reaches a steady value.

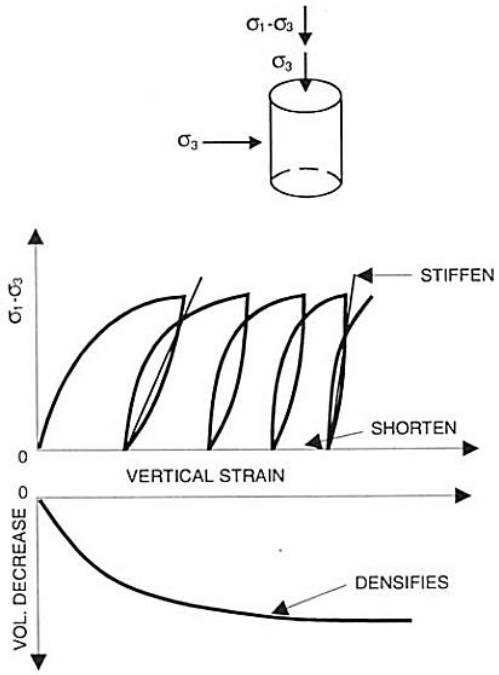


Figure 2-18 Behaviour of ballast under cyclic triaxial test (Selig and Waters, 1994)

As illustrated in Figure 2-19, Collins and Boulbibane (2000) explained the behaviour of granular material with the concept of shakedown in four stages. At the first stage, there are no permanent strains when first cyclic loading is adequately small. However, when the load exceeds the elastic load, the permanent strains will be made (Stage 2). The behaviour then becomes purely elastic after the finite number of cycles (no further permanent strain). As a result, the resilient strain becomes constant resulting in constant resilient modulus. At this stage, the material structure is expressed to have "shakedown". If the loads increase, the shakedown does not happen and the behaviour of granular material will be either "cyclic plasticity" where a closed cycle (loop) of permanent strain is formed (Stage 3) or "ratchetting" where permanent strain increases indefinitely (Stage 4). In the situation of either of stage 3 or stage 4, the structure will fail. The critical load below which the structure shakes down and above which can cause failure is called the shakedown load.

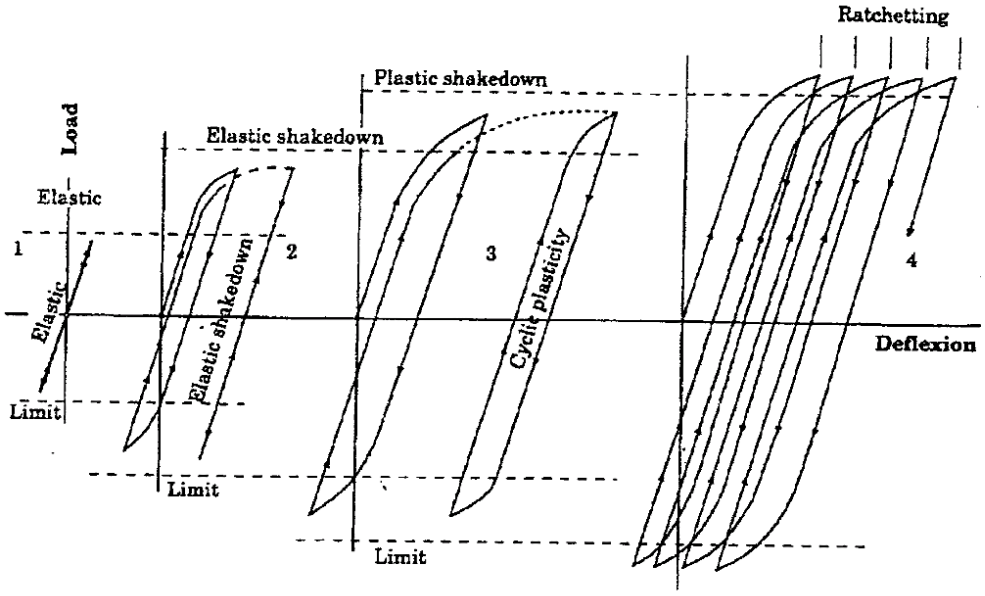


Figure 2-19 Concept of shakedown in four stages (Collins and Boulbibane, 2000)

2.5.2 PERMANENT STRAIN BEHAVIOUR OF GRANULAR MATERIAL

The plastic/permanent deformation of granular material is illustrated in Figure 2-17. The accumulation of such permanent strain under repeated loading occurs as a result of particle re-arrangement or particle breakage. This can be caused by several factors including stress level, initial density, frequency, number of cycles and sequence of loading. However, stress level is recognised widely as the most significant influence on permanent strain of granular material. Morgan (1966) reported that the accumulation of axial permanent deformation is directly associated with deviator stress and inversely related to confining pressure. Brown and Hyde

(1975) also showed permanent strain to be directly linked to the ratio of deviatoric stress to confining pressure. However, it is argued that identical ratios may give different permanent strains, especially for larger deviatoric stresses. As shown in Figure 2-20, Knutson (1976) found a ratio of 60/15 was associated with a larger permanent strain than 20/5 since it had a larger deviatoric stress.

A similar study was conducted by Lackenby et al. (2007) using a series of cyclic triaxial tests on ballast under various loading conditions. They also concluded that the permanent axial strain is related to the magnitudes of both maximum deviator stress and confining pressure. Their study showed that permanent axial strain decreased with decreasing maximum deviator stress and increasing confining pressure. Figure 2-21 shows the permanent axial strain and permanent volumetric strain response of granular materials as a function of number of cycles and confining pressure.

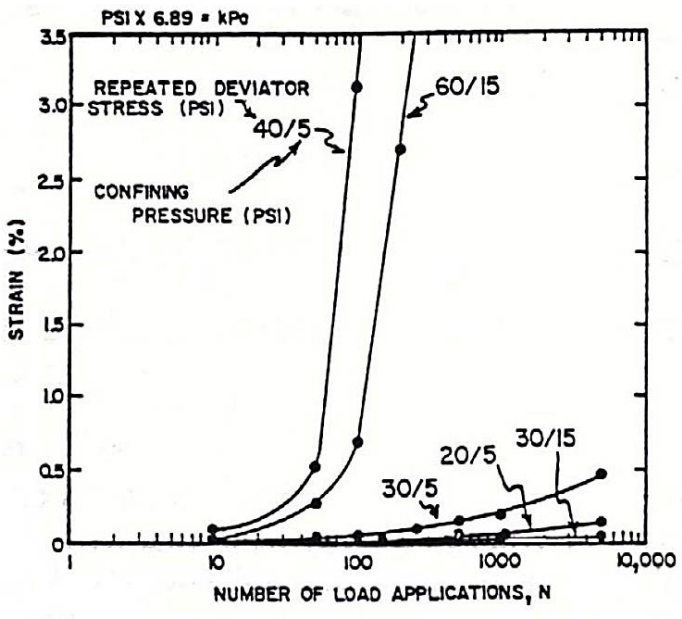


Figure 2-20 Effect of stress ratio on permanent strain (Knutson, 1976).

Another important factor affecting the accumulation of permanent strain is the initial density or the initial degree of compaction. Many studies have shown a small decrease in initial density to lead to a significant increase in the accumulated permanent strain (Holubec, 1969, Barksdale, 1972, Allen, 1973). This effect is more significant for angular aggregates as compared to rounded aggregates (Holubec, 1969) since the rounded aggregates have a higher initial relative density as compared to angular aggregates for the same compacting effort (Lekarp et al., 2000b). Shenton (1974b), Jeffs and Marich (1987), and Brown (1996) concluded that achieving

a high initial compacted density can significantly improve resistance to accumulation of permanent strain. Furthermore, the initial compaction of ballast improves the track quality by increasing the trackbed stiffness and reducing the differential settlement. However, according to Sharpe and Strange (2009), compaction of the lower part of ballast in the trackbed is unlikely to impact on the initial permanent strain of ballast, since the majority of the ballast deformation occurs directly under the sleeper where the stresses are high. Moreover, compaction of the lower ballast can reduce the ballast life by causing particle breakdown as well as reducing the void space (Sharpe and Strange, 2009).

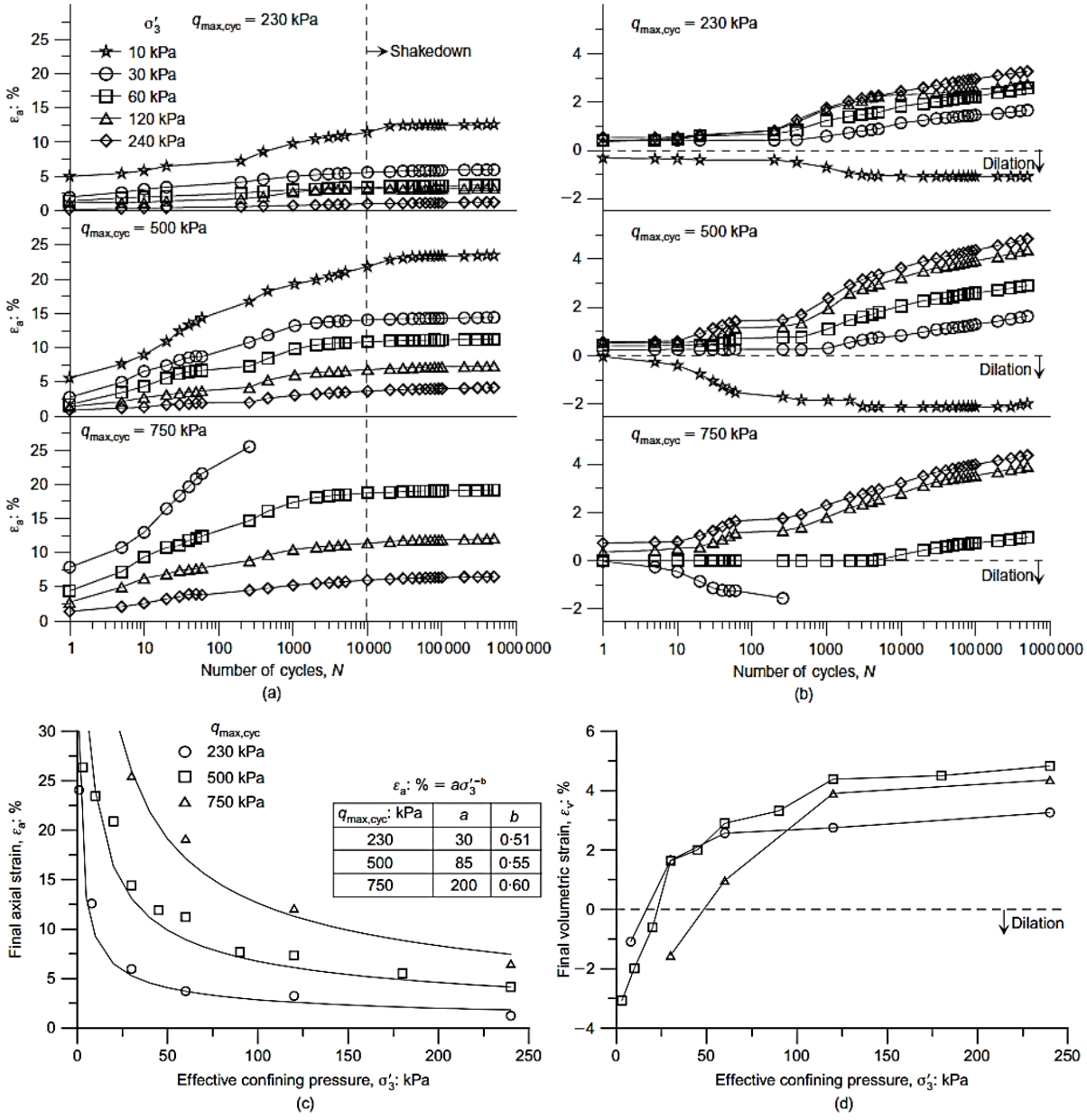


Figure 2-21 Strain response under cyclic loading: (a) axial strain ε_a as a function of the number of cycles N; (b) volumetric strain ε_v as a function of N; (c) final ε_a after 500,000 cycles and (d) final ε_v after 500,000 cycles (Lackenby et al., 2007).

2.6 RAILWAY SLEEPERS

A railway sleeper represents the start point of the superstructure of a railway track and its range of functions primarily includes rail gauge maintenance as well as the support and dissipation of live loads into track substructure. Sleepers are typically buttressed by ballast and anchor the track against vertical, longitudinal, and lateral movement. The railway industry uses a variety of sleeper types which can differ from one another in material composition, geometry and/or manner of installation within the track. Some of the common types include timber, steel, and concrete sleepers. Other less conventional types include concrete frame sleepers, concrete twin block sleepers, Y-shaped steel sleepers and the plastic composite sleepers. Each sleeper type has its own merits when used appropriately. However, the longevity of concrete, steel and timber sleeper types on the UK railway network indicates that they all satisfy certain basic criteria. From Profillidis (1995a), Manolo et al. (Manalo et al., 2010b), Sadeghi and Barati (2010) some of these criteria have been interpreted and graded as adequate, good or very good as presented in Table 2-3 with respect to their performance for the specified criteria. Those properties have been assigned equal grades in order to assume sleeper parity.

Table 2-3 Qualitative comparison of the main sleeper performance characteristics of common railway sleepers.

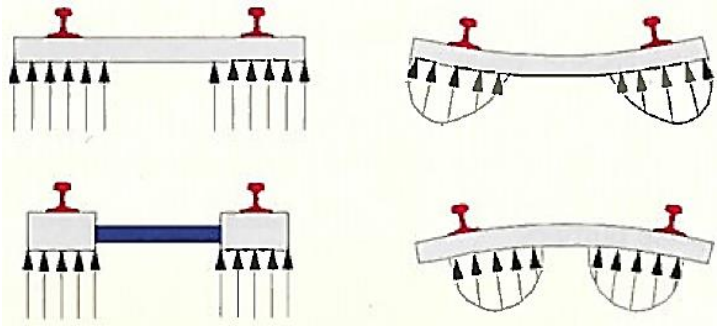
Property	Timber (Hardwood)	Steel	Concrete
Availability, Life Span	adequate	good	very good
Support heavy loads and high speeds	very good	good	very good
Handling and Installation	very good	good	adequate
Workability, Versatility	very good	good	good
Lighter Weight	good	very good	adequate
Residual value	adequate	very good	good
Moisture susceptibility, Fasteners	good	adequate	very good
Cost, Sleeper-ballast Interaction, Electric Conductivity	very good	adequate	good

Choice of railway sleepers varies widely geographically and sleepers can be differentiated with respect to their material type or geometry. However, the selection of sleeper types has been guided by financial considerations (Zarembski, 1993, Profillidis, 2000), logistical constraints (Qiao et al., 1998, Sonti et al., 1995) and experience (or rule of thumb) (Abbott, 1989) rather than research-derived evidence. Zarembski (1993) proposed strength (or survivability in a railway environment) and economic criteria as the only two basic sleeper acceptance criteria. Moreover, Profillidis (2000) recommended that a feasibility analysis be undertaken when choosing the most appropriate sleeper type for a line. This is suggested to include the following considerations:

- Construction or purchasing cost of sleeper
- Purchasing cost of fastenings and other disposable sleeper accessories
- Sleeper lifetime
- Maintenance cost
- Probable residual value of sleeper at the end of lifetime

Fundamentally, consideration of interactions of the different sleeper types with various track components has not been acknowledged; e.g. effect of different sleeper type on sleeper-ballast interaction (one the main causes of initial settlement of track) and on vertical subgrade stress.

When sleepers are newly placed or tamped, the contact area at the ballast-sleeper interface is idealised to be mainly below the rail seat (Raymond, 1978, Esveld, 2001). By packing ballast underneath the rail seats, the highest level of support would be provided directly underneath the rail. This can be done for a prismatic sleeper (e.g. timber and concrete mono-block sleeper) by packing the rail seat area beneath sleeper and leaving the centre free (Figure 2-22 (a)). In the case of sleepers such as concrete twin-block this is done naturally by the actual structure of the sleeper. According to Esveld (2001), if the sleeper support is very close to the inside or outside as a result of bad tamping, the sleeper will rotate under cyclic load. This alters the inclination of the rail and changes the gauge (Figure 2-22 (b)). The sleeper support can also vary after a number of loading cycles. For instance in the case where ballast is not well supported, it can lead to the development of a gap underneath the ends of a concrete mono-block sleeper. This condition according to Raymond (1978) is known as centre binding. In the case of centre binding, the sleeper is mainly supported at the centre and the rail loads are mainly transferred to the ballast in contact with the central section of the sleeper. The bending stiffness of a sleeper affects the tendency of a sleeper toward centre binding.



(a) **(b)**

Figure 2-22 Sleeper packing (a) correctly packed sleepers (b) incorrectly packed sleepers (Esveld, 2001)

Further details including advantages and disadvantages of using the different sleeper types investigated in this study will be discussed in subsequent sections.

2.6.1 TIMBER/WOOD SLEEPERS

Timber or wood is the first material to have been used for railway sleepers. Wood sleepers have a long history in terms of effectiveness and reliability of performance in the railway environment (Ellis, 2001). They are traditionally derived from hardwood (Rothlisberger, 2008), although softwood is also known to have been used where hardwood was not available. The latter is however, (even after treatment) more susceptible to wear in comparison to hardwood (Rothlisberger, 2008). Timber sleepers are prismatic in shape and as a result of being relatively lightweight (about 100kg), can be replaced by hand (Esveld, 2001). According to Profillidis (2000), the load distribution of timber sleepers is more effective than any other type of traditional sleeper. Therefore, if a railway track is built on a poor quality subgrade, timber can be a more suitable option than concrete, as concrete would need a deeper ballast layer in the aforementioned condition. Furthermore, timber sleepers are more suitable than concrete sleepers for special track works such as turnouts where the sleeper needs to be cut (Indraratna et al., 2011a).

The main advantage of wooden sleepers resides in their adaptability. In addition to being versatile with regard to their applicability, they are easy to handle and replace, with minimal installation requirements (Rothlisberger, 2008). This is in particular important for high speed or heavily trafficked lines, where the ability to bring in a large scale production team is constrained by time (Manalo et al., 2010b). In addition, wooden sleepers have other advantages, such as:

- Good damping properties (absorbs vibrations),
- Low manufacturing cost,
- Good track circuiting (electric insulation) quality, and
- Gauge adjustment is relatively easy

The main disadvantage of using wooden sleepers is their susceptibility to mechanical and biological degradation, which can lead to them failing (Qiao et al., 1998). Some common wood sleeper failure modes are shown in Figure 2-23.

The following additional disadvantages have also been documented:

- Relatively short lifetime, up to 25 years (Railway Technical, 2013),

- Difficulty in maintaining track gauge (hole widening problem),
- High maintenance cost (e.g. fire hazard and pest attack),
- Negligible scrap value, and
- Environmental concerns regarding the use and disposal of chemically-impregnated timber sleepers.



(a) **(b)**

Figure 2-23 Common types of timber sleeper failure. (a) fungal decay, (b) splitting at ends. (source: http://www.tubulartrack.co.za)(Manalo et al., 2010a)

2.6.2 STEEL SLEEPERS

Around 1880, the steel sleeper was introduced as a result of scarcity and sensitivity of timber materials (Profillidis, 2000). Since then, it has found widespread use in Europe, especially on secondary lines and industrial tracks (RAH, 2000). In Britain steel was first used the 1930s and there has been a standard for their usage since 1933 (BSI, 2013).

Steel sleepers have an inverted trough-shape with spade ends as shown in Figure 2-24. They are initially laid on the ballast sitting on the spade ends. Following this, under passage of a ballast train (Figure 2-25) or using a bespoke machine (Figure 2-26), the sleepers are pushed into the ballast resulting in partially filled troughs. Additionally, by means of a tamper, steel sleepers are tamped several times on installation to ensure that they are filled with ballast and led to an acceptable track geometry. Steel sleepers are generally lighter than their timber counterparts and have an average life of 50 years. They also provide significant lateral resistance of the track as a result of their spade ends. Additionally, steel sleepers are also faster to install with less site preparation requirements, offer greater stability and require less maintenance in comparison to timber sleepers. Steel sleepers also have the advantage of being 100% recyclable.

Steel sleepers are generally considered for use in tunnels and bridges where ballast depth is dictated by limited clearances. However, due to their light weight, they are not suitable for high speed lines and are restricted to lines with speeds of 100 mph (160 kph) or less (Railway Technical, 2013).

Steel sleepers have the following disadvantages:

- Corrosion prone (especially near industrial and coastal areas),
- Gauge change is not possible,
- Makes longitudinal and transverse track positioning difficult,
- Unsuitable for track circuiting areas, and
- Relatively high price.



Figure 2-24 Steel railway sleeper (TATA STEEL, 2014)



Figure 2-25 Installation of steel sleepers using ballast train



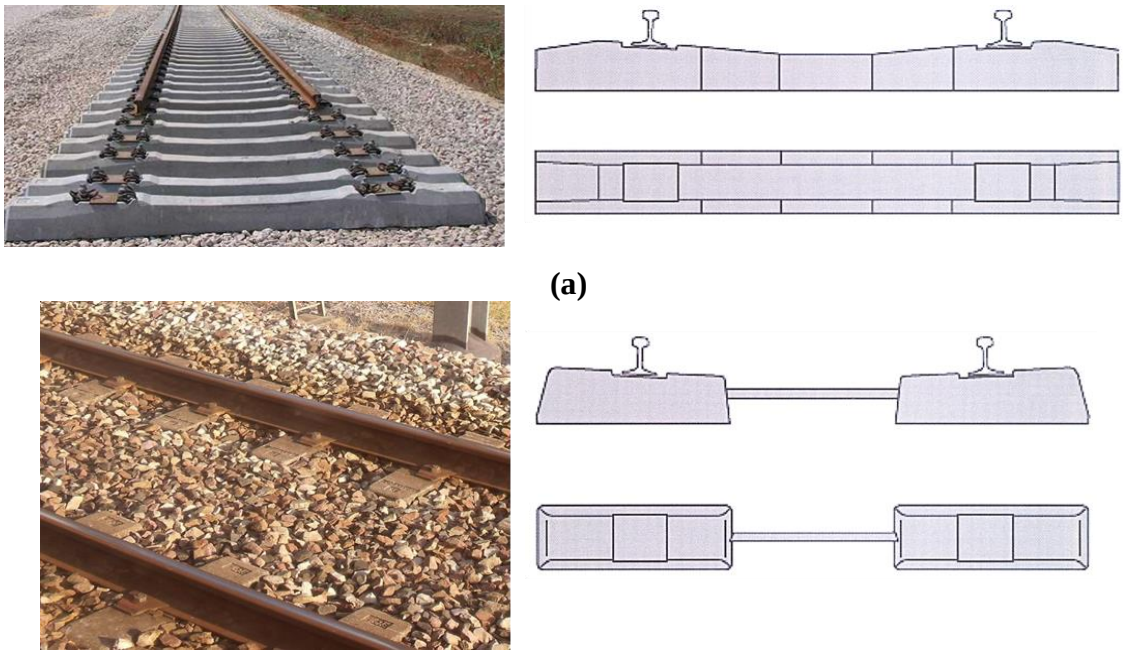
Figure 2-26 Machine for installation of steel sleepers

2.6.3 CONCRETE SLEEPERS

Use and development of concrete sleepers significantly increased after the Second World War, as a result of scarcity of wood, the introduction of CWR track, the advancement of technology related to concrete and pre-stressing techniques (Esveld, 2001). Concrete sleepers are typically made from reinforced, pre-stressed concrete. Mono-block and twin-block sleepers are the two main types of concrete sleeper. The twin-block sleeper comprises two trapezoidal concrete blocks joined by a connecting bar. Both types are shown in Figure 2-25.

The twin-block sleeper was developed in France, where it is predominantly used for high speed lines. The mono-block sleeper on the other hand was developed in the UK and the G44 is the most common variant.

Both mono-block and twin-block sleepers have a lifespan of roughly 50 years and perform similarly in their ability to maintain the track gauge. However mono-block sleepers distribute loads better than twin-block, although not as well as timber sleepers (Profillidis, 2000). The mono-block sleeper is able to stabilise dynamic loads more effectively than its counterparts due to a heavier weight advantage. With respect to lateral resistance, the twin-block is superior to the mono-block due to its possession of two additional vertical end faces in contact with the ballast. Generally however; compared to timber sleepers, both behave better in terms of their lateral resistance (Profillidis, 2000).



(a)

(b)

Figure 2-27 Concrete sleepers (a) mono-block, (b) twin-block (source: http://www.indiamart.com)

Looking at Figure 2-27 (b), the reduced footprint area of the twin-block sleeper suggests that it would require a sufficiently thick ballast layer to compensate for the loss of resistance to imposed loads, i.e. stresses developed in the ballast below the twin-block sleeper tend to be higher than under a mono-block. Twin-block sleepers also require extra maintenance to ensure that their component blocks do not tilt differentially and the track does not loose gauge due to movement of the flexible bar connecting the blocks.

In general, concrete sleepers have the advantage of being impervious to insect and moisture damage. Furthermore, they have been found by railroad Maintenance-of-Way (MOW) experts to maintain alignment well. It has also been reported that using clean, high quality ballast at installation stage in conjunction with concrete sleepers improves performance at the ballast-sleeper interface (Grant, 2005).

Concrete sleepers generally have the following disadvantages:

- Heavy to handle and require specialised machinery during laying and installation,
- Less flexible, as compared to a timber sleeper under heavy loading and susceptible to cracks (Railway Technical, 2013),
- The initial cost of concrete sleepers is almost double that of hardwood timber sleepers (Manalo et al., 2010b), however this can be varied from country to country,
- They have no scrap value, and

- Risk of damage from impacts (derailment, loading/unloading, tamping tines).

2.6.4 PLASTIC COMPOSITE SLEEPERS

Plastic sleepers (Figure 2-28) have increased in popularity in recent years with research into their use dating back to the mid-1990s (Grant, 2005). Various mixtures of raw materials, such as plastic, rubber from used tyres and waste fibreglass, are reported to have been used to produce synthetic sleepers with the flexibility and accessibility of wood, combined with the durability of concrete (Railway Technology, 2010).

Composite sleepers are documented to offer many potential advantages. For example, Wrap (2006) reported that the use of railway composite sleepers can increase the profitability of operations by reducing track maintenance costs, reducing downtime and improving performance. An investigation by Hoger (2000) showed that use of bulk recycled plastic for railway sleepers increases the strength of the sleeper. Furthermore, the study by Pattamaprom et al. (2005) showed that using engineered natural rubber for railway sleepers improves the compressive modulus and hardness of the sleeper. According to Grant (2005), plastic sleepers also offer increased lateral track stability when their sides and bottoms are appropriately roughened (Grant, 2005).



Figure 2-28 Plastic sleepers on a ballasted track (source: http://www.businessreport.co.uk/)

Wrap (2006) compared the features of a composite sleeper to a timber sleeper of equivalent size and weight and found the following:

- A composite sleeper lasts longer than a wooden sleeper, reportedly with better performance,

- A composite sleeper can be used interchangeably with wood or concrete sleepers,
- A composite sleeper can be nailed or spiked just like wood without splitting and with greater spike retention; or can be pre-drilled like concrete. Either way, identical "hardware" can be used for the remainder of the sleeper to track system,
- A composite sleeper can be installed with traditional wooden sleeper hardware and machinery,
- A composite sleeper experiences minimal deterioration due to climate, especially humidity or moisture,
- A composite sleeper is not susceptible to any insect damage, and
- A composite sleeper has no future disposal costs (unlike creosote wood sleepers) and can be recycled into new composite sleepers.

In the USA, plastic and composite sleepers are being trialled as environmentally friendly recyclable alternatives to conventional sleepers. In Japan, composite sleepers are used in areas of their rail network where maintenance or replacement is difficult. The Japanese synthetic sleepers are made of hard polyethylene foam and glass fibre, and are designed for a lengthy lifespan (more than 60 years) while maintaining the physical properties of timber sleepers (Miura et al., 1998). In the UK, successful track trials of plastic sleepers in 2009 led to Network Rail proposing to replace traditional wooden sleepers with plastic sleepers to meet a target of 23 percent recycled material use by 2012 (The Telegraph, 2009).

According to Indraratna et al. (2011a) plastic sleepers have gained limited acceptance, mainly due to the speed of mass production of stronger concrete sleepers. The main disadvantages of plastic sleepers are as follows:

- Uncertainty and poor understanding about the mechanical behaviour (Lampo et al., 2003, Ferdous et al., 2015),
- Low sleeper-ballast interaction that is greatly dependent on the surface characteristics of the sleeper (Lampo et al., 2003, Lotfy et al., 2016), and
- Possible stretching of the sleepers between the rails, leading to increasing gauge (Lampo et al., 2003).

2.7 UNDER SLEEPER PADS (USPS)

As discussed earlier in this chapter, static and dynamic loads imposed on a railway track result in ballast deformation of which one aspect is non-recoverable. As indicated by Indraratna et al. (2005), the sleeper-ballast interface is one of the most important parts of the track in terms of load distribution to the substructure. Vertical cyclic pressures at the interface can cause major particle rearrangement, ballast degradation and track settlement as a consequence. According to Sharpe and Strange (2009), the majority of the initial ballast deformation occurs directly under the sleeper where the stresses are high. These permanent deformations affect the geometry of the track and necessitate maintenance of the railway track after a number of imposed cycles of train passage. It is possible to influence track dynamics by incorporating an intervening elastic element at the sleeper-ballast interface, which its configuration, is able to alter the track dynamics and consequently, settlement performance of track.

With increasing demands for high operational speeds and loads combined with passenger comfort and minimal disruption to service, railway operators continue to face many technical and economic challenges. Under sleeper pads are one of many proprietary solutions that have been proposed to improve aspects of the railway track system, in achieving overall, a more efficient railway. Under sleeper pads (also known as sleeper soffit pads or sole pads) are resilient pads attached to the underside of sleepers to provide an intermediate (or intervening) elastic layer between the sleeper and the ballast (Lakušić et al., 2010a, Schneider et al., 2011).

While a relatively recent addition to the railway track structure, historically USPs have seen widespread applications in the area of noise and ground vibration mitigation and have been utilised in railway tracks for some 30 years (Plasek et al., 2007, Schneider et al., 2011). Very soft under sleeper pads with wooden sleepers were first used by the Swiss Railways (SBB) in 1986 for vibration reduction and in field trials, very few problems with track stability or material deterioration over a 20 year monitoring period were observed (Plasek et al., 2007). During the recent years, a broader application of USPs has developed, especially in Europe, notably in France (SNCF), Switzerland (SBB), Norway (NSB), Denmark (DSB), The Netherlands (ProRail), Spain (ADIF/RENFE), Hungary (MAV), Germany (DB) and Austria (ÖBB) (Garburg, 2007).

According to Johansson et al. (2008a) and Schneider et al. (2011), USPs typically possess a foam structure, usually with encapsulated air voids or cork particles and as reported by the

International Union of Railways (UIC) (Garburg, 2007), USPs have been manufactured from polyurethane, rubber, and Ethylene Vinyl Acetate (EVA) (Lupolen). USPs normally comprise two layers: an elastic spring layer and a bonding layer to facilitate attachment onto a preferred substrate, which may be any number of sleeper materials but often concrete. For softer pads there is usually an additional layer to prevent the ballast particles from puncturing the soft spring layer. This additional layer has been identified in literature to be usually a geotextile material Johansson et al. (2008a). The main methods of attaching a USP to the underside of a sleeper are summarised in UIC (2009a) as follows:

a) Coating by spraying or painting onto set concrete,
b) Gluing to hardened concrete,
c) Placing directly onto the unset concrete (during sleeper production), and
d) Placing onto unset concrete by using an interlocking layer (e.g. extruded knobs, wire mesh, geo-membrane, fine-grained gravel).

Standards, technical specifications, and recommendations on USPs have been released mainly in relation to how USP materials can be characterised and procedures to ensure that USPs achieve adequate attachment to the underside of sleepers; e.g. by Deutsche Bahn (DB)(Bahn-Norm, 2004, Bolmsvik, 2005), UIC (UIC, 2008), and London Underground (LU) (LU, 2015). However, no universally recognised guidance on USP selection in relation to other interrelated track elements or external factors such as maintenance requirements have been encountered in the literature thus far. There is a particular interest in finding a relationship that reconciles the stiffness of the USP with the stiffness of the other obvious elastic component of the railway track structure, namely rail pads. This is important as vertical sleeper accelerations are influenced by the interaction between stiffnesses of the USP and rail pad. For example, numerical tests by Johansson et al. (2008a) indicated that where a USP is considerably softer than a rail pad in a track system the USP preferentially or sacrificially functions to attenuate track vibration resulting in the increased sleeper vibrations mentioned earlier. This result is supported by a UIC report (Garburg, 2007), which surmises that a stiff rail pad with a soft USP is the poorest combination of the two materials as this induced cracks in the sleepers tested. It was further suggested that the converse would permit higher axle loads on the track.

Commercially several benefits have been associated with the use of USPs. Several of these claims have been researched, and while some have proven true, others still require further

research work to be validated, although intuitively they seem plausible. The railway administrations that partook in the UIC study of USPs and are trialling USPs on their networks collectively concur on the following benefits:

- Reduction of long pitch corrugation in tight radius curves,
- Substitution for under ballast mats (UBM) to reduce noise and vibrations,
- Less maintenance: prolongation of tamping intervention periods,
- Reduction of ballast depth (where there is inadequate clearance),
- Better load distribution leading to a reduction of rail and sleeper stresses,
- Improvement of track geometry,
- Improvement of track stability,
- Reduction in whole life costs, especially with heavier loading of tracks, and
- Increased passenger comfort.

It is the author's view that the aforementioned USP merits are (inferable or consequential) derivatives of some more fundamental (or primary) USP benefits. These primary benefits are referred to frequently in the literature and are outlined below:

a) Protection of ballast layer from inter-particle abrasion and sleeper-ballast attrition (Witt, 2008, Schneider et al., 2011),
b) Increased sleeper-ballast contact area (Witt, 2008, Riessberger, 2006, Schneider et al., 2011),
c) Improved track lateral resistance (Sol-Sánchez et al., 2014),
d) Reduction of stiffness variations between sleepers and at transition zones (Dahlberg, 2010), and
e) Improved load distribution on the subgrade (Dahlberg, 2010).

The adage 'the right tool for the job' applies when it comes to using USPs and although there are no explicit disadvantages of using USPs, they will arise as a result of selecting an inappropriate USP in any given application. It is therefore important for the USP to meet some basic criteria. These requirements are gleaned from Witt (2008) and other commercial sources as follows:

- Resistance to abrasion or puncture from sharp corners of ballast particles,
- Indefinite resistance to loads imposed by passing trains without compromising their elastic or resilient properties,

- The vibration attenuation properties of the USPs should also fit with the requirements for the track, and
- Track renewal or maintenance involving USP installation should be such that new machines that would incur additional cost are not needed although modifications may be necessary.

Using a finite element track model with optimised ballast/substructure and USPs, Dahlberg (2010) determined that it is possible use USPs to mitigate or smooth out stiffness disparities between transition zones e.g. the transition from a 'soft' embankment to a steel deck bridge. He also showed that the introduction of USPs into the track structure results in the axle load of a train being distributed over a greater number of sleepers. Consequently, this demonstrates that USPs are useful in reducing stress levels on the ballast layer. In agreement, Lakušić et al. (2010a) reports a stress reduction of up to 40% in the ballast bed. Consider Figure 2-29: here, using a finite element track model, Witt (2008) studied the influence of USPs on sleeper-ballast contact forces for a soft track section and a stiff track section. For the soft section, it is firstly noticed that there is not much difference in the maximum contact forces for the track with the stiff USP from that without a USP. On the other hand, as the USP becomes softer a decrease in the maximum ballast contact force (from 32kN to 19kN) is observable.

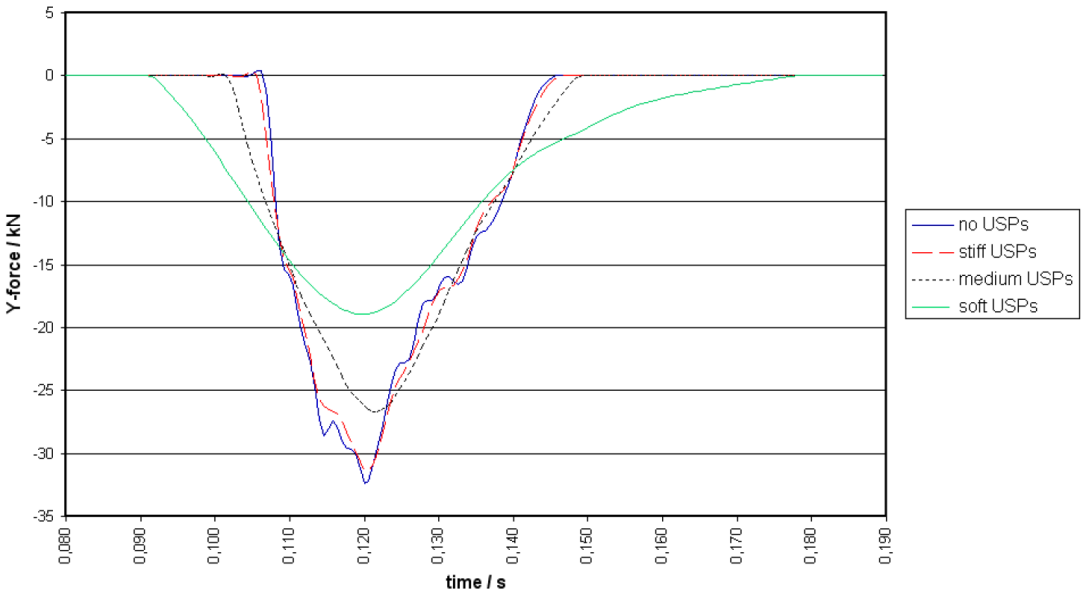


Figure 2-29 Effect of USP stiffness on ballast contact force Witt (2008)

Johansson et al. (2008a) investigated the influence of the properties of USPs have on dynamic train-track interaction and obtained several key results but acknowledged that due to the simplicity of the train-track interaction models used, the influence of track irregularities were

underestimated. From parameter tests for various combinations of dynamic stiffnesses of rail pad, USP and ballast/subgrade, it was determined that sleeper displacements, velocities and accelerations decrease with increasing USP stiffness. Like Witt (2008) and Dahlberg (2010), they also determined that the load at the sleeper-ballast interface increases with increasing USP stiffness. In addition, it was discovered that at frequencies above 250Hz the influence of USPs may not be significant. Overall the study concludes that choice of USP to improve track performance will be dependent on a compromise between the need to reduce sleeper-ballast interface contact forces (or track misalignment) on the one hand and reducing vibration levels on the other. Field studies on the influence of USPs on track dynamics carried out by Schneider et al. (2011) tie in with Johansson et al. (2008a)'s parameter tests in that the former reports higher sleeper accelerations with the use of USPs and this was increasingly so with decreasing USP stiffnesses. Schneider et al. (2011) also reported lower sleeper strains for softer USPs and vice versa for stiffer or no USPs. In summary it was therefore concluded that introducing a resilient layer at the sleeper-ballast interface influences both sleeper accelerations and flexural strains. From this it can again be inferred that the contact forces between the ballast bed and USPs are also influenced by the USP stiffness.

Like Johansson et al. (2008a) parameter studies were also conducted by Müller-Boruttau and Kleinert (2001). However, the difference between the two studies is that whereas Johansson et al. (2008a) considered combinations of dynamic stiffnesses of rail pad, USP and ballast/subgrade, Müller-Boruttau and Kleinert (2001) investigated combinations of rail pad, USPs and sleepers. Additionally, tests by Müller-Boruttau and Kleinert (2001) were for trackbed on concrete formation (i.e. within a tunnel). Despite the slight differences, the two studies demonstrate that increasing USP stiffness results in a decrease in sleeper displacement and an increase in ballast stress. On the other hand, dynamic measurements computed from accelerometer outputs from a field trial by Müller-Boruttau and Kleinert (2001) showed that a combined wider base sleeper and comparatively stiffer USP generates less ballast stress. In addition to this, Müller-Boruttau and Breitsamter (2001) determined using simulation models that attaching USPs to concrete sleepers imitates the compressibility of timber sleepers and that this combination may even outperform timber sleepers. Given the outcome of the studies by Müller-Boruttau and Kleinert (2001) and Müller-Boruttau and Breitsamter (2001), it can be inferred, albeit subtly, that the choice of USP is not a random one, and there are considerations to be made with respect to sleeper material and geometry, notwithstanding other elements of the structure of a track.

The UIC (2009a) field trial report details an investigation of the effect of USPs on track settlement. Using the ORE method, three USP sites were investigated for changes in geometry under increasing axle loads. The effect of the USP on track settlement is relatively uncharted as far as railway research is concerned and will form a significant part of this thesis. For similar track conditions it was concluded that the deterioration rates of the USP tracks were 40 to 70 percent less than that for an untreated track. However when track conditions are dissimilar, Figure 2-30 shows that although an improvement in settlement is the typical trend, this is not always the case, e.g. for the Paternion site; although substantial track improvement is initially apparent, this rapidly diminishes as the tonnage on the track increases.

It has been acknowledged in the UIC (2009a) summary that due to insufficient test data in the results pertaining to settlement, improvements may not be statistically robust enough.

As far as the benefits of USPs for the lateral stability of the track is concerned, it is at times claimed that the indentation of the ballast particles into the USP under loading offers some element of mechanical interlock at the sleeper-ballast interface which improves the lateral stability of the track. Lakušić et al. (2010a) reports a 9% increase in lateral resistance. However, Schneider et al. (2011) report that it has been shown via measurements on different test tracks in Switzerland that lateral resistance without USPs is better than when they are used. Additionally, contrary to the claims of increased lateral resistance, it was observed that when the USP stiffness was decreased, lateral track resistance decreased. These results resonate with the findings of Müller-Boruttau and Kleinert (2001) and who determined no improvement in lateral track resistance irrespective of USP stiffness.

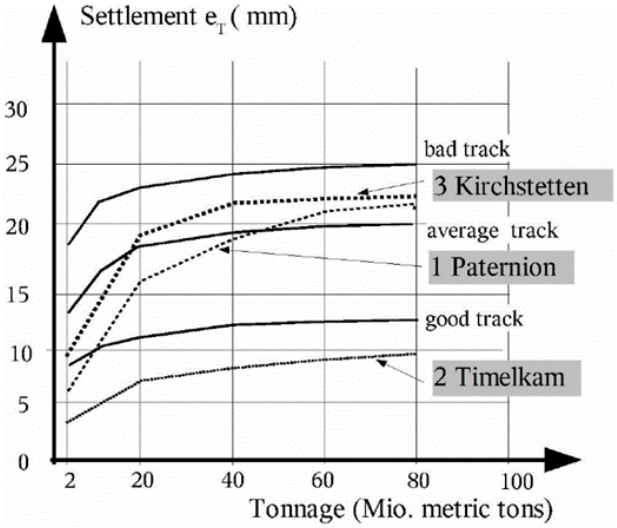


Figure 2-30 Effect of USP on track settlement for three sites UIC (2009a)

2.8 TRACK CATEGORIES AND DESIGN

2.8.1 TRACK CATEGORIES

The requirements for track design, maintenance, renewal, and inspection vary according to the track usage (tonnage) and speed. Railway tracks in the UK are therefore classified into seven categories based on usage (tonnage) and speed (Figure 2-31). The usage of a section of track is measured based on the traffic load carried per annum namely, the Equivalent Million Gross Tonnes per Annum (EMGTPA). The EMGTPA calculation is based on UIC leaflet 714R for "Theoretical Traffic Load" (Railway Group Standard, 1999), in which a fully seated loading condition is assumed for passenger vehicles and the variation in track damage caused by normal traffic has also been taken to account. The speed used to determine the track category should be the maximum permissible or enhanced permissible speed, which is the speed permitted over a section of line that applies to a specific type of train operating at cant deficiencies in excess of those permitted at the permissible speed (Railway Group Standard, 2001).

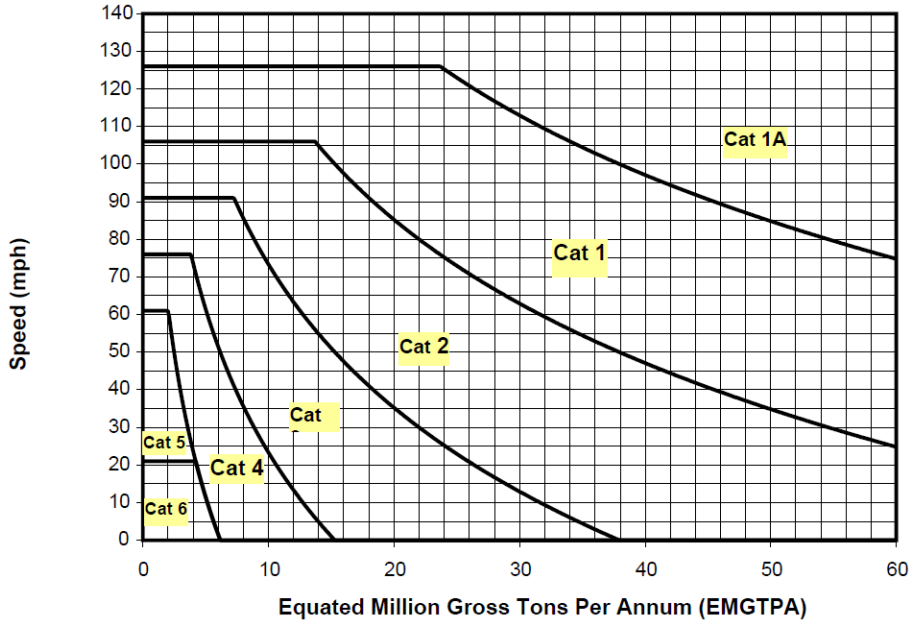


Figure 2-31 Track Categories based on EMGTPA and speed [1], NR/L2/TRK/2102 Issue 8 (NetworkRail, 2016)

[1] The horizontal portions of the track Category boundaries are based on the maximum speeds of the principal traffic types. The curved portions of the track category boundaries up to 25 EMGTP are based on current and past experience. Above 25 EMGTPA, the curves have been extrapolated to reflect the expected track damage for very heavily used lines NETWORKRAIL 2016. Design and construction of track. London: NR/L2/TRK/2102, issu 8, NetworkRail.

The International Union of Railways (UIC, Union Internationale des Chemins de fer) classified railway tracks into six categories (Table 2-4) based on the daily traffic (UIC code 714R) (UIC, 2009b). In this track categorisation, traffic is calculated on the basis of a theoretical traffic load and is the amount of load from the train passing along the rail track and results from the conversion of various train loadings into equivalent passenger train loads in which speed and wear effects have been taken into account.

Table 2-4 UIC Track categorisation (UIC, 2009b)

UIC group	Daily traffic (T_L) (Gross tonnes)
1	T_L > 130,000 tonnes/day
2	80,000 tonnes/day < T_L< 130,000 tonnes/day
3	40,000 tonnes/day < T_L < 80,000 tonnes/day
4	20,000 tonnes/day < T_L < 40,000 tonnes/day
5	5,000 tonnes/day < TL < 20,000 tonnes/
6	TL < 5,000 tonnes/day

2.8.2 TRACK DESIGN

Methods of track design are historically based on provision of a thickness of trackbed layer sufficient to reduce the stresses to an acceptable level, bearable for the subgrade. The first steps to design a railway track are to set the designated EMGTPA (for maintenance planning) and the intended track category. A line speed corresponding to the two aforementioned parameters can be then defined using Figure 2-31. To achieve the defined speed, the trackbed stiffness should be sufficient to provide a support. This can be defined by Dynamic Sleeper Support Stiffness (DSSS). The required DSSS (K) and corresponding depth of trackbed layer are presented in Table 2-5 and Figure 2-32 respectively according to Network Rail Standard NR/L2/TRK/4239 (NetworkRail, 2015), for both existing and new lines. The required DSSS values are the minimum to guarantee that the track quality can be maintained to an adequate standard (NetworkRail, 2015).

An optimum level of trackbed stiffness is an important factor for track quality and reduced maintenance intervention (NetworkRail, 2015). For existing lines the DSSS can be measured using a Falling Weight Deflectometer (FWD). The DSSS according to the Railway Group Standard (2011) (GC/RT5021) is defined as the peak load divided by the peak deflection of the underside of a rail seat area of an unclipped sleeper, when the sleeper is subjected to an approximately sinusoidal pulse load at each rail seat. The pulse load which is applied by the

FWD is representative in magnitude and duration of the passage of a heavy axle load at high speed, typically 20 tonnes at 100 mph.

Table 2-5 Required dynamic sleeper support stiffness, NR/L2/TRK/4239 after (NetworkRail, 2015)

		Minimum Dynamic Sleeper Support Stiffness (K) - kN/mm/sleeper end
Existing lines, line speed <50mph		30
Existing lines, line speed 50 to 125mph	With geogrid reinforcement	30
	Without geogrid reinforcement	60
New track	Up to 100mph	60
	Above 100mph	100

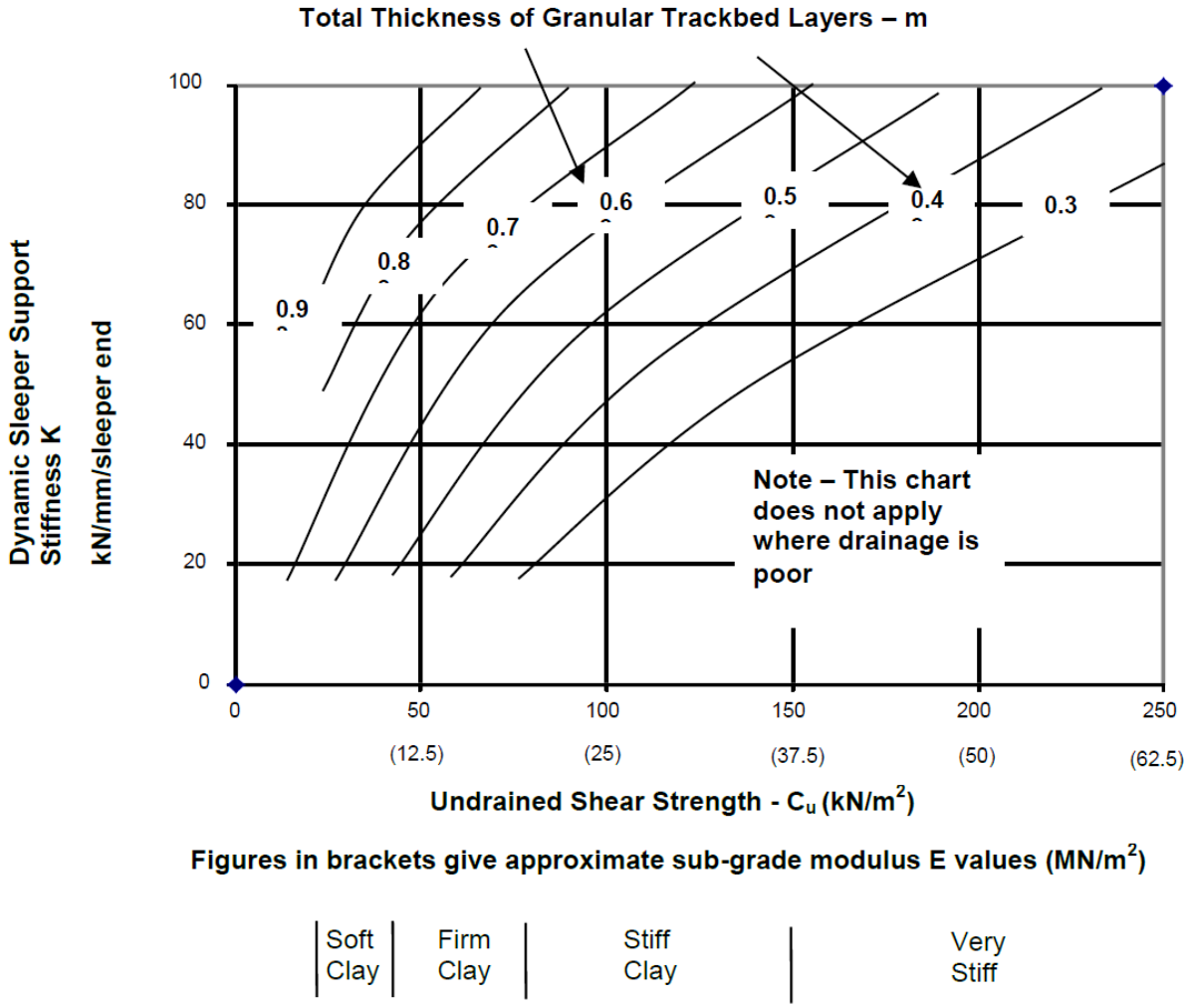


Figure 2-32 Required total depth of granular trackbed layers on cohesive soils, NR/L2/TRK/4239 (NetworkRail, 2015)

2.9 SUMMARY

In chapter 2, the main components of a railway track including substructure and superstructure have been described as well as the main types of force experienced by those components during the in-service life of a railway track.

The mechanism of load transfer from train wheels to the components underneath was explained as well as measurement methods for stresses distributed at the sleeper-ballast and ballast-subgrade interfaces. With regard to measurement of maximum stresses at the formation level, the measurement methods found in the literature were found to be useful to approximate the maximum stresses at formation level; however, the effect of different sleeper types or USP inclusion on the stresses can not be measured by them. This has not been acknowledged in previous experimental studies either.

Factors influencing the settlement of track as a result of permanent deformation of ballast have been explained as well as existing common ballast maintenance methods. However, it has been found in the literature that stoneblowing may create less disturbance and damage to the existing ballast compared to ballast tamping. Therefore improving the settlement performance of a railway track to avoid/minimise the need for ballast tamping and introducing a track structure better coordinated with stoneblowing could be advantageous.

It was found in the literature that the resilient and permanent deformation behaviour of granular material is affected by several factors. Stress was found to be the most influential factor on granular material resilient and permanent deformation behaviour. It was also found in the literature that the initial degree of compaction has important influence on permanent deformation behaviour of granular material and it can help improve the track quality by increasing the trackbed stiffness and reducing the differential settlement.

Different types of railway sleeper being used around the world were described including the advantages and disadvantages of using them in a railway track. It was found in the literature that the selection of sleeper types has not been guided by research-derived evidence and therefore as a result of lack of performance data, there is no clarity for the selection of railway sleepers and over which sleeper performs optimally on track.

Under Sleeper Pads (USP) were described as a relatively recent addition to the railway track structure to improve the overall performance of it. Different types of USP in respect to stiffness,

representing soft and stiff pads, were explained including the benefits of using each and their effect on track performance. The majority of studies found in the literature on use of USPs using numerical modelling and field/laboratory experiments were focused on the effect of USPs on track noise and vibration and the benefits of USPs in transition zones and turnouts. However, the effect of using USP on track vertical settlement especially in combination with different types of sleeper has not been fully researched, nor the optimum stiffness of USP, and possible alteration of the USP with respect to track settlement improvement.

Railway track categories and design have been described. Dynamic Sleeper Support Stiffness (DSSS) as a way of measuring trackbed stiffness has also been explained. In this study DSSS will be calculated for all laboratory full-scale tests and will be compared with the NR standard requirement described in Section 2.8.2.

It should be noted that the literature background for the phase 2 tests of this research will be presented in the corresponding sections for the box test (Chapter 3) and CET (Chapter 4).

3. MONOTONIC LARGE-SCALED TRIAXIAL TEST

3.1 INTRODUCTION

Much research has been done in the past using large triaxial tests on ballast to study its properties e.g. (Shenton, 1974a, Raymond and Bathurst, 1994, Skoglund, 2002, Fair, 2004, Aingaran, 2014). The strength of the ballast material is determined by monotonic (static) traxial tests. In this investigation, a series of drained monotonic triaxial tests were carried out to determine and compare the ballast strength of two different ballast sample sizes including full-size and 2/3 scaled size Network Rail (NR) ballast respectfully.

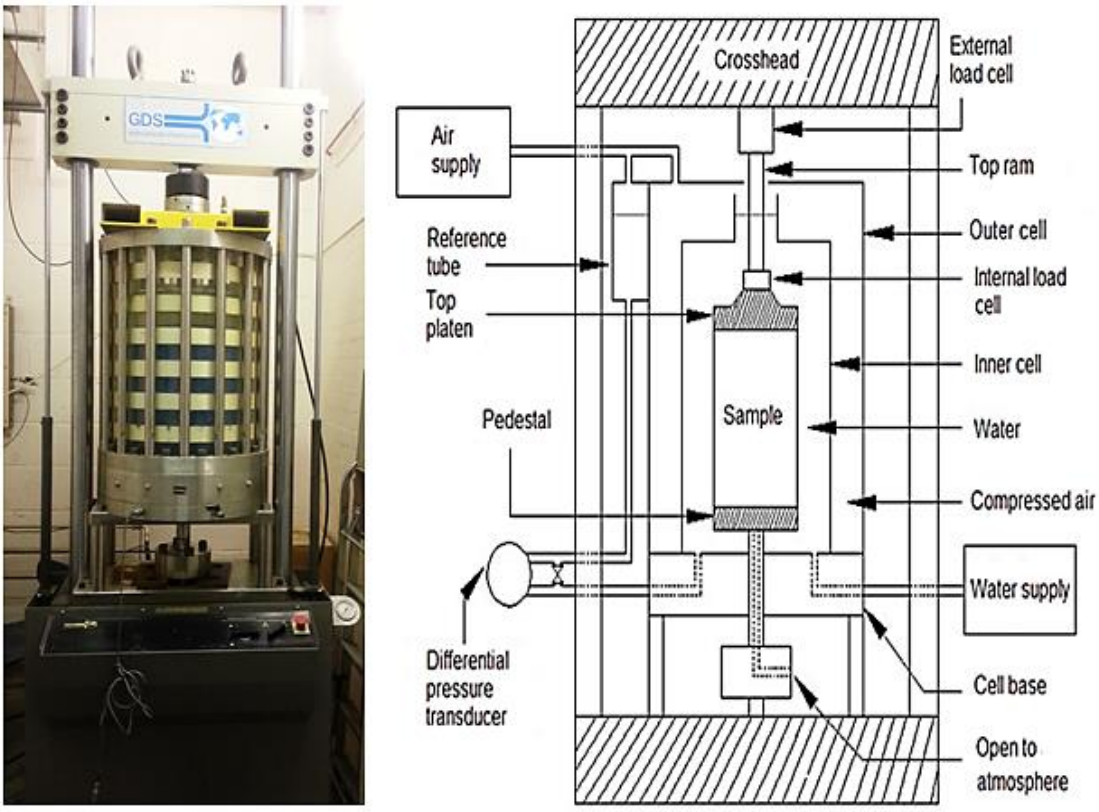
3.2 TRIAXIAL TEST APPARATUS

The large triaxial apparatus used in this project to investigate the strength of ballast under static loading and different stress conditions is shown in Figure 3-1(a). The apparatus was manufactured by GDS Instruments Ltd. The apparatus can accommodate a sample of 300-mm diameter and 450-mm height. A schematic diagram of the apparatus is shown in Figure 3-1(b). The system used in the Nottingham large-scale triaxial apparatus is similar to the one presented in Ng et al. (2002) and is based on a differential pressure measurement for volume change rather than on sample radial displacement measurements. This system was chosen as the large size and angular shape of the ballast causes an irregular outer surface profile making radial displacement measurement inappropriate and unreliable.

As shown in Figure 3-1(b), in this system, water is used in an inner cell in direct contact with the sample. Then water in the neck is pressurised by air (in an outer cell) at a specified pressure. This applies a specified confining pressure to the sample. When the pressure is identical in the inner and outer cell, the volume of water in the inner cell stays constant and the level of water at the neck becomes steady. Any changes in volume of the sample result in equivalent changes in the volume of water in the inner cell .

The change in volume is measured by recording the differential pressure between the inner cell water at the neck and the water inside a reference tube by means of a differential pressure transducer. The levels of the water in both inner cell and reference tube are equal at the start of each test. This means that in case of sample expansion, the level of the water in the inner cell

neck increases whereas it is constant in the reference tube. The recorded readings by the differential transducer are used to calculate the volume changes, knowing the cross-sectional area of the inner cell at the neck and in the reference tube. Moreover, using the axial and volumetric strains, the average radial strain can be determined.



(a) **(b)**

Figure 3-1 (a) The large triaxial apparatus (b) Schematic diagram of the large triaxial apparatus

At the base of the apparatus, an actuator was installed to apply axial load to the sample by means of a shaft connected to a pedestal supporting the sample. The axial load was controlled by two load cells, a 100kN external load cell and a 64kN internal load cell. The internal load cell was only installed to double check the readings from the external load cell as it was independent of the friction of the shaft passing through the cell top. The readings from both loads cells confirmed that the frictional effect was negligible. Therefore, the external load cell readings were used for all tests as both internal and external load cell readings were equal.

A 100-mm Linear Variable Differential Transformer (LVDT) was connected to the loading actuator to measure the axial deformation/displacement.

Further details about the apparatus can be found in Aursudkij (2007). His investigation showed that triaxial tests done by means of the Nottingham apparatus are repeatable and are reproducible with negligible differences.

3.3 TEST SAMPLE

3.3.1 BALLAST MATERIAL AND GRADING

The ballast used for both sample sizes, full-size and 2/3 scale size Network Rail (NR) specification, was a Granite aggregate commonly known as Markfieldite and sourced from Cliffe Hill Quarry in Leicestershire. Markfieldite's primary constituent is microdiorite, an intrusive igneous material of excellent uniformity, strength and durability and thus an appropriate choice to railway ballast material (Whateley and Barrett, 2006). The physical properties of the ballast are shown in Table 3-1. As can be seen in the table, the physical properties of the ballast used for this investigation conform to the NR specifications (NR/L2/TRK/8100) (Network Rail, 2009).

The particle size distribution curves for both aforementioned samples are shown in Figure 3-2. As can be seen in the figure, the particle size distribution of the full-size sample is within the NR specifications. The preparation of the ballast samples is explained in the next section.

Table 3-1 The physical properties of Cliffe Hill Quarry granite ballast

Physical Property	Test Standard	Test Result	Network Rail Specification - Max
***Los Angeles Abrasion**	BS EN 1097-2	14	20
***Micro Deval**	BS EN 1097-1	6	7%
Flakiness Index	BS EN 933 -3	0.18	35%
Length Index	**	0.02	4%
Fine Particle Content	BS EN 933 – 1	0.4	0.6%
Fines Content	BS EN 933 – 1	0.3	0.5%

*Los Angeles & Micro Deval testing modified in accordance with prEN 13450 Annex C and Annex E.
** Length Index testing carried out in accordance with Appendix E of RailTrack Specification RT/CE/S/006.

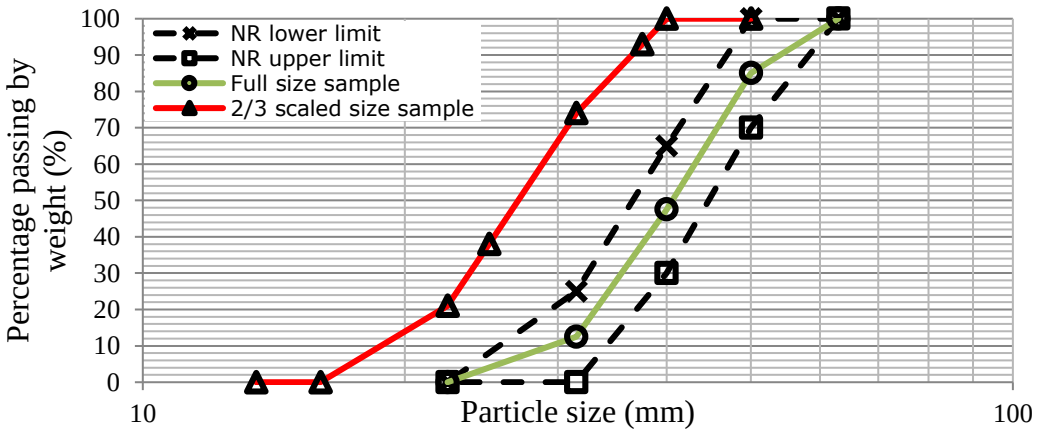


Figure 3-2 Particle size distribution curve for full size and 2/3 scaled size sample

3.3.2 PREPARATION OF BALLAST SAMPLES

As mentioned earlier the cylindrical ballast sample in the triaxial apparatus has a height H= 450-mm, a diameter D=300-mm. This results in an aspect ratio H/D=1.5. The ratio suggested by Bishop and Green (1965) for cohesionless soils is H/D=2. According to them, the end friction leads to a higher value for the angle of shearing resistance. In the case of the ballast triaxial test the diameter would not be identical all over the sample height and this would affect the volumetric strain calculation. Therefore, the volumetric strain was measured by using a differential pressure transducer instead of using the diametral strain. According to Aursudkij (2007), using an aspect ratio of 1.5 led to a simpler and more economical design for the aforementioned apparatus cell.

The cell diameter also should be large enough to accommodate adequately a representative sample. As reported by Fair (2004), the D/d_{max} ratios (where D is specimen diameter and d_{max} is the maximum particle size) used by different researches were between 4.7 and 10. This suggests that both full-size and 2/3 scale size NR samples were within the reasonable range, corresponding to D/d_{max} of 300/50=6 and 300/37.5=8, according to Fair (2004). This is also consistent with the suggested minimum value of 5-7 by Skoglund (2002) for the full-size NR sample and the 2/3 scaled ballast.

As mentioned in section 3.3.1, a similar granite ballast material was used for all the tests in order to ensure the repeatability and validity of the results. To prepare the samples, ballast was sieved and blended by weight back into the designed gradation. The resulting material was mixed to ensure a random distribution of particle sizes. The gradation for the full size sample conforms to Network Rail specifications (NR/L2/TRK/8100) (Network Rail, 2009). The graduation of 2/3 scale sample was designed based on the closest available sieves to the actual 2/3 NR graduation. As shown in Figure 3-2, the full-size NR ballast graduation ranges between 22.4-mm and 50-mm and it varies between 16-mm and 37.5-mm for the 2/3 scale NR ballast. Table 3-2 and Table 3-3 show the proportion of ballast sizes used for both aforementioned graduation.

Table 3-2 Proportion of ballast sizes used for full-size NR ballast

Network Rail full size sample		% by weight for interval
% passing by weight	sieve size (mm)	
85	50	15
47.5	40	37.5
12.5	31.5	35
1.5	22.4	11
0	<22.4	1.5

Table 3-3 Proportion of ballast sizes used for 2/3 scaled size NR ballast

2/3 scaled ballast sample		% by weight for interval
% passing by weight	sieve size (mm)	
93	37.5	7
74	31.5	19
38	25	36
21	22.4	17
0	16	21

Since a ballast sample is made of large irregularly shaped particles, there are large gaps (voids) in between them. In order to mimic the density of ballast in the field, ballast samples were compacted using a vibrating plate compactor. To achieve this, the sample was constructed in three equal layers of 150-mm thickness each and compacted for 30 seconds. After the 450-mm depth was achieved, the surface was manually levelled to ensure sufficient contact between ballast particles and the top cap.

The membrane used for the triaxial tests was a 4-mm latex membrane manufactured by Precision Dippings Marketing Ltd. Although using a thick membrane offers slightly extra confining pressure to the triaxial test sample, previous research done by Indraratna et al. (1998) and Key (1998) showed that the confinement provided by a thick membrane was negligible compared to the applied confining pressure. According to Indraratna et al. (1998), in the case of a test with 1kPa confining pressure, the maximum correction was less than 8%. Membrane penetration is also another matter that should to be considered in addition to the membrane thickness. It can cause errors in the effective confining stress during an undrained triaxial test on a coarse grained granular material (Kramer et al., 1990, Miura and Kawamura, 1996). However, since in this project relatively dry material was used, the error caused by membrane penetration was generally small. This is also supported by Hjortnaes-Pedersen and Molenkamp (1982). Their research showed that membrane penetration affected the deviator strain by 0.0003 (mm/mm). Therefore, the triaxial test results in this investigation were not corrected for membrane penetration. Membrane penetration can also affect the reading for volume change if there are pressure cell changes, this was not the case in this study.

Once the ballast sample is prepared it needs to be placed in the membrane within the split cell mould and compacted. The quantity of ballast used needs to be determined so that the bulk/dry density and other related soil properties can be calculated. Typically the membrane is placed within the split cell mould and held against the sides using suction. The general method of triaxial test preparation in this study was as follows:

A 4-mm thick latex membrane was pulled up and rolled over a steel split mould (with an internal diameter of 304-mm) and held in place by two neoprene O-rings at the top and the base. Additionally, a seal tape and steel O-ring were also later used on top of the neoprene O-rings to avoid air leaks. To reduce the friction between bottom and top of specimen, and the base and top, two circular latex sheets sandwiching silicon grease were place at each end.

As mentioned before, the split mould was filled by ballast in three equal thickness layers and each layer was compacted for 30sec using a vibrating plate compactor. The top of the specimen was levelled by hand placement of particles up to the top of the mould. The mass of the specimen was then calculated from the remaining test material. Specimens prepared by this compaction method had an average bulk density of 1525 kg/m^3.

Throughout the building of the sample, the membrane was held against the steel compaction mould. The top cap was then connected on the top of the sample. Once the ballast sample was sealed, the compaction mould was removed while a temporary internal suction of approximately 20kPa was applied to the specimen. This was through the valve at the bottom of the cell base using a vacuum pump. The suction helding the specimen in place after the compaction mould was removed.

The internal and external chambers were then connected and the cell was positioned and bolted onto the loading frame. Subsequently, the cell was filled with water. When the water reached the sample mid height the pressure sensors were zeroed. This was done to measure/collect pressure readings at the centre of the sample. Once the cell was filled with water, a constant confining air pressure of at least 15kPa was applied to the external chamber. This confined the sample and allowed the suction to be removed. The confining air pressure was measured by using a pressure gauge and the pressure system connected to a computer. These were both set to read zero when the water was at mid-height of the sample. The pressure control was provided via an external pressure control system. The drainage of the specimen was controlled by valves under the apparatus base plate. Throughout the test, the valves were opened to atmospheric pressure and consequently the pressure inside the specimen was equal to atmospheric pressure.

3.4 TEST PROCEDURES AND PROGRAMME

The in-situ confining pressure of self-standing ballast perpendicular to the rail is approximately 5-50kPa, as reported by Selig and Alva-Hurtado (1982). This was based on an assumed ballast

coefficient of lateral earth pressure. The box test study by Selig and Waters (1994) also showed a horizontal stress of approximately 15kPa to 60kPa in the unloaded and loaded conditions (Figure 3-3).

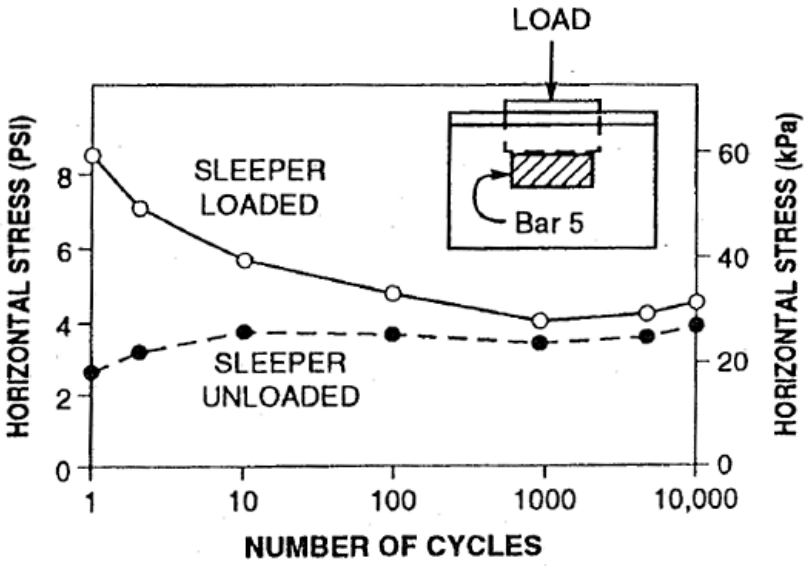


Figure 3-3 Effect of repeated load on horizontal stress in box test (Selig and Waters, 1994)

In this investigation, monotonic triaxial tests were carried out to determine and compare the ballast strength of two different ballast sample sizes including full-size and 2/3 scale size NR ballast. Therefore, in order to mimic in-situ confining pressure conditions, three monotonic tests were carried out for each aforementioned sample in a displacement controlled manner with different cell pressures of 15kPa, 30kPa and 60kPa. A summary of the triaxial test plan is presented in Table 3-4. The dry unit weight of the ballast samples presented in Table 3-4 was on average 1525 kN/m^3, calculated from the mass of the ballast used and the dimensions of the mould.

All the traiaxial tests were carried out up to approximately 12% axial strain. This was done for safety reasons and to avoid the sample touching the inner cell in accordance with previous experience by Aursudkij (2007) and Shi (2009).

Once the sample preparation and instrumentation set-up were done, the sample was loaded at a rate of 0.5mm/min (~0.11% per minute) up to approximately 12% axial strain (54-mm).

Table 3-4 Summary of triaxial test plan

	Confining Pressure kPa	Initial Weight (kg)	Compaction Density (kg/m^3)	Initial Voids Ratio
2/3 Scale size Ballast	15	49	1540	0.54
	30	49.5	1556	0.55
	60	48	1509	0.53
Full-Size Ballast	15	49.6	1559	0.55
	30	49.8	1565	0.55
	60	45.9	1443	0.51

It should be noted that before starting the tests, all parts of the triaxial apparatus were precisely checked for functionality and calibrated by an engineer from GDS Instruments Ltd doing a two-day visit. As a result of it, some parts were replaced (including the internal load cell) or if possible were repaired.

3.5 DATA ANALYSIS METHODS

The axial stress, axial strain and volumetric strain were recorded at a rate of 0.2Hz (every 5 seconds) for every triaxial test. The recorded file then was transferred to an excel spreadsheet for the purpose of post-processing. Using the recorded data, peak compressive strength and friction angle were calculated as described in the following subsections.

3.5.1 PEAK COMPRESSIVE STRENGTH

The peak compressive strength was the greatest axial compressive stress throughout the test. This value is dependent on the value of the confining pressure.

3.5.2 FRICTION ANGLE

Using the peak principal stress ratio from the triaxial test results and the Mohr-Coulomb failure criterion, it is possible to calculate the peak friction angle of ballast, see Equation 3-1 below:

$$\left(\frac{\sigma_1'}{\sigma_3'}\right)_P = \frac{1+\sin\phi_P}{1-\sin\phi_P} \qquad \textbf{Equation 3-1}$$

where:

ϕ_P = peak friction angle
σ_1' = major principal stress
σ_3' = minor principal stress

The value of cohesion was assumed to be zero. Knowing $\sigma_1' = q_{peak} + \sigma_3'$ (where q_{peak} is maximum deviator stress), the following equation can be derived:

$$\phi_P = \sin^{-1}(q_{peak}/(2\sigma'_3 + q_{peak})) \qquad \textbf{Equation 3-2}$$

As shown in equation 3-2, the peak friction angle is related to the maximum deviator stress.

3.6 TEST RESULTS

A total of six monotonic triaxial tests were undertaken, using the confining pressures of 15kPa, 30kPa, and 60kPa. All tests were undertaken up to 12% axial strains. Figure 3-4 and Figure 3-5 present respectively the deviator stress variations ($q = \sigma'_1 - \sigma'_3$) and volumetric strain ($\varepsilon_v = \Delta V/V_0$) against axial strain ($\varepsilon_a = \Delta h/h_0$) of all aforementioned ballast samples under monotonic triaxial loading.

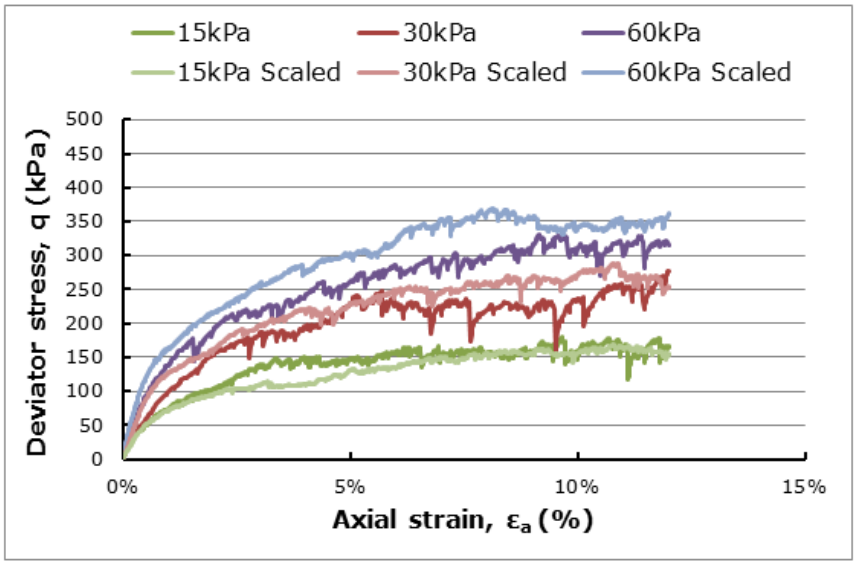


Figure 3-4 Deviator stress – axial strain behaviour of ballast under monotonic loading

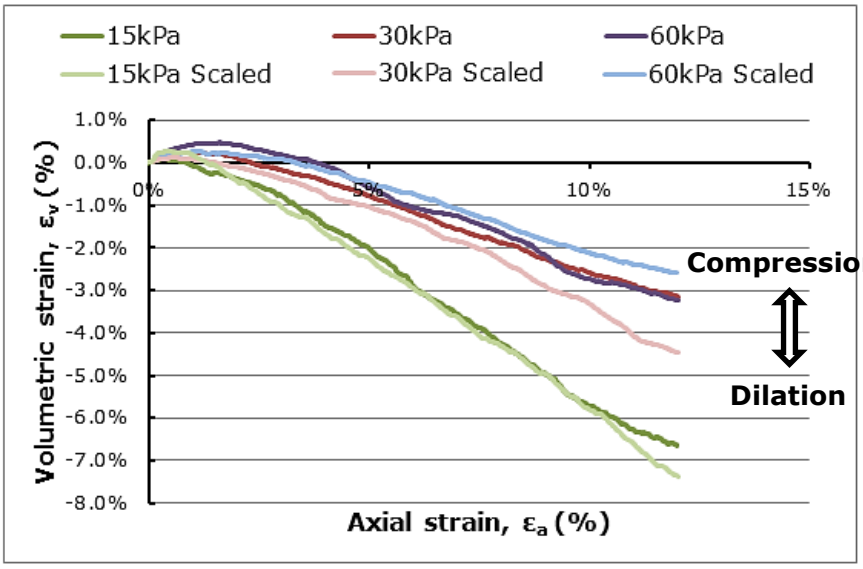


Figure 3-5 Volume change behaviour of ballast under monotonic loading

As can be seen in Figure 3-4, an increase in confining pressure increases the deviator stress at a given strain level for both full-size and 2/3 scale size ballast samples. It can also be seen in the figure that in a higher confining pressure there are more fluctuations compare to a lower confining pressure, which can be as a result of higher particle breakage in the case of higher confining pressure. The deviator stress ultimately achieves a peak level and this peak stress (q_{peak}) is considered as failure for the ballast samples. As expected, a higher confining pressure results in a higher peak deviator stress. The values of q_{peak} at different confining pressures for both sample sizes are presented in the Table 3-5. The test results showed that the value of peak deviator stress (q_{peak}) for the 2/3 scaled size sample is similar to (in the case of 15kPa confining pressure) or slightly higher than the for full size sample (in the case of 30kPa and 60kPa confining pressure).

Table 3-5 Peak deviator stress under monotonic loading

	Confining Pressure (kPa)	Peak deviator Stress (kPa)
2/3 scaled size ballast	15	168
	30	287
	60	370
Full size ballast	15	179
	30	277
	60	330

In Figure 3-5, for both sample sizes, the volume of the ballast decreases initially (compression, $\varepsilon_v \geq 0$) and as tests continue the volumetric strain changed towards dilation ($\varepsilon_v \leq 0$). The test with least confining pressure (15kPa in this investigation) has the largest dilation (least compression) corresponding to its largest $(q/p')_{max}$ as can be seen in Figure 3-6 (where p' is the mean effective stress). The changes in deviator stress ratio (q/p') versus increasing axial strain (ε_a) are presented in Figure 3-6. As can be seen in the figure, there is a prompt initial increase in the deviator stress ratio and then at higher strain levels it stays steady. It can also be seen in the figure that a lower confining pressure resulted in a higher peak stress ratio regardless of sample size.

The q-p' stress paths outlining the triaxial tests for both sample sizes are presented in Figure 3-7. The chosen stress paths for the test represent the in-situ stress conditions of the ballast perpendicular to the rail. It is notable that an increase in confining pressure increases the mean effective stress which results in higher peak deviator stress.

Table 3-6 presents the peak friction angles of both ballast sample sizes under monotonic triaxial loading. It can be seen in the table that the peak friction angle decreases as the confining pressure increases regardless of ballast sample sizes. Similar to the results for the peak deviator stress (q_{peak}), the test results showed that the value of peak friction angle for the 2/3 scale size sample is comparable to (in the case of 15kPa confining pressure) or slightly higher than the full size sample (in the case of 30kPa and 60kPa confining pressure). The slight difference is because of asperities breaking for large ballast particles which are more difficult for smaller particles. The resistance to fracture is inversely proportional to the size.

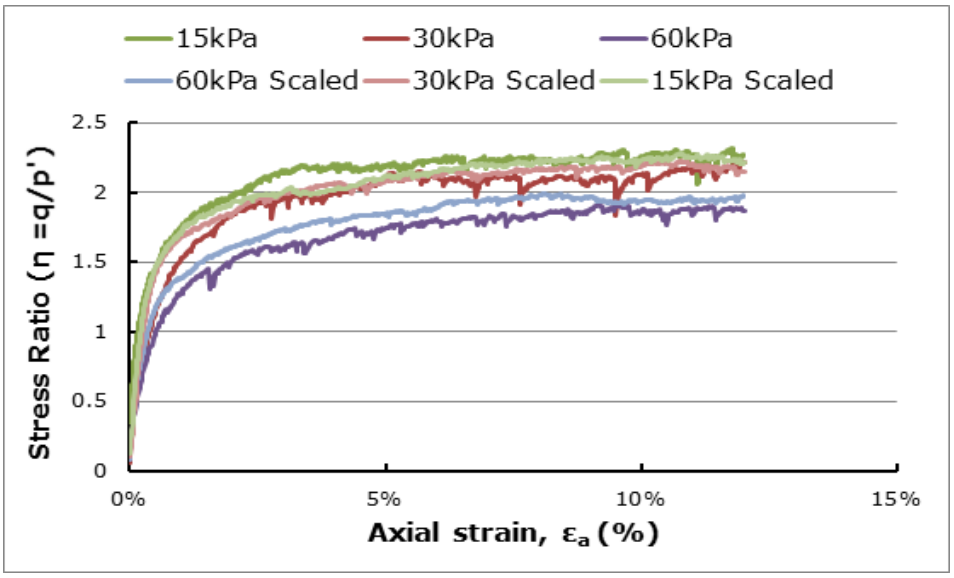


Figure 3-6 Stress ratio (η) versus axial strain (ε_a) plot for ballast under monotonic loading

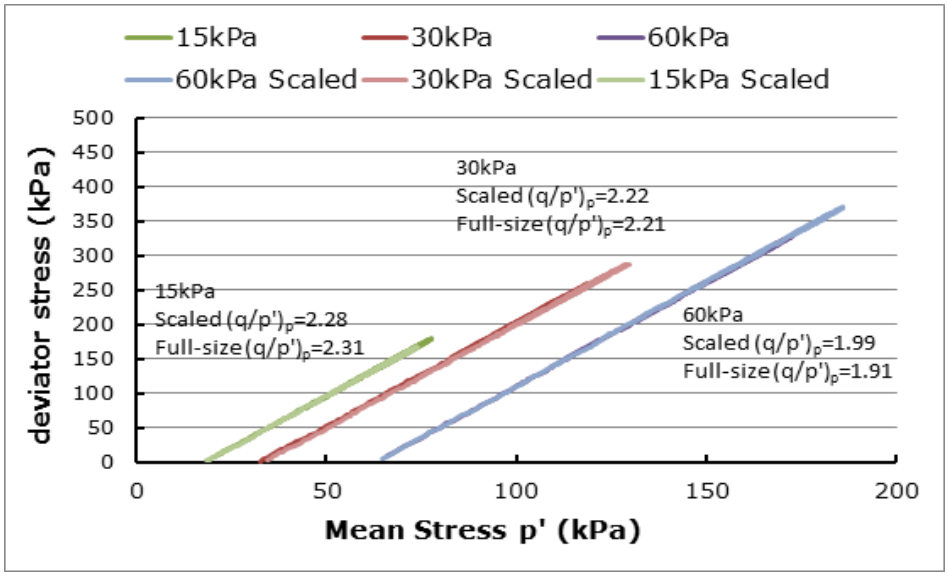


Figure 3-7 Deviator stress vs. mean effective stress in monotonic triaxial tests

Table 3-6 Peak friction angle for both sample sizes

	Confining Pressure (kPa)	Peak Friction Angle (degree)
2/3 scale size ballast	15	55.7
	30	54.3
	60	48.4
Full size ballast	15	56.6
	30	53.9
	60	46.4

3.7 SUMMARY

The aim of large-scale triaxial tests was to determine and compare the ballast strength of two different ballast sample sizes including full-size and 2/3 scale size NR ballast. Conventionally, the triaxial apparatus measures axial and radial displacement. However, in case of the Nottingham large-scale triaxial apparatus, axial displacement and volume change were measured instead. A differential pressure transducer was used to measure the volume changes and the data was compared with the water level reading in the neck of the inner cell.

In total six monotonic triaxial tests using confining pressure of 15kPa, 30kPa and 60kPa were conducted. Peak strength and peak angle of friction were calculated for both full-size and 2/3 scale size NR samples. The test results showed that the values of peak strength and peak angle of friction for 2/3 scale size sample were generally comparable to or slightly higher than these of the full-size sample.

4. BOX TEST

4.1 INTRODUCTION

Laboratory experimentation is often compared to assessing in-service railway track performance for research purposes, because it is a relatively low-cost but effective method of replicating field conditions. In addition, using laboratory experimentation provides an opportunity for simulating different conditions, controlling a range of variables and accessing extensive readily available experimental resources.

In this chapter, preliminary tests are described, undertaken using a box test apparatus prior to laboratory full-scale tests. These were carried out to obtain an insight into the performance of different sleeper section types with regard to settlement and trackbed stiffness with and without use of Under Sleeper Pads (USPs). Once the preliminary tests were completed (box tests phase 1), full-scale tests were undertaken using the Nottingham Railway Test Facility (RTF), the results and details of which will be presented in chapter 6. Comparison of test results from preliminary box tests and full-scale RTF tests showed consistency in all test cases. This demonstrated the reliability of the box test apparatus. Therefore, additional tests were conducted using the box test apparatus (box tests phase 2). The aim was to use the box test to allow a relatively large number of simplified tests to be carried out within a reasonable cost and time frame. The tests in phase 2 involved use of frictional sleepers and USPs, ballast reinforcement using polystyrene particles, sleeper section types and USPs using 2/3 scale ballast, use of two-layered ballast systems, and use of USP types with various stiffnesses. The details and results of the aforementioned tests will be presented later in this chapter.

A description of the box test apparatus, materials used and test procedures are also presented in this chapter followed by presentation of key outcomes of tests undertaken.

4.2 BOX TEST APPARATUS

The box test is a relatively quick and reliable laboratory experimentation equipment replicating railway track conditions in a controlled environment. It was therefore considered suitable for undertaking the preliminary tests in this study (details are described later in this chapter).

One of the first attempts of using the box test was by Norman (1982) at the University of Massachusetts for strength and settlement testing of railway ballast. Box tests have also been used in several studies since Norman with different box dimensions for investigation of a range of railway track variables (e.g. effect of load level, subgrade stiffness, and wet and dry ballast (Raymond and Bathurst, 1987), effect of load level and ballast fouling (Selig and Devulapally, 1991), and effect of stoneblowing (Anderson and Key, 2000)). All aforementioned studies demonstrated the box test to be a reliable test to simulate the impact of different track variables with remarkably consistent results and good repeatability.

The box test apparatus used in this study was basically a box with dimensions 700-mm long, 300-mm wide and 450-mm deep representing a confined element of a loaded track as highlighted in Figure 4-1. Previous studies (McDowell et al., 2005, McDowell and Stickley, 2006) have shown the box test used in this study to be a useful method of simulating realistic track loads on railway ballast in order to assess the effect of specific track variables. Besides a wooden base and one reinforced Perspex face reserved for observation, the remainder of the box is of case-hardened steel.

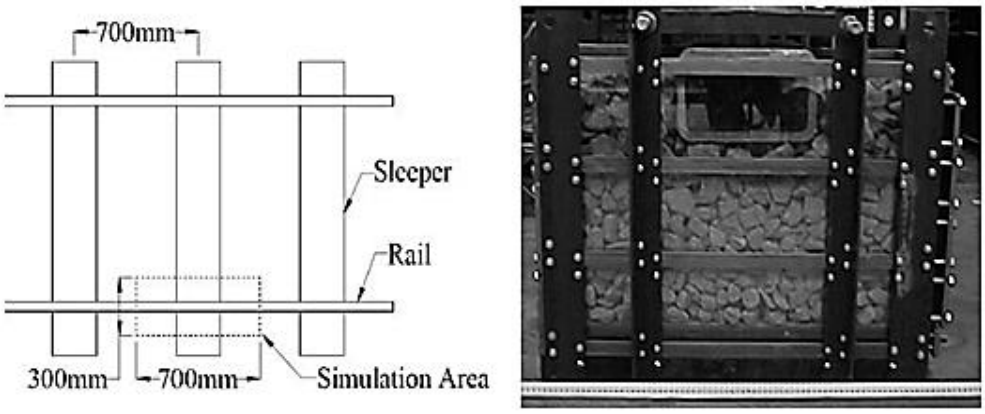


Figure 4-1 Box test apparatus

A 6-mm thick rubber mat is placed on top of the wooden base of the box to simulate a typical subgrade reaction. The suitability of a rubber mat for this purpose was assessed by compressing a 3-mm thick mat with an incremental static load on a 100-cm^2 platen area. For a maximum stress of 190kPa at the base of the box and a deflection of 0.25mm as per the aforementioned compression test, if the platen is assumed to be a rigid foundation on an infinite elastic half-space, an equivalent elastic subgrade modulus can be obtained by solving for E_s in equation 4-1 (Brown, 1969b, Brown, 1969a):

$$\boldsymbol{\rho = \frac{qdI(1-v^2)}{E_s}}$$ **Equation 4-1**

where:

ρ =foundation settlement; q = applied stress; d = circular area diameter; υ = poisons ratio of the soil; E_s = equivalent elastic soil modulus; and I = displacement influence factor.

Taking I and υ to be 0.79 (Schleicher, 1926) and 0.2 respectively, an equivalent elastic modulus of 32.8MPa is estimated for the formation.

4.3 MATERIALS

4.3.1 SLEEPER SECTIONS

Sleeper section types used to conduct box test experiments were steel, plastic, timber and 'concrete' sleeper sections. The plastic and timber sleeper sections (Figure 4-2) were derived (by cutting) from full-sized sleepers but due to technical limitations a rectangular steel box and a U-shaped steel section were used to represent concrete and steel sleepers respectively. However after conducting the full-scale tests, results showed that the rectangular steel box was not a proper representation of a concrete sleeper; therefore, it was suggested to cast a concrete section (Figure 4-3) to have a close match to a concrete sleeper in terms of material and geometry. However, the results from testing the rectangular steel box are still presented alongside the rest of the sleeper sections because it is still considered as a close representation of a hollow sleeper used in the industry for securing cable crossings (Figure 4-4). The rectangular steel box and a U-shaped steel section are shown in Figure 4-4. Moreover, the dimensions of the sections investigated are given in Table 4-1 below.

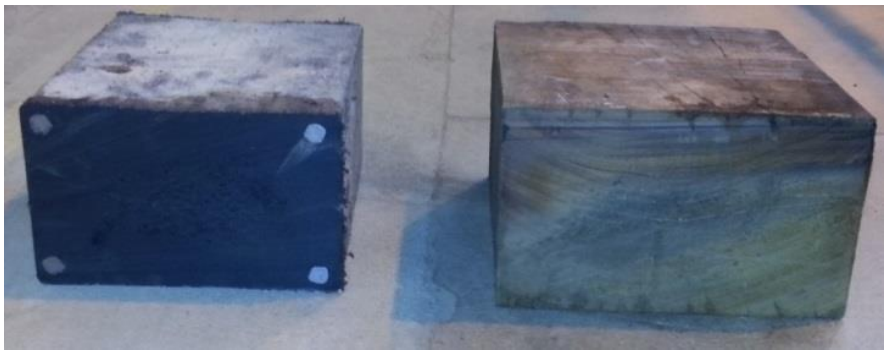
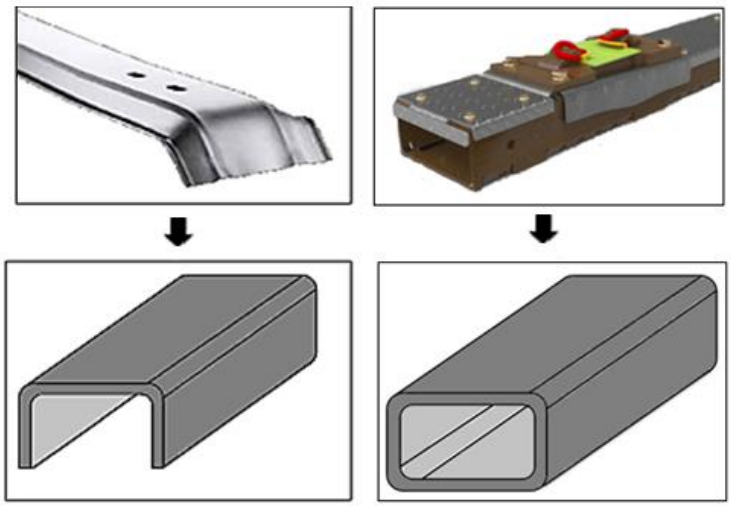


Figure 4-2 Plastic and timber sleeper sections



Figure 4-3 Concrete section



(a) **(b)**

Figure 4-4 (a) Inverted U-shape sleeper section (b) Rectangular sleeper section

Table 4-1 Dimensions of representative sleeper sections

	Width (mm)	Depth (mm)	Length (mm)	Weight (kg)	Wall thickness (mm)	Percentage of full size (%)	
						length	weight
U section	250	150	300	13	10	12.5	15.5
Rectangular Steel box	250	150	300	17.5	10	12	5.8
Concrete	285	210	300	43	-	12	14
Plastic	250	150	300	11	-	11.5	11.1
Timber	290	170	300	16	-	11.5	9.6

4.3.2 UNDER SLEEPER PADS

With the exception of the U-section sleeper, each sleeper section was additionally tested in turn with a USP affixed to it. The under sleeper pads used in the investigations were provided courtesy of Tiflex limited. Two types of pads were investigated: a 4mm thick (Type1) USP and

a 9mm thick (Type2) USP, representing stiff and soft variants respectively. Images and details of the USP types investigated are shown in Figure 4-5 and Table 4-2 respectively.



(a) **(b)**

Figure 4-5 Under Sleeper Pads (a) type1 (stiff) (b) type2 (soft)

Table 4-2 Under Sleeper Pad specifications

Property	Under Sleeper Pad variant	
	Type1 (stiff)	**Type2 (soft)**
Technical ID	FC500	FC208GF
Thickness	4 mm	9 mm
Weight	6 kg/m^2	5.6 kg/m^2
Stiffness (C_{Stat})	0.228-0.311 N/mm^3	0.079-0.105 N/mm^3
Core material	Trackelast FC500	Bonded cork

4.3.3 BALLAST

The ballast source and specification for the box tests were the same as the full-size and 2/3 scale size ballast that has been previously described for the large-scale triaxial tests in the previous chapter.

4.4 SAMPLE PREPARATION AND TESTING PROCEDURE

For each test case, the ballast bed was constructed in 100-mm layers using a vibrating plate compactor and by manually levelling the ballast surface till a 300-mm deep ballast bed was achieved. An initial voids ratio (based on a typical granite ballast density of 2650 kg/m^3) of 0.67 was also ensured in each test (using the same amount of ballast and compaction underneath the sleeper section). Apart from the U-section sleeper, sleeper sections were positioned centrally and at the top of the ballast bed and the vacant cribs filled with ballast for lateral stability. The depth of crib ballast was dependent on the sleeper section height. The U-section sleeper on the other hand was positioned on a 450-mm thick ballast bed and then pushed in with an incremental static load till its hollow underside was filled with ballast. This was done to simulate the typical industry practice of ensuring steel sleepers are properly embedded using a passing ballast train. Although the application of a an incremental static preload during the

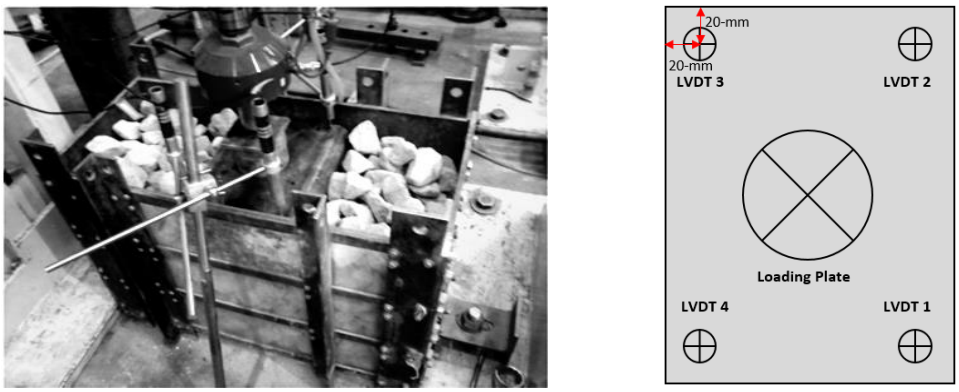
installation of the U-section sleeper is likely to have resulted in early stiffening of the ballast ahead of cyclic loading, it is in keeping with standard industry practice.

The sleeper sections for the tests including USPs utilised an epoxy resin which was allowed to cure for a minimum of 12hours as a means of bonding the two elements. Separate sections of the same sleeper type were used for each USP type to ensure that the contact area would not be compromised by the residue from a previous test.

To avoid loss of contact between the loading piston and the sleeper section, a seating load of 3kN was first applied to the sleeper section at the start of each test. The ballast was then cyclically loaded for almost 30,000 cycles between 3kN and 40kN by transmitting sinusoidal pulses at a frequency of 3Hz. Although a sleeper in the field over its lifetime is subjected to more traffic compared to that used in this study, 30,000 cycles was chosen with consideration given to key differences between the field and the laboratory apparatus. Unlike in the field where ballast can be considered to be poorly confined (laterally) the box test is highly confined. In this regard it is reasonable to expect the ballast in the box test to attain close to a pure resilient response much earlier. The results showed this to be justified, with little change in the settlements that developed in the later stages of each test. Moreover, the results showed no deviation in the order of settlements between sleepers beyond 1000 cycles indicating that for the purposes of comparison 30,000 cycles was adequate.

Past studies (Lekarp et al., 2000a) have shown that the effect of frequency of loading on the resilient behaviour of granular materials is negligible or non-existent unless an increased frequency due to wheel flats or rail imperfections alters existing dynamic loading conditions. The maximum sleeper-ballast interface stress varied between the tests according to sleeper section dimensions which made different sleeper contact areas with ballast (between 460kPa and 533kPa) although the loading conditions were constant for all tests. Vertical sleeper movement over the test duration was tracked continuously with four Linealy Variable Differential Transformers (LVDTs) mounted at the four corners of each sleeper section tested as shown in Figure 4-6. This arrangement was considered appropriate to allow for tilt that could occur because of the inherently random ballast structure. The LVDTs were LDC1000A series and they were purchased form RDP Electronics and each had a permissible stroke range of ±25-mm. All the LVTDs were tested for linearity using the setup shown in the Figure 4-7.

A plot example for calibration of one of the LVDTs used is presented in Figure 4-8. The load cell of the box test actuator was also calibrated and showed a linear behaviour.



(a) **(b)**

Figure 4-6 (a) General setup of the box test (b)Plan view of mounted LVTDs on sleeper section

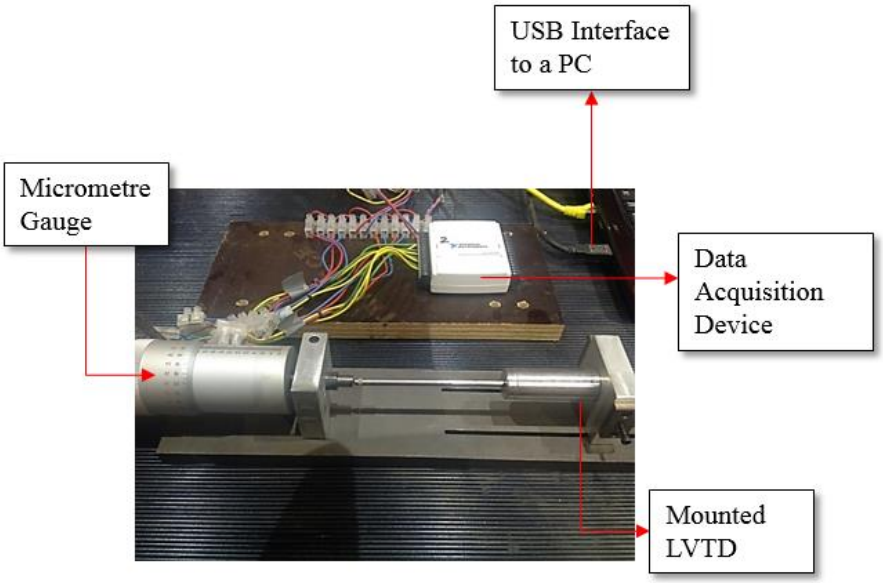


Figure 4-7 Calibration setup for LVDTs

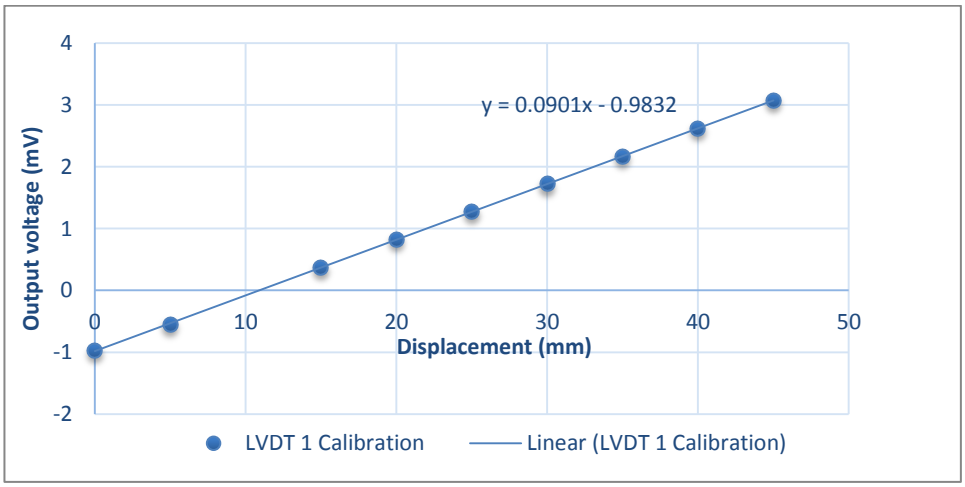


Figure 4-8 LVDT Calibration Plot Example

Test data was acquired and part-analysed using a graphical programme written with LabVIEW software. The front panel view of the LabVIEW programme used is shown Figure 4-9. The front panel showed a real-time digital display of the LVDT outputs during the test. The sleeper section displacements were sampled at a rate of 60Hz, equating to approximately 20 data points per cycle per transducer. This rate of data sampling conformed to the Nyquist criterion (Shannon, 1949). According to him, to have enough data to capture a full cycle (including peaks and troughs), a signal should be sampled no less than twice the highest frequency of interest within the signal.

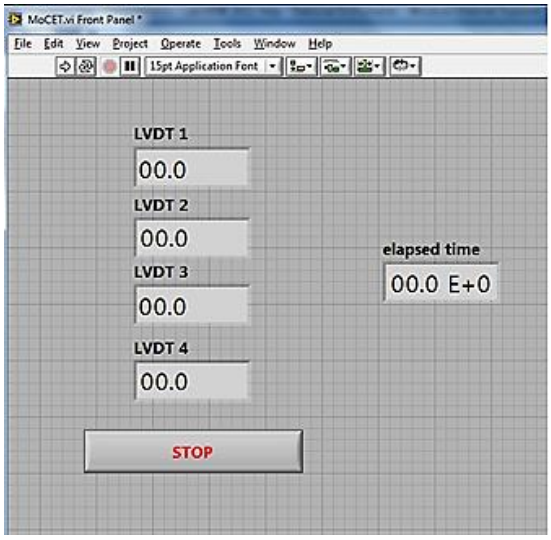


Figure 4-9 LabVIEW front panel

An average minimum displacement value was obtained for each loading cycle for the four LVDTs for the test duration. The general arrangement of the box test setup is shown in Figure 4-6 (a).

4.5 TESTING PROGRAMME

A total of 41 unique box test set-ups were undertaken over two phases in this chapter (Table 4-3). All the tests were repeated twice (minimum) in order to verify the repeatability of the results and avoid any systematic and random errors. In all cases, the difference was less than 6% except in the case of the U-section where it was 9%. This could be due to the complex installation method of the U-section compared to the rest of the sleeper sections. The test results presented in this chapter represent an average value from the repeated test results in each case. In total 82 box tests were conducted including 26 tests in phase 1 and 56 tests in phase 2 of the box test study. In phase 1, the performance of different sleeper section types with and without

USPs was studied using full-size ballast. Phase 2 was conducted after full-scale tests undertaken on standard sleeper types with and without USPs using the Nottingham Railway Test Facility (RTF). Once the consistency of the box test and the full-scale results was confirmed, phase 2 tests were conducted using the box test. Phase 2 involved use of frictional sleepers and USPs, ballast reinforcement using expanded polystyrene particles (small size: 30-mm x 30-mm x 30-mm, large size: 60-mm x 60-mm x 60-mm), sleeper section types and USPs using 2/3 scale ballast, use of two-layered ballast systems, and use of USP types with various stiffnesses. The Programme of all tests undertaken using the box test apparatus is presented in Table 4-3.

Table 4-3 Programme of tests undertaken using box test apparatus

Phase	Test Title	Test description
Phase 1: Initial Experiments	Sleeper Section Type	**Sleeper section used:** hollow, concrete, U-section , plastic and timber
	Under Sleeper Pads	**USP type1 with**: hollow, concrete, plastic and timber section **USP type 2 with:** hollow, concrete, plastic and timber section
Phase 2: Additional Experiments	Frictional Sleeper & USPs	**Concrete section with:** mesh only USP-Mesh USP-Geo
	Ballast Reinforcement (Expanded Polystyrene Particle)	**Concrete section with polystyrene particle:** large %10 small %10 large %5 small %5 large %2.5 small %2.5 large %1 small %1
	2/3 Scaled Ballast	**Sleeper section used:** concrete, plastic and timber **USP Type1 with:** concrete, plastic and timber section **USP Type 2 with**: concrete, plastic and timber section
	Two-Layered Ballast System	**Concrete section with:** ballast sample1 ballast sample2
	Multi-Layer USP	**Concrete section with:** single-layer USP type1 double-layered USP type1 triple-layered USP type1 single-layer USP type2 double-layered USP type2 triple-layered USP type2

4.6 FRICTION TEST

To help interpreting the settlement results from the box test, friction tests were undertaken on all sleeper sections and USPs. The initial assumption was that greater coefficient of friction at the ballast-sleeper interface could result in higher resistance to ballast travel and rearrangement, and consequently could result in lower overall settlement.

The friction test was conducted by application of a vertical constant load to the bottom of a ballast plate and a horizontal variable load to the front side of the sleeper section. The ballast plate was in contact with sleeper section underside and it was made by using epoxy resin to glue ballast particles to each other and to the inside of the plate. In construction of the ballast plate, the selection of particles was carried out in such a way as to achieve a fairly levelled surface with an appropriate contact at the ballast-sleeper interface, similar to the levelled ballast layer in contact with the bottom side of the sleeper section in the box test set-up. Figure 4-10 shows the ballast plate used for the friction test; the top surface of ballast were in contact with the sleeper section.

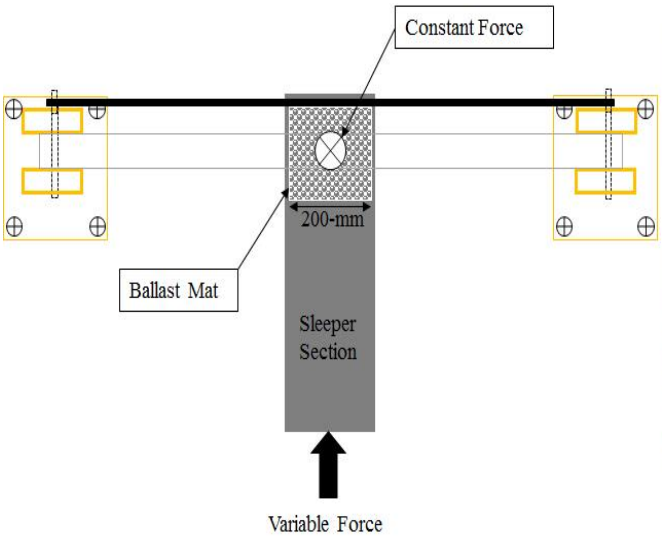
In each test a constant vertical load and a horizontal variable load under controlled conditions were applied and tests were continued until the LVDT positioned at the front face of the sleeper section showed a movement. Figure 4-11 presents the friction test set-up.

To do the friction test for the USP type1 and USP type2, each USP type was separately attached to a sleeper section using epoxy resin and was allowed to cure for a minimum of 12hours as a means of bonding the two elements. In this case, the ballast plate was in contact with USP underside.

The results of friction test for sleeper types and USPs are presented in Table 4-4. The coefficient of friction was calculated as the ratio of horizontal variable force required to make the sleeper section move (resisting load), to the constant vertical load applied. As can be seen in the table, the highest value for coefficient of friction was in the case of USP2 and the lowest value was in the case of the rectangular section. The difference between the measured values for coefficient of friction in the case of USP type1 and USP type2 was very small; however, the value was slightly higher for the USP type2.



Figure 4-10 Plate of ballast





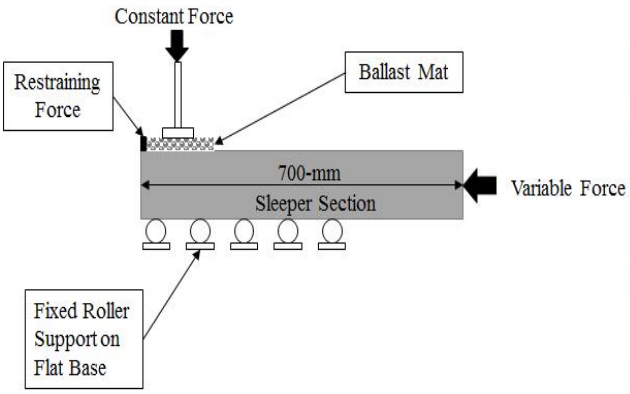




Figure 4-11 Friction test set-up

Table 4-4 Friction test results

Sleeper section	Constant load (kN)	Resisting load (kN)	Coefficient of friction
Rectangular section	10	2.90	0.29
	30	9.00	0.30
	45	14.40	0.32
Concrete	10	7.50	0.75
	30	21.30	0.71
	45	33.30	0.74
Timber	10	5.80	0.58
	30	18.00	0.60
	45	27.45	0.61
Plastic	10	4.70	0.47
	30	13.50	0.45
	45	21.60	0.48
USP Type1	10	8.00	0.80
	30	24.90	0.83
	45	36.45	0.81
USP Type2	10	8.90	0.89
	30	26.10	0.87
	45	40.50	0.90

As can be seen in Table 4-4, out of all sleeper section types tested, the concrete section showed the highest value of friction coefficient. This can be the result of the rougher surface of the concrete section underside (in contact with ballast), compare to the rest of the sleeper section types investigated.

4.7 SLEEPER SECTION TYPES AND USP EXPERIMENTS USING FULL-SIZE BALLAST (BOX TEST PHASE ONE)

In this section, box test investigations were undertaken using all the sleeper sections described in section 4.3.1 with and without USPs. From the data, it was possible to acquire and/or compute values for sleeper section displacement and trackbed stiffness. Images were also acquired for visual inspection of the sleeper section bearing area and USPs after tests were completed.

The first 10 cycles of the data were also ignored in the presentation of all the settlement and stiffness results, as they effectively represent bedding into the ballast of the sleeper section and were not reproducible between tests.

4.7.1 VISUAL INSPECTION

For each test, images of the ballast bed were acquired for examination and comparison purposes. A comparison of ballast bed images before and after the tests with and without USPs, showed little or no evidence of lateral movement of ballast or ballast breakage as a result of the

cyclic loading. As this was the case for all the sleeper sections investigated, a single example is given (in Figure 4-12 (a-c)) of the ballast bed, for the plastic sleeper section. In Figure 4-12 (a-c), there is no significant change in the configuration or appearance of ballast located directly beneath the sleeper section with and without the use of a USPs after the tests. McDowell and Stickley (2006) showed in their box tests that strong ballast such as granite showed insignificant breakage in the area underneath the sleeper section after installing geogrids. Considering that USPs were also resilient and attached directly to the base of the sleeper section, it was not unreasonable to suggest that breakage was also limited and perhaps even more so than in the case of when geogrids were installed. This is because the USP material was likely to have resulted in a 'softer' more evenly distributed ballast contact beneath the sleeper section. From a micro-mechanical perspective, the effect of ballast size and coordination number (McDowell et al., 1996) could also offer a viable explanation; however this was not the focus of the present study.

The USPs used for the plastic sleeper section were inspected for puncture, indentation or ballast abrasion. It was clear that greater sleeper coverage area was achieved when the softer type2 USP was used compared to the type1 USP as shown in Figure 4-12(d). This was unsurprising when it was considered that USP type2 was at least three times less stiff than USP type 1. Ballast scuffs on the USPs (after the test) were evidence of prospective areas of ballast-sleeper attrition or lateral (surface ballast) translation. The ballast marks were often accompanied by indentations (especially in the case of USP2) which would have likely improved ballast-sleeper interaction and possibly the lateral stability of the sleeper section in the box. While of significance, visual inspections of the bases of the sleeper sections were not conducted as the bases had been compromised due to previous use.

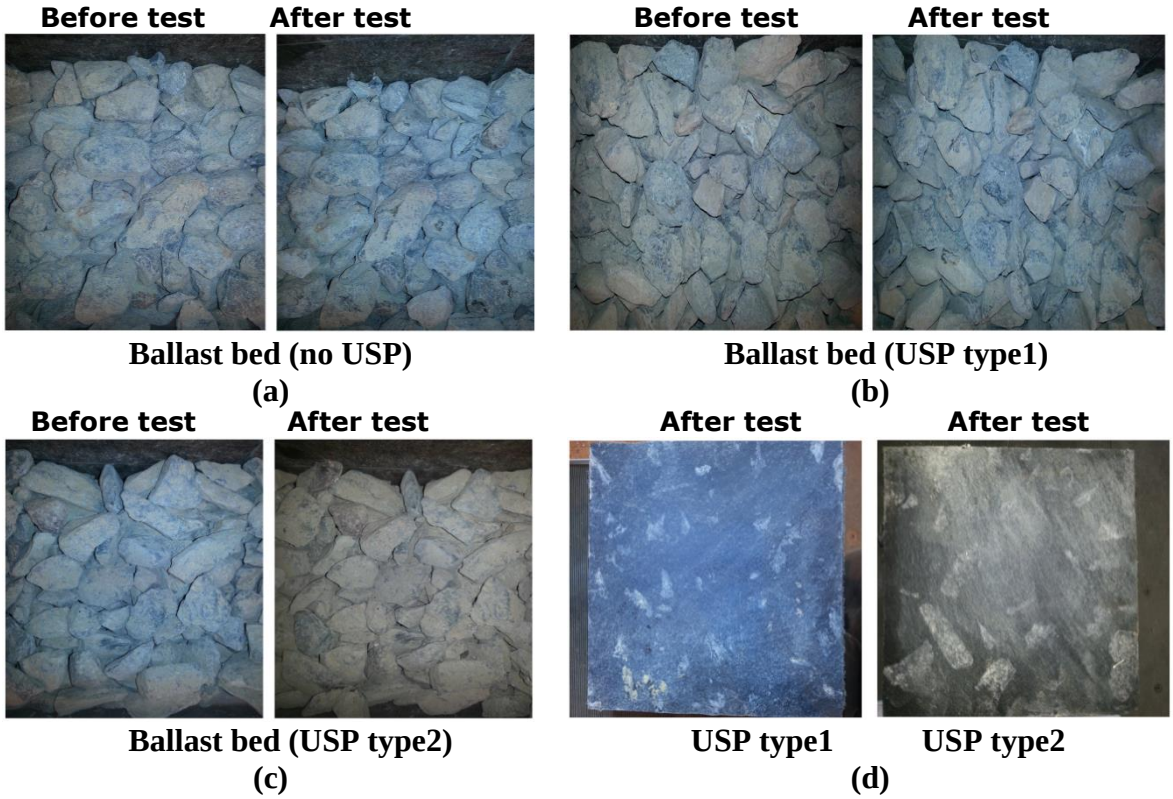


Figure 4-12 Images of ballast bed and USP before and after 30,000 cycles

4.7.2 SETTLEMENTS AND STIFFNESS

The evolution of settlement and trackbed stiffness for each sleeper section investigated with and without soft and stiff USPs are shown in Figure 4-13 to Figure 4-16. Trackbed stiffness was calculated by the heavy-light load method (Equation 4-2) used by Lim (2004).

$$k = \frac{F_{max} - F_{min}}{\delta_{max} - \delta_{\min}} \qquad \textbf{Equation 4-2}$$

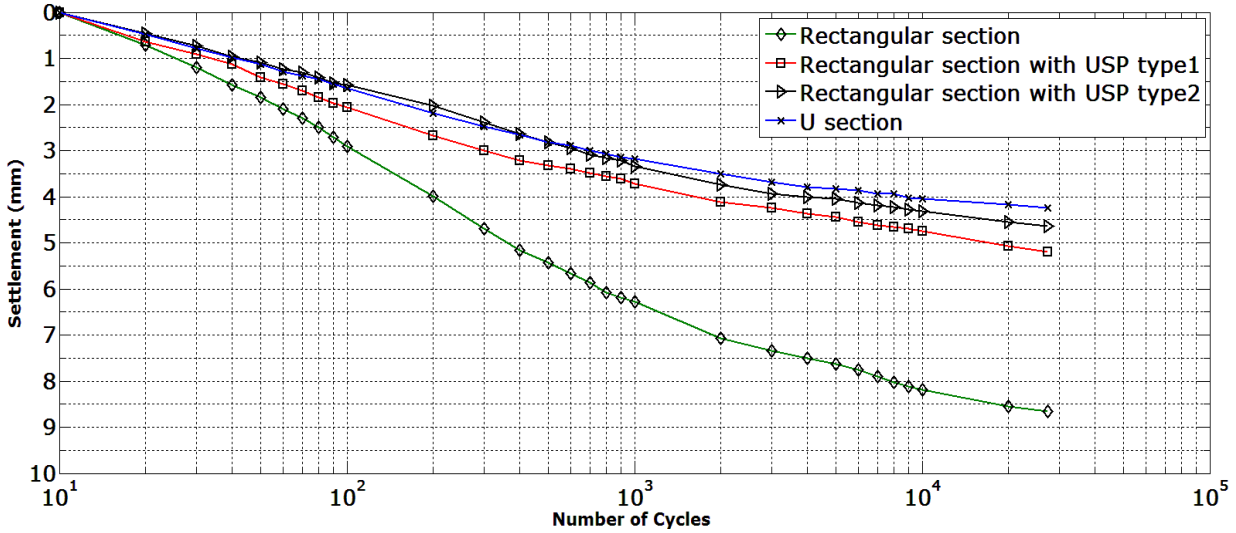
where:

k = stiffness of trackbed
$F_{max} - F_{min}$ = change in applied vertical force
$\delta_{max} - \delta_{min}$ = resilient vertical displacement

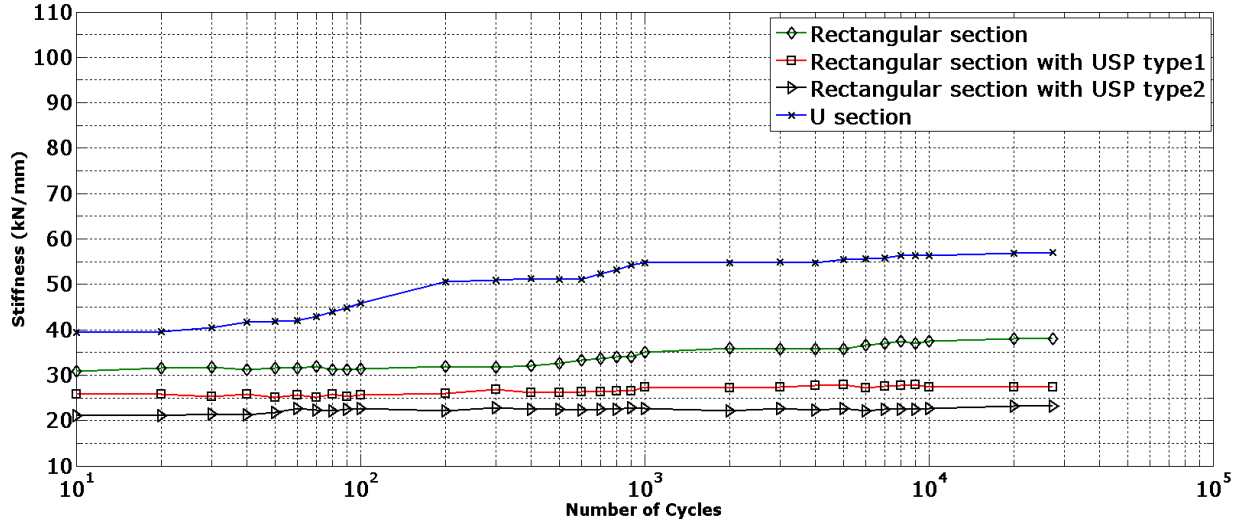
In Figure 4-13 to Figure 4-16, the effects of USPs on the individual sleeper section types with respect to settlement therand trackbed stiffness are shown.

In the case of the rectangular section (Figure 4-13 (a) and (b)), it was observed that although both USP type1 and USP type2 reduced settlement, the difference between them was marginal, giving comparable settlement to that of the U-section. However, the differences between U-section and rectangular section with USP in the stiffnesses that resulted were more evident.

When the concrete section was used (Figure 4-14 (a) and (b)), results showed that application of USPs reduced the concrete section settlement irrespective of USP types. However, slightly better performance was observed in the case of the concrete section with USP type2. With regard to the stiffness, application of USPs resulted in trackbed stiffness reductions. However, the difference in the resulting stiffness was more evident when USP type2 was used.

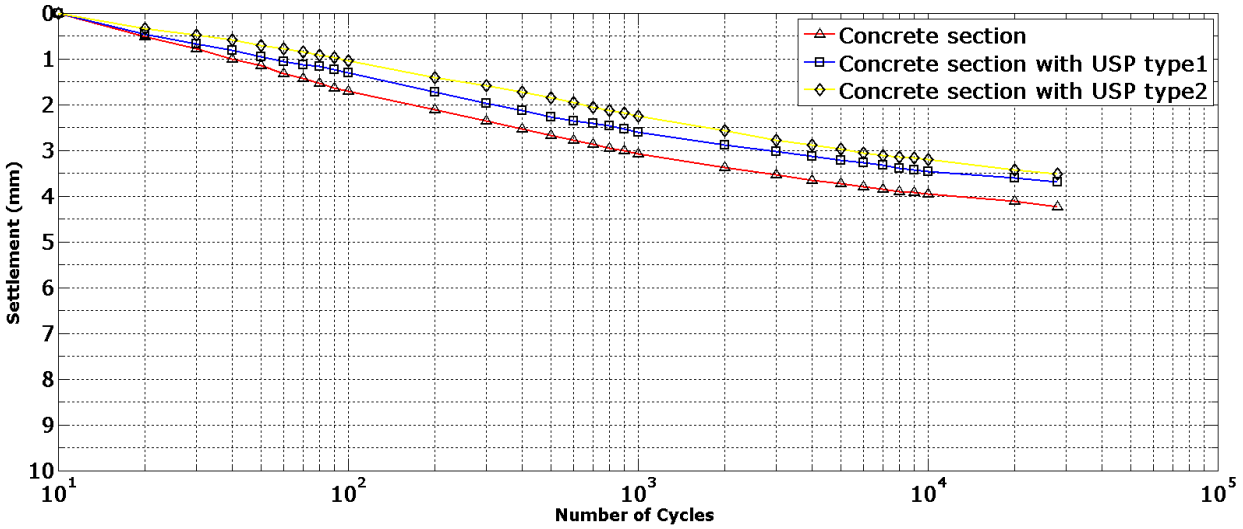


(a)

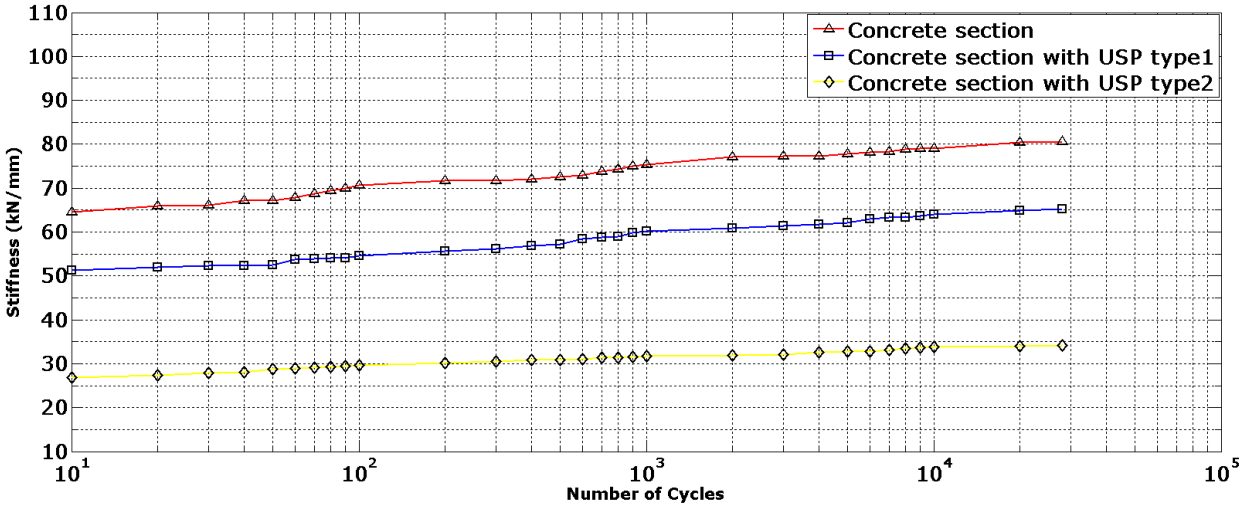


(b)

Figure 4-13 (b) Effect of USP inclusion on rectangular section performance (a) settlement (b) trackbed stiffness



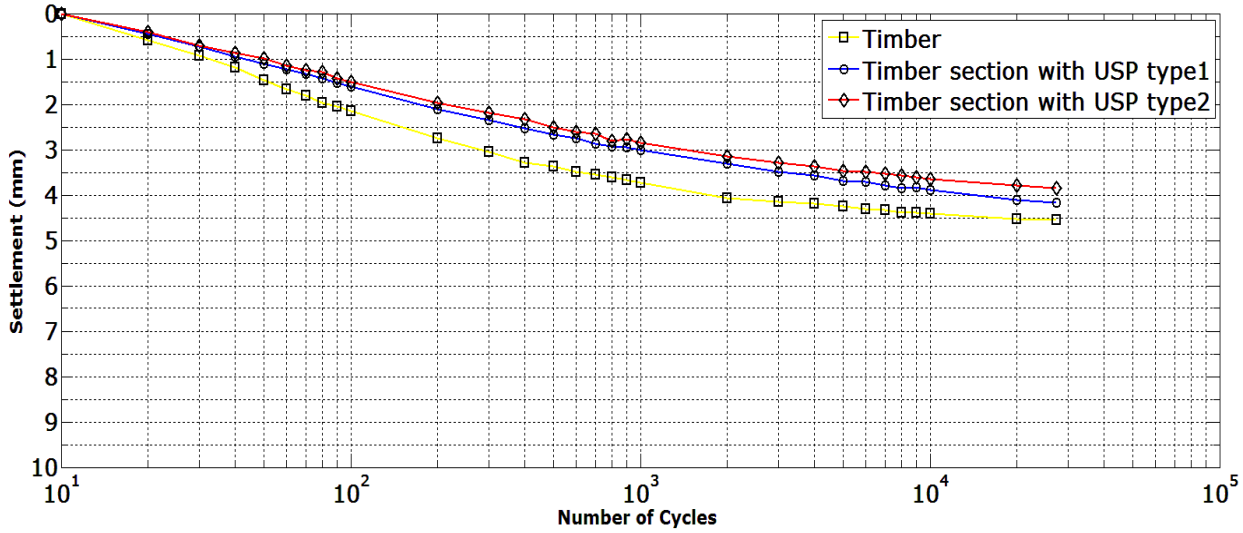
(a)



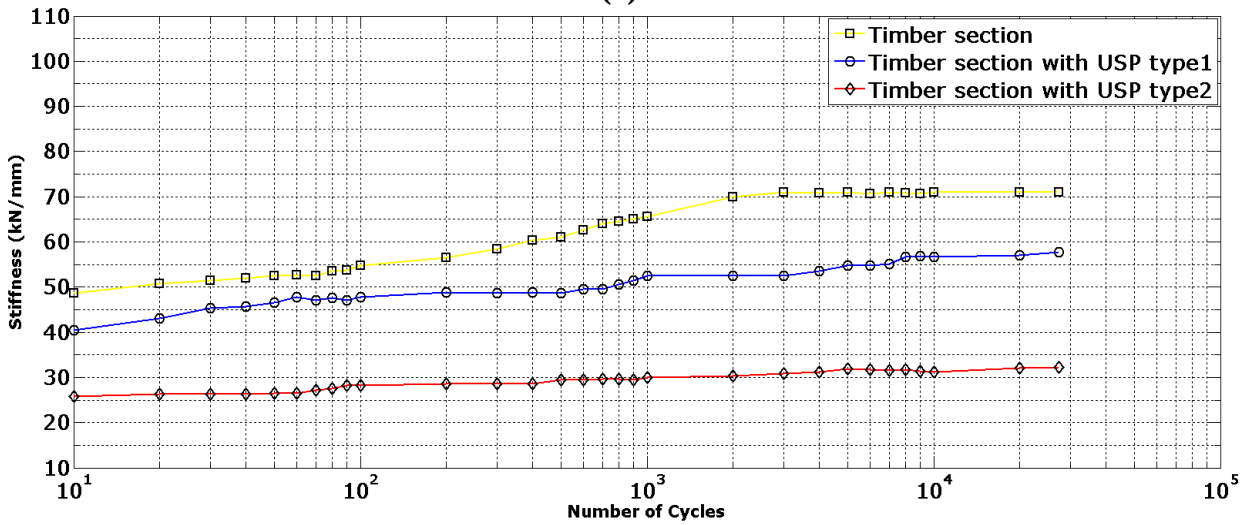
(b)

Figure 4-14 Effect of USP inclusion on concrete section performance (a) settlement (b) trackbed stiffness

For the timber section (Figures Figure 4-15 (a) and (b)), the least settlement and stiffness was observed for USP type2. It can be seen that in the long term the use of USP types had marginal influence on settlement compared to 'no USP'. However, where trackbed stiffness was concerned, a clear reduction in stiffness was observable.



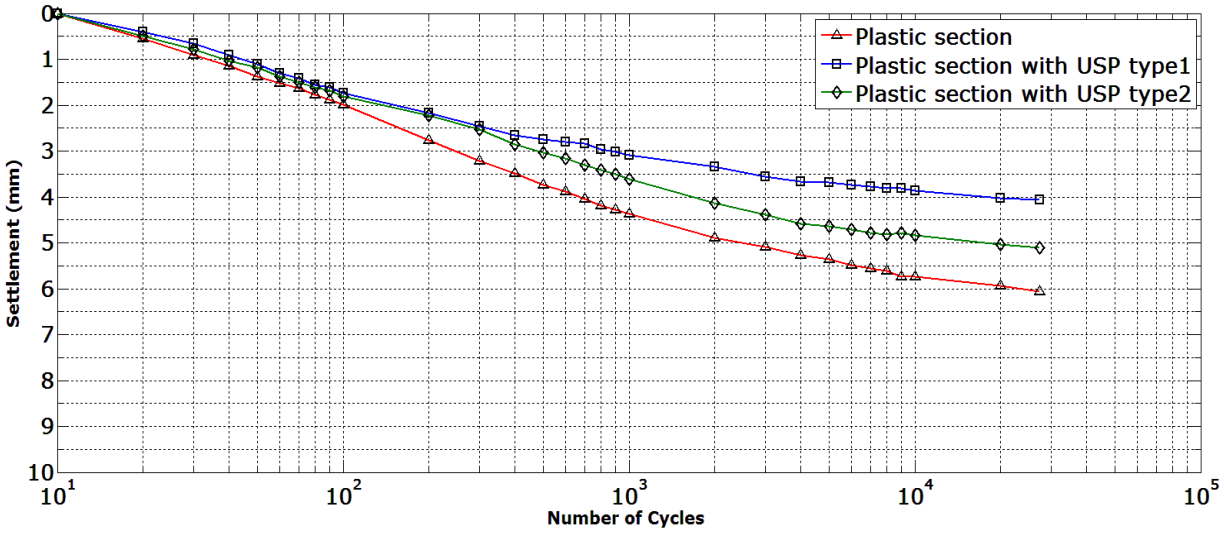
(a)



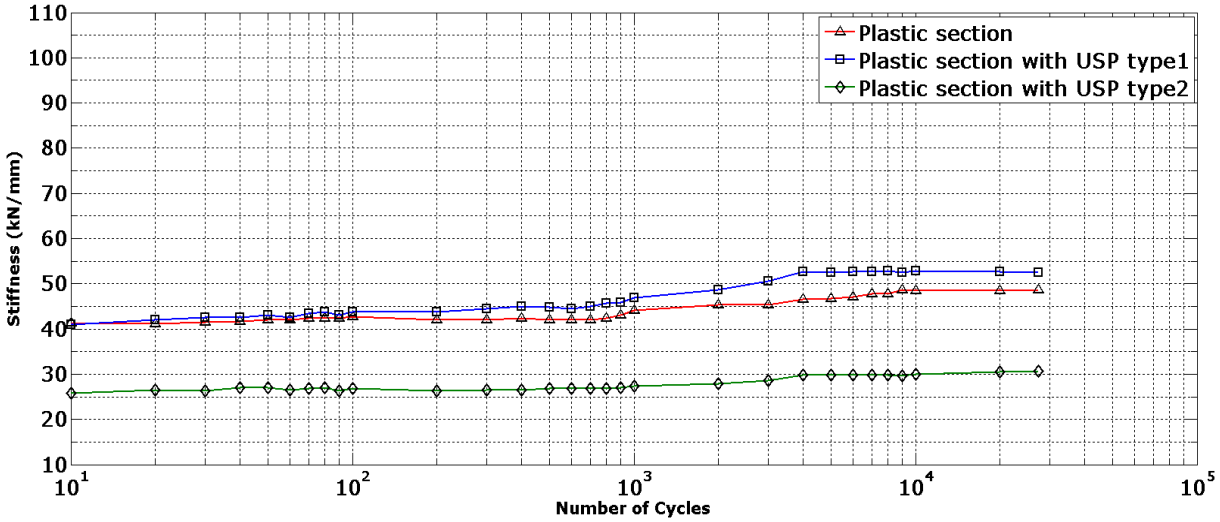
(b)

Figure 4-15 Effect of USP inclusion on timber section performance (a) settlement (b) trackbed stiffness

For the plastic sleeper section (Figure 4-16 (a) and (b)), using both types of USP resulted in reduced settlements but the USP type1 improved slightly more compared to USP type2. With regard to stiffness the USP type1 seemed to have slightly increased the trackbed stiffness compared to 'no USP' whereas USP type 2 resulted in the lowest trackbed stiffness.



(a)



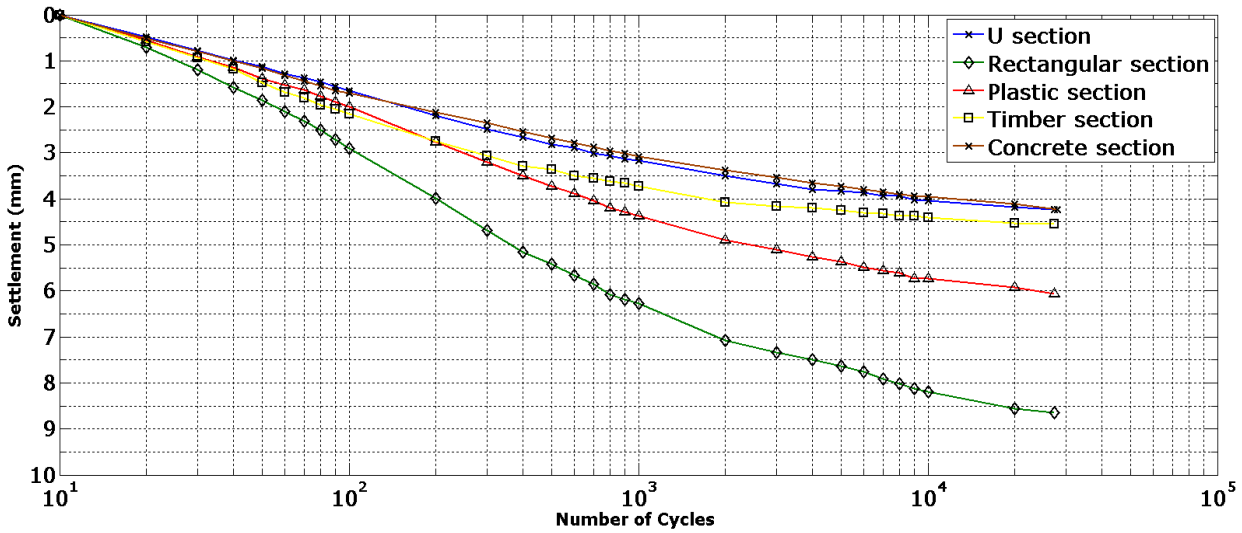
(b)

Figure 4-16 Effect of USP inclusion on plastic section performance (a) settlement (b) trackbed stiffness

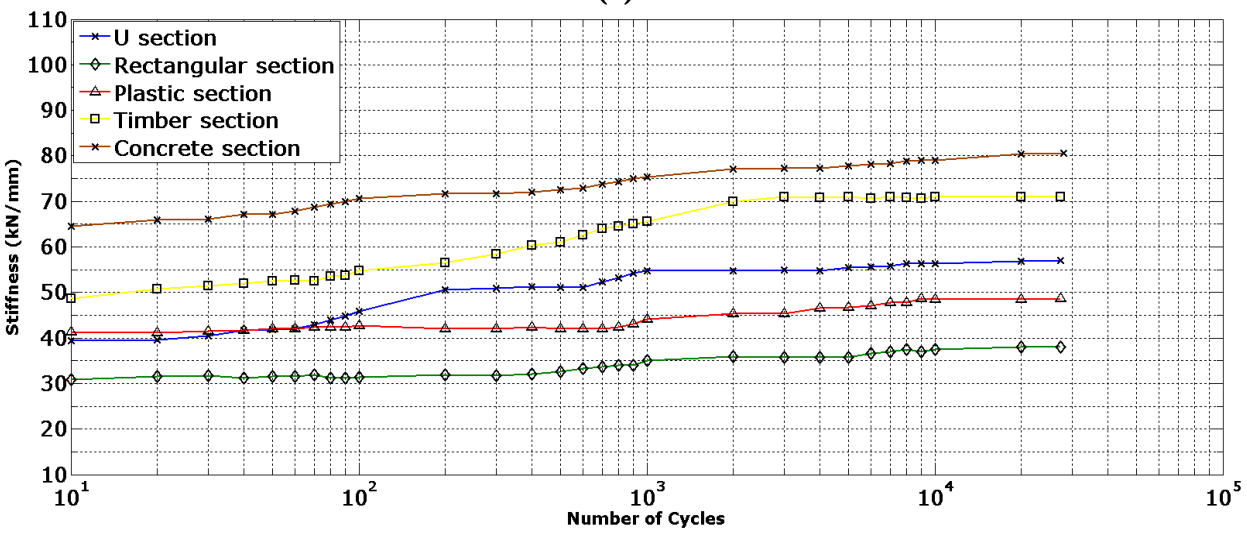
Figure 4-17 to Figure 4-19 compare the performance of the sleeper section types to one another with and without the use of USPs.

When USPs were not present, the main observation from the settlement results is that of the five sleeper sections, the U-section sleeper and concrete section equally yielded the least settlement and the rectangular section sleeper the most (Figure 4-17 (a)). The highest change in stiffness during the test in the case of 'no USP' (Figure 4-17 (b)) was observed for the timber sleeper section and the least were observed for the rectangular section and the plastic sleeper section. In terms of the magnitude of the stiffness, the concrete section had the highest and the rectangular section, the lowest. The latter was explained by the rounded geometry of the base of the rectangular section that effectively reduced the contact area between the section and the

ballast, thereby concentrating imposed loads over a smaller area of ballast. Overall, all the sections showed a tendency towards constant stiffness.



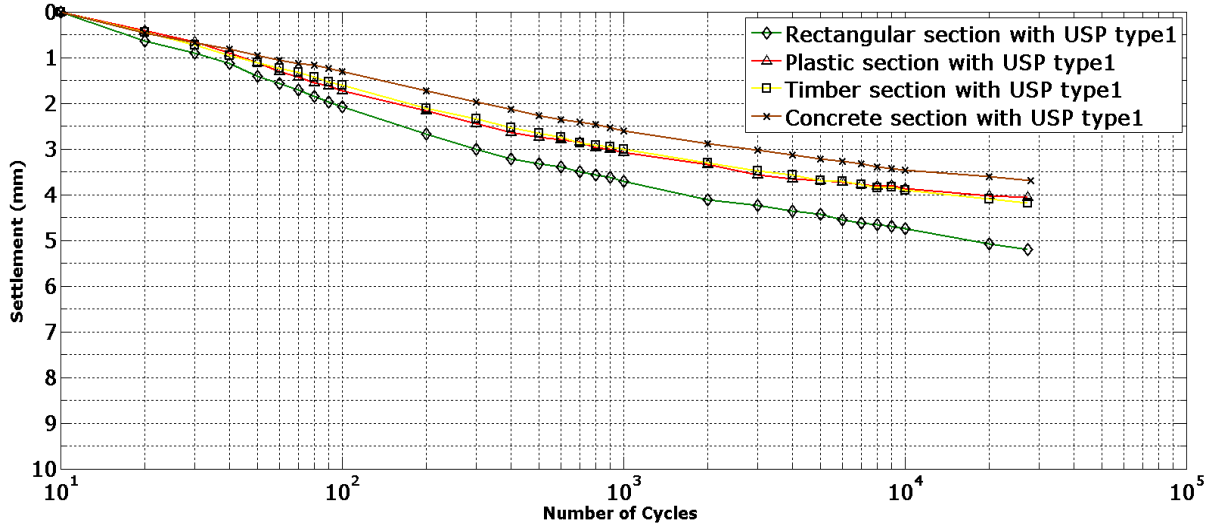
(a)



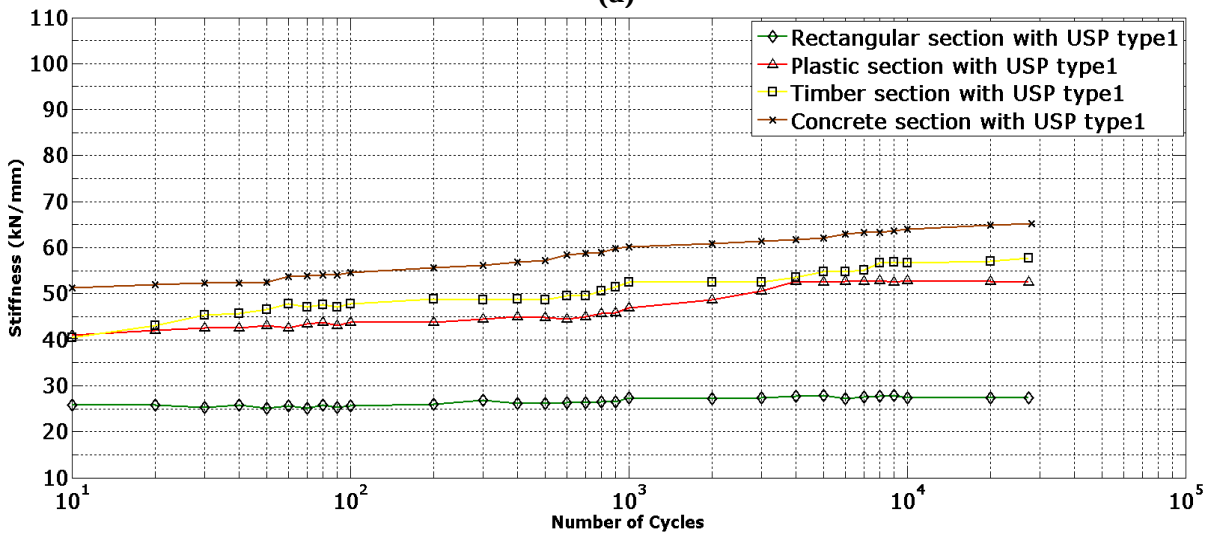
(b)

Figure 4-17 (a) Evolution of sleeper section displacement for sleeper sections without USPs (b) Evolution of trackbed stiffness for sleepers without USPs

It is clear that when USP type1 is considered (Figure 4-18 (a) and (b)), there was generally less variability in settlement between the sleeper sections compared to the 'no USP' scenario. Although similar performances were observed, the results showed slightly better performance for the concrete section. With regard to stiffness, similar stiffnesses were recorded for the timber and plastic sleeper sections with USP type1; for the rectangular section, the stiffness was the lowest and for the concrete section it was the highest.



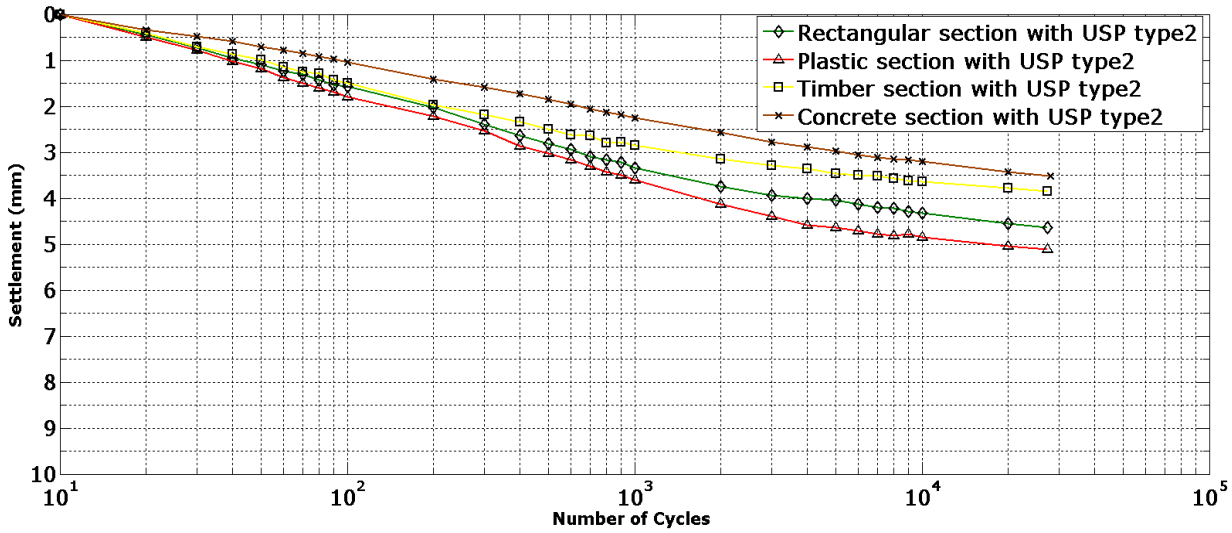
(a)



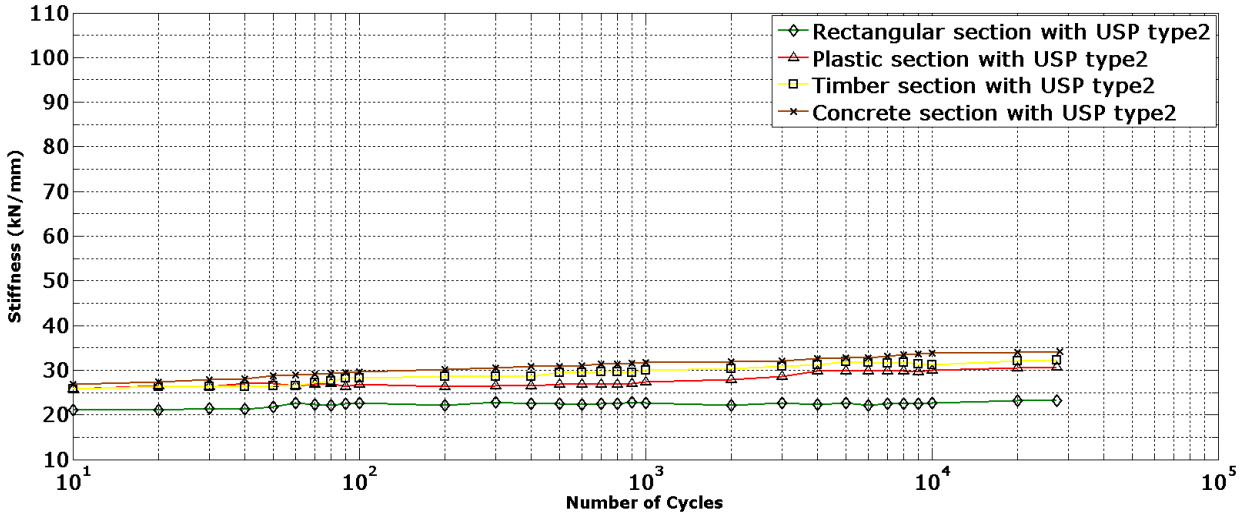
(b)

Figure 4-18 (a) Evolution of sleeper section displacement for sleeper sections with USP type1 (b) Evolution of trackbed stiffness for sleepers with USP type1

In the case of USP type2 (Figure 4-19 (a) and (b)), a similarly small variability in settlement was observed as for USP type1 cases. In terms of stiffness the softer USP (type2) lead to a reduction in variability. In the case of the plastic sleeper section a reduction in stiffness was observed (compare to 'no USP' case) in contrast to the increase seen for USP type1 possibly indicating stiffness of USP type2 caused the USP to interact with the underlying ballast to a greater extent than the sleeper.



(a)



(b)

Figure 4-19 (a) Evolution of sleeper section displacement for sleeper sections with USP type2 (b) Evolution of trackbed stiffness for sleepers with USP type2

The above results showed that there was not much differences in settlement performance for a given USP, i.e. the sleeper material became almost irrelevant. The differences in performance in Figure 4-18 (a) Figure 4-19 (a) are relatively small and were considered to be within the limits of experimental variability.

The sleeper settlements and trackbed stiffnesses (shown as averages over 30,000 cycles), for all the tests undertaken in phase 1 of box test are summarised in Figure 4-20 and Figure 4-21 respectively.

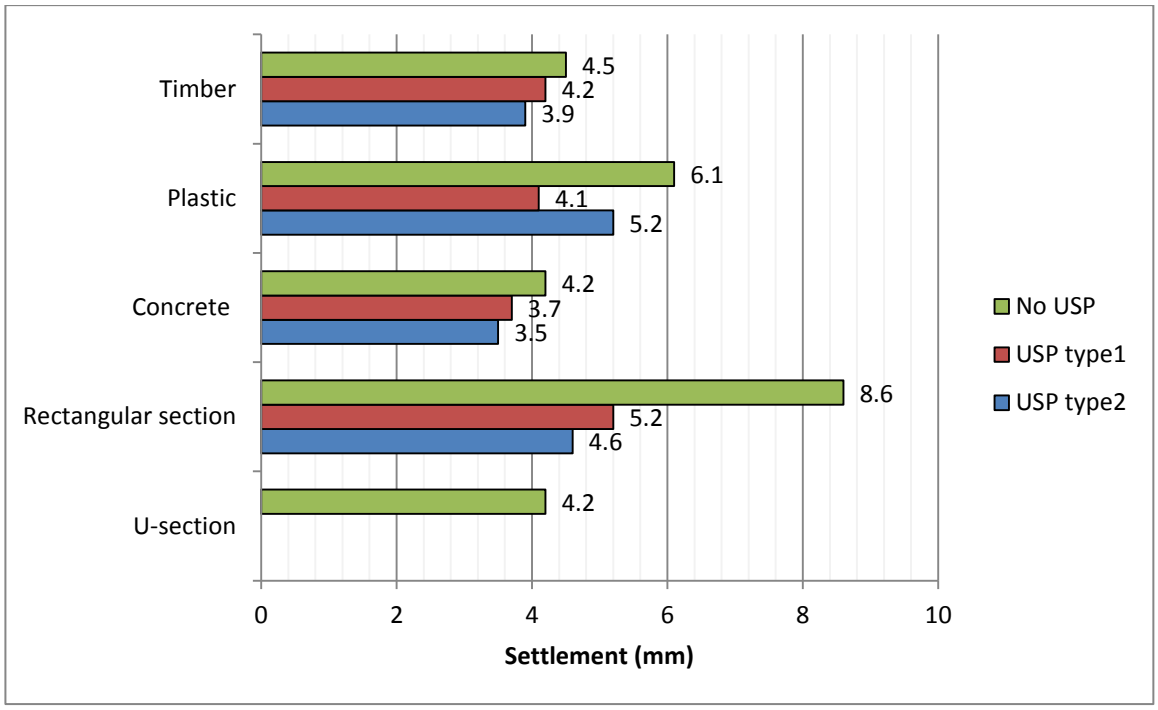


Figure 4-20 Sleeper section tests settlement at 30,000 cycles

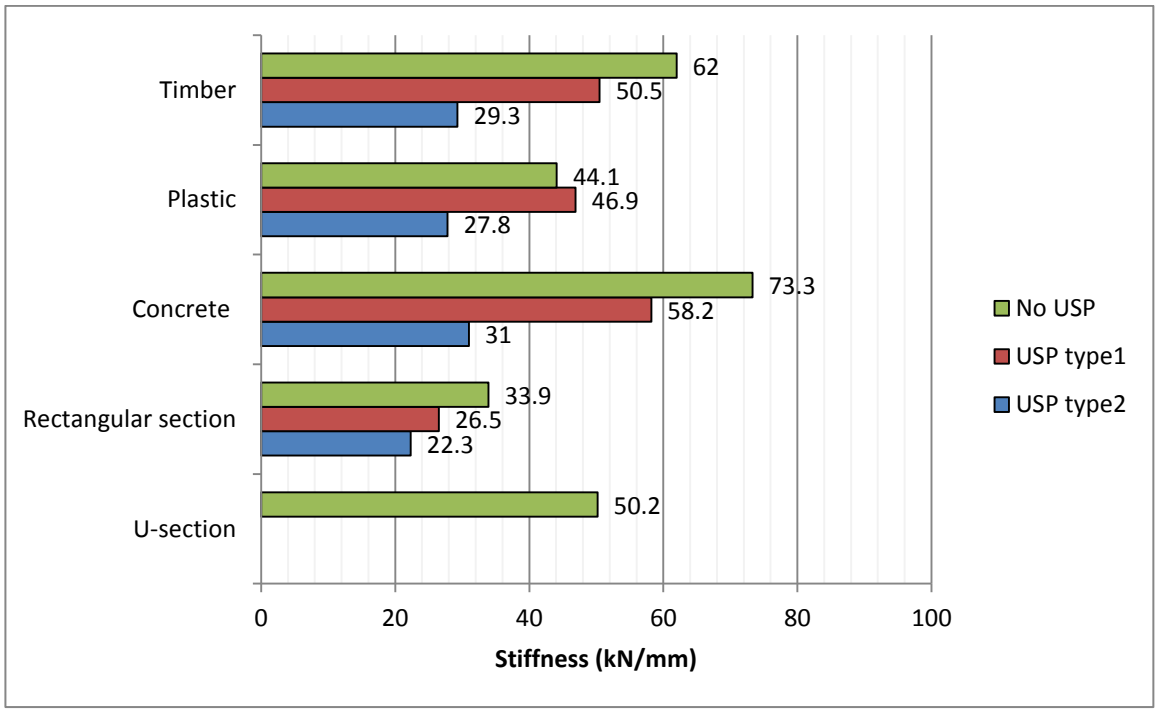


Figure 4-21 Sleeper section tests average trackbed stiffness over 30,000 cycles

It can be seen from Figure 4-20 that in the case of sleepers without USPs a greater coefficient of friction at the ballast-sleeper interface (Table 4-4) resulted in lower settlement, which can be due to the higher resistance to ballast travel and rearrangement. In the case of USPs, it can be seen in the figure that irrespective of the type of USP used, the variability in the measured settlement between the different sleeper section types was less compared to when USPs were

not used; this is consistent with the low variability between the measured coefficient of friction values at the ballast-sleeper interface for the USP types.

A similar trend is also noted for the measured stiffness in the case of the soft (type2) under sleeper pad (Figure 4-19b); the variability in stiffness with the soft USP was less compared to when USPs were not used and when the stiffer USP was used. The effect of the USPs on settlement and average settlement is qualitatively presented in Table 4-5 and

Table 4-6 as a form of sleeper selection guidance based on settlement and average stiffness.

The effect of the USPs on the settlements and stiffnesses measured generally agreed with previous studies. However this study was significant because it has highlighted that the differences between sleeper types are marginal with respect to settlement when USPs are introduced into the track system. Reductions in observed settlements appear to stem from increased interaction between the sleeper and ballast at their interface (whether or not a USP is present). The increased interaction leads to lower individual ballast particle stress concentrations and improved the load spreading ability of the ballast. From a broader perspective, this could extend the duration between maintenance cycles and the life of a railway track.

Table 4-5 Sleeper selection guidance based on settlement

	No USP	USP Type1	USP Type2
↓ Increasing settlement	U-section & Concrete	Concrete	Concrete
	Timber	Plastic	Timber
	Plastic	Timber	Rectangular
	Rectangular	Rectangular	Plastic

Table 4-6 Sleeper selection guidance based on average ballast stiffness

	No USP	USP Type1	USP Type2
↓ Increasing average stiffness	Rectangular		
	Plastic	Rectangular	Rectangular
	U-section	Plastic	Plastic
	Timber	Timber	Timber
	Concrete	Concrete	Concrete

4.8 ADDITIONAL EXPERIMENTS (BOX TEST PHASE TWO)

Once testing the performance of the sleeper section types with and without USPs had been completed using the box test, the full-scale test facility was used to conduct similar tests using standard sized railway sleepers. Comparison of the results showed consistency between the box test and the full-scale results. Therefore, additional tests were undertaken using the box test. As

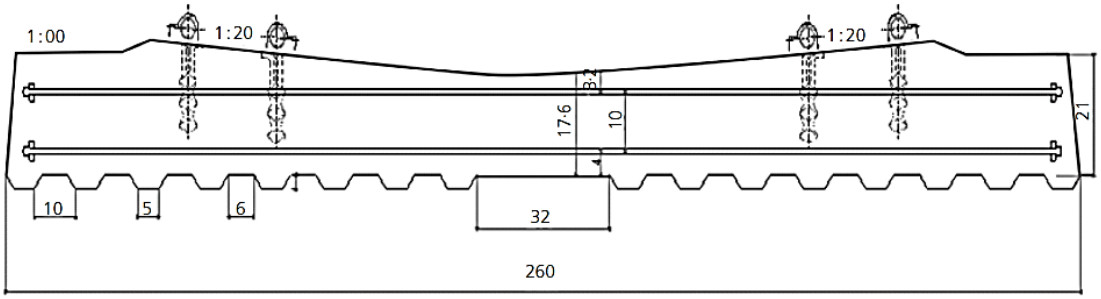
stated previously, the aim was to use the box test to allow a relatively large number of simplified tests to be carried out within a reasonable cost and time frame. The details and outcomes of these tests are presented in the following section.

4.8.1 FRICTIONAL RAILWAY SLEEPER AND USP

Railway tracks must be resistant against lateral loads caused by trains as a result of cornering, wind loading, and vehicle dynamics effects, and by temperature due to the thermal expansion of steel rails. Lateral loads are transferred to the sleepers from rails through fastenings in traditional ballasted railway track. The lateral resistance of a sleeper in a ballasted track comes from contact with ballast at the base, shoulder, and crib; however, in the case of mono-block concrete sleeper, it is predominantly from the frictional forces between the ballast particles and the sleeper underside (Zakeri, 2012, Zakeri and Bakhtiary, 2014). Several studies have been conducted on methods of evaluating and measuring the lateral resistance of sleepers in ballasted track (e.g. (Koike et al., 2014, Le Pen et al., 2014, Zakeri and Bakhtiary, 2014, Zakeri, 2012). There are also studies that have recommended use of concrete lateral resistance sleepers (frictional sleepers) and compared them with standard concrete sleepers in terms of resistance to lateral loads (Zakeri et al., 2012, Zakeri and Fakhari, 2014, Montalbán Domingo et al., 2014). The frictional sleeper suggested by Zakeri et al. (2012) is presented in Figure 4-22. Their study showed that use of a frictional concrete sleeper improved the lateral resistance of the ballasted railway track by 69%. This was consistent with a Finite Element (FE) study done by Montalbán Domingo et al. (2014) that showed 53% improvement in lateral resistance of ballasted track by using lateral resistance sleepers. Figure 4-23 shows the concrete resistance sleeper used in the Montalbán Domingo et al. (2014) study. Although the study by Montalbán Domingo et al. (2014) showed improvement in lateral resistance of concrete sleppers, the installation method of this sleeper with the geometry given in Figure 4-23 was not disscussed.

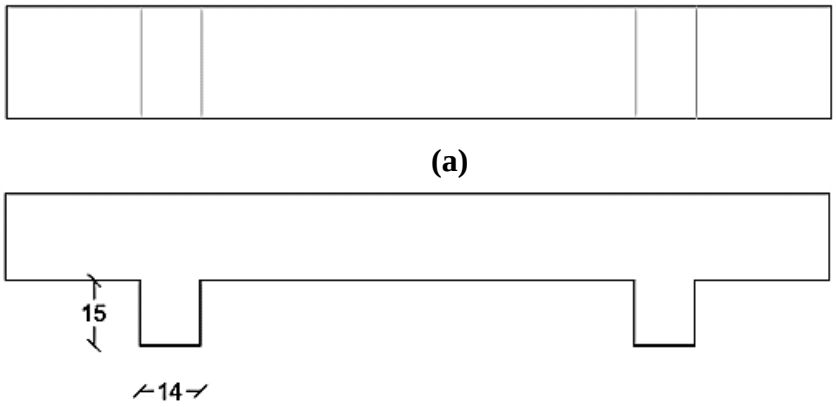


Figure 4-22 (a) Frictional sleeper bottom (Zakeri, 2012, Zakeri et al., 2012)



(b)

Figure 4-22 Frictional sleeper (a) bottom (b) cross-section (Zakeri, 2012, Zakeri et al., 2012)



(a)

(b)

Figure 4-23 Designed frictional sleeper (a) plan (b) cross-section (Montalbán Domingo et al., 2014)

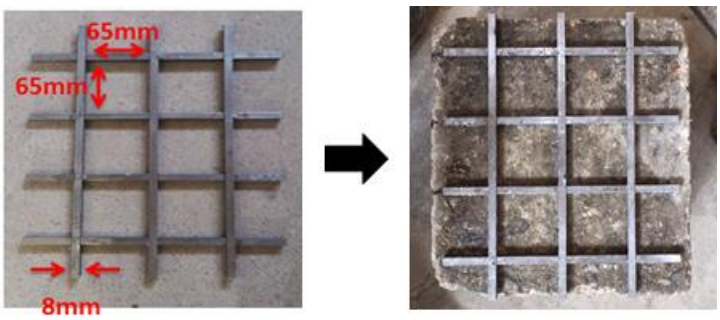
In this section, box test experiments were conducted using a type of frictional concrete sleeper and USP type2 to investigate their influences on the settlement performance and stiffness of the ballasted track. The results were then compared with the standard concrete section and USP type2 presented before in section 4.7.2.

The frictional sleeper section used in this study, was made by modifying the bottom of the concrete sleeper section so as to increase the frictional forces at the ballast-sleeper interface. To do this, a steel mesh with dimensions given in Figure 4-24(a) was attached to the concrete section (Figure 4-24(b)). The steel mesh was made of an 8-mm Square Bright Drawn Mild Steel (BDMS) and was attached to the concrete sleeper using an epoxy resin which was allowed to cure for a minimum of 12hours as a means of bonding the two elements. The grid size used for the steel mesh was 65-mm to accommodate the biggest ballast particle size of the full-size sample used in this test.

The steel mesh (Figure 4-25(a)) was also attached to the bottom of a USP type2 (Figure 4-25(b)) to produce a frictional USP and the combination was connected to the concrete sleeper section using an epoxy resin. To create the frictional USP, the USP type2 was used instead of the USP type1 because the friction at the ballast-sleeper interface of the USP type2 was slightly higher than that of the USP type1 (Table 4-4). Moreover, the USP type2 was softer that type1, and the possibility of impeding ballast particle migration by penetration into it was also higher compared to the USP type1. Therefore use of USP type2 could increase the frictional forces at the ballast-sleeper interface and consequently the resistance to ballast travel and rearrangement. A frictional USP was also made by attaching a Tensar biaxial geogrid (Figure 4-26(a)) to a USP type2 as a mesh, with a grid size of 65-mm (Figure 4-26(b)).

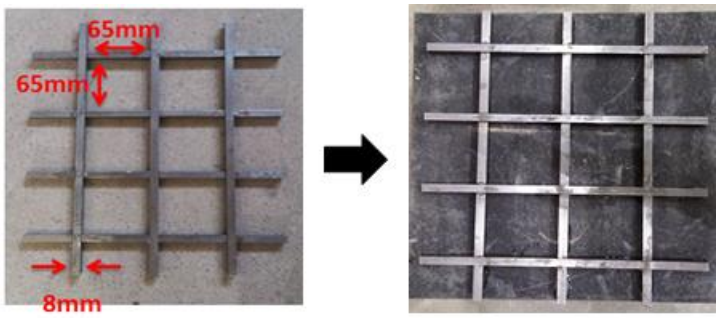
The aforementioned tests described in this section were conducted to investigate the potential for settlement performance improvement of a concrete sleeper and a USP by increasing the frictional forces at the ballast-sleeper interface. It is not proposed that a steel mesh or a biaxial geogrid would actually be the correct means of improving performance.

Sample preparation of the full-size ballast used and the setup of the box tests for the frictional concrete sleeper section and the USP were similar to the method described in section 4.4.



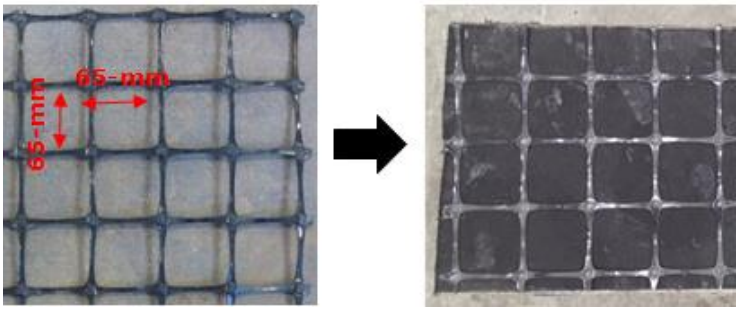
(a) (b)

Figure 4-24 (a) Steel mesh (b) Frictional sleeper - concrete Section with attached steel Mesh



(a) (b)

Figure 4-25 (a) Steel mesh (b) Resistant USP - USP type2 with attached steel mesh



(a) **(b)**

Figure 4-26 (a) Biaxial geogrid (b) Resistant USP - USP type2 with attached biaxial geogrid

In Figure 4-27, a comparison of the performance of the frictional concrete section plus or minus USP against the performance of normal concrete section and USP type2 is shown, with respect to settlement and trackbed stiffness.

In the case of the frictional sleeper (Figure 4-27(a) and (b)), it was observed that although it reduced the settlement, the difference compared to the normal concrete section was marginal. However, the difference in the resulting settlement was more evident when frictional USP type2 was used. Of the two frictional USPs, the one made using steel mesh resulted in higher settlement reductions. This may be as a result of the higher thickness of steel mesh compared to the biaxial geogrid mesh which created a higher interlock between the steel mesh and ballast particles at the sleeper-ballast interface. With regard to stiffness, the differences in results were marginal between the frictional concrete section and normal concrete section; however, the differences between the frictional USP and the normal USP type2 were more evident. The increase in stiffness was more evident in the case of frictional USP made with steel mesh compared to that of made with biaxial geogrid.

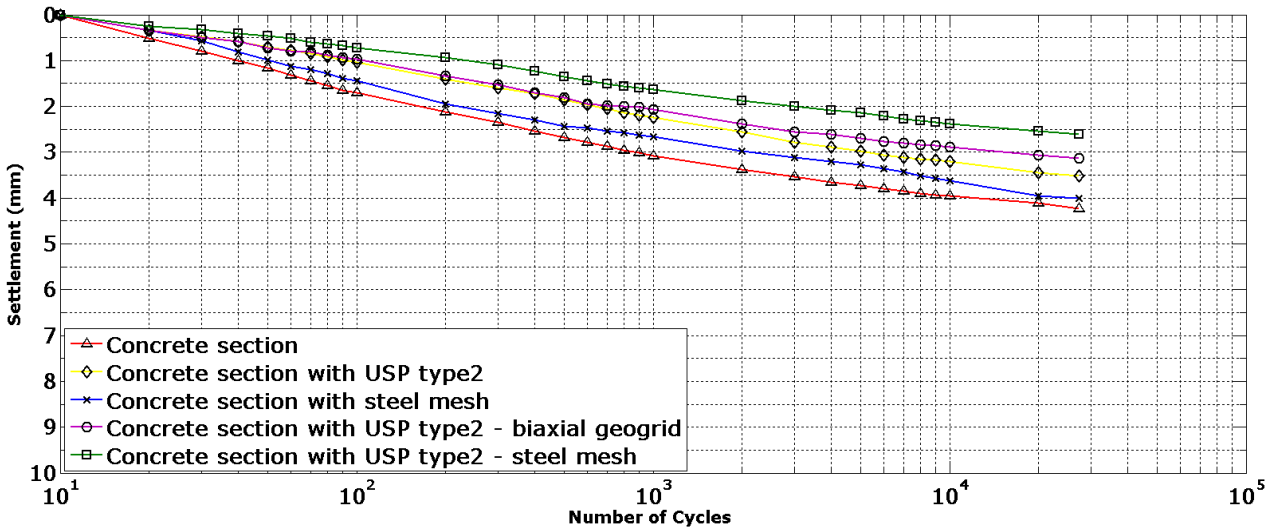
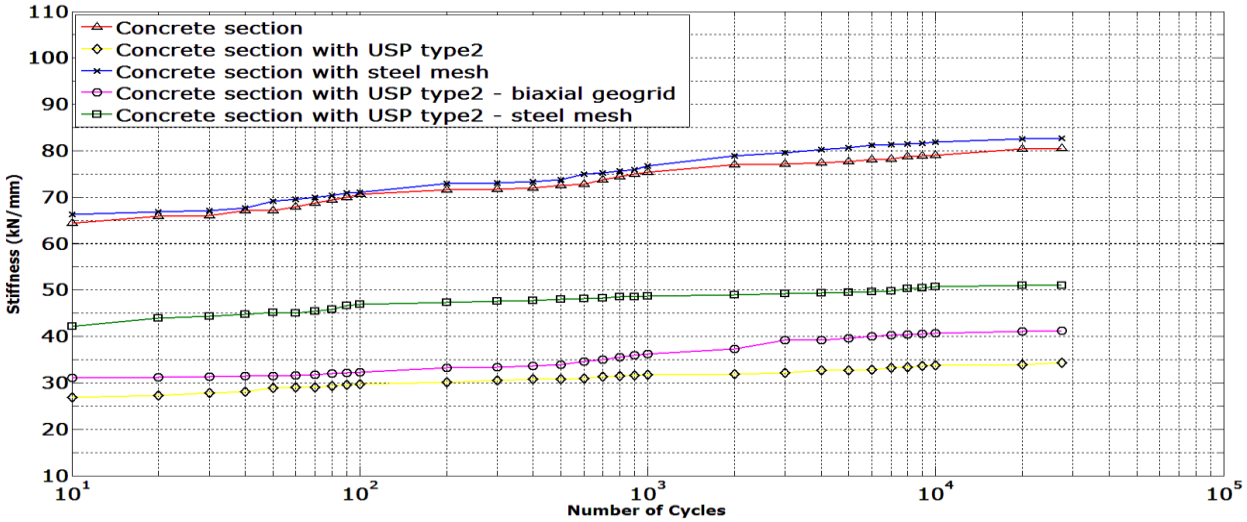


Figure 4-27 (a) Performance comparison between frictional and normal concrete section & USP type2 in terms of settlement



(b)

Figure 4-27 Performance comparison between frictional and normal concrete section & USP type2 in terms of (a) settlement (b) trackbed stiffness

The final settlement and average ballast stiffness results for frictional and normal concrete sections with and without USPs are presented in Figure 4-28 and Figure 4-29. The results of this investigation suggest that there is some potential for improving the settlement performance of concrete sleepers and USPs by modifying the underside of the sleeper and/or a USP and thereby increasing the frictional forces at ballast-sleeper interface.

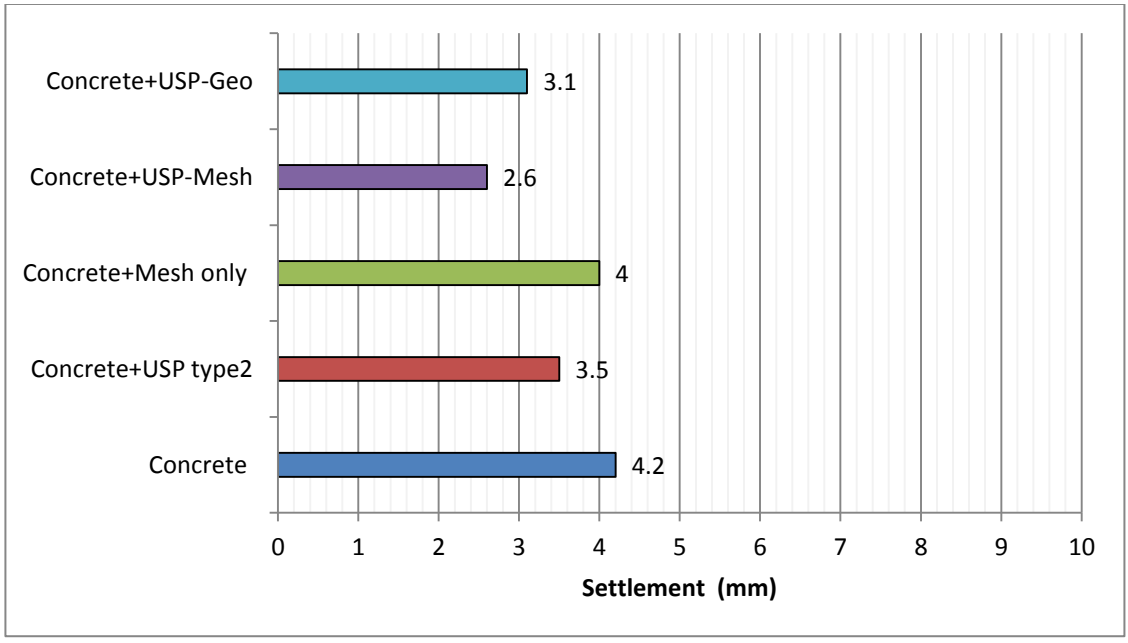


Figure 4-28 Settlement performance comparison at 30,000 between frictional and normal concrete section & USP type2

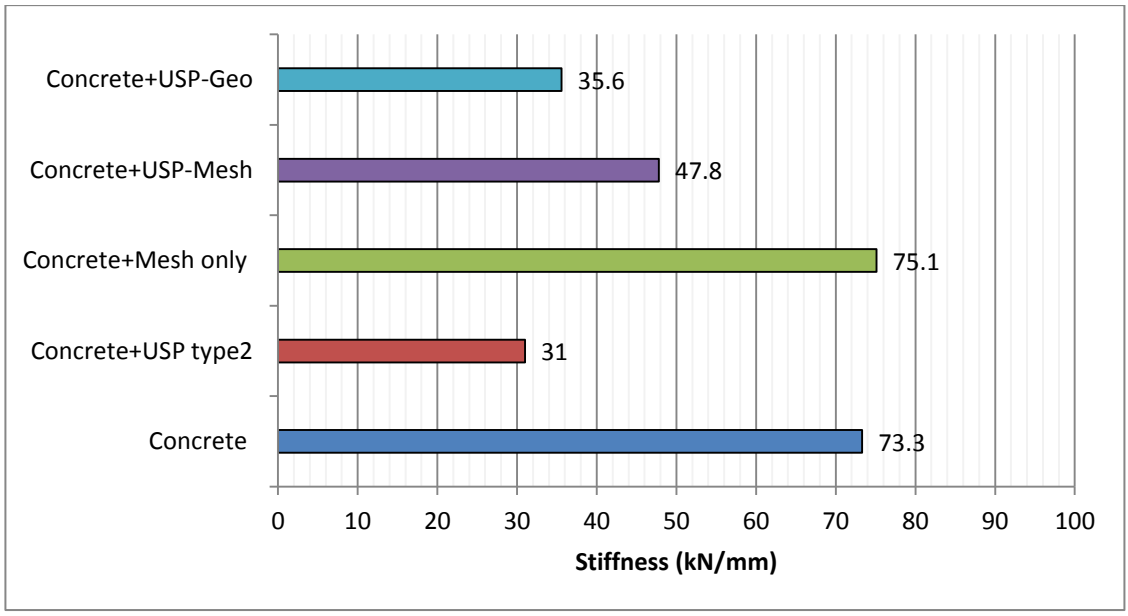


Figure 4-29 Average trackbed performance comparison over 30,000 between frictional and normal concrete section & USP type2

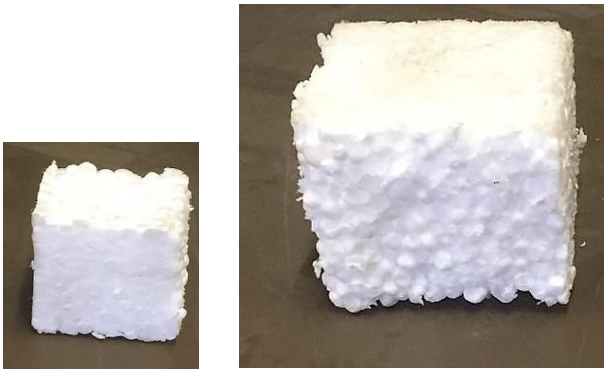
4.8.2 REINFORCED BALLAST USING EXPANDED POLYSTYRENE PARTICLES

To minimise ballasted track settlement and consequently to increase its service life, several innovative solutions have been applied in recent decades to reinforce the ballast particles. Application of elastic elements between ballast particles was one example of these solutions with the purpose of improving ballast performance and durability. Studies by Keene et al. (2012) and Kennedy et al. (2013) showed that use of polymers as an elastic element can improve the elasticity and resistance to plastic deformation of the ballast layers. Application of crumb rubber made from used tires as an elastic element, was also studied by Sol-Sánchez et al. (2015). The results of their study showed that the use of 10% of crumb rubber (by volume) can reduce ballast degradation; however, it resulted in a reduction of the ballast layer stiffness.

In this section, standard end-of-life protective packaging polystyrene was used to provide elastic elements between ballast particles, and the effect of its application on plastic deformation and stiffness of the ballast layer was studied. The expanded polystyrene has been used as a shock-absorbing element in packaging industry to minimise damages to goods during the distribution process. Having this in mind, expanded polystyrene was added to the ballast sample to decrease the contact forces between ballast particles and consequently minimising ballast bed deterioration and settlement. Using a hand saw, polystyrene sheets were cut up to two different sizes and separately were used in full-size ballast samples. The sizes of polystyrene

particles used were: small size with dimensions of 30-mm x 30-mm x 30-mm (Figure 4-30(a)) close to the size of a small ballast particle, and alternative large size with dimensions of 60-mm x 60-mm x 60-mm (Figure 4-30(b)) close to the size of a large ballast particle in the full-size ballast sample. In this study the influence of inclusion of different polystyrene percentages on the ballast behaviour for both polystyrene (as elastic element) was investigated separately. The percentages studied for both polystyrene sizes were 1%, 2.5%, 5%, and 10% by volume using the box test apparatus described in section 4.2. Figure 4-31 shows an example of a ballast sample mixed with polystyrene; the ballast sample shown in the figure was mixed with 5% large polystyrene particles.

All the box tests presented in this section were conducted using the concrete sleeper section (Figure 4-3) and full-size ballast.



(a) **(b)**

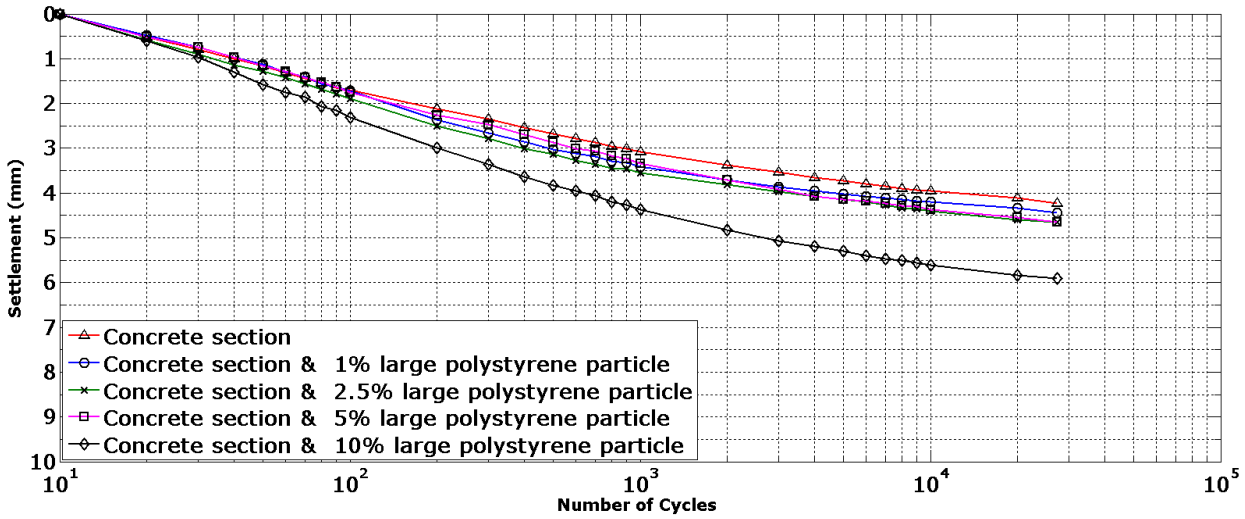
Figure 4-30 Polystyrene Particles (a) small size particle (b) large size particle



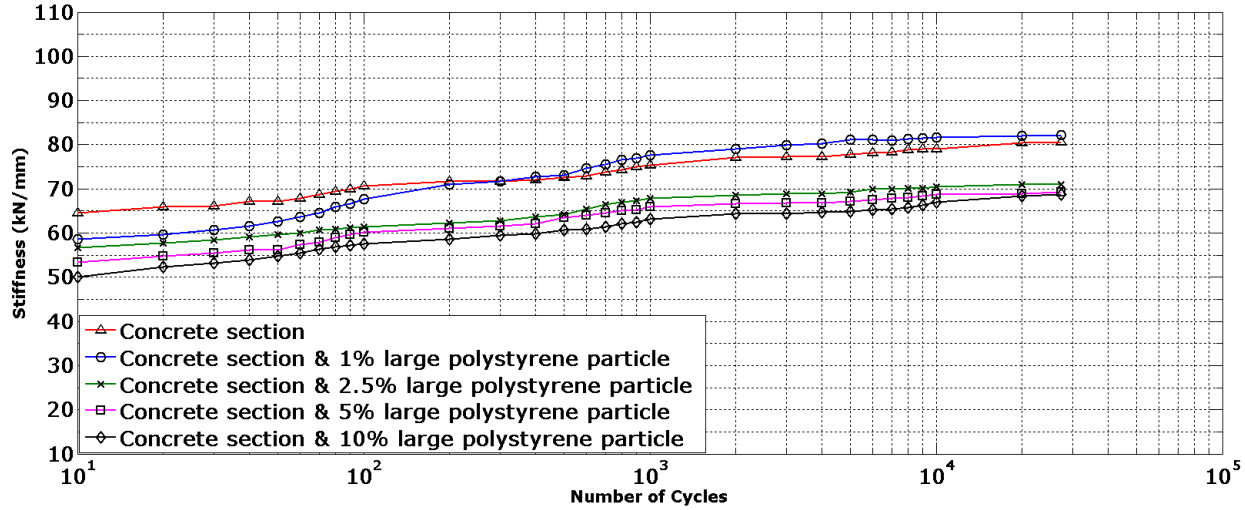
Figure 4-31 Full-scale ballast sample mixed with 5% large polystyrene particles

The evolution of ballast settlement recorded for ballast only (control test) and ballast mixed with different percentages of large polystyrene particles is presented in Figure 4-32(a). As settlement results indicated, the inclusion of large particle polystyrene as an elastic element increased ballast settlement. This could be due to higher vertical dynamic movements and

acceleration. As a result, the addition of a higher percentage of large polystyrene particles resulted in a higher settlement in the ballast layer. As can be seen, the increase in settlement was more obvious in the case of ballast mixed with 10% large size polystyrene particles. With regard to stiffness Figure 4-32(b), it was observed that an increase in percentage of large size polystyrene particles resulted in stiffness reductions in the majority of the tests with the exception of the 1% large polystyrene. In the case of 1% large polystyrene, stiffness results measured were in the same range as those recorded for the ballast only case.



(a)

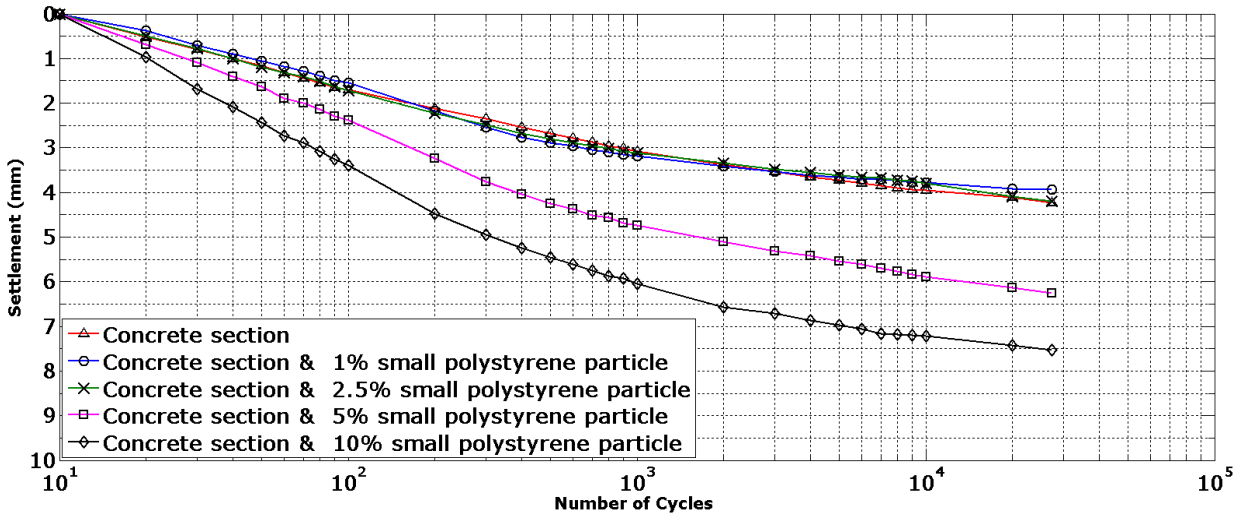


(b)

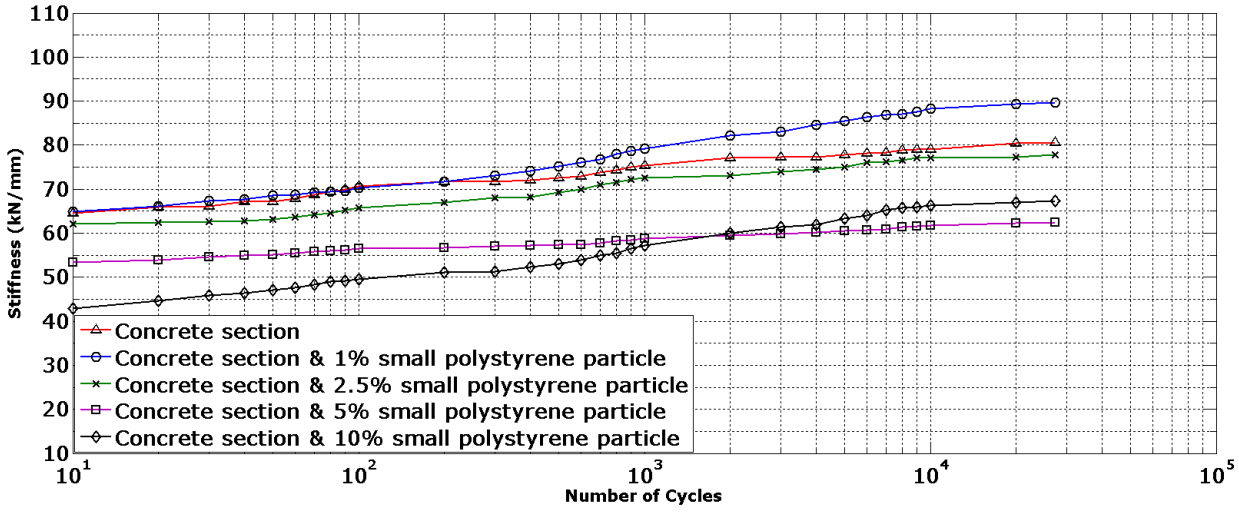
Figure 4-32 Effect of large size polystyrene particle inclusion on ballast layer performance (a) settlement (b) trackbed stiffness

Following testing of the large size polystyrene particles as an elastic element, the polystyrene sheets were cut to the smaller dimensions and tested mixed with the full-size ballast sample at different percentages (1%, 2.5%, 5%, and 10%).

In the case of small size polystyrene particles (Figure 4-33 (a) and (b)), it was observed that use of 1% small size polystyrene particles had a minor influence on settlement compared to ballast only. However, a clear increase in stiffness was observable. In the case of 2.5% small polystyrene particles, the recorded settlement and stiffness results were in the same range as those recoded for the ballast only case, whereas 5% and 10% showed an increase in settlement and reduction in stiffness.



(a)



(b)

Figure 4-33 Effect of small size polystyrene particle inclusion on ballast layer performance (a) settlement (b) trackbed stiffness

Figure 4-34 and Figure 4-35 show the settlement and average respectively stiffness results for the control test (ballast only) and for both large and small size polystyrene particles mixed with ballast. With the exception of 1% and 2.5% small size polystyrene, the tests showed generally no improvement in settlement when polystyrene was added to ballast, regardless of polystyrene

inclusion sizes. As can be seen, irrespective of polystyrene size, higher settlement results were recorded when higher percentages of polystyrene were used. In the case of 1% and 2.5% polystyrene particles, the small size resulted in lower settlement of the ballast layer and higher stiffness values compared to large size polystyrene. However, in the case of 5% and 10% polystyrene particles, the large size lead to lower settlement of the ballast layer and greater stiffness values compared to small sized polystyrene.

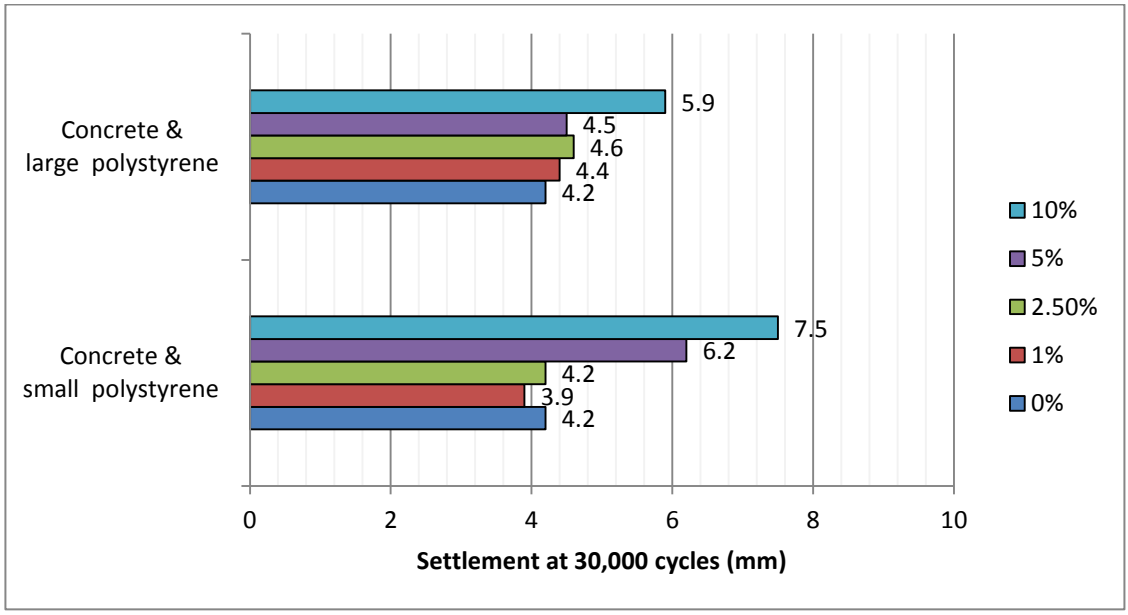


Figure 4-34 Trackbed settlement at 30,000 cycles for large and small polystyrene particle tests

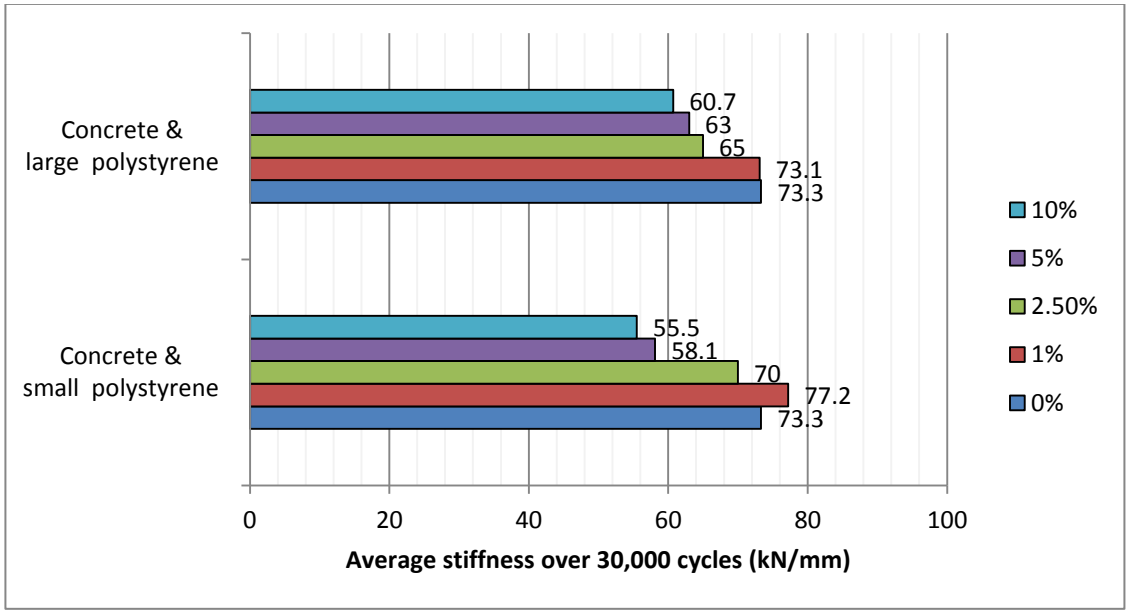


Figure 4-35 Trackbed average stiffness over 30,000 cycles for large and small polystyrene particle tests

4.8.3 SLEEPER SECTION TYPES AND USPS USING 2/3 SCALED BALLAST

Over the years, different ballast sizes have been used and the choice of size of ballast has been based on experience. According to a historical study by Claisse and Calla (2006) on development of ballast specifications from the early 1900s, a change to ballast of smaller size is recommended for the current ballast specifications. According to their study, alteration to a smaller size would: reduce considerably maintenance requirements for ballasted railway track; mitigate noise levels; improve quality of ride; and extend life of ballast. Therefore, size and grading are key factors for railway ballast.

In this section, the 2/3 scaled size Network Rail (NR) ballast which was tested for strength in chapter 3, was used for box test experiments. The settlement and stiffness results were then compared to those from testing of full-size ballast. In chapter 3, the monotonic large-scale triaxial tests were conducted on full-size and 2/3 scaled size NR samples, and the peak strength and peak angle of friction calculated for both of the samples. The calculated results showed that the values of peak strength and peak angle of friction for the 2/3 scaled size samples were generally comparable to or slightly higher than those of the full-size sample. In terms of permeability of 2/3 scale ballast compared to full-size ballast, although the 2/3 scaled ballast can offer less, it was still within the satisfactory drainage range based on NR/L2/TRK/4239 (NetworkRail, 2015).

The concrete, plastic and timber sleeper sections used in this section 0 were similar to those presented in section using the full-size ballast.

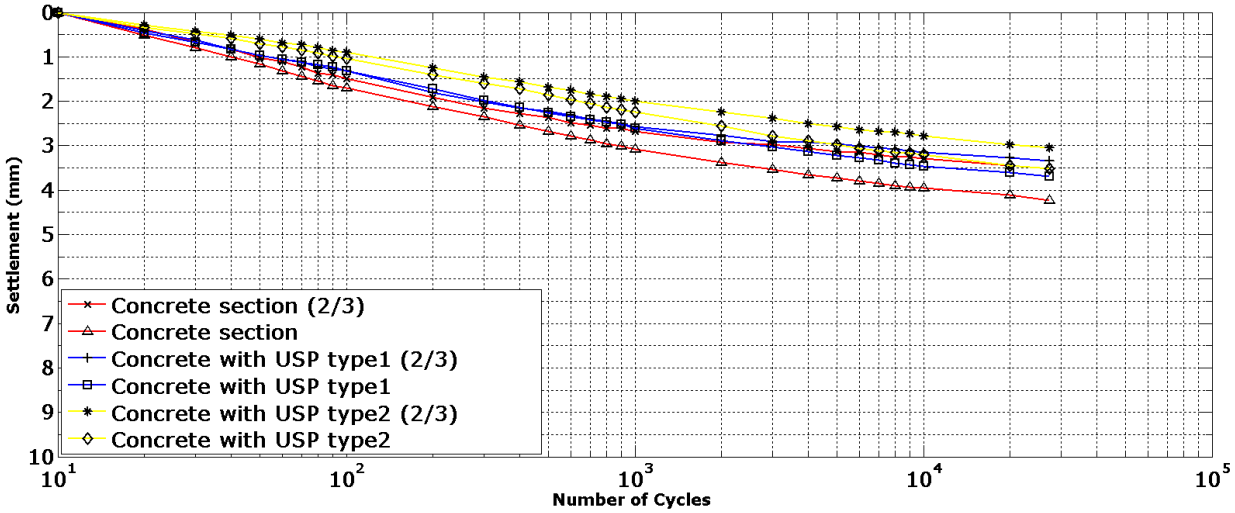
Results

Figure 4-36 to Figure 4-38 show the evolution of settlement and ballast bed stiffness for each sleeper section using 2/3 scaled ballast with and without USP type1 and type2 (stiff and soft). In the figures, the 2/3 scaled ballast results were also compared with those from the full-size ballast tests.

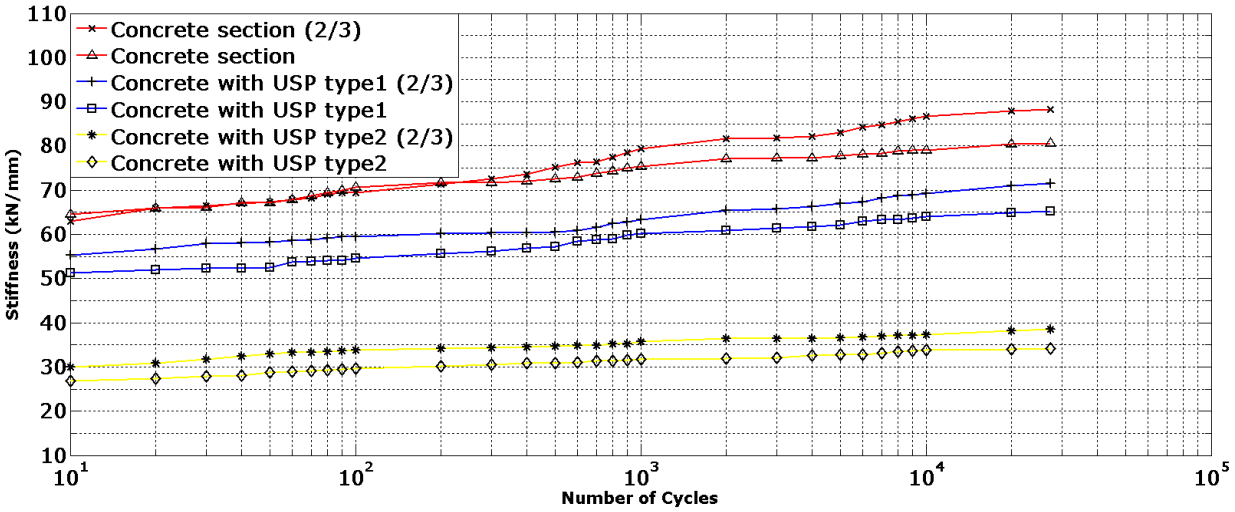
In the case of the concrete section using 2/3 scaled ballast (Figure 4-36 (a)), a reduction in settlement was observed for both USP type1 and type2; however, the resulting settlement when full-size ballast was used was higher compared to that of 2/3 scaled ballast. It is also noticeable in Figure 4-36 (a) that the difference in the settlement results was marginal when 2/3 scaled ballast was used both with and without USPs. This could be due to the increase in the number

of contact points between ballast particles and sleeper section base as a result of using smaller ballast particles compared to the full-scale ballast.

When stiffness is considered in the case of the concrete section with 2/3 scaled ballast (Figure 4-36 (b)), similar to full-scale ballast, inclusion of USPs (regardless of type) reduced the stiffness; however comparison of the results showed higher stiffness when 2/3 scaled ballast was used in all the concrete section test cases.



(a)



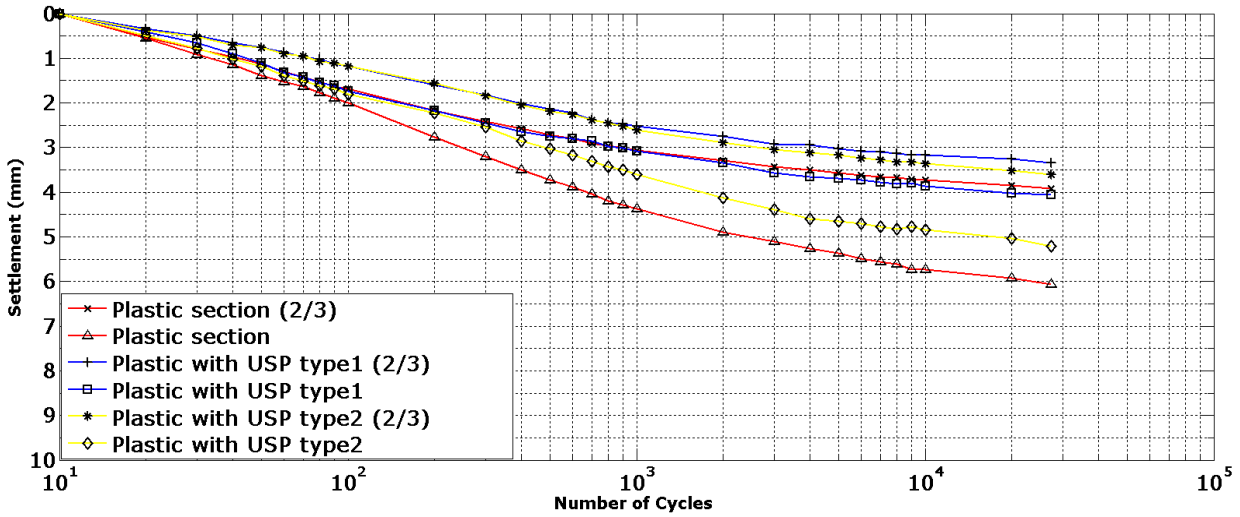
(b)

Figure 4-36 Effect of USP inclusion on concrete section performance using full-size and 2/3 scaled size ballast (a) settlement (b) trackbed stiffness

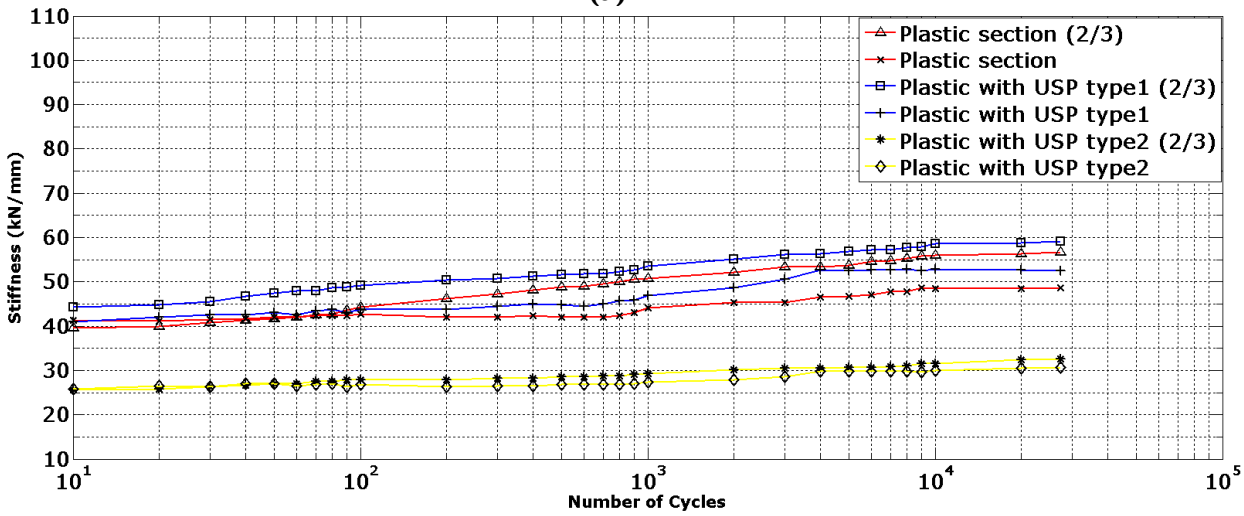
For the plastic section (Figure 4-37(a)), inclusion of both USP types resulted in settlement reductions when 2/3 scaled ballast was used but the improvement was slightly higher in the case USP type1 compared to type2. When the full-size ballast was compared to 2/3 scaled size in the case of the plastic sleeper, a difference in resulting settlement was evident. It was

observed that use of 2/3 scaled ballast (in the case of 'no USP') improved the settlement performance of the plastic sleeper by 56%. The settlement performance of the plastic section in the case of 'no USP' using 2/3 scaled ballast was comparable to that of the plastic section with USP type1 using full-size ballast.

With regard to stiffness (Figure 4-37 (b)), similar to the full-size ballast, in the case of 2/3 scaled ballast, use of the USP type1 increased slightly the stiffness of trackbed compared to 'no USP'; however, less trackbed stiffness resulted when the USP type2 was used. The use of 2/3 scaled ballast with the plastic section increased the stiffness in all test cases when compared to those from using of full-size ballast. However, the differences in stiffness results were not as evident as of those in the settlement results.



(a)

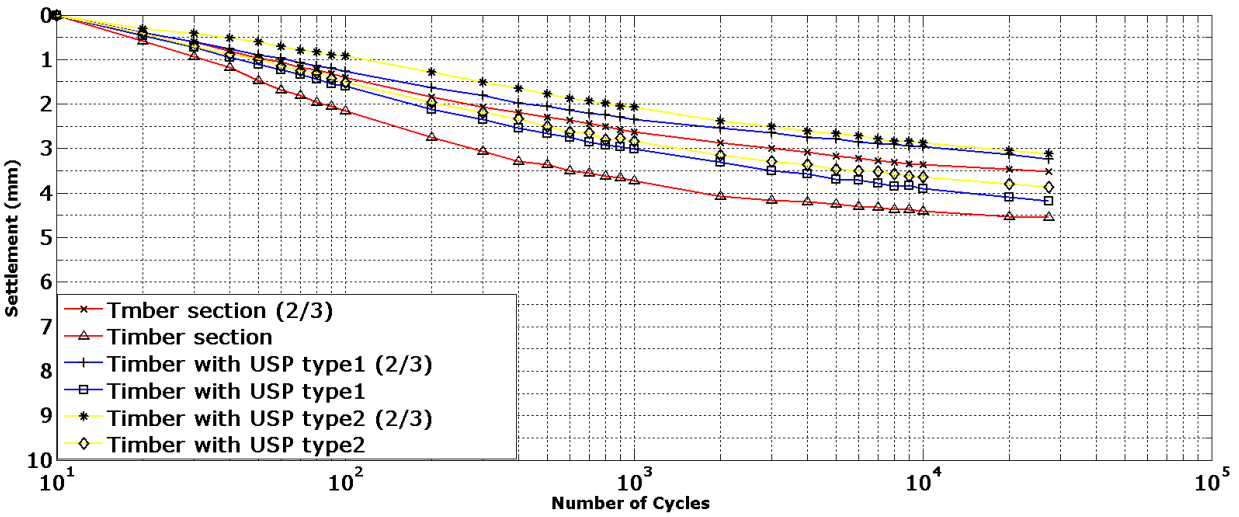


(b)

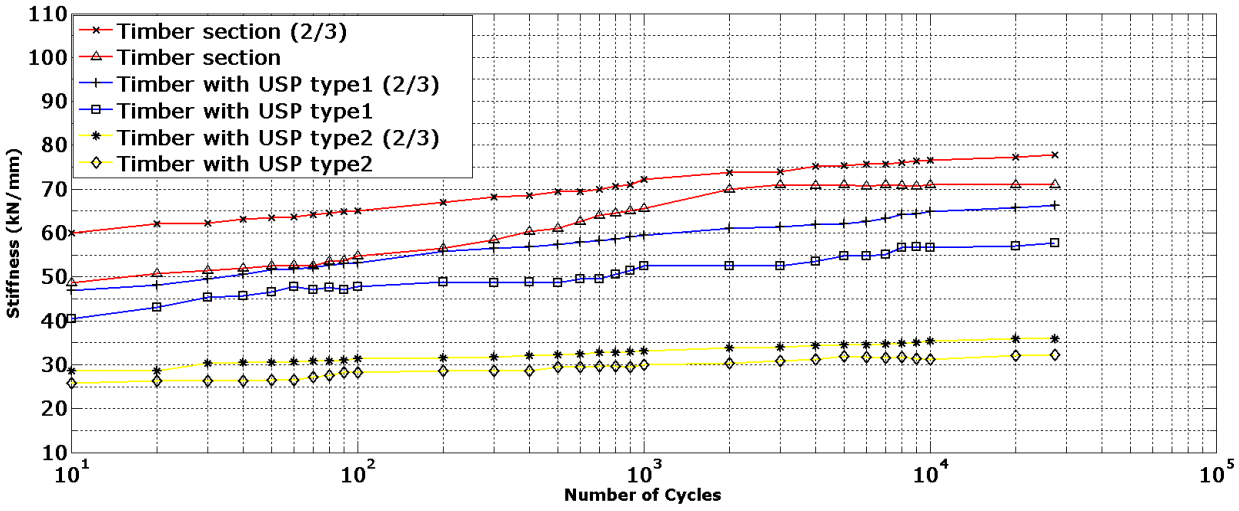
Figure 4-37 Effect of USP inclusion on plastic section performance using full-size and 2/3 scaled size ballast (a) settlement (b) trackbed stiffness

For the timber section (Figure 4-38(a)), the USP type2 resulted in the least settlement reduction when 2/3 scaled ballast was used, consistent with the settlement results for timber section tests with USPs and full-size ballast. As can be seen in Figure 4-38 (a), the timber section in the case 'no USP' using 2/3 scaled ballast performed better than the timber section with USP type2 when full-size ballast was used (timber gave the best settlement performance when full-size ballast was used).

Where trackbed stiffness is concerned (Figure 4-38 (b)), use of USP caused a clear reduction in stiffness in the case of timber section with 2/3 scaled ballast. However, these values compared to stiffness values when full-size ballast was used showed an increase in stiffness for all the tests.



(a)



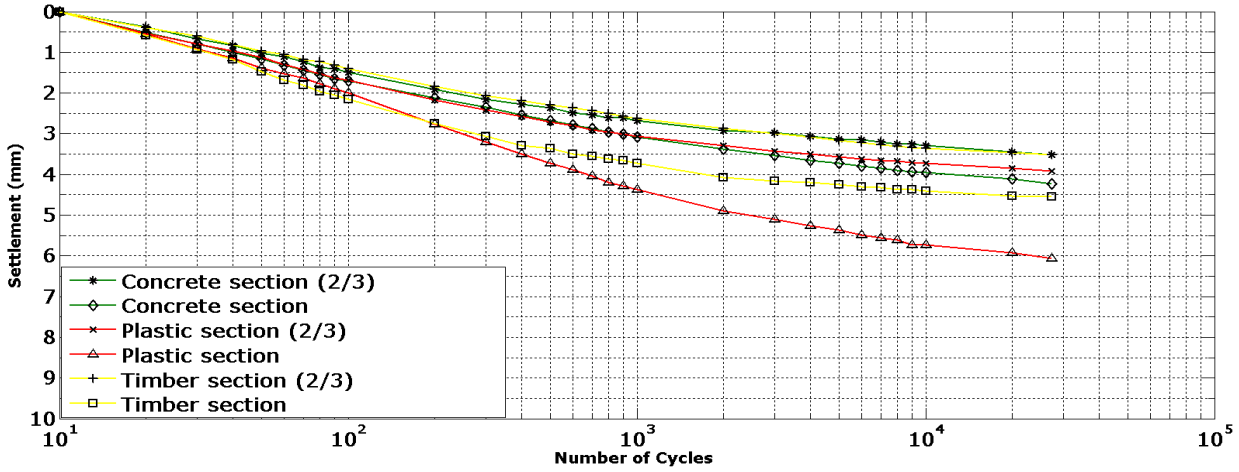
(b)

Figure 4-38 Effect of USP inclusion on timber section performance using full-size and 2/3 scaled size ballast (a) settlement (b) trackbed stiffness

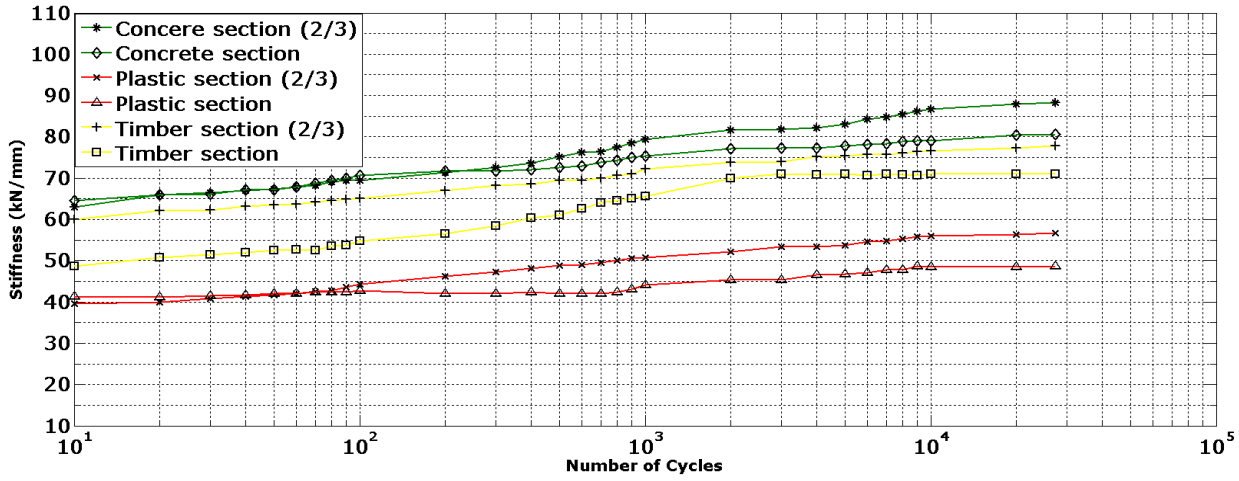
Figure 4-39 to Figure 4-41, compare the performance of the sleeper section types to one another with and without the use of USPs in both 2/3 scale and full-size cases.

The main observation from the settlement results (Figure 4-39 (a)) without USPs is that the 2/3 scale ballast tests yielded lower settlement compared to those from testing of full-size ballast (regardless of sleeper section type). However, the highest improvement in settlement was observed in the case of the plastic sleeper section (approximately 36%), and the least improvement in the case of the concrete section (approximately 17%), when 2/3 scale ballast was used.

With regard to the stiffness results (Figure 4-39 (b)), when the USPs were not considered, the 2/3 scale ballast sample tests showed higher stiffness (regardless of sleeper section type) than in full-size ballast tests. However, the increases in stiffness in the case of 2/3 scale ballast tests were almost comparable when different sleeper sections were tested.



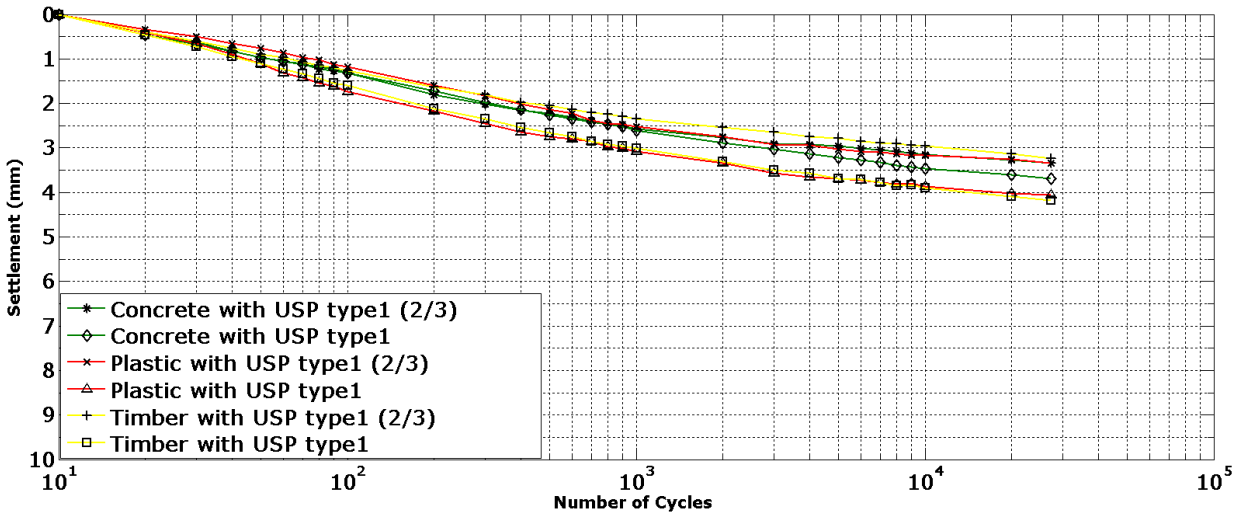
(a)



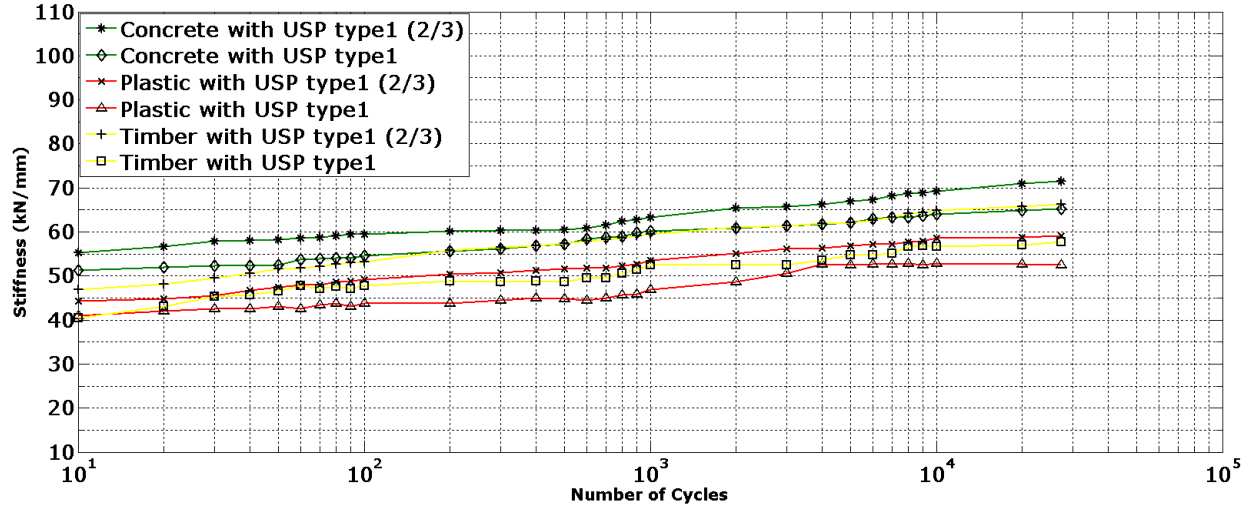
(b)

Figure 4-39 (a) Evolution of sleeper section displacement for sleeper sections using full-size and 2/3 scaled size ballast (b) Evolution of trackbed stiffness using full-size and 2/3 scaled size ballast

When USP type1 was considered (Figure 4-40 (a) and (b)), better settlement performance was recorded when a 2/3 scale ballast sample was used compared to full-size ballast (regardless of sleeper section type) but the improvement in settlement was slightly lower than in the 'no USP' scenario. Of all the sleeper sections, timber resulted in the largest improvement in settlement (approximately 23%) comparable with the plastic section (approximately 19%) and the concrete section showed the least settlement improvement (approximately 11%). In terms of stiffness when USP type1 was used, similar to the 'no USP' scenario, higher stiffness results were recorded in the case of 2/3 scale ballast compared to the full-size ballast irrespective of sleeper section type.



(a)

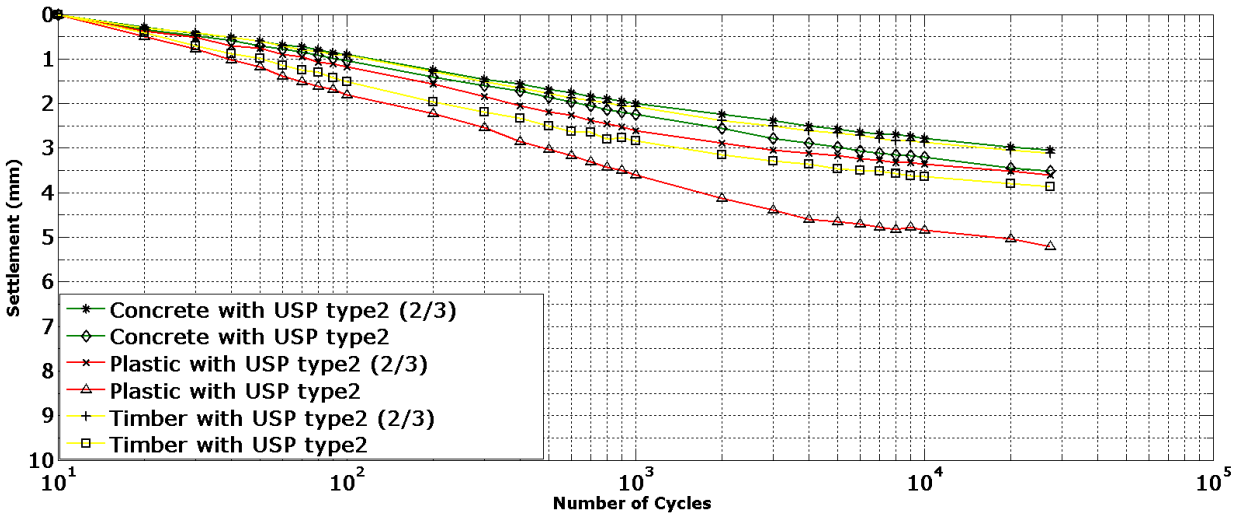


(b)

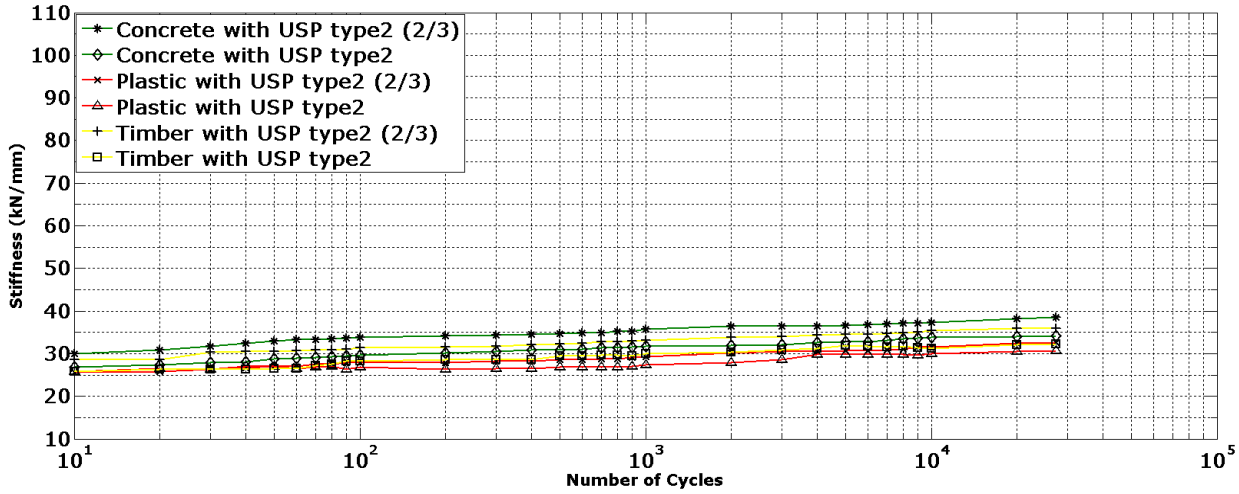
Figure 4-40 (a) Evolution of sleeper section displacement for sleeper sections with USP type1 using full-size and 2/3 scaled size ballast (b) Evolution of trackbed stiffness for sleepers with USP type1 using full-size and 2/3 scaled size ballast

In the case of USP type2 (Figure 4-41 (a) and (b)), similarly improved settlement performance was observed when 2/3 scale ballast was used compared to full-sized ballast (irrespective of sleeper section type). However, comparable to the 'no USP' scenario, the highest improvement in settlement was observed in the case of the plastic sleeper section (approximately 31%), and the least improvement was observed in the case of the concrete section sleeper (approximately 14%), when 2/3 scale ballast was used.

Where trackbed stiffness for USP type2 is concerned, little variability was observed in the case of 2/3 scale (similar to full-size ballast), regardless of sleeper section type. Nevertheless, the 2/3 scale ballast slightly increased the trackbed stiffness compared to the full-size ballast in all tests.



(a)



(b)

Figure 4-41 (a) Evolution of sleeper section displacement for sleeper sections with USP type2 using full-size and 2/3 scaled size ballast (b) Evolution of trackbed stiffness for sleepers with USP type2 using full-size and 2/3 scaled size ballast

The sleeper settlements and trackbed stiffnesses (shown as averages over 30,000 cycles), are presented in Figure 4-42 and Figure 4-43 respectively.

As can be seen in Figure 4-42, regardless of sleeper and USP types used, the variability in measured settlements between the different sleeper sections when 2/3 scale ballast was used, was very low compared to full-sized ballast. The settlement results in all the tests undertaken showed the 2/3 scale ballast to give improved overall settlement performance. Of all the tests carried out, the highest improvement in settlement was recorded for the plastic sleeper section 'no USP' (36%) and the least was observed for the concrete section with USP type1 (11%). Furthermore, of all the tests undertaken, the lowest settlement was given by the concrete section with USP type2 and 2/3 scale ballast, comparable to that of the timber sleeper section with USP type2 and 2/3 scale ballast. On the other hand, the highest settlement was given by plastic sleeper section 'no USP' with full-size ballast.

As shown Figure 4-43, irrespective of the type of USP and sleeper section used, the measured trackbed stiffness for the 2/3 scale ballast was comparable to that of the full-size. However, the 2/3 scale ballast slightly increased the trackbed stiffnesses in all the test cases.

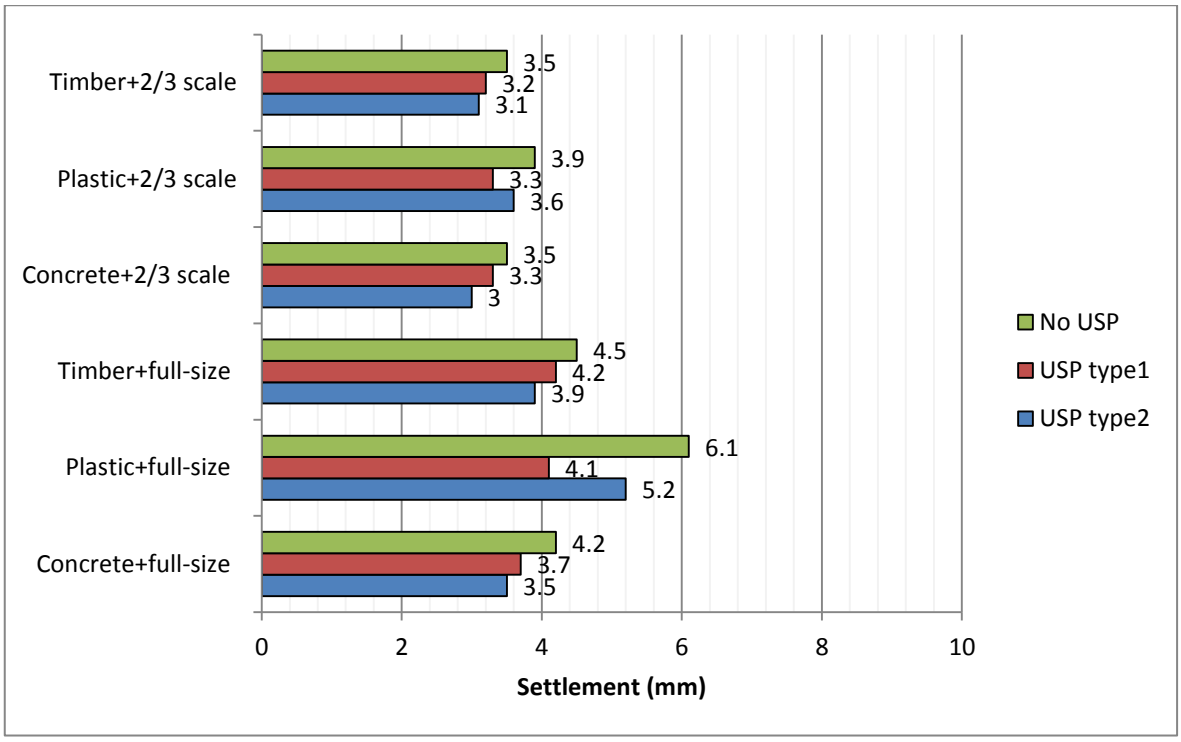


Figure 4-42 Sleeper section settlement at 30,000 cycles with and without USP using full-size and 2/3 scaled size ballast

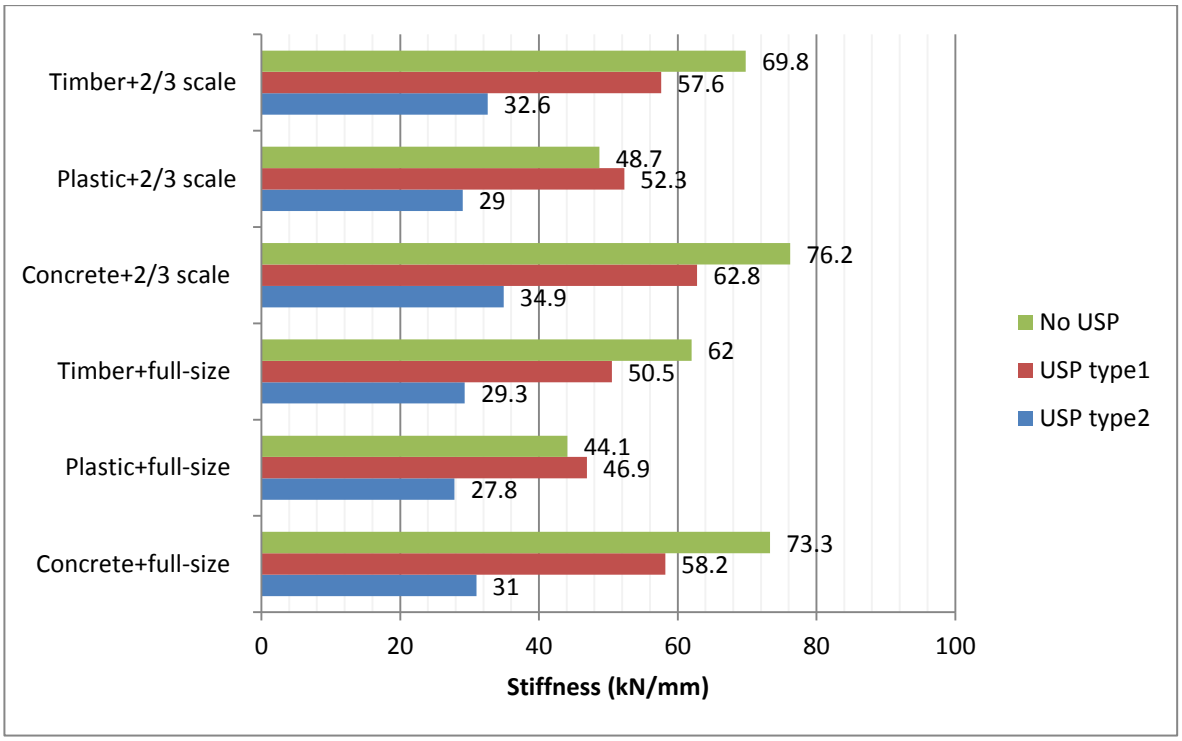


Figure 4-43 Average trackbed stiffness over 30,000 cycles with and without USP using full-size and 2/3 scaled size ballast

4.8.4 TWO-LAYERED BALLAST SYSTEM

In the previous section, results showed that use of the 2/3 scale ballast can improve settlement performance of a sleeper section up to 36% compared to the full-size ballast, depending on the sleeper section type used. In this section, two-layered ballast systems, namely sample1 (Figure 4-44) and sample2 (Figure 4-45), were tested in terms of settlement and trackbed stiffness performance using the box test apparatus. Sample1 consisted of a 200-mm full-size ballast layer underneath the sleeper followed by a 100-mm 2/3 scale ballast layer at the bottom. Sample 2 contained a 100-mm 2/3 scale ballast layer underneath the sleeper followed by a 200-mm full-size ballast layer at the bottom. In the case of both these ballast samples, the type of ballast used for the crib ballast was identical to the ballast type used directly underneath the sleeper section. Sample 1 was similar to a typical ballast-subballast system that includes large aggregate at the top and small aggregate at the bottom. Sample 2 was proposed with the aim of increasing the number of contact points between ballast and sleeper at the ballast-sleeper interface. In this section, performance of sample 2 was compared against that of sample 1 in terms of settlement and trackbed stiffness using the box test apparatus and the concrete section as a sleeper. The aforementioned two-layered ballast systems were also compared with the single-layered systems using full-size and 2/3 scaled size ballast separately.

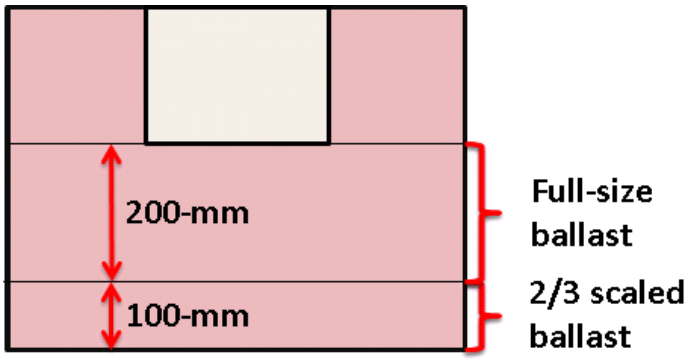


Figure 4-44: Two-layered ballast Sample 1

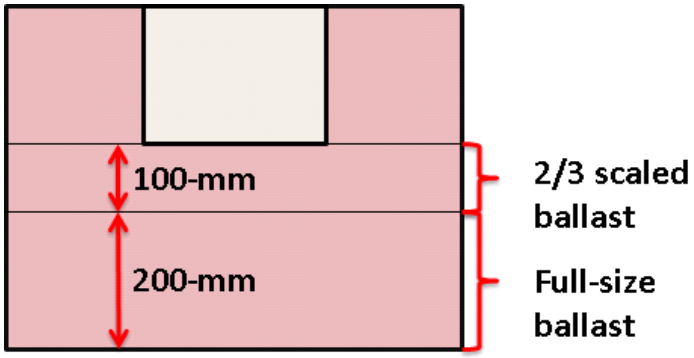
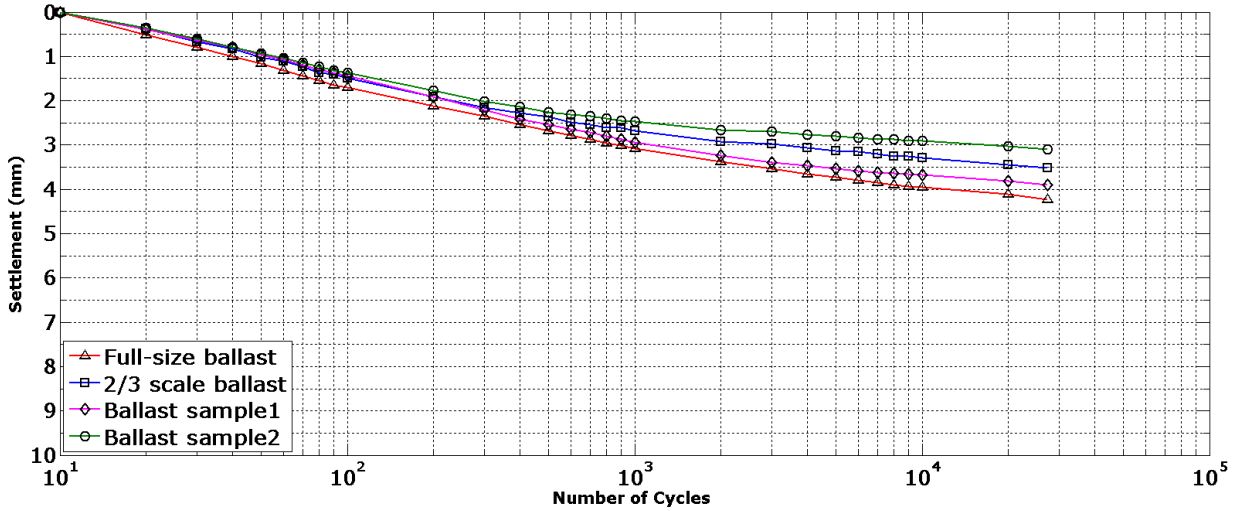


Figure 4-45: Two-layered ballast Sample 2

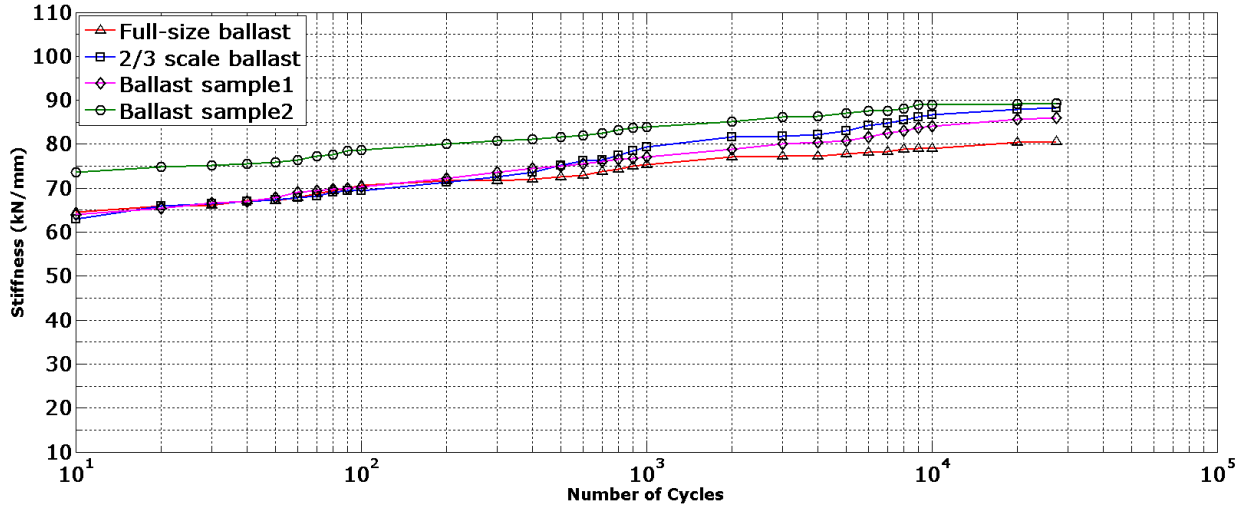
Results

Figure 4-46 (a) and (b) shows the evolution of settlement and ballast bed stiffness respectively for different ballast sample systems including single-layered and two-layered systems. In the case of two-layered ballast samples, sample 2 showed better performance in terms of settlement results compared to sample 1. As can be observed in Figure 4-46 (b), the settlement performance of ballast sample 2 was also better than both single-layered ballast samples namely full-sized and 2/3 scale ballast. When ballast sample1 was used, higher trackbed settlement was observed compared to the 2/3 scale ballast sample; however it was less than the settlement recorded for the full-sized ballast sample.

With regard to trackbed stiffness, of the two-layered ballast samples used, higher stiffness results were observed for ballast sample 2 compared to ballast sample 1; however recorded stiffness for ballast sample 2 was comparable to that of the 2/3 scale ballast sample.



(a)



(b)

Figure 4-46 Evolution of different ballast systems in terms of (a) settlement (b) trackbed stiffness

Figure 4-47 and Figure 4-48 show settlement and average stiffness results for all types of ballast systems used, including single-layered and two-layered systems. As can be observed, of all the single-layered and two-layered tests undertaken, the lowest settlement results were recorded for two-layered system sample 2 and the highest was measured for the single-layered system full-size ballast. In terms of stiffness, the lowest value was observed for single-layered system using full-size ballast was used.

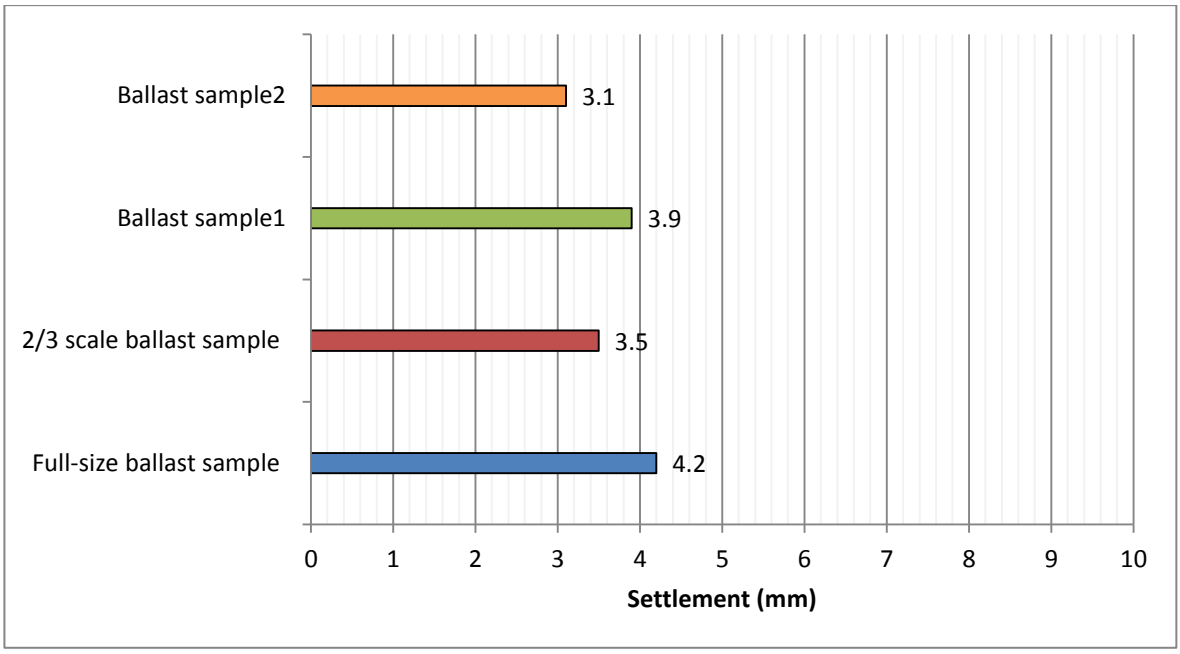


Figure 4-47 Settlement at 30,000 cycles for different ballast systems

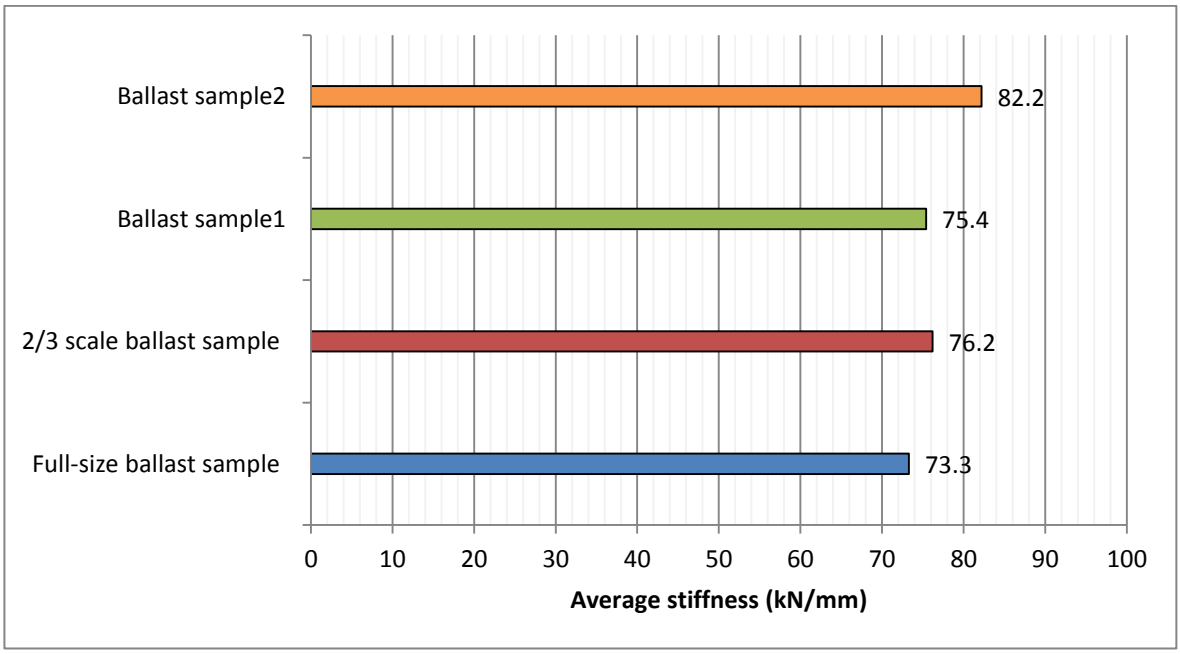


Figure 4-48 Average stiffness over 30,000 cycles for different ballast systems

4.8.5 EFFECT OF USP STIFFNESS ON TRACKBED SETTLEMENT AND STIFFNESS

In section 0, each sleeper section (except the U-section) was tested in turn with USP type1 and USP type2, using full-size ballast. Regardless of USP type, in all test cases, results showed that use of USPs reduced the trackbed settlement. As mentioned in section 0, USP type1 and USP type2 have different thickness and stiffness, representing stiff and soft variants respectively. In

this section, an extra layer of USPs were used to obtain USPs with a greater range of stiffness and affixed to the concrete section and tested using the box test apparatus. The aim was to investigate the effect of USP stiffness on trackbed settlement and stiffness, and to obtain an indicative understanding of optimal USP stiffness in order to improve trackbed settlement.

The first test of each USP type was started by attaching a double-layer USP to the underside of the concrete section. Subsequent tests were then carried out using an additional layer of USP. This was continued for each USP type until no improvement was observed in measured trackbed settlement.

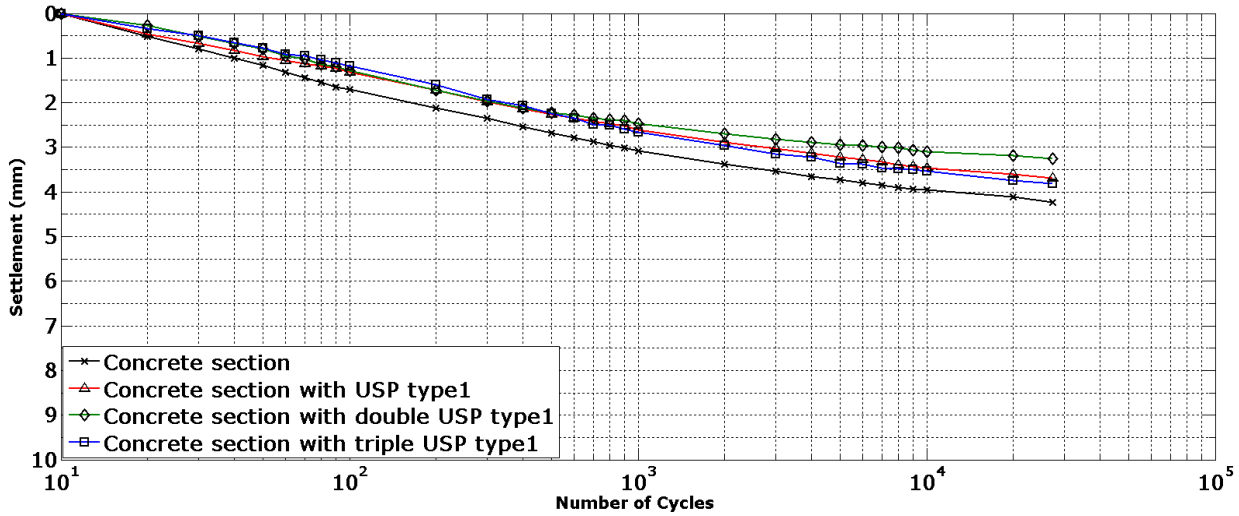
The stiffness for multi-layer USPs was calculated with the assumption that USP layers behave as a series of springs. Therefore the stiffness of a double-layer USP would be half of the stiffness of a single-layer USP.

Results

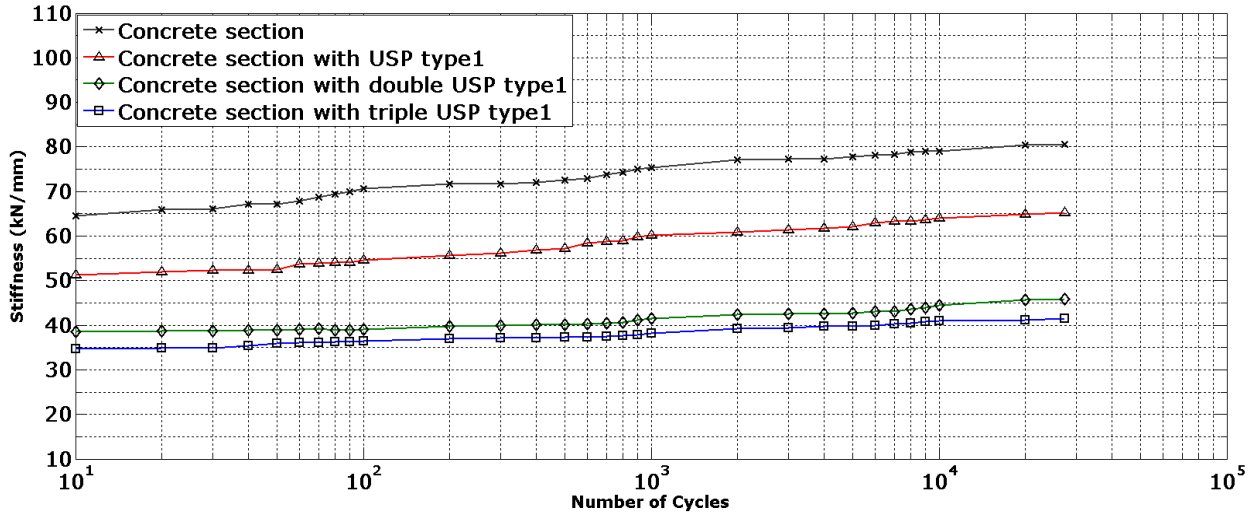
In Figure 4-49 and Figure 4-50, the effects of multi-layer USPs on the concrete section sleeper with respect to settlement and trackbed stiffness are shown.

As it can be seen in Figure 4-49 (a), it was observed that use of a double-layer USP type1 reduced the concrete section settlement compared to the single-layer and ‘no-USP’ scenarios. Moreover, when an additional layer of USP type1 was attached to make a triple-layer USP type1, higher settlement results were recorded compared to the double-layer USP type1. However, the settlement results recorded were still lower than the ‘no USP’ scenario and comparable to the single-layer USP type1 results. Therefore, no extra tests were conducted on additional layers of USP type1.

The results of trackbed stiffness for the multi-layer USP type1 tests are shown in Figure 4-49 (b). It was observed in the figure that attaching additional layers of USP type1 reduced the track bed stiffness. However, the differences in stiffness reduction between the ‘No USP’ scenario and single-layer USP were higher than those between the double-layer and triple-layer USP.



(a)

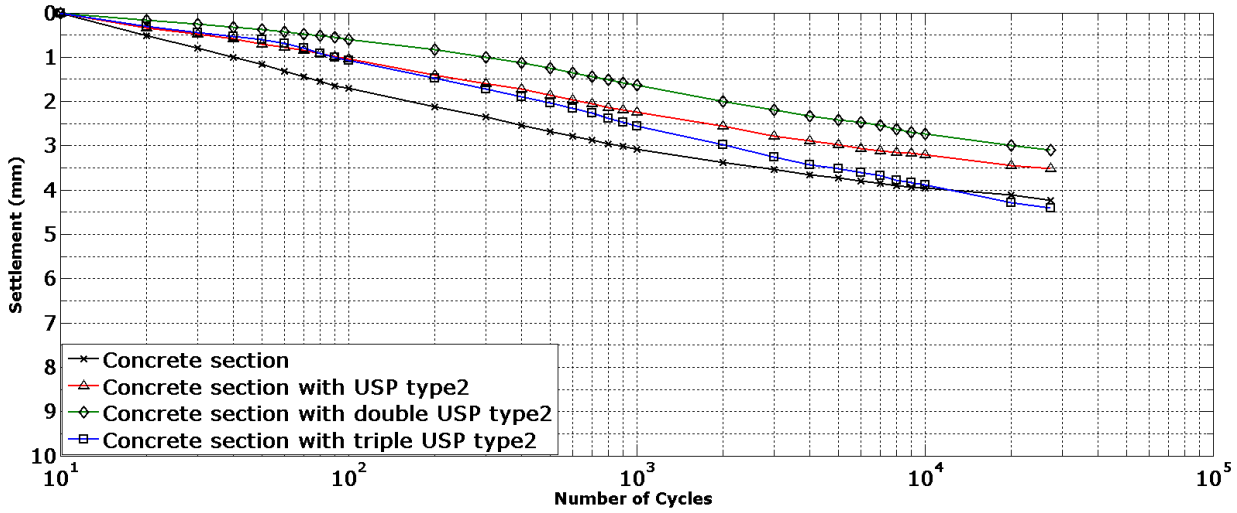


(b)

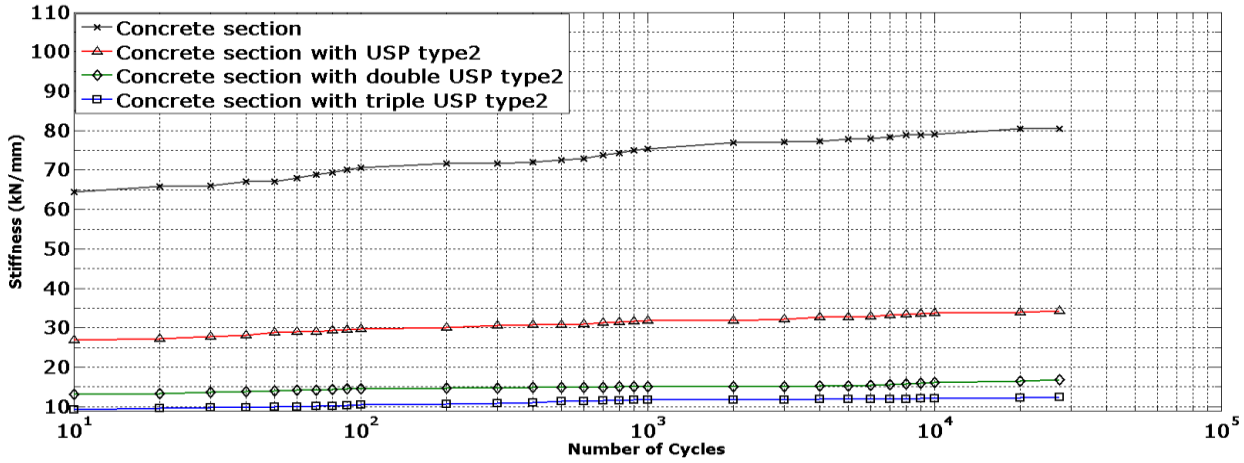
Figure 4-49 Effect of multiple-layer USP type1 inclusion on concrete section performance (a) settlement (b) trackbed stiffness

In the case of USP type2 (Figure 4-50 (a) and (b)), a double-layer USP type2 resulted in higher settlement reduction compared to the single-layer USP and 'no USP' scenarios. However, when the triple-layer USP type2 was considered, higher settlement was observed compared to the double-layer, single-layer, and the 'no USP' scenario. Therefore no further tests on additional layers of USP were carried out.

With regard to trackbed stiffness (Figure 4-50 (b)), multi-layer USP type2 showed reductions in stiffness, and compared to the USP type1 higher reductions in stiffness were observed.



(a)



(b)

Figure 4-50 Effect of multiple-layer USP type2 inclusion on concrete section performance (a) settlement (b) trackbed stiffness

In Table 4-7, the stiffness and thickness details of all the USP multi-layer type1 and type2 tests are presented. Figure 4-51 and Figure 4-52 show settlement and average stiffness results. As can be seen in the figures, the lowest settlement result was observed in the case of the double-layer USP type2, comparable to the result for double-layer USP type1. Therefore, the stiffness range for double-layer USP can be considered as an indication of the optimal stiffness range to improve the trackbed settlement.

Table 4-7 Details of multi-layer USP type1 and type2

Box Test Experiment	USP Stiffness (N/mm^3)	USP thickness (mm)
Concrete Section	-	-
Concrete with USP type1	0.228-0.311	4
Concrete with USP type2	0.079-0.105	9
Concrete with Double USP type1	0.114-0.155	8
Concrete with Double USP type2	0.0395-0.0525	18
Concrete with Triple USP type1	0.076-0.103	12
Concrete with Triple USP type2	0.0263-0.0.035	27

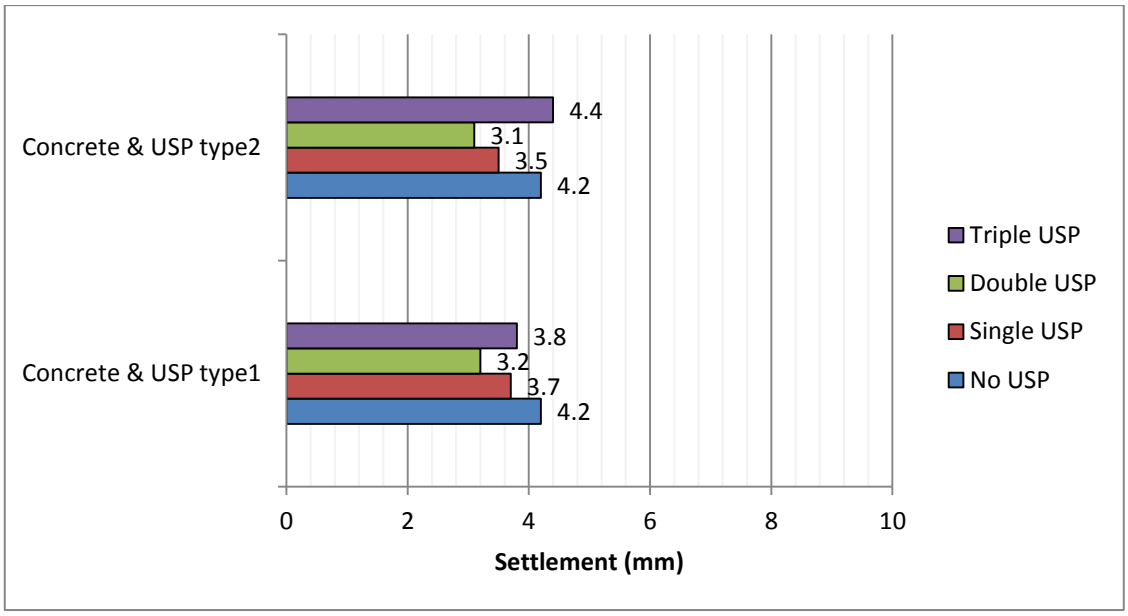


Figure 4-51 Settlement at 30,000 cycles for multi-layer USP type1 and type2

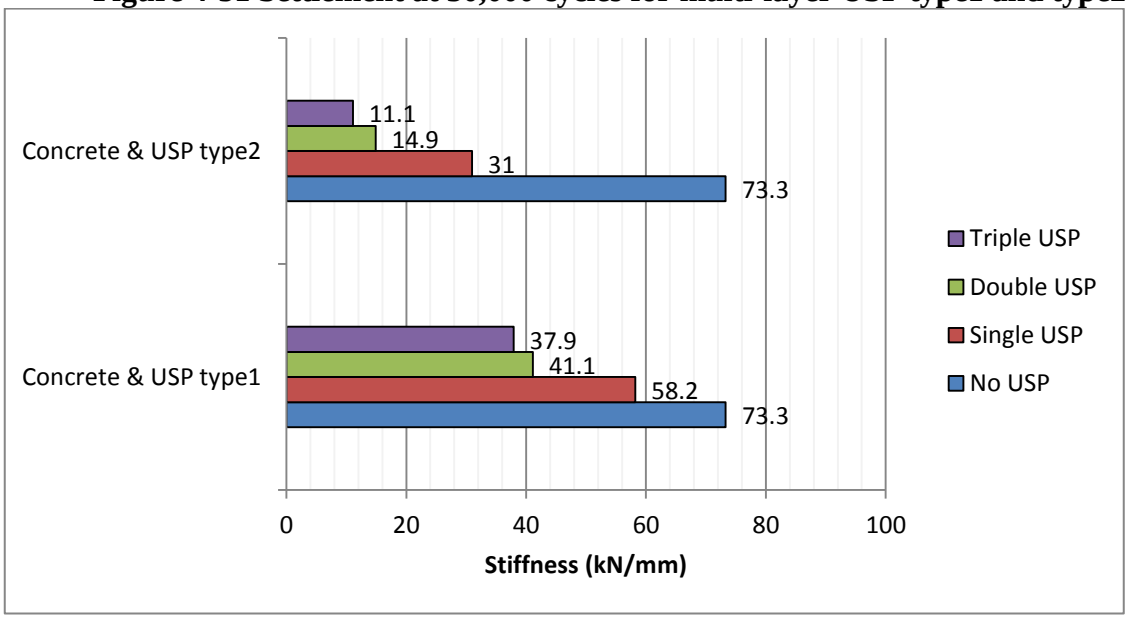


Figure 4-52 Average trackbed stiffness over 30,000 cycles for multi-layer USP type1 and type2

4.9 SUMMARY

A total of 41 unique tests were conducted over two phases using the box test, 13 tests in phase 1 and 28 tests in phase 2, a summary of which is as follows:

- The U-section (representing a steel sleeper) and concrete section accumulated the least permanent settlement in the case of 'no USP' and full-size ballast.

- When USPs were included (regardless of type) in the case of full-size ballast, all of the tests showed reduced trackbed settlement. However, there were no great differences in settlement performance for a given USP, i.e. the sleeper material becomes almost irrelevant. Additionally, the application of USPs appeared to generally confer greater resiliency to the trackbed and as evidenced by the results has potential USPs have application in track areas requiring spot re-sleepering (where one sleeper section type without a USP could be just as effective as another sleeper section type that has a USP attached). In other words, USPs may be used to offset differences in the performance of different sleeper types.
- Visual inspection of the ballast beds (in the case of full-size ballast) before and after all tests undertaken with and without USPs showed no obvious ballast deterioration.
- Use of frictional sleepers and USPs generally showed improvement in settlement. However, the improvement in settlement in the case of a frictional USP was higher compared to that of a frictional sleeper. Trackbed stiffness was also increased slightly when frictional sleepers and USPs were used compared to those of the normal concrete section and USP.
- The inclusion of polystyrene particles (regardless of size) did not improve the trackbed settlement (with the exception of up to 2.5% small size polystyrene inclusions) and it is therefore not suggested. Of the large and small size polystyrene inclusions, the small-sized particles resulted in lower ballast layer settlements and higher stiffness values compared to large inclusions when 1% and 2.5% polystyrene inclusions were used. However, in the case of 5% and 10% polystyrene inclusions, the large size led to lower ballast layer settlements and greater stiffness values compared to small size.
- Investigation of sleeper section types and USPs using 2/3 scale ballast showed improvement in settlement performance in all of the test cases when it was compared to similar tests using full-sized ballast. Therefore, the use of finer ballast grading is suggested. Of all the tests carried out with 2/3 scale ballast and full-size ballast, the largest improvement in settlement was recorded for the plastic sleeper section (36%) and the lowest was observed for the concrete section with USP type1 (11%). However. The best settlement performance was observed in the case of a concrete section with USP2 and 2/3 scale ballast, comparable to that of the timber sleeper section with USP2 and 2/3 scale ballast. On the other hand, the largest settlement was given by the plastic sleeper section with full-sized ballast. With regard to the measured trackbed stiffness, the 2/3 scale ballast test results were comparable to those of the full-sized ballast. However, use of 2/3 scale ballast slightly increased the trackbed stiffness in all the test cases compared to the full-size ballast.

- In the case of two-layered ballast systems compared with single-layered ballast systems, results showed the lowest settlement for ballast two-layered ballas sample2 (finder ballast grading on top) and the highest was measured for the single-layered ballast system using full-size ballast. In terms of stiffness, the highest value was recorded when the two-layered ballast sample2 was used and the lowest was recorded in the case of the two-layered ballast sample1. There for the use of two-layered ballast system with fine ballast grading on top is recommended.
- So far as use of USP types with various stiffnesses is concerned, the double-layer USPs showed a better settlement performance compared to single-layer and triple-layer USPs (irrespective of type). However, the lowest settlement result was observed in the case of double-layer USP type2, comparable to the settlement result double-layer USP type1. Therefore, the USP stiffness range of double-layer USPs can be considered as an indication of the optimal stiffness range to improve trackbed settlement.

5. COMPOSITE ELEMENT TEST (CET)

5.1 INTRODUCTION

In this chapter using the Composite Element Test (CET) apparatus laboratory experiments were conducted to validate and check repeatability of the box test results in a condition with reduced degree of ballast confinement. The CET apparatus was also used to conduct experiments that were difficult to perform using the box test apparatus, e.g. gluing ballast.

Similar to the box tests, the CET tests were carried out in two phases. The first series of tests was performed after the box tests in phase one. In this CET phase, the performance of sleeper section types with and without USPs was investigated (in terms of permanent settlement and stiffness). Then similar tests were conducted using the full-scale test facility (RTF) which will be presented in Chapter 6. Once the consistency of the results were confirmed by comparing the results from box tests and CET tests (in phase one) with those from the RTF tests, the second series of CET tests were conducted. The tests in phase 2 of CET involved ballast gluing, use of sleeper section types and USPs using 2/3 scale ballast, and use of USP types with various stiffnesses. The details and results of the aforementioned tests will be presented later in this chapter.

Descriptions of the CET apparatus, materials used and test procedures are also presented in this chapter followed by presentation of key outcomes of tests carried out.

5.2 COMPOSITE ELEMENT TEST (CET) APPARATUS

The CET was first used by Kwan (2006) to investigate the effect of ballast reinforcement on trackbed performance using biaxial Tensar geogrid. His research included the geogrid geometry, geogrid placement and the effect of subgrade stiffness variables. The CET apparatus was originally designed to approximately simulate a simplified format of a underside a sleeper condition under cyclic loading. The test can provide meaningful direct data on the comparative performance of track component variables, e.g. sleeper types and USPs. Figure 5-1 and Figure 5-2 show the CET setups Kwan (2006) used in his study. However, in this investigation, the aforementioned setups were amended to create a condition more closely representing the RTF and in-situ. It should be noted that the role of the CET was not direct track simulation but an

element test to assess the effects of the key track variables. Figure 5-3 presents a schematic of the CET setup used in this study. After Kwan (2006), the CET has been used in a number of studies, since it allowed conducting of a fairly large number of tests under simplified conditions to investigate the effect of different track component variables, within a reasonable time and cost (Brown et al., 2006, Brown et al., 2007b, Das, 2010). Shi (2009) used the CET to monitor the ballast layer deformation to predict the trackbed permanent deformation. Moreover, Chen et al. (2013) simulated the CET to study the interlock characteristics of ballast-geogrid under various boundary conditions.

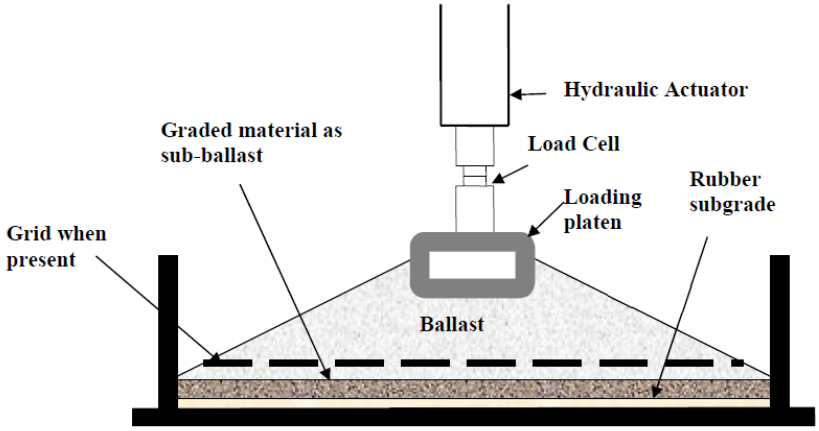


Figure 5-1 First CET setup used by Kwan (2006)

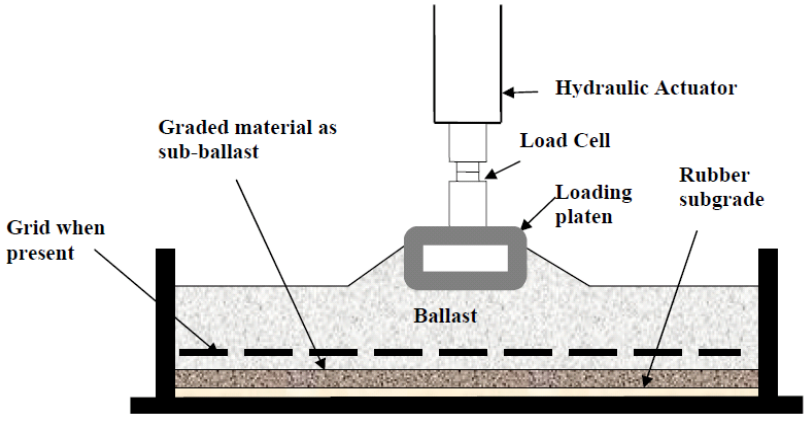


Figure 5-2 Second CET setup used by Kwan (2006)

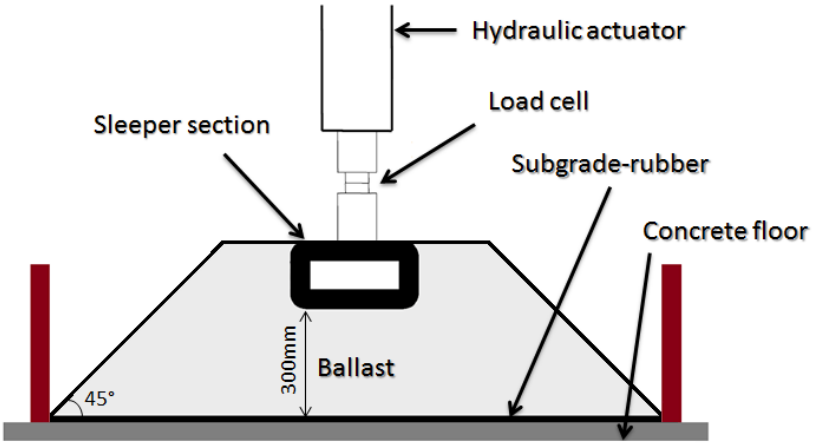


Figure 5-3 Schematic of the CET setup used in this investigation

The CET apparatus used in this study, was basically a trough-like area with 0.7m width and 1.4m length dimensions. The trough was made of two wooden walls the inner sides of which were covered by a 5-mm thick steel sheet to decrease the friction with ballast particles. The walls were sufficiently high to place 300-mm ballast underneath the sleeper section. Similar to the box test, rubber mats were used to simulate a typical subgrade reaction.

5.3 MATERIALS

The full-size and 2/3 scale size ballast used for the CET tests were similar to those used for box tests and the large-scale triaxial tests, in terms of both source and specifications.

The sleeper sections used were steel rectangular, concrete, plastic and timber sections; they were similar to those used for box test but with 700-mm lengths (Table 5-1). The U section was not studied in the CET apparatus due to installation difficulties that required pushing the section into the ballast bed. This was not possible due to the reduced confinement condition of the ballast layer in the CET compared to the box test.

Similar to the box tests, the USPs which were used for the CET tests were from Tiflex limited namely USP type1 (stiff) and USP type2 (soft).

The same polystyrene as used in the study presented in the previous chapter (section 4.8.2), was used together with D-Limonene (orange terpene oil) (>98% purity) to produce the glue for the purpose of gluing ballast, the details of which will be presented later in this chapter.

Table 5-1 Dimensions of representative sleeper sections

`	Width (mm)	Depth (mm)	Length (mm)	Weight (kg)	Wall thickness (mm)	Percentage of full size (%)	
						length	weight
Rectangular Steel box	250	150	700	40.8	10	28	13.5*
Concrete	285	210	700	100.3	-	28	32.6
Plastic	250	150	700	25.6	-	26.8	25.9
Timber	290	170	700	37.3	-	26.8	22.4

* With respect to a full-sized concrete sleeper

5.4 SAMPLE PREPARATION AND TESTING PROCEDURE

The ballast bed for each CET test was constructed by placing 300-mm layers of ballast and compacting using a vibrating plate compactor. The top surface of the ballast was then manually levelled prior to positioning the sleeper section on top and placing the crib ballast. Since the CET arrangement provided less confinement for the ballast bed compared to the box test, it was not possible to compact the ballast bed in layers after every 100-mm of ballast placement, as was done for the box test. Therefore, it was performed once the 300-mm ballast had been placed in order to avoid ballast falling down from the sides and to achieve 300-mm layers of ballast underneath the sleeper section. The same aforementioned profile with side shoulder slopes of 45 degrees were ensured for each test by marking on the inner wall of the trough. The arrangement set-up for the CET is presented in Figure 5-4.

Cyclic loading was applied by a servo-hydraulic actuator at a frequency of 2Hz, for 30,000 cycles applying a sinusoidal load between 2kN and 20kN. The 2kN seating load was to avoid loss of contact between loading piston and the sleeper section. The 20kN load and 2Hz frequency were the maximum achievable levels; trial CET tests carried in advance showed that higher frequency and cyclic loading than aforementioned values, would not maintain the ballast profile; since there is a lower level of confinement compare to the box test. Application of 20kN using the rectangular hollow section induces a contact stress of 114 kPa underneath the sleeper section, which is approximately half of the maximum expected on an actual track.

As can be seen in Figure 5-4, four LVTDs at the sides of sleeper section top surface were used to measure the settlement data. The data acquisition system used for the CET was identical to the one used for the box test, but the LabVIEW programme was adapted for the CET loading and frequency condition. The collected data was then analysed using MATLAB and presented at the following intervals: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 200, 300, 400, 500, 600, 700,

800, 900, 1000, 2000, 3000, 4000, 5000, 6000, 7000, 8000, 9000, 10000 20000 and 30000 cycles.

Similar to box test, the first 10 cycles of the data were also ignored in the presentation of all the settlement and stiffness results, as they effectively represent bedding into the ballast of the sleeper section and were not reproducible between tests.



Figure 5-4 The CET test setup used in this investigation

5.5 TESTING PROGRAMME

As presented in Table 5-2, a total of 31 unique CET tests were conducted in two phases. In order to verify the repeatability of the results and avoid any systematic and random errors, the first few tests (sleeper type tests) were repeated, subsequently only a single test was done on each case. The difference was less than 3% in all cases. The presented data in this chapter are from the average value of the repeated test results in each case. In phase 1, 12 CET tests were undertaken investigating performance of different sleeper section types with and without USPs using full-sized ballast. After completion of full-scale tests using the Nottingham Railway Test Facility (RTF) and checking the consistency of the results of CET and RTF, phase 2 of CET testing was undertaken. This phase of CET testing involved use of ballast gluing, sleeper section types and USPs using 2/3 scale ballast, and use of USP types with various stiffnesses.

Table 5-2 Programme of tests undertaken using CET apparatus

Phase	Test Title	Test description
Phase 1	Sleeper Section Type	**Sleeper section used:** hollow, concrete, plastic and timber
	Under Sleeper Pads	**USP type1 with**: hollow, concrete, plastic and timber section **USP type 2 with:** hollow, concrete, plastic and timber section
Phase 2	Ballast Gluing	**Concrete section with:** glued ballast glued ballast and USP type2 broken-glue ballast broken-glue ballast with USP type2
	2/3 Scaled Ballast	**Sleeper section used:** concrete, plastic and timber **USP Type1 with:** concrete, plastic and timber section **USP Type 2 with**: concrete, plastic and timber section
	Multi-Layer USP	**Concrete section with:** single-layer USP type1 double-layered USP type1 triple-layered USP type1 single-layer USP type2 double-layered USP type2 triple-layered USP type2

5.6 SLEEPER SECTION TYPES AND USP EXPERIMENTS USING FULL-SIZE BALLAST (CET PHASE ONE)

Twelve CET tests were conducted on different sleeper section types described in section 5.3 with and without USPs using full-sized ballast. The sleeper section settlement and the trackbed stiffness results for all the test are presented as plots against the number of cycles (over 30,000 cycles). Trackbed stiffness was calculated using the same method explained in section 4.7.2.

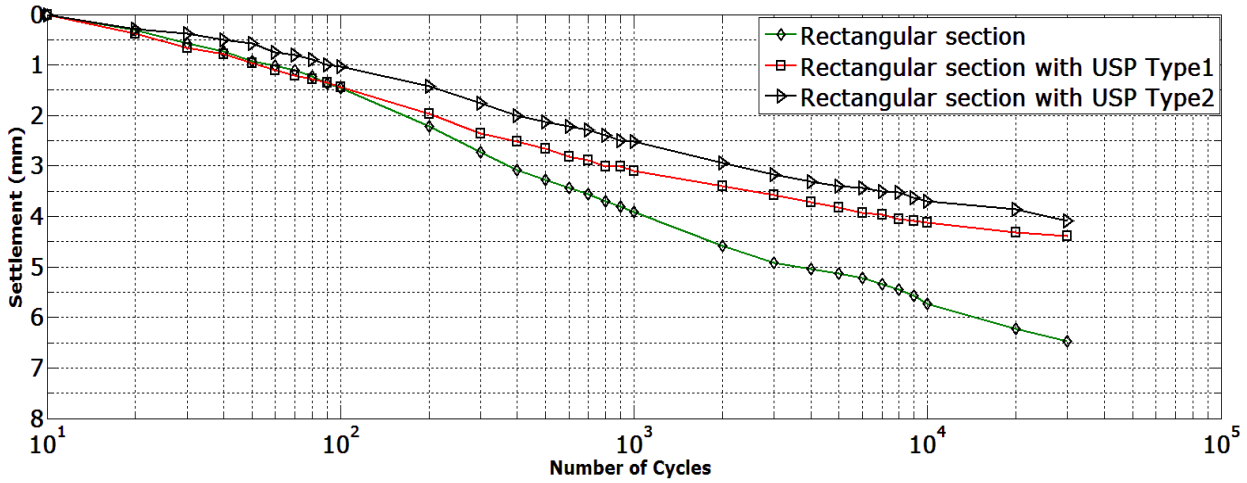
Both CET settlement and stiffness results showed a similar trend compared to those of the box test in terms of performance ranking for sleeper section types with and without USPs. However, the magnitude of the results was less when compared with the box test results. The CET offered less confinement for the ballast layers compared to the box test, meaning greater freedom for ballast movement; however, the magnitude of the applied load in the case of the CET was half of that in the case of the box test. Furthermore, sleeper sections in the CET tests were 700-mm long whereas they were 300-mm in box tests, meaning the stress level in the case of the box test was about 2.3 times higher compared to the CET. Therefore, notwithstanding the

differences in the nature of the box test and CET apparatus, the arguments for the observed permanent deformations and trackbed stiffness resulting from the use of different sleeper sections with and without USP are similar to those discussed for the box tests.

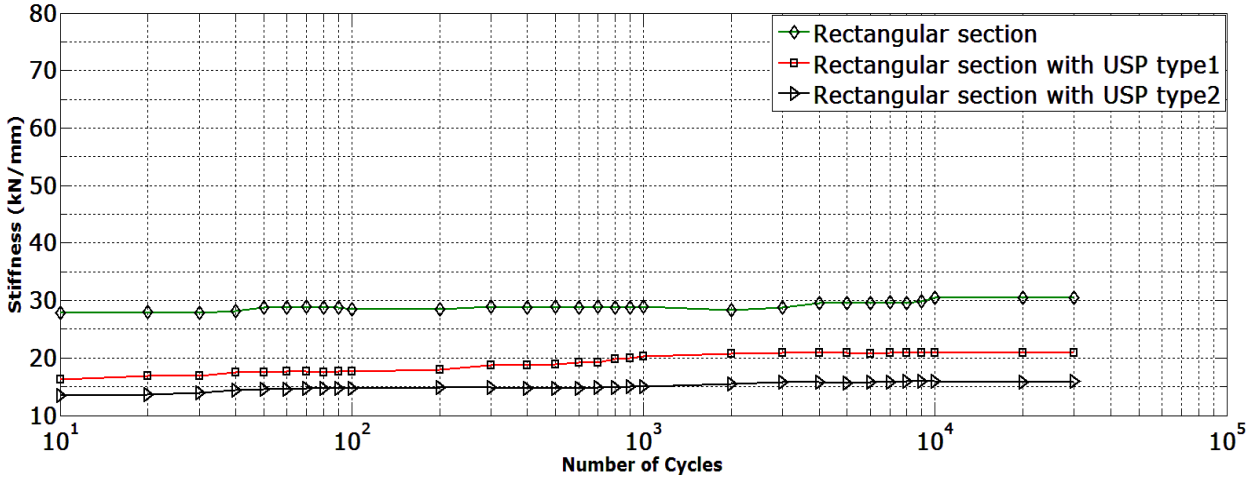
5.6.1 RESULTS

Figure 5-5 to Figure 5-8 present the evolution of settlement and trackbed stiffness for each sleeper with and without USPs.

In the case of the rectangular section (Figure 5-5), the difference between use of USPs (regardless of type) and 'no USP' was evident in settlement results. This can be a result of the distinct friction difference between the very rigid surface of the steel section and the flexible surface of the USPs (Table 4-4) that increased the number of contacts at the ballast-sleeper interface and hence resulted a better load spread and lower settlement.



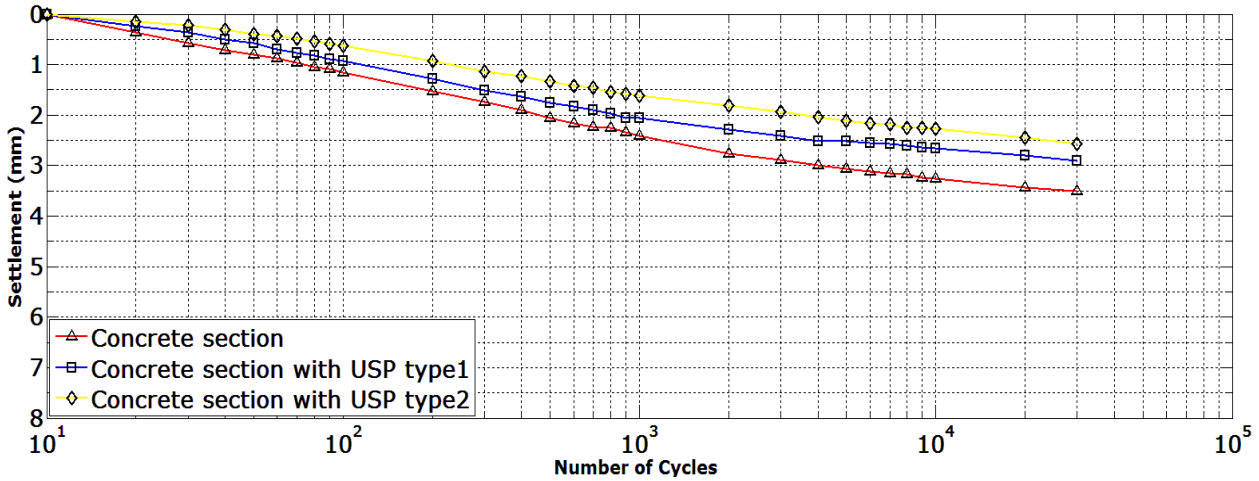
(a)



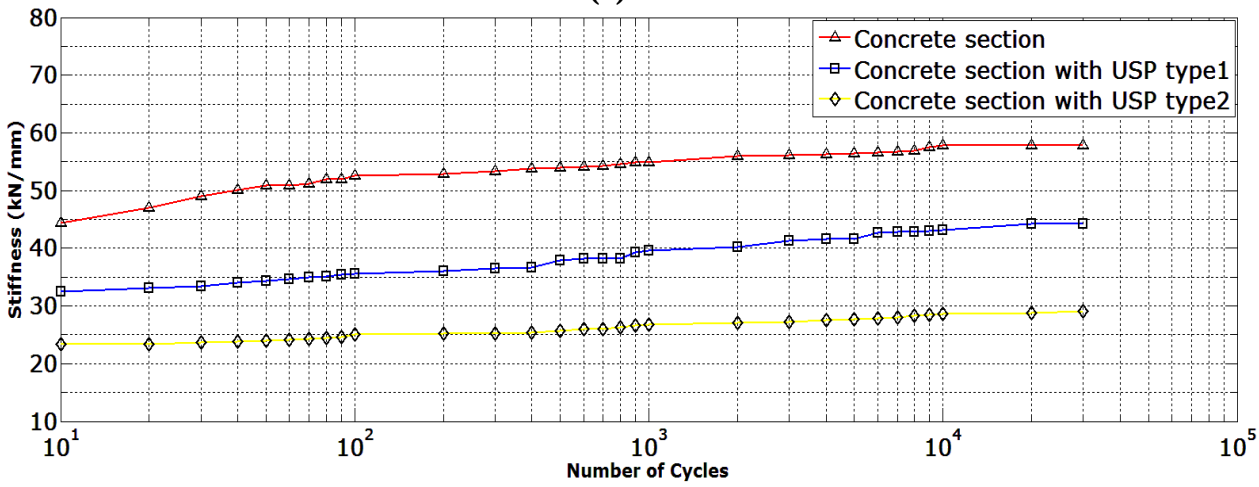
(b)

Figure 5-5 Effect of USP inclusion on rectangular section performance (a) settlement (b) trackbed stiffness

For the concrete section (Figure 5-6), improvement in settlement was not as evident as for the rectangular section; however, the difference in stiffness results between use of USP and 'no USP' was obvious. The comparable settlement for the USP cases and 'no USP' cases is consistent with the comparable coefficient of friction at the ballast-sleeper interface for these cases. The evident difference in stiffness can be a resulted of the elastic behaviour of the USP, especially in the case of USP type 2, which decreased the stiffness of the overall trackbed.



(a)

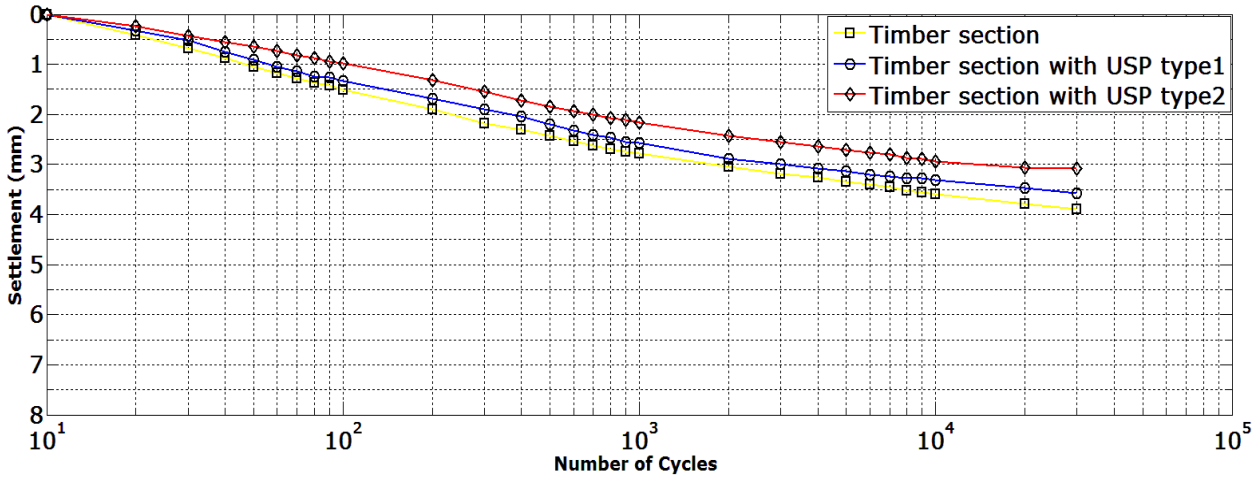


(b)

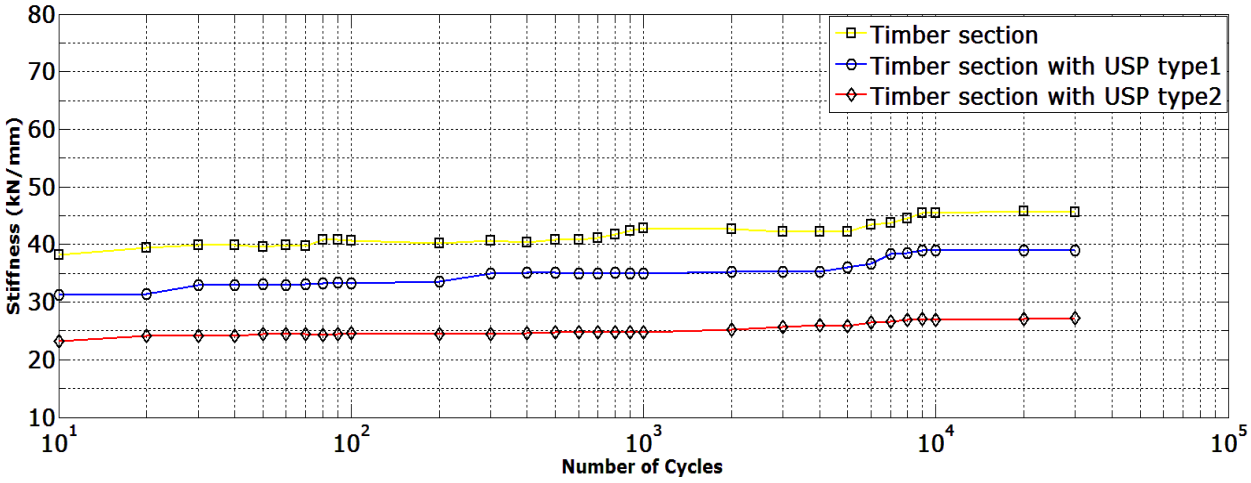
Figure 5-6 Effect of USP inclusion on concrete section performance (a) settlement (b) trackbed stiffness

The performance of the timber section in terms of both settlement and stiffness (Figure 5-7) was close to the performance of concrete. However, when USPs were used (irrespective of type) the timber section performed slightly better than concrete. This can be due to the slightly wider dimension of the timber section compared to concrete, which increased the number of contacts

at the ballast-sleeper interface and also decreased the stress at the interface and the overall settlement.



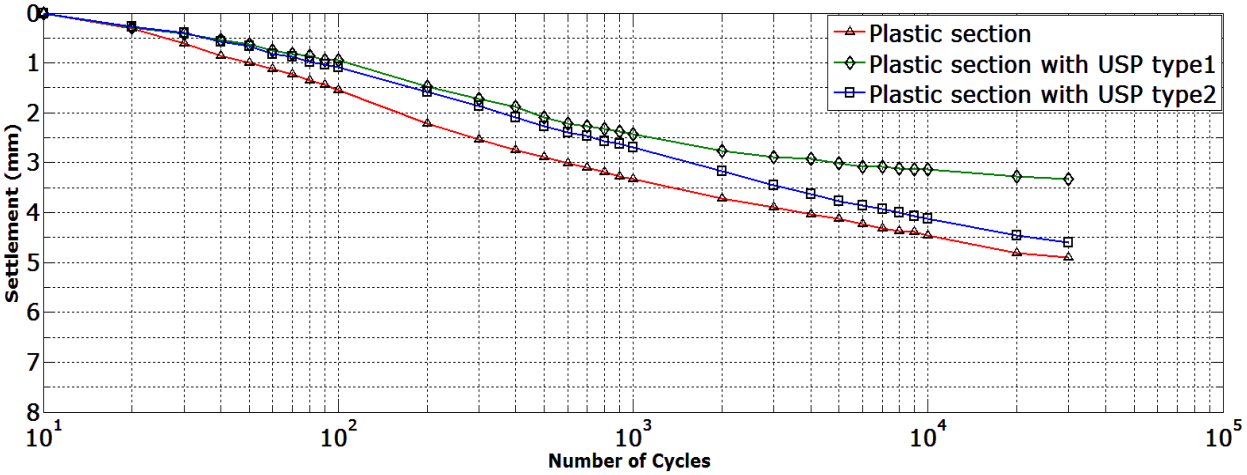
(a)



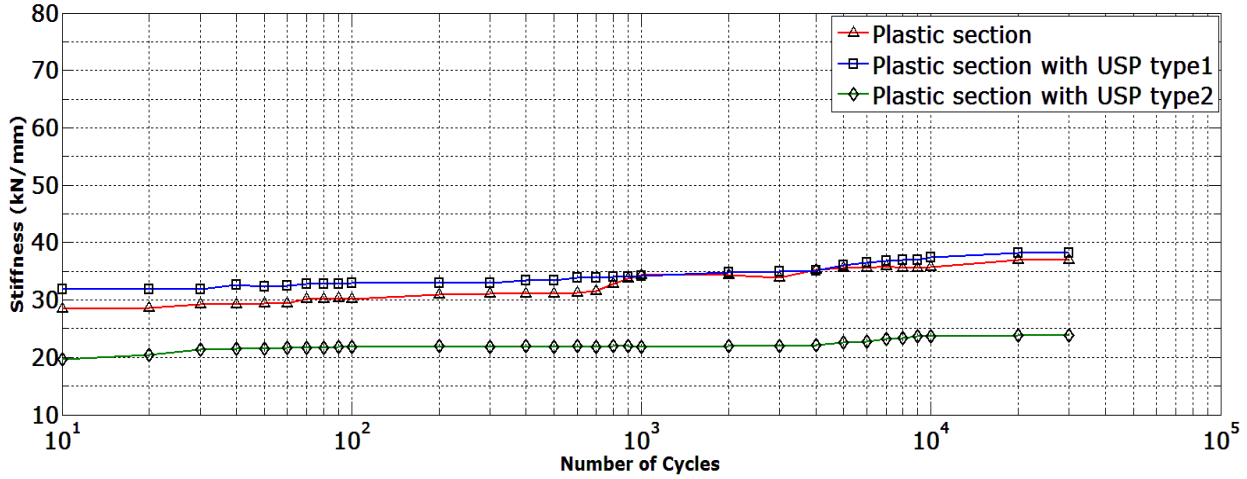
(b)

Figure 5-7 Effect of USP inclusion on timber section performance (a) settlement (b) trackbed stiffness

For the plastic sleeper (Figure 5-8), the overall settlement value was higher compared to that for the concrete and timber sections. This can be a result of the smaller dimensions of plastic in both width and depth compared to the concrete and timber sections, which caused greater stress at the ballast-sleeper interface. Similar to the box test, the plastic section and USP type1 performed better in terms of overall settlement compared to that for the plastic section and USP type2.



(a)



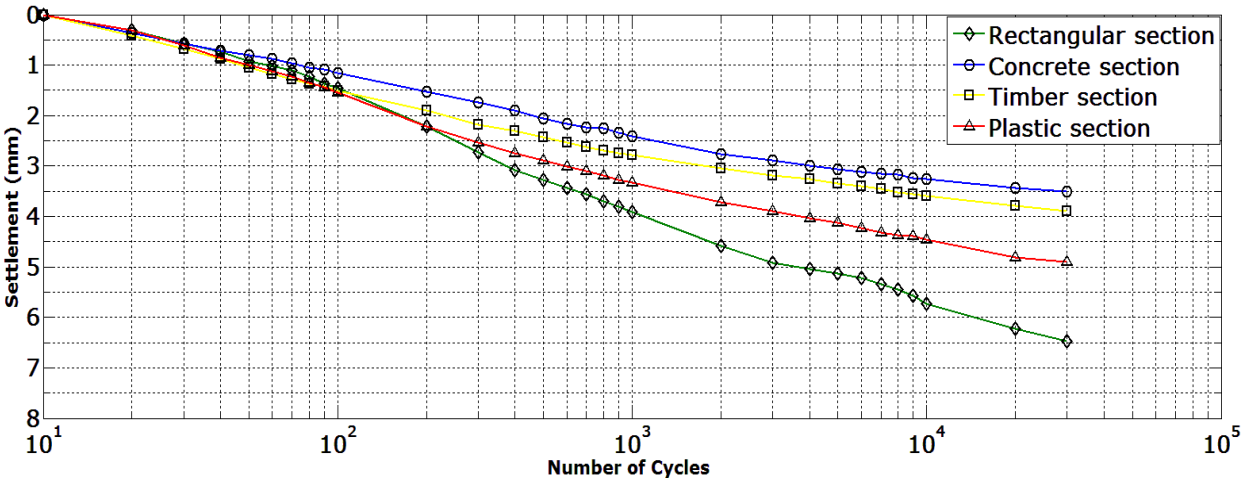
(b)

Figure 5-8 Effect of USP inclusion on plastic section performance (a) settlement (b) trackbed stiffness

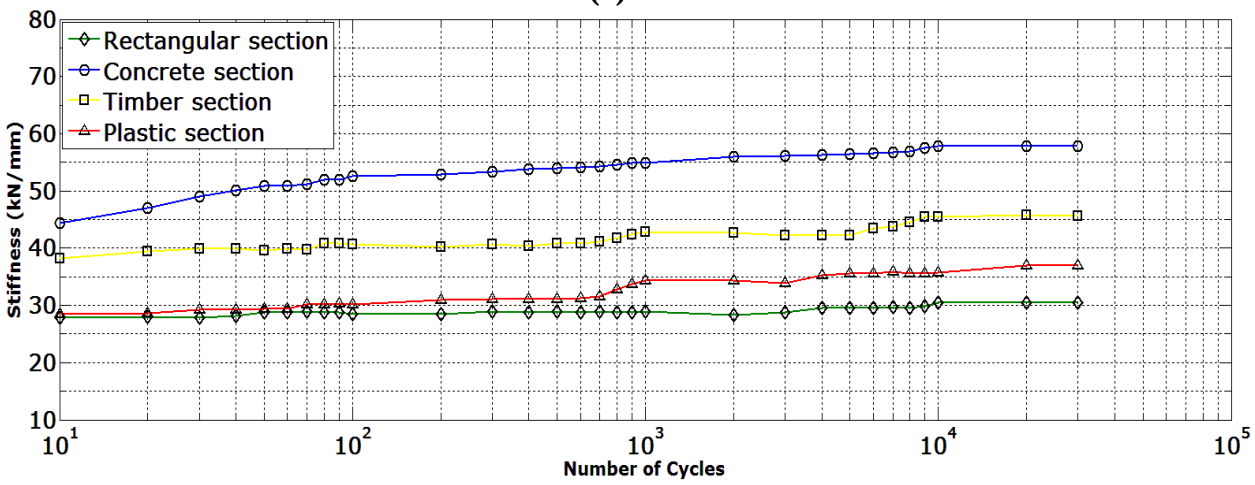
Figure 5-9 shows the settlement of all sleeper section types in the case of 'no USP'. As can be seen, the rectangular section yielded the most settlement and the concrete section the least, comparable to the timber section (consistent with box test results). So far as stiffness is concerned, the lowest stiffness was observed in the case of the rectangular section and the highest in the case of the concrete section. Similar to the box test results, in the case of CET tests for the sleeper type sections, a higher trackbed stiffness resulted in a lower overall sleeper section settlement.

When sleeper section types are considered with USP type1 and type2 (Figure 5-10 and Figure 5-11 respectively), less variability is observed in terms of both sleeper settlement performance and trackbed stiffness (consistent with the box test), in particular in the case of concrete, timber and plastic sections. The slightly different settlement performance of the rectangular section compared to the rest of the sleeper sections (although it had similar dimensions to the plastic

sleeper), can be explained by the rounded geometry of the base of the rectangular section (Figure 4-4 (b)) that effectively reduced the contact area between the section and the ballast, thereby applying a higher stress over a smaller area of ballast.



(a)



(b)

Figure 5-9 (a) Evolution of sleeper section displacement for sleeper sections without USPs (b) Evolution of trackbed stiffness for sleepers without USPs

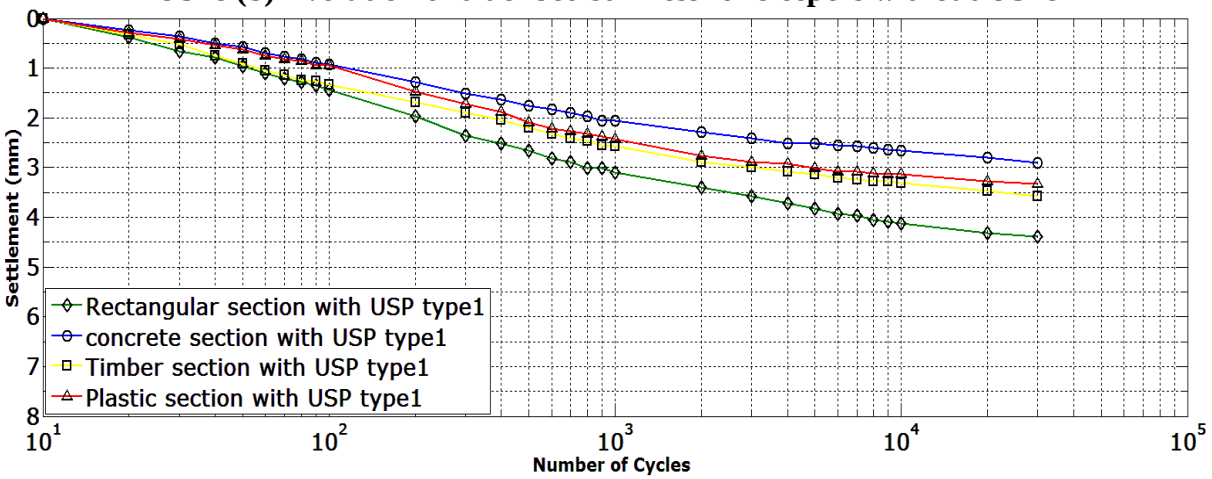
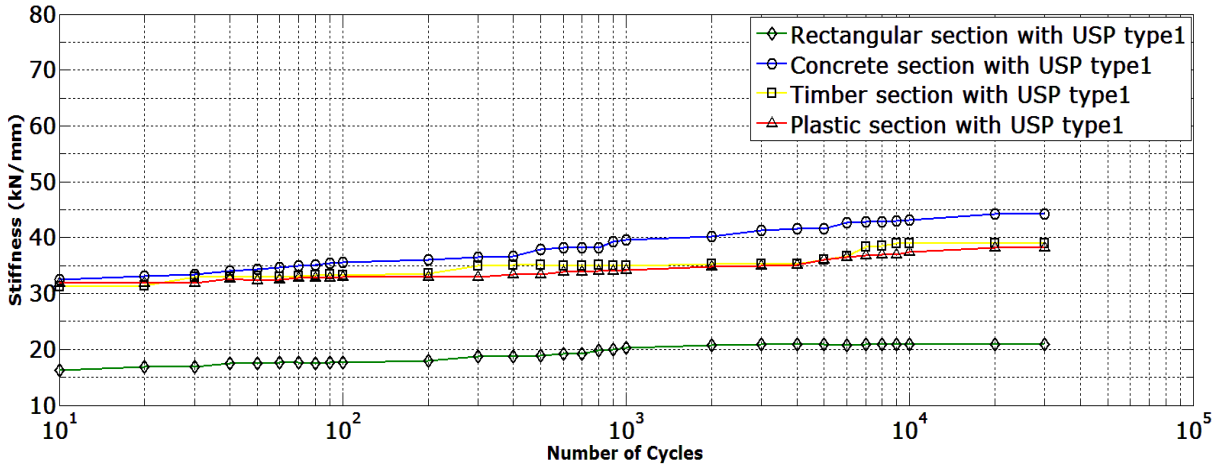
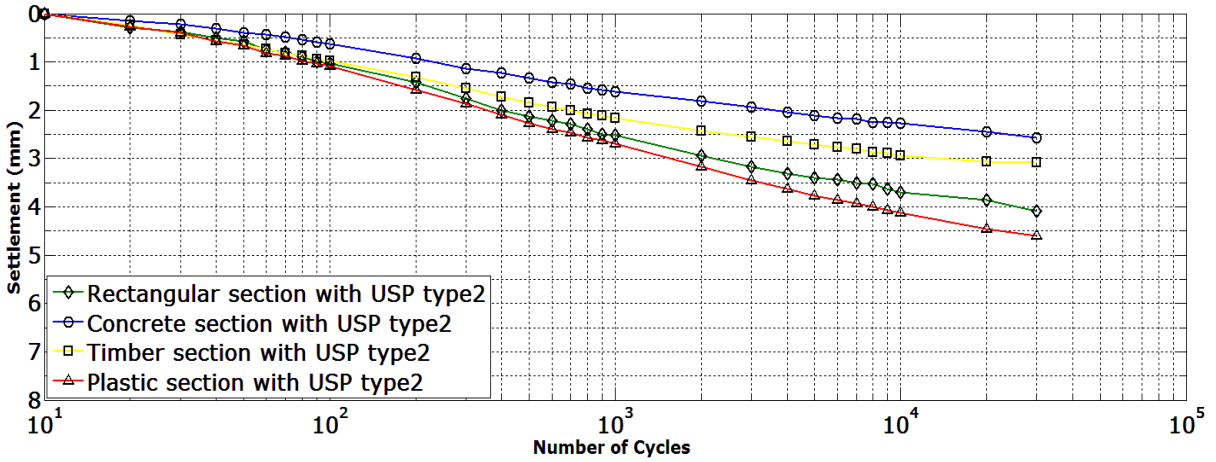


Figure 5-10 (a) Evolution of sleeper section displacement for sleeper sections with USP

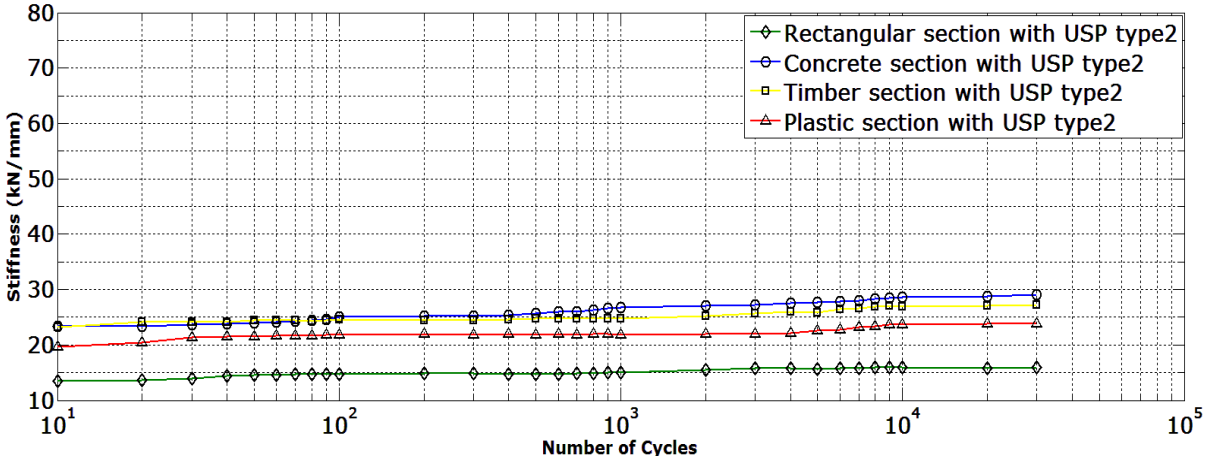


(b)

Figure 5-10 (b) Evolution of trackbed stiffness for sleepers with USP type1



(a)



(b)

Figure 5-11 (a) Evolution of sleeper section displacement for sleeper sections with USP type2 (b) Evolution of trackbed stiffness for sleepers with USP type2

A summary of the sleeper section settlements (at 30,000 cycles) and trackbed stiffnesses (as averages over 30,000 cycles), for all the tests conducted in phase 1 of CET testing are presented in Figure 5-12 and Figure 5-13 respectively.

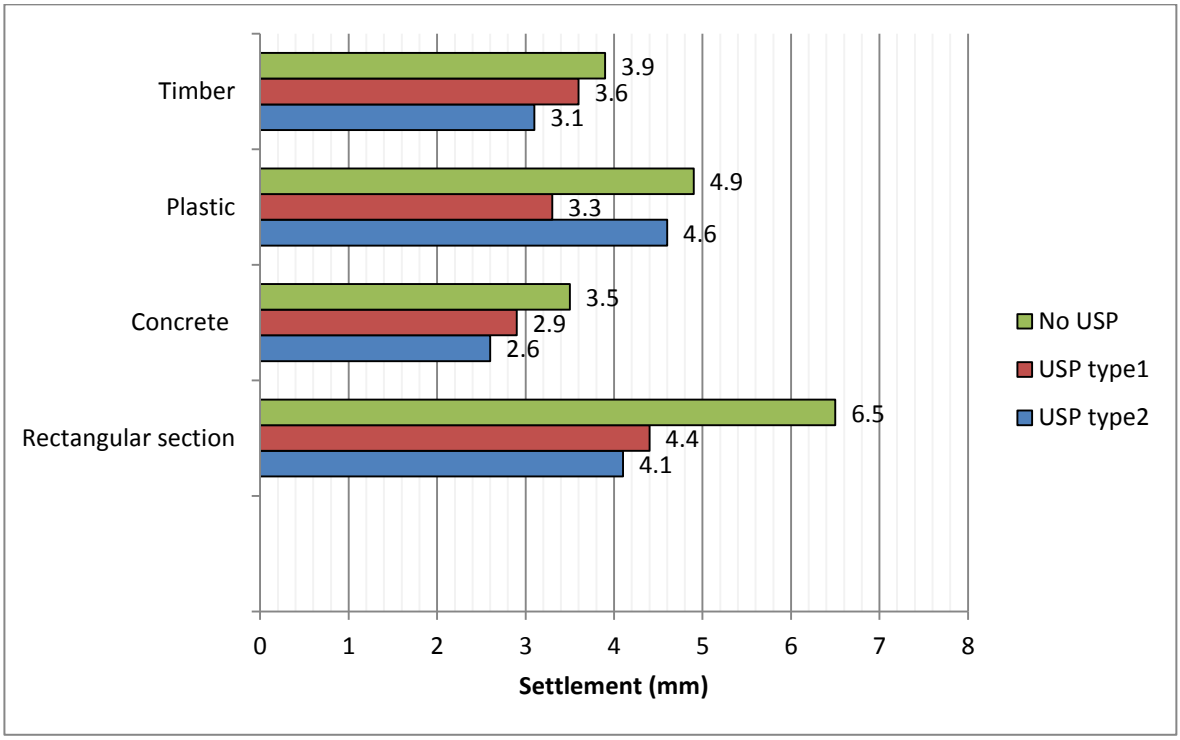
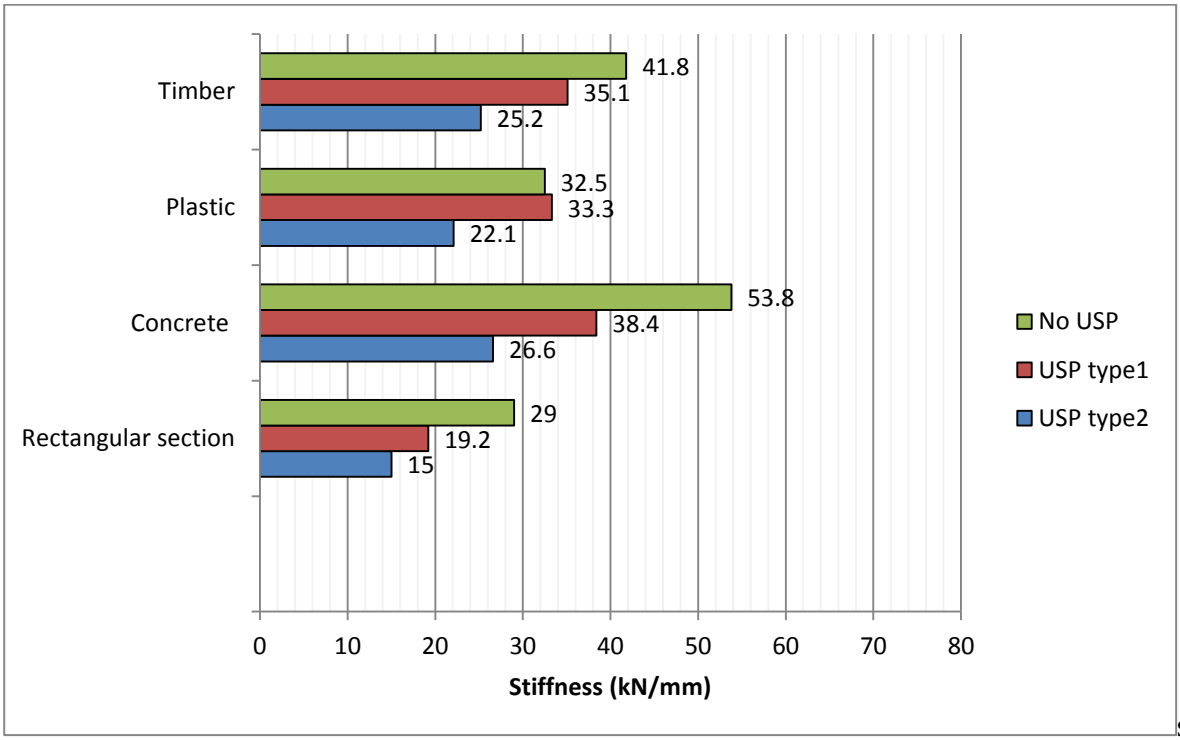


Figure 5-12 Sleeper section tests settlement at 30,000 cycles



s

Figure 5-13 Sleeper section tests average trackbed stiffness over 30,000 cycles

5.7 ADDITIONAL EXPERIMENTS (CET PHASE TWO)

Phase two of the CET tests was carried out after completion of CET phase 1 and the full-scale tests using the RTF and confirmation of consistency of CET phase 1 results with those of the RTF. Ballast gluing, use of sleeper section types and USPs using 2/3 scale ballast, and use of USP types with various stiffnesses were investigated in Phase two of the CET testing, the details and outcome of which will be presented in the following sections of this chapter.

5.7.1 BALLAST GLUING (BONDING)

Increase in speed of train, weight of axle, and track capacity impose a significant load increase on the superstructure and consequently the ballast bed. It is therefore important to improve the lateral stability of the track and minimise the track settlement to avoid the need for frequent track maintenance works. These are particularly important in the transition zones, turnouts and glued insulated joint track and track weld areas. One of the ways of stabilising railway ballast bed is ballast bonding/gluing, which was approved by Network Rail (NR) as a way of improving the lateral resistance of track and a standard was published for it by NR in 1996 (NR/SP/TRK/127). According to Lakušić et al. (2010b) ballast gluing is useful in the following cases:

- Protection of track under traffic during construction;
- Enhancement of lateral stability in curves;
- Stabilisation of railway track in transition areas;
- Stabilisation of ballast under rail joints;
- Stabilisation of ballast under turnouts and crossings.

Ballast gluing bonds the particles at their contact points, and it is not considered as injecting since it does not fill the voids between the particles. Therefore it does not influence the trackbed drainage adversely. Studies by Lakušić et al. (2010b) at the Technical University Budapest demonstrated that drainage speed was higher in the case of glued ballast compared to the dry ballast. This was explained by the fact that in the case of glued ballast the particles were covered by a thin adhesive film which decreased the roughness compared to the non-glued ballast. This consequently reduced the resistance of particles to drainage of water. The methods for implementation of ballast gluing are explained in detail in NR standards (NR/SP/TRK/127) and in a publication by Lakušić et al. (2010b). According to the aforementioned documents, the ballast bed needs to be very well compacted and levelled for the application of ballast gulling.

Examples of ballast gluing implementation are presented in Figure 5-14 and Figure 5-15 for mechanical and manual methods respectively.

Ballast layers can be glued up to different depths underneath a sleeper to produce trackbed with various stiffnesses (structural bonding); which is especially useful at transition zones. Ballast beds can also be glued at surface level only (surface bonding) to avoid lifting of small particles in the trackbed by a high speed train passing and to facilitate the cleaning of the surface layer of the track using industrial vacuums. Ballast bonding methods have been used in European countries such as Germany, Belgium, Czech Republic, Hungary, and the UK.



(a) **(b)**

Figure 5-14 Ballast gluing application (a) using a wagon (b) using a small machine (Lakušić et al., 2010b)



Figure 5-15 Manual application ballast gluing

Although application of ballast gluing is beneficial for the ballasted track in terms of reducing the frequency of maintenance works, the overall cost of super glues used for this method is high. It would therefore be valuable to identify a way of producing a low cost glue and testing its performance under conditions similar to real track. In this section, by using end-of-life

protective packaging polystyrene and pure d-Limonene (orange terpene oil) (>98% purity), a low cost glue was produced and CET tests were conducted to investigate the performance of the tackbed in respect to settlement and stiffness when the glue was applied to the ballast layers. A glued ballast bed was tested using the concrete section with and without USP (type2) and compared to the non-glued ballast bed. The glued ballast was then taken apart to form single particles and mixed and tested again with and without USP (type2). The aim was to investigate the reusability of the glued ballast and to investigate the performance of the glued ballast in a condition when for maintenance works or any other reasons, the bonds between the particles are broken.

Discarded polystyrene commonly used in the packaging industry, does not bio-degrade for years, and therefore it is important to find ways of recycling this material to reduce the environmental impact caused by it. The method of recycling polystyrene using d-Limonene was first used by Noguchi et al. (1998) in the Sony Corporation Research Centre. D-Limonene is a natural vegetable oil that is a good natural solvent of polystyrene and is extracted from the rinds of citrus fruits. Producing a well-performing glue using D-Limonene and discarded packaging polystyrene can be done through simple steps. The final product is a non-toxic adhesive with a strong smell of oranges.

To produce the aforementioned glue, the polystyrene sheets were first cut into small pieces and then 1000-ml of d-Limonene was poured into a large container and the polystyrene pieces were added and mixed up to the saturation level for d-Limonene. The process was carried out in a well ventilated area. The final product was a transparent viscous liquid which was stored in a tightly-sealed container to prevent it from drying. The glue was first tested on a small quantity of ballast (Figure 5-16). The glue became relatively solid a couple of minutes after application to the ballast but full drying took up to 12 hours.



Figure 5-16 Application of the glue to a small quantity of ballast particles

The process of preparing the CET ballast bed was similar to the method described in section 4.4 but during the ballast bed preparation the glue was manually applied to alongside of the ballast at different levels using a large paint brush (Figure 5-17). Once the CET test set up was completed, it was then allowed to cure for a minimum of 12 hours before the test was conducted. The glue produced out of 1000-ml of d-Limonene was enough to conduct two CET tests. For each CET, approximately 500kg of ballast was used.



Figure 5-17 Application of ballast gluing to CET ballast bed

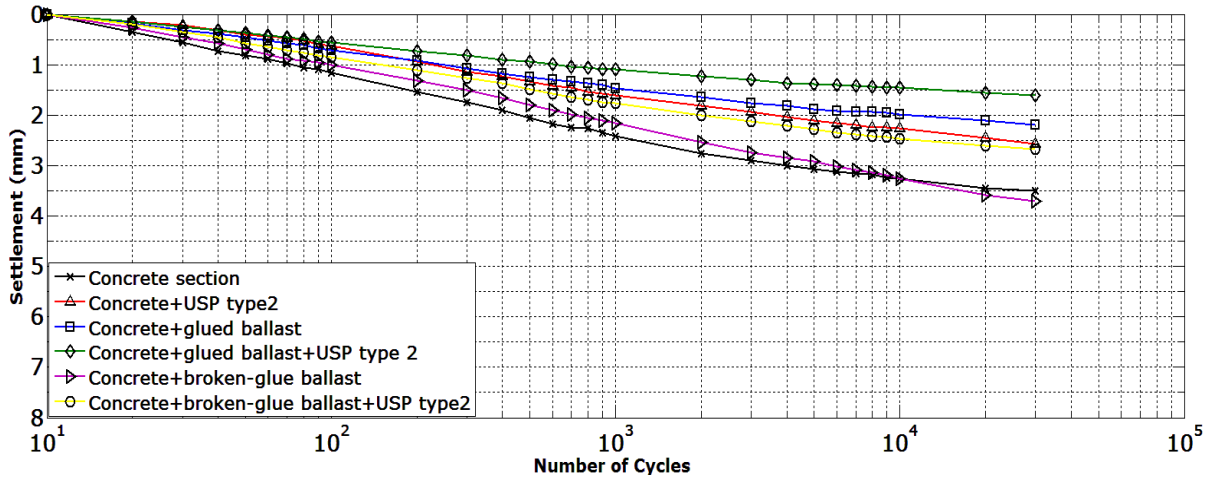
Results

A comparison of performance of glued ballast bed using concrete section plus or minus USP type2 against the performance of the non-glued ballast bed is presented in Figure 5-18 with respect to settlement and trackbed stiffness. Additionally, the performance of the broken-glue ballast bed was compared against the aforementioned tests. As can be seen in Figure 5-18 (a), improvement in settlement of the ballast bed was evident in the case of glued ballast compared to non-glued ballast. The settlement performance of glued ballast with the concrete section was also better than that of the non-glued ballast with the concrete section and USP type2, which yielded the least settlement in the sleeper section tests with or without USP (Figure 5-12). When the glued ballast was taken apart to single particles and mixed and tested again, it was observed that the overall settlement increased compared to that of glued ballast bed; however, it was still comparable with non-glued ballast. Similarly, when broken-glue ballast was used with the concrete section and USP type2, the settlement results were comparable to those of the non-glued ballast with the concrete section and USP type2. This may be as a result of glued ballast holding the particles together and reducing the concertation of stresses between stones at the

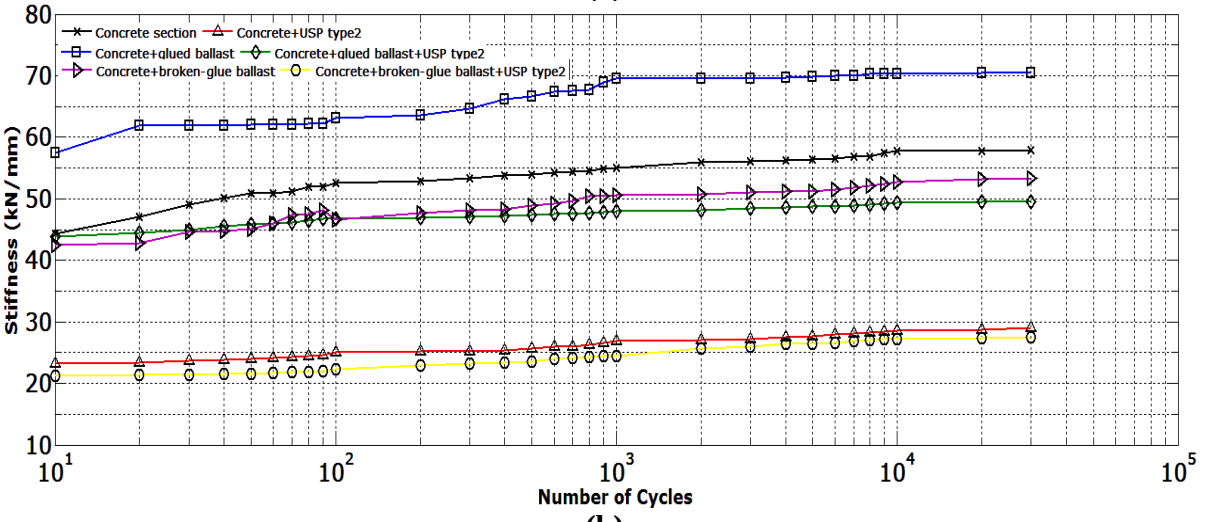
corners. Therefore when the particles were taken apart and tested again, the angularity of particles was still preserved and they performed similarly to the fresh non-glued ballast.

In the case of glued ballast and USP type2, the observation showed reduction in overall settlement compared to that of the glued ballast without USP. This may be as a result of the increase in load spreading ability of the sleeper at the ballast-sleeper interface when USP type2 was used compare to the 'No USP' case, which consequently resulted in less overall settlement.

With regard to the stiffness results (Figure 5-18 (b)), application of ballast gluing increased the stiffness of the ballast bed compared to non-glued ballast bed stiffness. Observation of stiffness results for broken-glue ballast showed a reduction in stiffness for both the USP type2 and 'no USP' cases when compared with glued ballast; however, the differences were insignificant when they were compared to those of the non-glued ballast; in particular in the case of USP type2.



(a)



(b)

Figure 5-18 Performance comparison between normal ballast, glued ballast, and broken-glue ballast sample with and without USP type2 in terms of (a) settlement (b) trackbed stiffness

The sleeper settlements (shown at 30,000 cycles) and trackbed stiffnesses (shown as averages over 30,000 cycles), are summarised in Figure 5-19 and Figure 5-20 respectively. for all the concrete tests with glued ballast, non-glued ballast, and broken glued ballast with and without USP2. Application of ballast gluing improved the settlement performance of the concrete section by 37% in the case of 'no USP' and by 38% in the case of USP type2 compared to those of the non-glued ballast. Comparison of settlement results between the case of the concrete section with non-glued ballast (without USP) and the case of the concrete section with glued ballast and USP type2 showed 54% improvement in settlement.

Where stiffness results are concerned, gluing ballast increased the average stiffness of the trackbed by 23% in the case of 'no USP' and by 78% in the case USP type2.

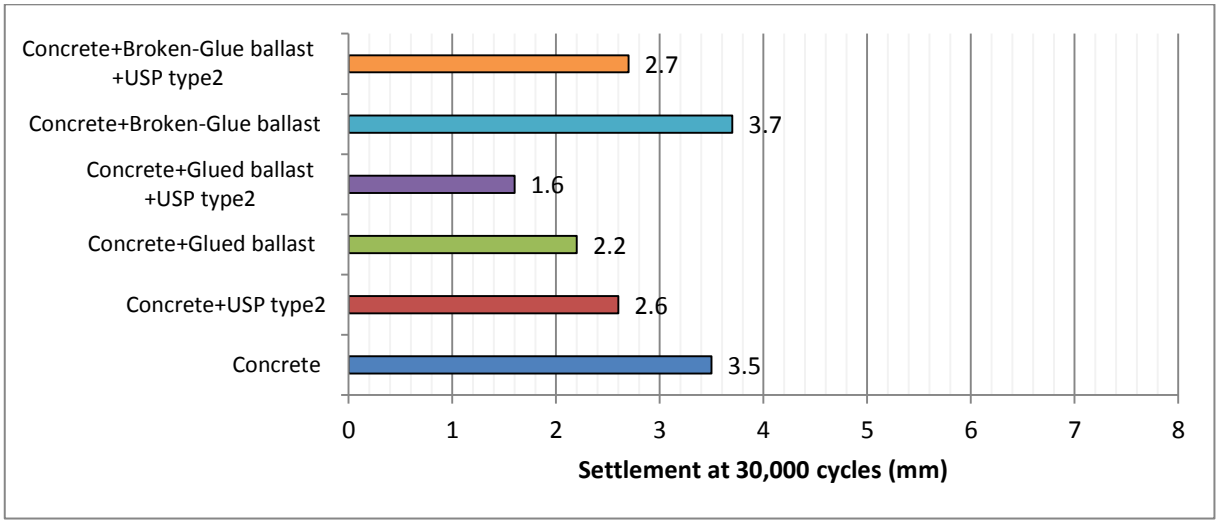


Figure 5-19 Settlement performance comparison at 30,000 between normal ballast, glued ballast, and broken-glue ballast sample with and without USP type2

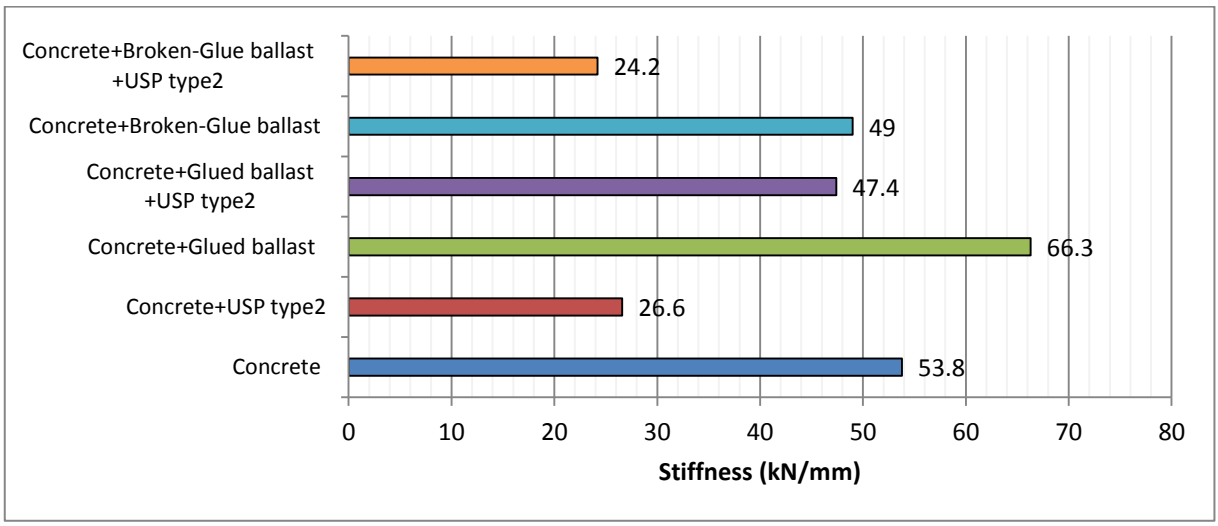


Figure 5-20 Average trackbed performance comparison over 30,000 between normal ballast, glued ballast, and broken-glue ballast sample with and without USP type2

5.7.2 SLEEPER SECTION TYPES AND USPS USING 2/3 SCALED BALLAST

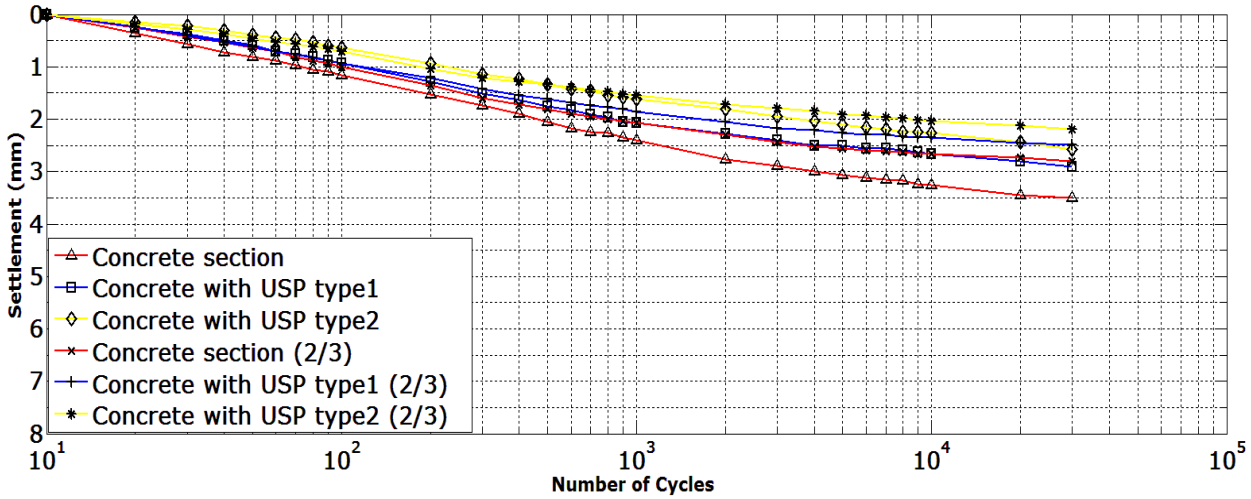
In this section, similar to section 4.8.3 for the box test, 9 tests were conducted on different sleeper section types namely concrete, timber, and plastic sections (described in section 5.3) with and without USPs using 2/3 scaled ballast. The performance of the section types was investigated with respect to settlement and trackbed stiffness. The settlement and stiffness results were then compared to those from testing of full-sized ballast (section 5.6). In terms of comparison of the box test and CET test results, both apparatus results showed better settlement performance and stiffness when 2/3 scaled ballast was used compared to full-sized ballast. The CET results also showed a similar trend compared to those of the box test in terms of performance ranking of sleeper section types with and without USPs using 2/3 scale ballast. However, the magnitude of settlement and stiffness results was less when compared with the box test results, as elaborated in section 5.6. The CET test discussions for the observed permanent deformations and trackbed stiffness resulting from the use of different sleeper sections with and without USP using full-size or 2/3 scale ballast are therefore similar to those discussed for the box tests, notwithstanding the differences in the nature of the box test and CET apparatus, e.g. load, loading frequency, confinement level, sleeper section size and etc.

Results

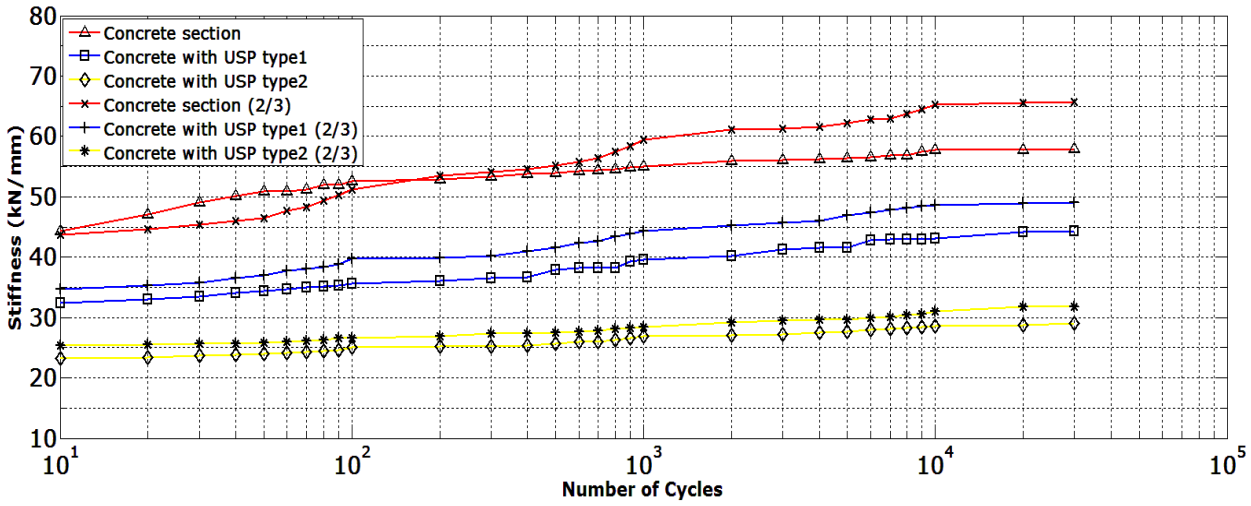
In Figure 5-21 to Figure 5-23, the performance of each sleeper section separately with and without USP type1 and type2 (stiff and soft) using 2/3 scaled ballast is shown with respect to settlement and trackbed stiffness. The results from full-sized ballast tests are also presented in the aforementioned figures for the purpose of comparing against the 2/3 scaled ballast results.

In the case the concrete section and 2/3 scale ballast (Figure 5-21), although inclusion of USPs (regardless of type) reduced the overall settlement, the differences in resulting settlements were not as noticeable as when full-sized ballast was used. Use of USPs or 2/3 scale ballast both increased the number of contacts and the ability to spread load at the sleeper-ballast interface, and it can be seen in Figure 5-21 (a) that all the tests conducted using 2/3 scale ballast or USPs (stiff or soft) had comparable performance in terms of settlement.

With respect to stiffness results when the concrete section and 2/3 scale ballast were used, use of USPs reduced the stiffness of the trackbed; however in the case of USP type2 the reduction in stiffness was higher than with USP type1. Comparing the 2/3 scale ballast and full-sized ballast test results showed slightly higher trackbed stiffness in the case of 2/3 scale ballast.



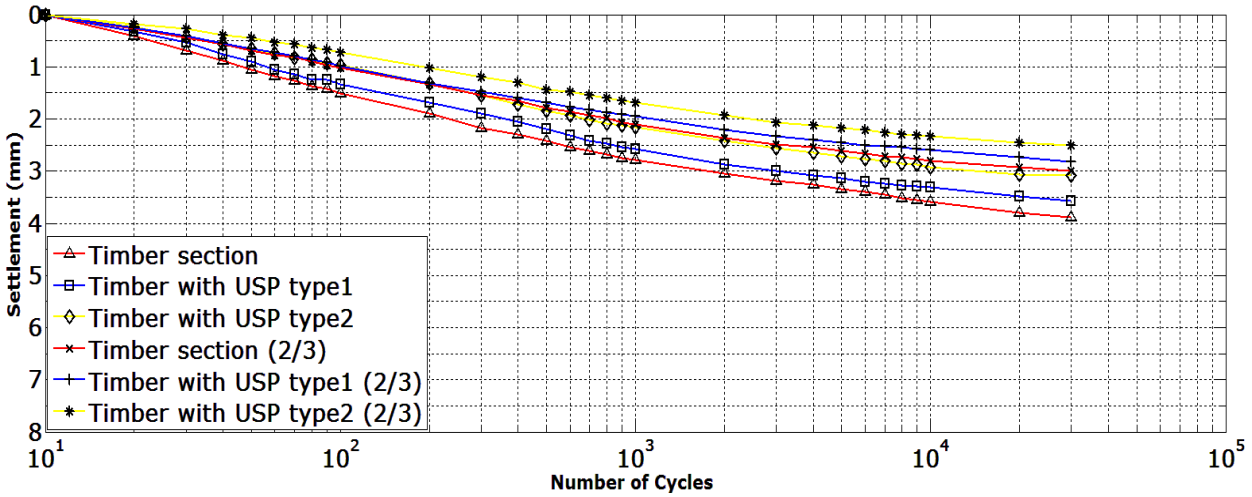
(a)



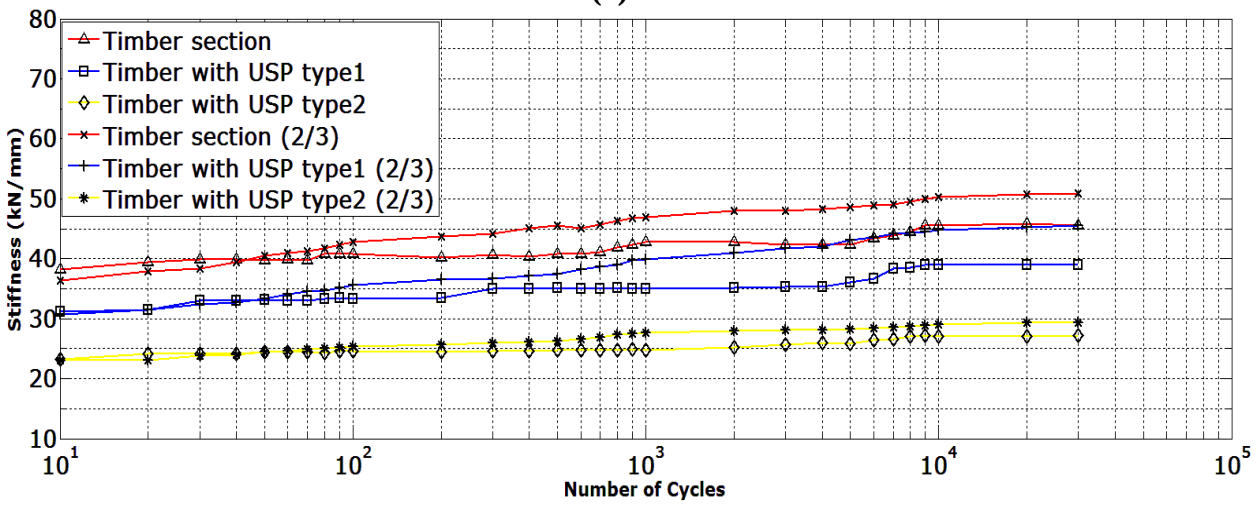
(b)

Figure 5-21 Effect of USP inclusion on concrete section performance using full-sized and 2/3 scaled size ballast (a) settlement (b) trackbed stiffness

For the timber section and 2/3 scale ballast (Figure 5-22), similar performance compared to the concrete section was observed; however the concrete section showed slightly better performance with respect to settlement and trackbed stiffness. This can be explained by the greater height of the concrete section compared to the timber section that provided the sleeper with more crib ballast at the sides and consequently more support.



(a)

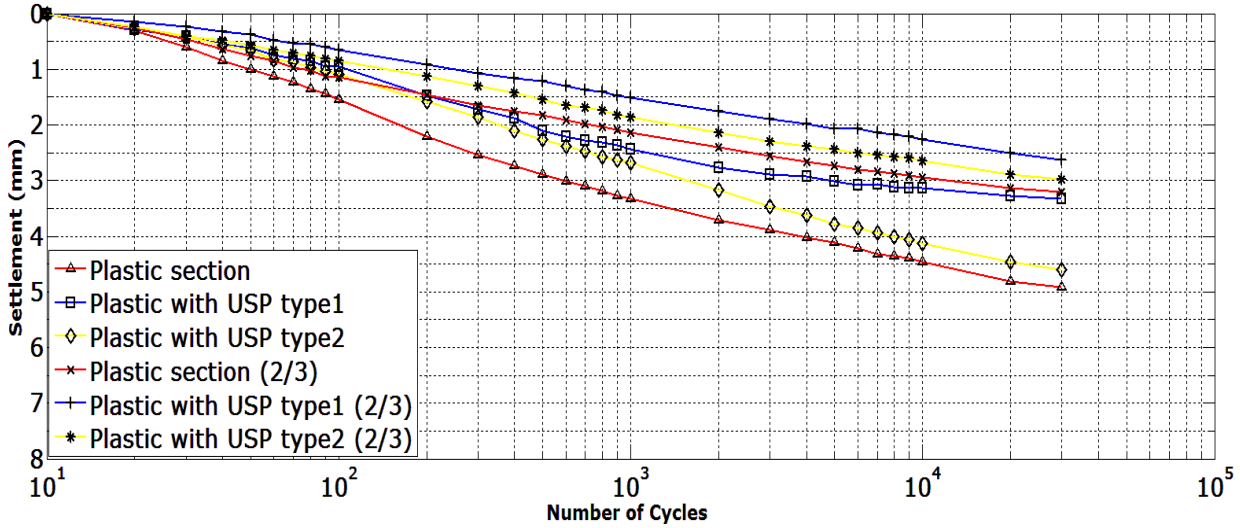


(b)

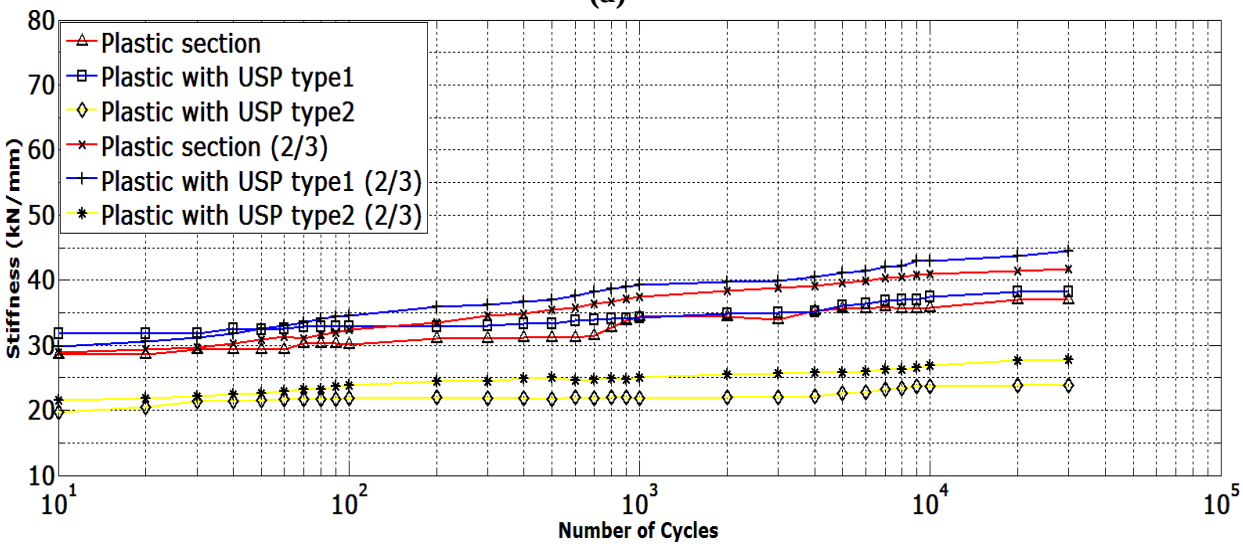
Figure 5-22 Effect of USP inclusion on timber section performance using full-sized and 2/3 scaled size ballast (a) settlement (b) trackbed stiffness

In the case of the plastic section (Figure 5-23), the settlement performance of the sleeper section was better when USP type1 was used compared to USP type2 regardless of ballast size. Moreover, when settlement performance of 2/3 scale ballast and full-sized ballast was compared for the plastic section in the case of the 'no USP' scenario, significant improvement was observed when 2/3 scale ballast was used. This suggested that the plastic sleeper should be used with smaller size ballast to perform best with respect to settlement.

With regard to stiffness (Figure 5-23 (b)), USP type1 increased the trackbed stiffness compared the 'no USP' case for both 2/3 scale and full-sized ballast. All the results with the plastic section are consistent with those of the box test.



(a)

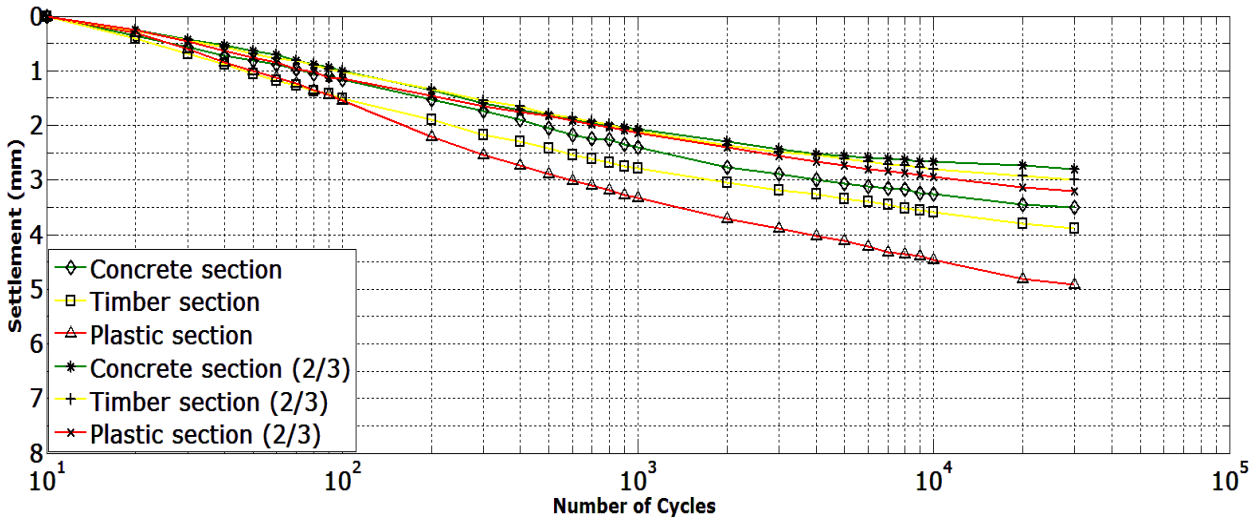


(b)

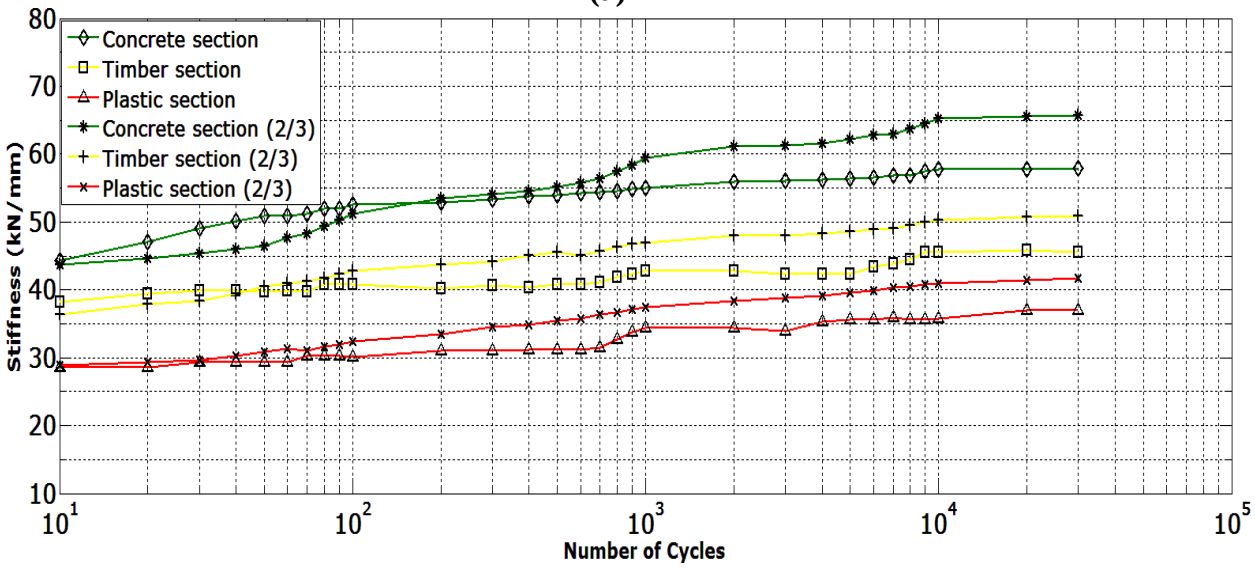
Figure 5-23 Effect of USP inclusion on plastic section performance using full-sized and 2/3 scaled size ballast (a) settlement (b) trackbed stiffness

In the case of the 'no USP' scenario (Figure 5-24), the variability in settlement performance of sleeper sections was low when 2/3 scale ballast was used compared to full-sized ballast, although the concrete section performed slightly better. Improvement in settlement performance of sleeper sections in all cases was also observed when 2/3 scale ballast was used; however, the plastic section resulted in larger improvement in settlement compared to the rest of the sleeper sections.

So far as the trackbed stiffness is concerned (Figure 5-24 (b)), for both sample types in the case of 'no USP' the concrete section yielded the highest stiffness and the plastic the lowest. However, for each sleeper section 2/3 scale ballast increased the stiffness compared to the full-sized ballast.



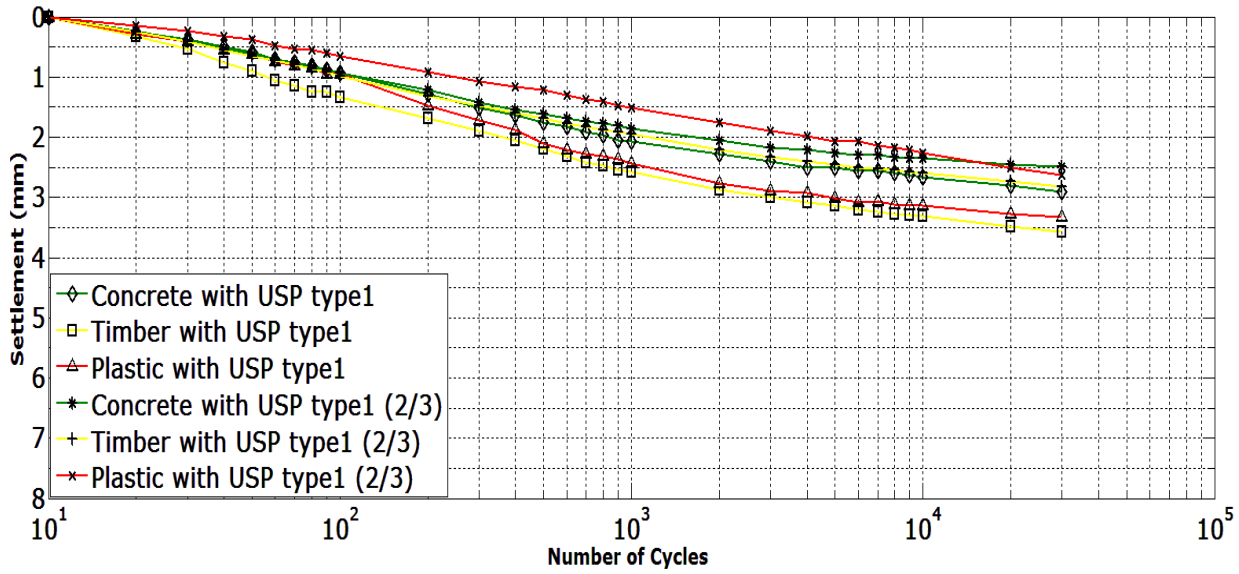
(a)



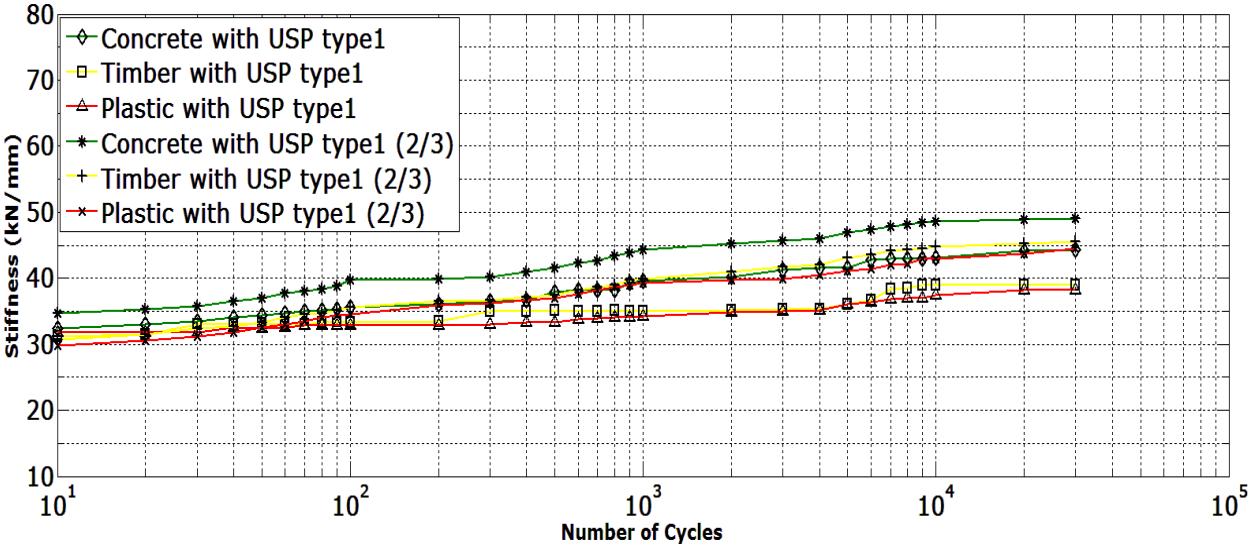
(b)

Figure 5-24 Evolution of sleeper section displacement for sleeper sections using full-size and 2/3 scaled size ballast (b) Evolution of trackbed stiffness using full-size and 2/3 scaled size ballast

When USP type1 was used (Figure 5-25), the difference in settlement and trackbed stiffness results was low compared to the 'no USP' scenario (irrespective of ballast sample size). However, the 2/3 scale ballast performed slightly better compared to the full-sized ballast, regardless of sleeper section type.



(a)

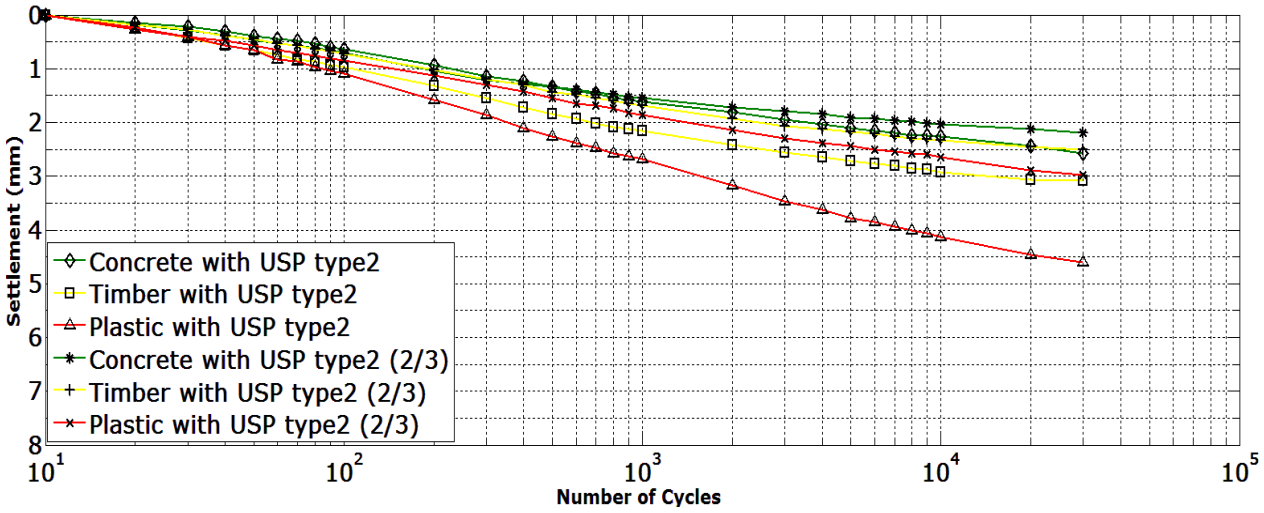


(b)

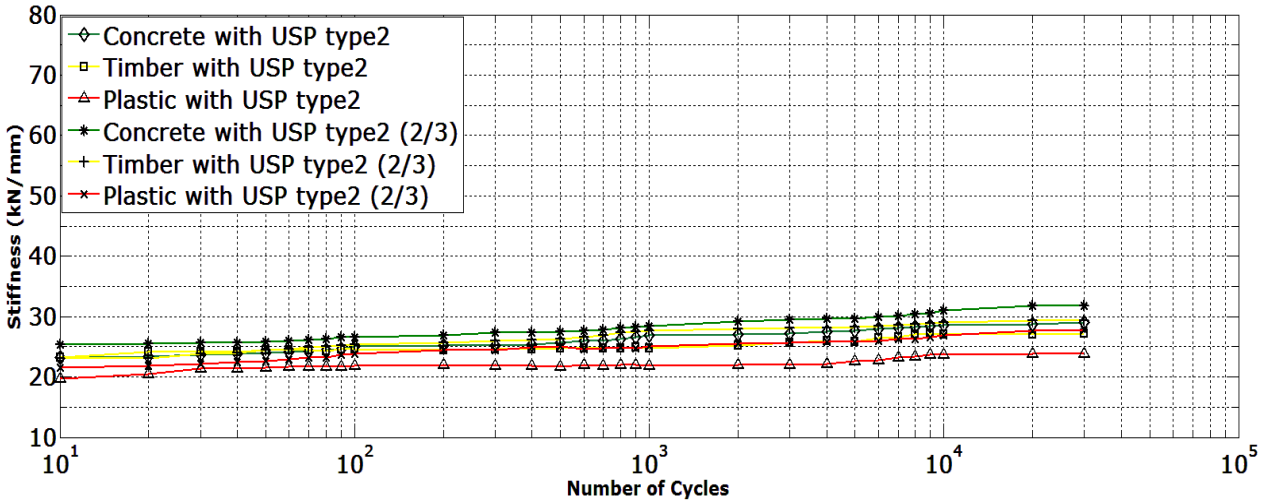
Figure 5-25 Evolution of sleeper section displacement for sleeper sections with USP type1 using full-size and 2/3 scaled size ballast (b) Evolution of trackbed stiffness for sleepers with USP type1 using full-size and 2/3 scaled size ballast

In the case of USP type2, use of 2/3 scale ballast significantly improved the settlement performance of the plastic sleeper compared to that of the full-sized ballast. Apart from the plastic section with full-sized ballast, there was little variability in settlement results for the sleeper section types (regardless of ballast size). However, 2/3 scale ballast showed slightly better settlement performance compared to the full-sized ballast.

With regard to stiffness, the performances of sleeper sections were very comparable irrespective of ballast sample size. However, use of 2/3 scale size ballast increased the stiffness result for each sleeper section compared to those of full-sized ballast.



(a)



(b)

Figure 5-26 (a) Evolution of sleeper section displacement for sleeper sections with USP type2 using full-size and 2/3 scaled size ballast (b) Evolution of trackbed stiffness for sleepers with USP type2 using full-size and 2/3 scaled size ballast

Figure 5-27 and Figure 5-28 show the sleeper section settlements (at 30,000 cycles) and trackbed stiffnesses (as averages over 30,000 cycles), respectively for both 2/3 scale and full-sized ballast. Of the all tests undertaken, the concrete section with USP type2 and 2/3 scale ballast yielded the least settlement and the plastic section with no USP and full-sized ballast the most. As can be seen, there were smaller differences in settlement performance of sleeper sections in all cases (with or without USP) when 2/3 scale ballast was used compared to those of the full-sized ballast. It was also observed that use of 2/3 scale ballast improved the settlement performance of sleeper sections (with or without USP). However, in the case of the 'no USP' scenario, the largest improvement was recorded in the case of the plastic section (approximately 39%). Furthermore, a settlement result comparison with the plastic section

between ‘no USP’ and full-sized ballast scenario, and USP type1 and 2/3 scale ballast showed approximately 47% improvement. This became approximately 37% for the concrete section between ‘no USP’ and full-sized ballast case, and USP type2 and 2/3 scale ballast. For the timber section it was approximately 36% between ‘no USP’ and full-sized ballast scenario and USP type2 and 2/3 scale ballast case.

In the case of stiffness (Figure 5-28), less variability was observed when 2/3 scale ballast was used compared to full-sized ballast. Moreover, use of 2/3 scale ballast increased the stiffness of trackbed in all cases compared to the full-sized ballast. Of all the tests presented in this section, the highest trackbed stiffness value was recorded in the case the concrete section with 2/3 scale ballast.

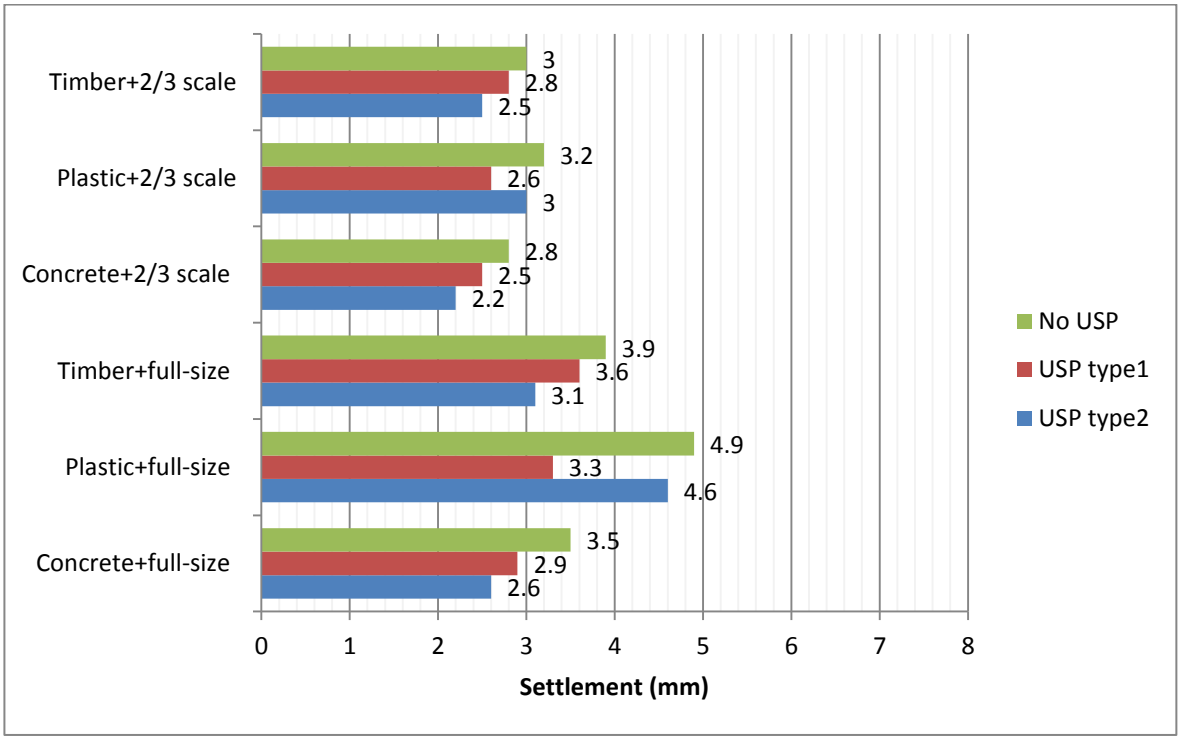


Figure 5-27 Sleeper section settlement at 30,000 cycles with and without USP using full-size and 2/3 scaled size ballast

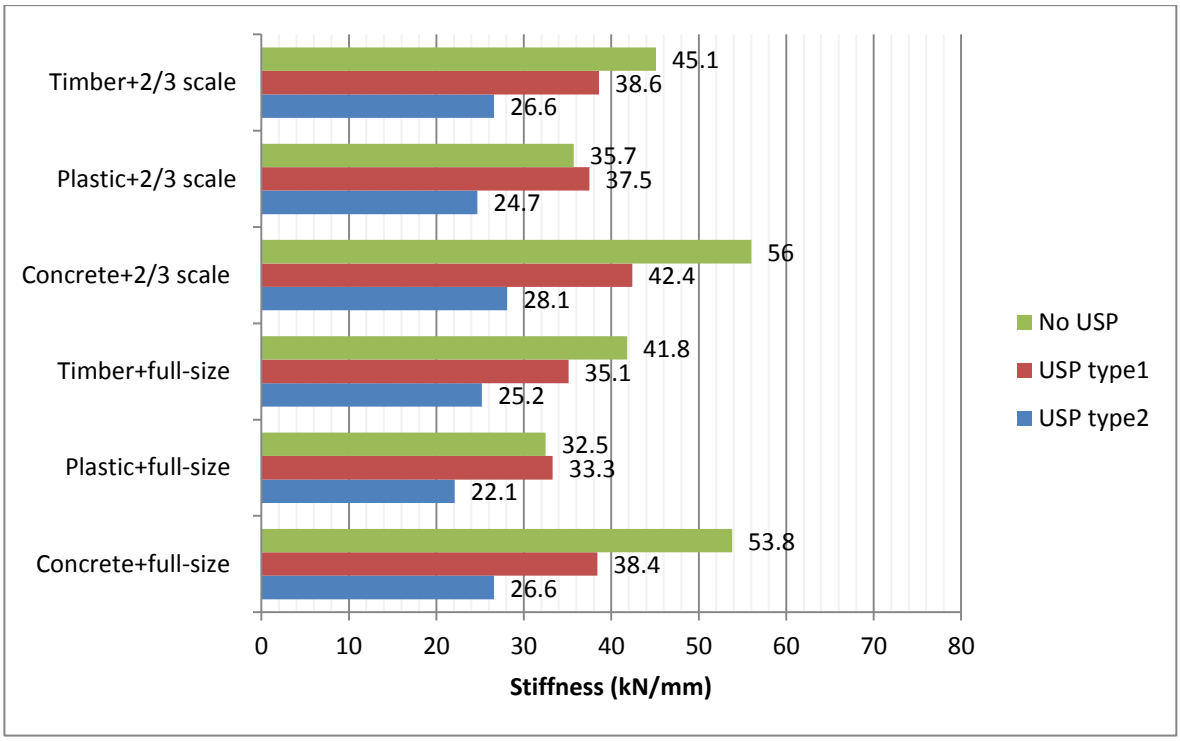


Figure 5-28 Average trackbed stiffness over 30,000 cycles with and without USP using full-size and 2/3 scaled size ballast

5.7.3 EFFECT OF USP STIFFNESS ON TRACKBED SETTLEMENT AND STIFFNESS

In this section, similar to the tests described in section 4.8.5 using the box test, extra layers of USP were used to make USPs with a greater range of stiffness and affixed to the concrete section and tested using the CET apparatus. Consistent with the box test results, an optimal stiffness range to improve the trackbed settlement was achieved by using a double-layer USP type2, comparable to double-layer USP type1. Therefore, the arguments presented in section 4.8.5 for the box test results are similar to those for the CET test. The details of multi-layer USPs used in this investigation are shown in Table 5-3.

Table 5-3 Details of multi-layer USP type1 and type2

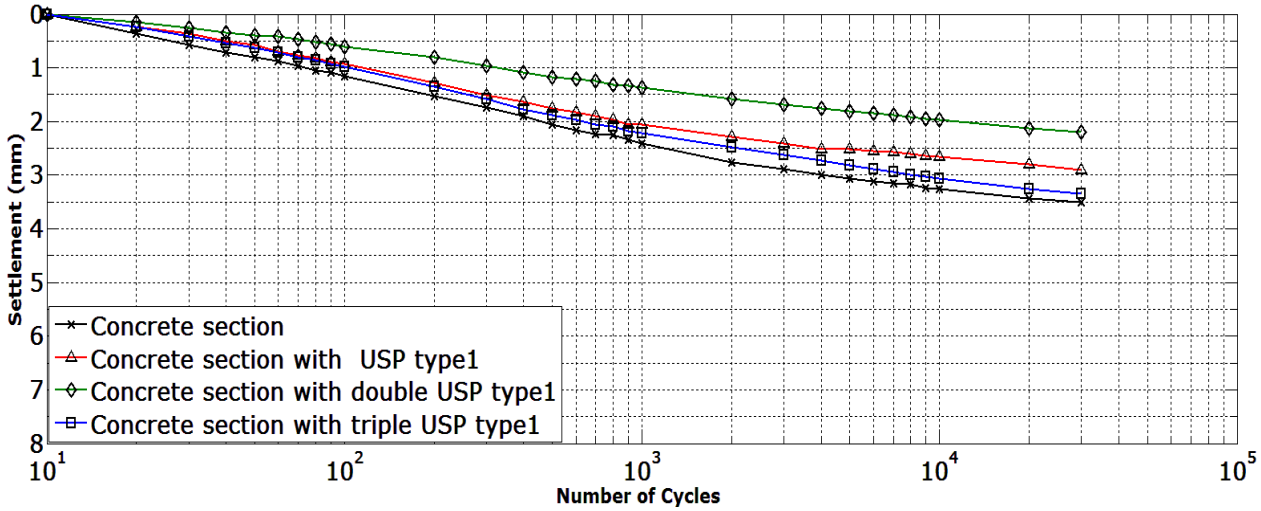
Box Test Experiment	USP Stiffness (N/mm^3)	USP thickness (mm)
Concrete Section	-	-
Concrete with USP type1	0.228-0.311	4
Concrete with USP type2	0.079-0.105	9
Concrete with Double USP type1	0.114-0.155	8
Concrete with Double USP type2	0.0395-0.0525	18
Concrete with Triple USP type1	0.076-0.103	12
Concrete with Triple USP type2	0.0263-0.0.035	27

Results

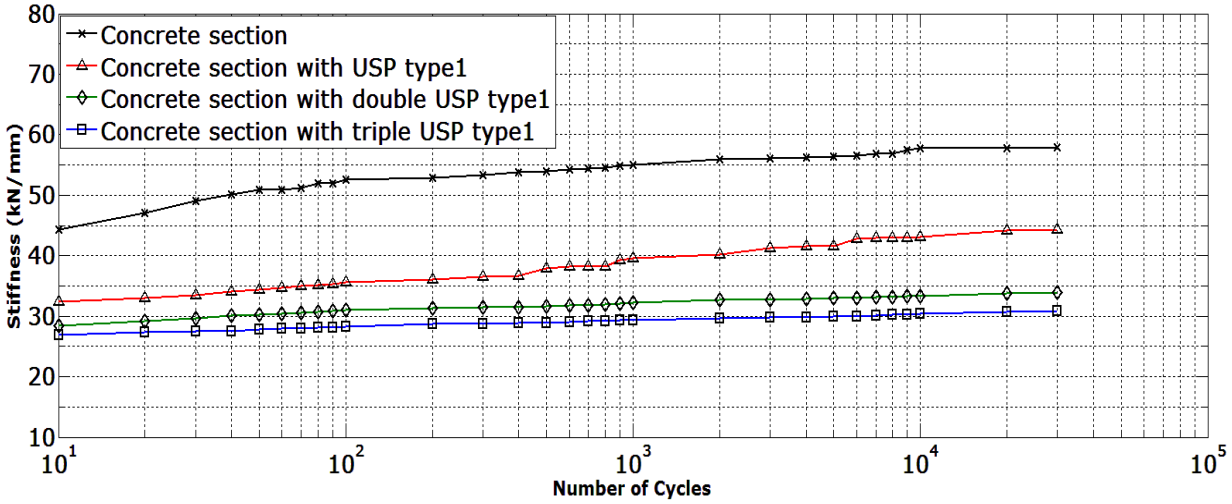
The settlement and trackbed stiffness results for inclusion of multi-layer USPs on the concrete section are presented in Figure 5-29 and Figure 5-30.

In the case of multi-layer USP type1 (Figure 5-29), the best settlement performance was observed when a double-layer USP was used. The triple-layer USP increased the settlement compared to the double-layer USP; however, the settlement performance of the triple-layer was still slightly better than the 'no USP' scenario.

As expected, with respect to the trackbed stiffness (Figure 5-29 (b)), the greater the number of USP layers, the greater the value of the trackbed stiffness was reduced.



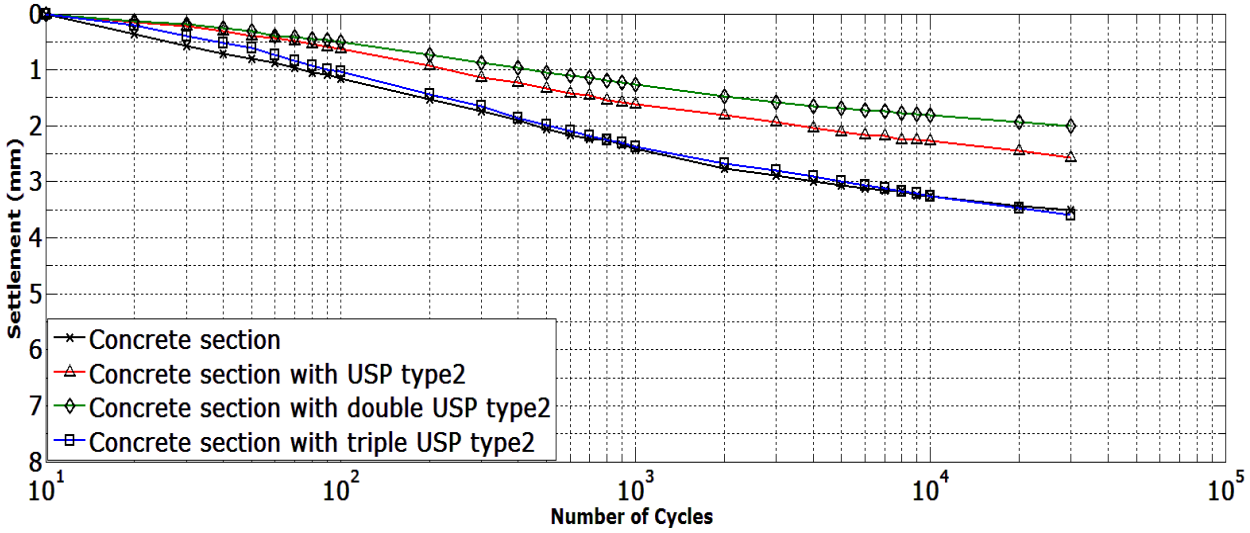
(a)



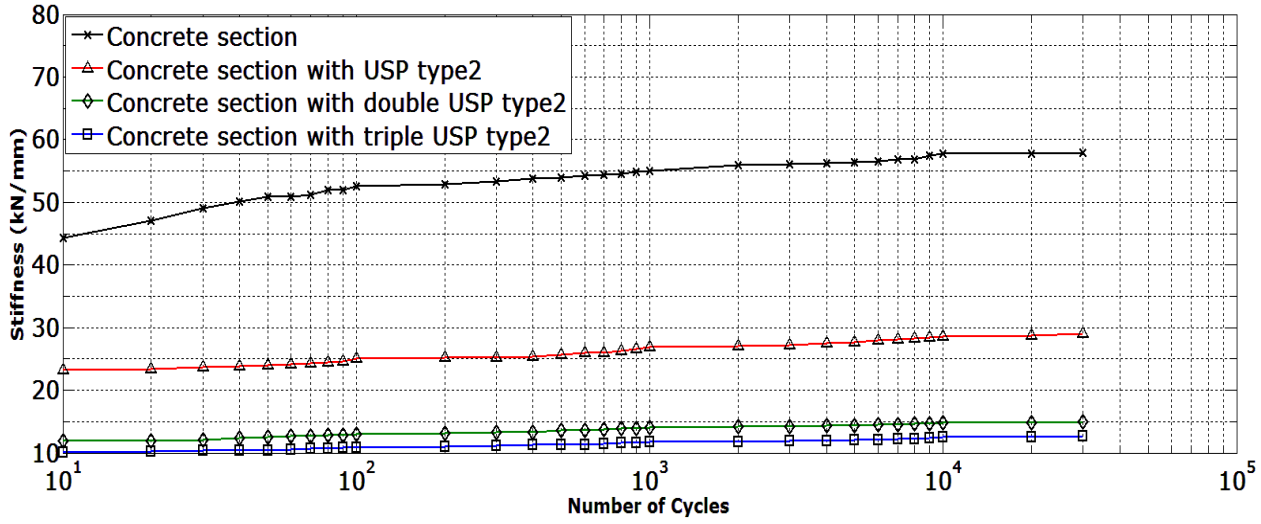
(b)

Figure 5-29 Effect of multiple-layer USP type1 inclusion on concrete section performance (a) settlement (b) trackbed stiffness

For multi-layer USP type2 (Figure 5-30), similar to multi-layer USP type1, the double-layer performed the best in respect to sleeper settlement, comparable to that of double-layer USP type1.



(a)



(b)

Figure 5-30 Effect of multiple-layer USP type2 inclusion on concrete section performance (a) settlement (b) trackbed stiffness

Figure 5-31 and Figure 5-32 summarise the settlement and trackbed stiffness for all the CET multi-layer tests conducted. Use of double-layer USP type2 improved the settlement performance of the concrete section by approximately 43%. This was 37% in the case of the double-layer USP type1. Use of multi-layer USP (regardless of type) reduced the trackbed stiffness; however, multi-layer USP type2 resulted in higher stiffness reduction compare to multi-layer USP type1.

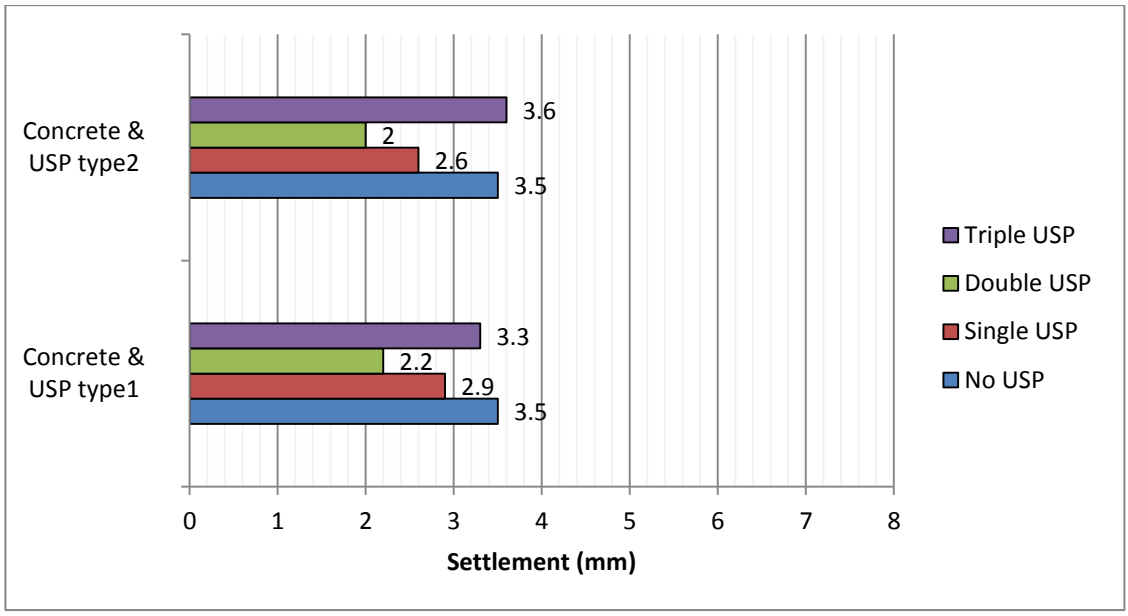


Figure 5-31 Settlement at 30,000 cycles for multi-layer USP type1 and type2

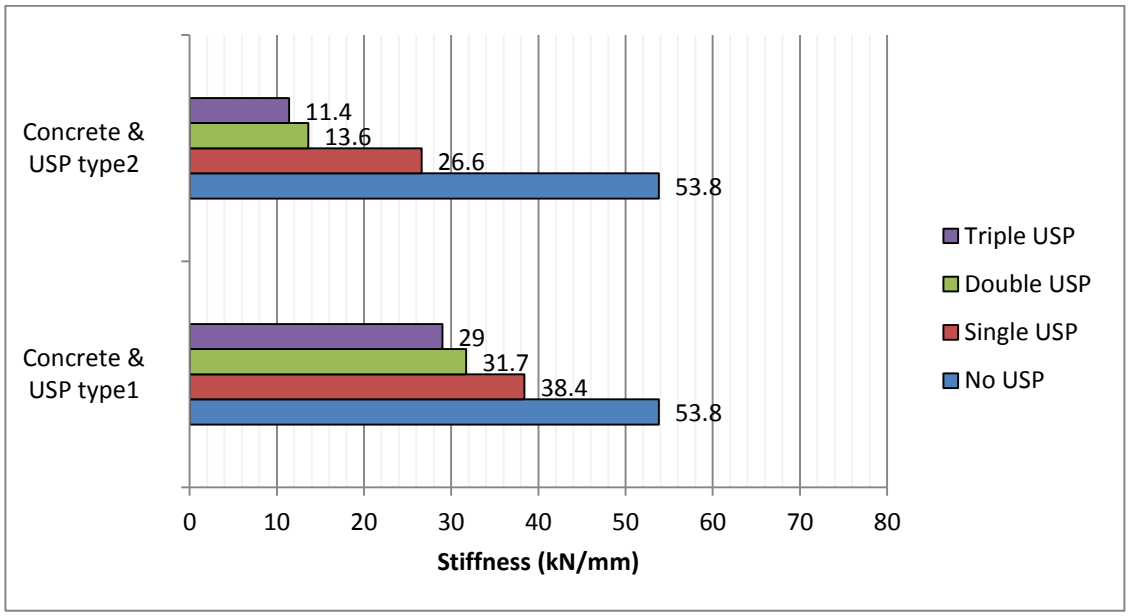


Figure 5-32 Average trackbed stiffness over 30,000 cycles for multi-layer USP type1 and type2

5.8 SUMMARY

A total of 31 tests were conducted using the CET, 12 tests in phase 1 and 19 tests in phase 2. The results of the tests carried out in both the box test and the CET apparatus were consistent in performance ranking in respect to settlement and trackbed stiffness. However, the magnitude of recorded settlement and trackbed stiffness results for the box tests was higher than in the CET tests due to the differences in test condition (e.g. load, frequency of loading, level of

confinement, size of sleeper, etc.). A summary of the results from the CET tests conducted in phase 1 and 2 is as follows:

- In the case of 'no USP' and full-size ballast, of all the sleeper sections used, timber and concrete sections outperformed the plastic and rectangular sections;
- When USPs were used (regardless of USP type), all of the tests showed reduced trackbed settlement;
- Although USP type2 (the softer of the two) in the majority of tests performed better than USP type1, there were no great differences in settlement performance for a given USP;
- Application of USPs offset differences in the performance of different sleeper types;
- Application of a low cost glue, made from end-of-service packing polystyrene, to the ballast bed improved the settlement performance of the concrete section by 37% and increased the stiffness performance of the ballast bed by 23% in the case of the 'no USP' scenario;
- In the case USP type2 and the concrete section, glued ballast performed 38% better than non-glued ballast in terms of settlement and it increased the track bed stiffness by 78%;
- In the case of glued ballast and USP type2, the settlement performance of the concrete section reduced by 54% compared to the 'no USP' scenario and non-glued ballast.
- When 2/3 scale ballast was used, in all the test cases, settlement performance of the sleeper sections improved and the trackbed stiffness increased;
- Regardless of USP type, smaller differences in settlement performance of sleeper sections in all tests were observed when 2/3 scale ballast was used compared to full-sized ballast;
- In the case of multi-layer USP, double-layer USP type 1 and type2 performed the best.

6. RAILWAY TEST FACILITY

6.1 INTRODUCTION

The box test and the Composite Element Test (CET) apparatus in previous chapters were used ahead of the full-scale apparatus as preliminary tests to obtain an insight into the performance of different sleeper types with and without the use of Under Sleeper Pads (USPs). The aforementioned apparatus were also used for comparison with the full-scale apparatus to assess the consistency of the sleeper settlement results. The purpose of using the box test and CET was to allow a relatively large number of simplified tests to be carried out within a reasonable cost and time frame.

In this chapter the Nottingham full-scale railway test facility (RTF) was used to achieve better understanding of the performance of standard size sleeper types with and without USPs in terms of track settlement as well as their influence on the levels of diffused stresses at the ballast-subgrade interface. In this chapter, a description of the original design, modification and equipment of the RTF as well as instrumentation, materials, methodologies and procedures related to this study are also presented.

6.2 ORIGINAL DESIGN

The Railway Test Facility at the University of Nottingham was used for the full-scale tests in this study. The facility was designed and assembled to aid the development of the design and construction of trackbeds (Brown et al., 2007a). The RTF is capable of simulating the passage of a 28km/hr train with an axle load of 250kN sequentially over three full size sleepers (Brown et al., 2007a). The RTF is located in a concrete pit 1.9-m deep, 4.1-m long and 2.1-m wide. An independent loading frame provides a reaction to the three sequential vertical 100kN loads applied centrally to the three sleepers (Figure 6-1). The loads are delivered by actuators attached to load cells onto the centres of spreader beams supported where the rails would normally have been attached. The actuators are aligned to correspond with the 0.65m spacing of the three sleepers, which is in keeping with current UK practice by Network Rail NR/L2/TRK/2102 (NetworkRail, 2016). The main components of the RTF are shown in Figure 6-2.

Vertical settlement readings are provided by displacement transducers attached to the actuators but these are often augmented by Linearly Variable Differential Transformers (LVDTs) at targeted locations on the sleepers. Pressure cells are also reported to have been used in the RTF (Kwan, 2006, Aursudkij, 2007) to record the stress transmitted from the ballast to a silt subgrade.

A comprehensive description of the original design process of the RTF is given in Brown et al. (2007a). However, it is important to note that the configuration of the track superstructure/substructure in the RTF can be changed depending on test requirements as has been the case for this study.



Figure 6-1: The RTF Loading Frame

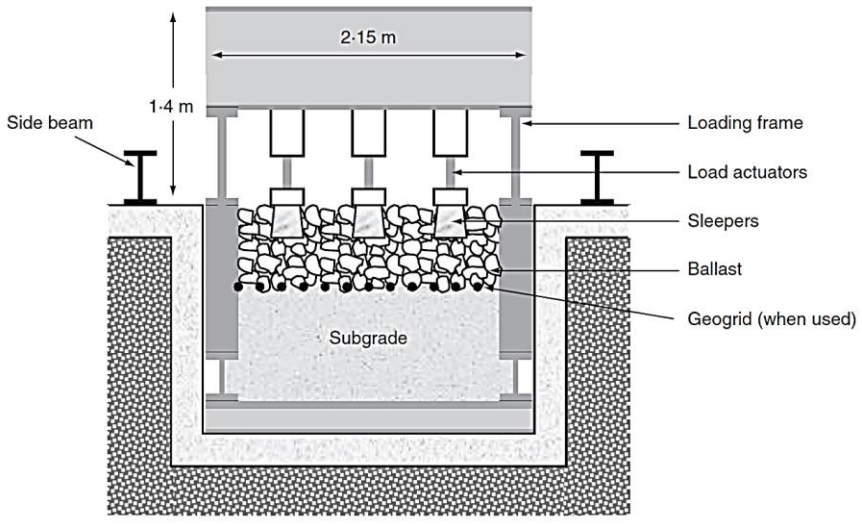


Figure 6-2: Cross-section of the facility (Brown et al., 2007a)

6.3 MODIFICATION TO THE RTF

As described in the previous section, the RTF was designed to house 'soil material' as a subgrade, and all the previous RTF studies (Brown et al., 2007a, Kwan, 2006, Aursudkij, 2007, Shi, 2009) have used compacted silt subgrade as the formation beneath the trackbed. However, for the purpose of this investigation, it was modified to minimise the effect of trackbed stiffness variations between all tests and to achieve a more homogenous and consistent foundation stiffness. The previous silt subgrade was removed completely and the base of the pit elevated by casting a rigid removable reinforced concrete slab to achieve a total height of 900-mm (Figure 6-3 and Figure 6-4). As illustrated in Figure 6-4, the slab was cast as close as practically possible to the RTF steel frame. However, there were still small gaps left at the sides which were filled and compacted using pea gravel and small sized ballast until the same level as the top of the concrete slab was achieved. Following this, similar to the box and CET, a commercial rubber mat was used to simulate a typical subgrade reaction (Figure 6-5). It was believed that this would ensure uniform stiffness across the entire ballast footprint and reduce the inconsistent behaviour one might have experienced with silt. Moreover, the use of a rubber mat with a known stiffness gives the additional flexibility of being able to change the subgrade stiffness by altering the thickness of the rubber mat.



Figure 6-3 Construction of reinforced concrete formation

In the original RTF pit design, the simulated track was confined by concrete walls but this was modified in the present study such that the trackbed was constrained only in the direction of travel (Figure 6-6). A ballast shoulder length of 375-mm (conforming to NR/L2/TRK/2102)

was created at both ends of the sleepers. The shoulders provide adequate lateral support to the sleepers and replicate a typical unrestrained shoulder found in a real track. A 1:1 (45 degree) slope was created after the ballast shoulders. This configuration (illustrated in Figure 6-6), and is more representative of a real-life track as it allows lateral migration of ballast under cyclic loading. In addition, it provides the necessary clearance to achieve a 45 degree load spread.



Figure 6-4 Cast rigid concrete trackbed foundation

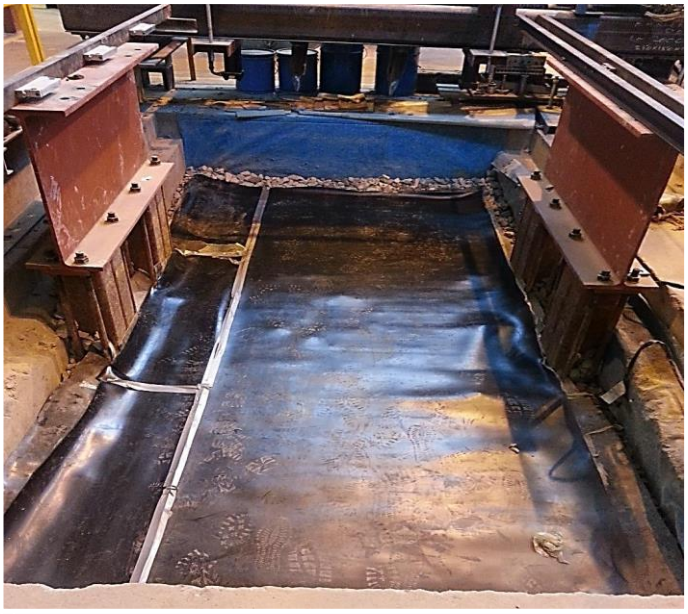


Figure 6-5 Commercial rubber mat

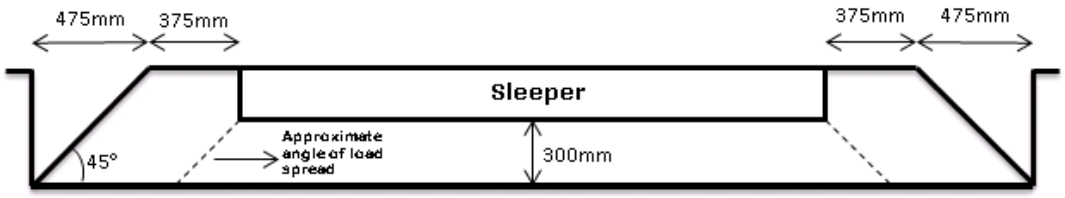


Figure 6-6 Proposed track set-up

6.4 THE TEST FACILITY EQUIPMENT

As shown earlier in Figure 6-2, the RTF has a loading frame designed to constrain three 100kN vertical loads which simulate the dynamic loading of a passing train. The vertical loads are delivered via the arms of the three servo hydraulic actuators shown in Figure 6-7.



Figure 6-7 servo hydraulic actuators

The actuator loads are applied via load cells to the centres of the spreader beams supported on two rollers at each end. The rollers are secured on the sleepers at the position where rails would otherwise have been fastened with a separation of 1500-mm centre to centre (1435-mm inside edges of the rails) conforming to the current UK standard (Figure 6-8).

A three-channel controller stores control functions and collects the data from the in-built displacement transducers and load cells (Figure 6-9). The collected data can be obtained and stored using an appropriate data acquisition device connected to a data storage appliance, i.e. PC. The details about the data acquisition setup will be presented in section 6-6.

The simulated cyclic loading of a passing train is achieved by applying sinusoidal loading between 5kN and 97kN with a 90° (Π/2) phase lag between each actuator (Figure 6-10). This means that actuator one would start its loading cycle first and actuator two would be activated once actuator one completes a quarter of the loading cycle. This pattern would continue to actuator three and hence achieve sequential loading across the three actuators with a frequency (f) of 3Hz, which simulates a train with a speed (V) of 28km/hr, calculated as follows using Equation 6-1:

$$\mathbf{V = \frac{\Delta x}{\Delta t} = \frac{Sleeper\ Spacing}{\Delta t}\ where\ \Delta t = \frac{\varphi^{\circ}}{360\,.\,f}} \qquad \textbf{Equation 6-1}$$

Where in the above equations:
V = Speed
φ° = Phase Angle
Δt = Time Shift
f = Frequency



Figure 6-8 RTF loading arrangement

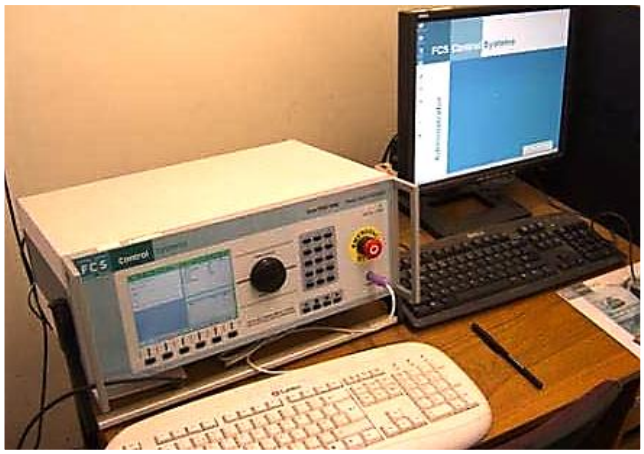


Figure 6-9 Three-channel controller

The 3Hz loading frequency and the resulting speed of 28km/hr were the maximum achievable based on the pressure and flow capacity of the laboratory hydraulic pump. Although the value of achievable loading frequency in this study seems relatively low, past studies e.g. Lekarp et al. (2000a) have shown that loading frequency has negligible influence on the resilient behaviour of granular material unless increased frequency causes dynamic loading through wheel flats or rail imperfections which are not the case in this study.

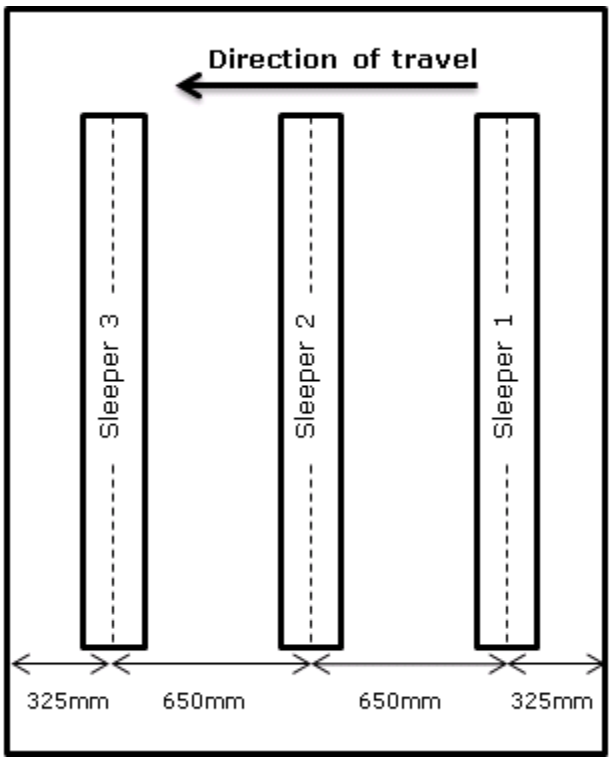


Figure 6-10: Arrangement of sleepers in the RTF

The RTF has been used for several studies in the past (Kwan, 2006, Aursudkij, 2007, Shi, 2009), due to its versatility, for testing and investigation of track related variables. These studies looked at the effect of subgrade type, geogrid reinforcement, formation level stress and seasonal variation. In some cases the effect of ballast tamping was also investigated e.g. Aursudkij (2007) and Shi (2009) used photogrametry to monitor the movement of railway ballast particles. In all aforementioned studies, the middle sleeper (sleeper number 2) was considered as the closest representative of a real life railway sleeper due to the realistic boundary condition it has in conjunction with the two outer sleepers. The outer sleepers are not restricted symmetrically in the direction of travel and they are both limited by the concrete pit wall (Figure 6-10). Moreover, in all the previously mentioned studies, the outer sleepers settled more than the middle one. This can happen due to the lack of rails to connect the three sleepers together

causing the individual sleepers to operate independently. Lack of rails also allows the outer sleepers to tilt away from the centre as they are not symmetrically constrained. Outer sleeper tilting was also explained by Kwan (2006) and Brown et al. (2007a) to happen as a result of poor compaction and the closeness to the smooth, polythene-covered walls of the pit. Kwan also made an effort to avoid the outer sleepers tilting by connecting them to the frame by means of a thin wire rope to keep them vertical; however, the attempt was not successful.

Furthermore, the author is of the opinion that the outer sleepers tilt away from the centre as a result of non-symmetrical conditions at the sides of the sleepers (Figure 6-11(b)). As can be seen in Figure 6-11 (a), at the maximum load the middle sleeper has symmetrical conditions in terms of confinement and loading influences from adjacent sleepers. However, the outer sleepers are under non- symmetrical conditions and these result in more settlement at the less confined and also possibly less well compacted side of the sleeper compared to the other side (Figure 6-11 (b)).

It should be noted that the purpose of the outer sleepers was not to represent the performance of the real sleepers. However it was to recreate the correct in-situ conditions for the middle sleeper.

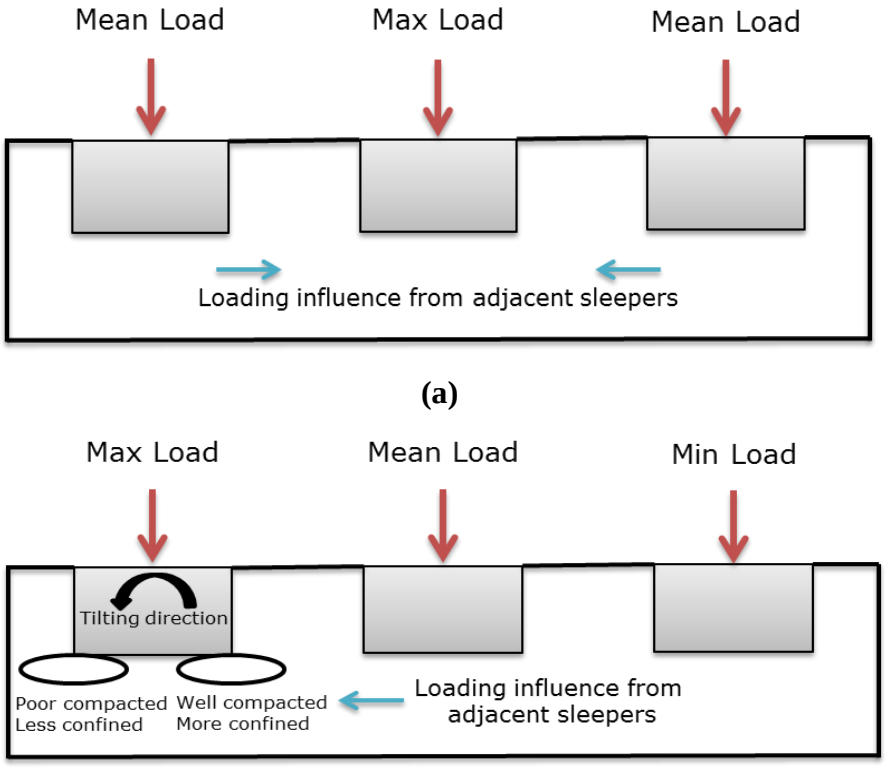


Figure 6-11: Loading influence of adjacent sleepers (a) middle sleeper at max load (b) outer sleeper at max load

In this study very careful consideration was made of the track construction process for all the tests to minimise potential tilting by providing consistent support conditions for all three sleepers (the track bed construction process will be explained later in this chapter).

Figure 6-12 shows the load distributions along three successive sleepers in the RTF, as determined by Brown et al. (2007a) in accordance with the load pattern suggested by Awoleye (1993) (Figure 6-13). The testing facility simulates an axle load of approximately 20 tonnes which is approximately comparable to a typical heavy freight axle load (Aursudkij, 2007).

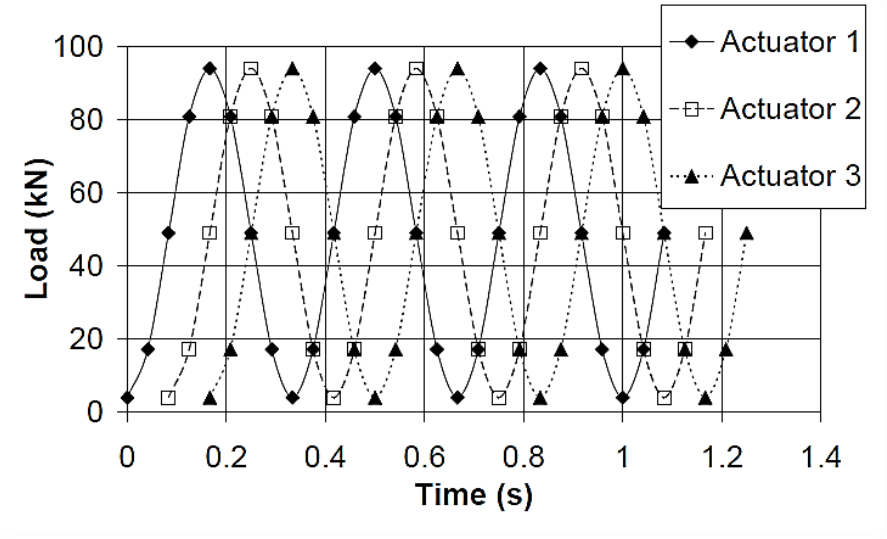


Figure 6-12 Phase loading used in the RTF

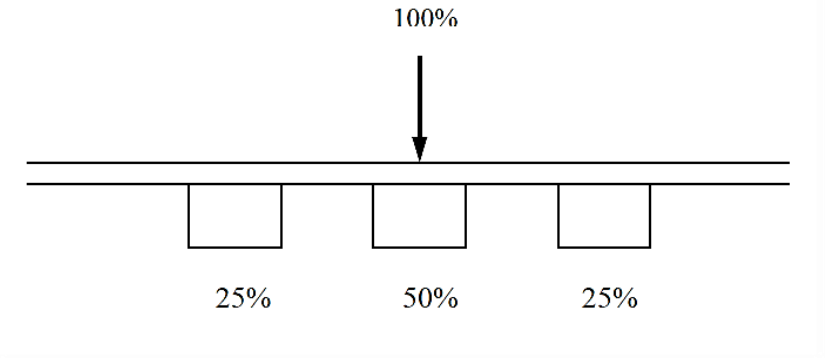


Figure 6-13 The load distributions along successive sleepers in the RTF (Awoleye, 1993)

6.5 RTF INSTRUMENTATION

The full instrumentation suite used in the RTF setup for every test comprised fourteen LVDTs, three 100kN load cells and two pressure plates. The LVDTs were DCTH1000 (Figure 6-14), with a range of ±25-mm, with <±0.5/±0.25/±0.1 linearity, and were purchased from RDP electronics Ltd (RDP Electronics Ltd, 2012). The LVDTs were positioned in pairs at the ends

of individual sleepers except for the middle sleeper for which an extra pair of transducers were located mid-sleeper (Figure 6-15). Measurements from the additional centrally located pair of LVDTs were considered a useful indicator of the mode of bending of the sleepers over the duration of the test. As per the preliminary tests, all the LVDTs were individually calibrated and found to be linear over their respective ranges.

The RTF load cells were F254 fatigue rated ‘‘pancake’’ load cells purchased from Novatech Measurements Limited and designed for high levels of cyclic loading (Figure 6-16). Each load cell had a capacity of 100kN (10 tonnes) with a 50% safe overload and a 200% ultimate overload (Novatech, 2015). All three load cells were calibrated individually in compression and it was found that they all had a rated output of 2mV/V over capacity of the load cell. Therefore using a 10V DC excitation voltage source, the load cell will output 20mV at its full 100kN capacity.

The pressure plates were installed at locations on the concrete slab vertically corresponding to the location of the rail seat and middle of sleeper 2 (Figure 6-17). Further explanations about design, manufacture, and installation of the pressure plates are presented in the next section. The location of all instrumentation is shown in Figure 6-15.



Figure 6-14 External LVDTs model DCTH1000 (RDP Electronics Ltd, 2012)

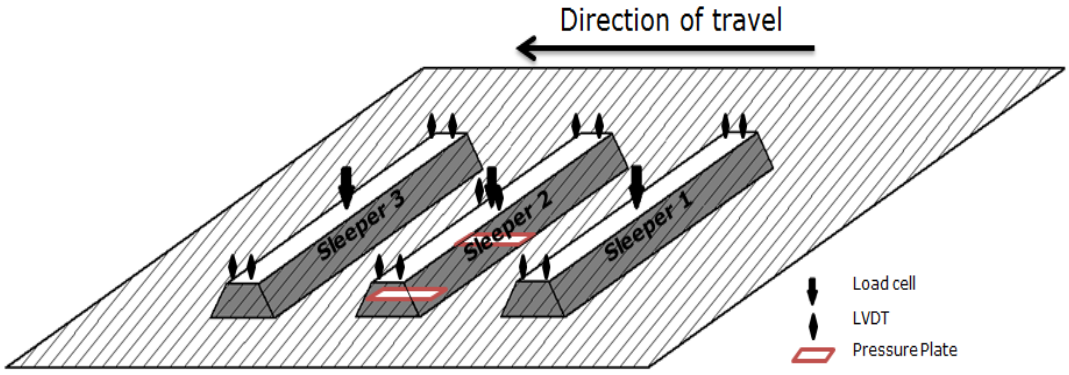


Figure 6-15 The RTF instrumentation locations



Figure 6-16: F254 fatigue rated pancake load cell (Novatech, 2015)

6.5.1 PRESSURE PLATES

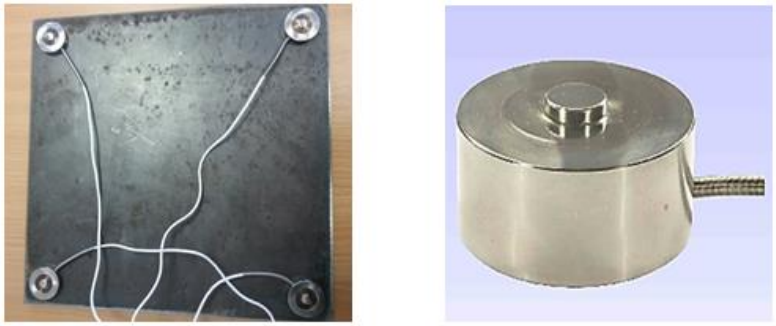
Pressure plates were designed and installed to measure pressure at a 300-mm depth (at the ballast-foundation interface), coinciding with the midpoint and one rail seat of the middle sleeper. Each pressure plate comprised a 300-mm by 300-mm, 12-mm thick rigid steel square plate instrumented with a load cell at each corner (Figure 6-17a). The load cells are model 53 (Figure 6-18 (b)), 2000lbs (8.9kN) compression load cells with +/-0.25% linearity and were purchased from RDP electronics Ltd (RDP Electronics Ltd, 2012). They were connected to a data acquisition device and the summation of the outputs of the load cells was divided by the area of the plate to obtain a pressure measurement.

It was initially proposed to use three pressure plates equally spaced on the concrete slab 300-mm below the middle sleeper at locations corresponding to the minimum and maximum expected pressures. However due to the symmetry of the setup it was decided that two plates would suffice to assess the suitability of the pressure plates on the concrete slab, without influencing the existing geometry of the experimental set-up. Two square recesses with dimensions of 300-mm × 300-mm × 22-mm were created in the concrete to accommodate the two pressure plates (Figure 6-17).

Both pressure plates as a system of four load cells were calibrated and checked for linearity both in loading and unloading stages. The measured calibration factors were used as inputs for partial data processing using LabVIEW software.



Figure 6-17 Pressure plates accommodated into the subgrade



(a) **(b)**

Figure 6-18 (a) Pressure plate (b) Load cell model 53 (RDP Electronics Ltd, 2012)

The pressure plates were designed with consideration to the maximum pressure expected at 300-mm below the middle sleeper using the 2:1 trapezoidal rule as follows:

Application of the 2:1 Trapezoidal Rule

The 2:1 approximation is a simple method to estimate vertical stresses at any given horizontal plane within the loaded area below the sleeper. The method assumes that stress disperses with depth in the form of a trapezoid with a 2:1(vertical:horizontal) ratio as shown in Figure 2-12. The vertical dissipated stresses reduce with increasing depth due to the increase in the area over which the load is spread. According to Jeffs and Tew (1991a) the load spread method provides an average value of the vertical stress. The maximum vertical pressure at a given depth below a rectangular sleeper was estimated as follows:

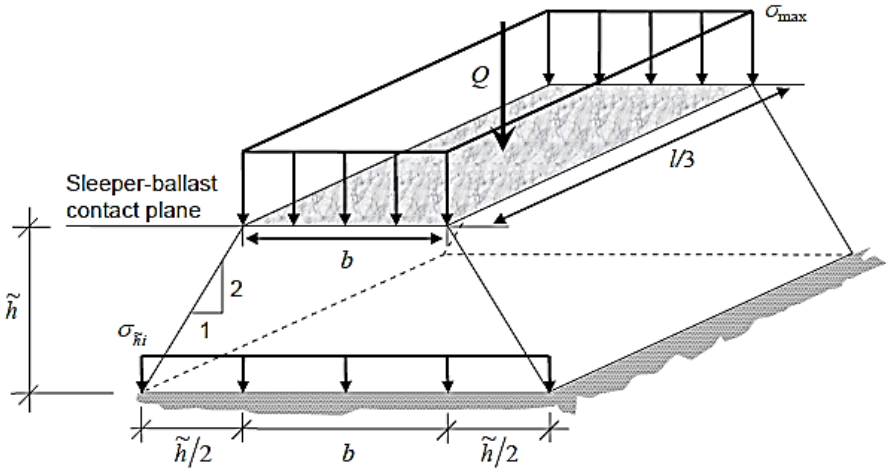


Figure 6-19 Approximation by 2:1 method for the calculation of the induced vertical stress at depth due to an applied load Q

$$\boldsymbol{\sigma_{z\,max} = \frac{Q}{(b+\tilde{h})\left(\frac{l}{3}+\tilde{h}\right)}} \qquad \textbf{Equation 6-2}$$

$$\boldsymbol{\sigma_{max} = \frac{Q}{A_{sb}}} \qquad \textbf{Equation 6-3}$$

Where in the above equations:

Q = static wheel load for a third of the sleeper
$\tilde{h}$ = thickness of ballast
$\sigma_{z\,max}$ = maximum stress on top of rubber mat
σ_{max} = maximum sleeper − ballast contact stress
l = sleeper length
b = sleeper width
A_{sb} = contact area between the sleeper and ballast for a third of the sleeper

Q=100kN
$\tilde{h}$= 0.3m
b= 0.285m
l= 2.5m
A_{sb}=0.2375m^2
Therefore:
σ_{max} = 421.0526kPa
$\sigma_{z\,max}$= 150.8739kpa
Pressure Plate Area= 0.09m^2
Maximum Load applied on each pressure plate= 13.57kN < (4× 8.9) kN

Assuming a rectangular shape for the concrete sleeper with l and b of 2500-mm and 285-mm respectively σ_{max} is calculated as 421kPa and $\sigma_{z\,max}$ at 300-mm below sleeper bottom is

calculated as 150kPa. The maximum load applied on each pressure plate is accordingly calculated as 6.78kN which is less than the maximum capacity of the chosen load cells.

The calculated value was consistent with the value reported by Brough et al. (2003) which showed that subgrade stresses 300-mm below sleeper bottom ranged between 71 and 196 kPa depending on traffic type. However, it was not consistent with the Talbot Committee (1918) who found that the subgrade pressure, when a centre sleeper accommodates 50 percent of the wheel load, is approximately 386 kPa. The value reported by Talbot Committee (1918) is consistent with the stress expected at the sleeper-ballast interface (Shenton, 1975, Esveld, 2001, Indraratna et al., 2011a).

6.6 DATA ACQUISITION

All the data from LVTDs, pressure cells and the RTF compression load cells were acquired using the data acquisition setup shown in Figure 6-20.

The instrumentation signals were acquired using National Instruments (NI) software and signal conditioning hardware in the form of the LabVIEW graphical programming language and a National Instruments SCXI based data acquisition platform. The details of the data acquisition setup are presented in Table 6-1.

The data recording for all the instrumentation was acquired at a rate of 60Hz (20 samples per cycle). A low pass filter was integrated into the signal acquisition setup in order to avoid a random fluctuation in the signals. The fluctuations in signals were caused by the high volume of signals being acquired and the environment of the laboratory.

Table 6-1 Data acquisition details

Instrumentation	Front mounting block - connection terminal	Excitation Source
Pressure plate load cells	SCXI 1314	SCXI 1000 – 10V
RTF load cells	SCXI 1314	Control system – 10V
External displacement transducers (RDP LVDTs)	SCXI 1300	External (Voltmeter) – 25V

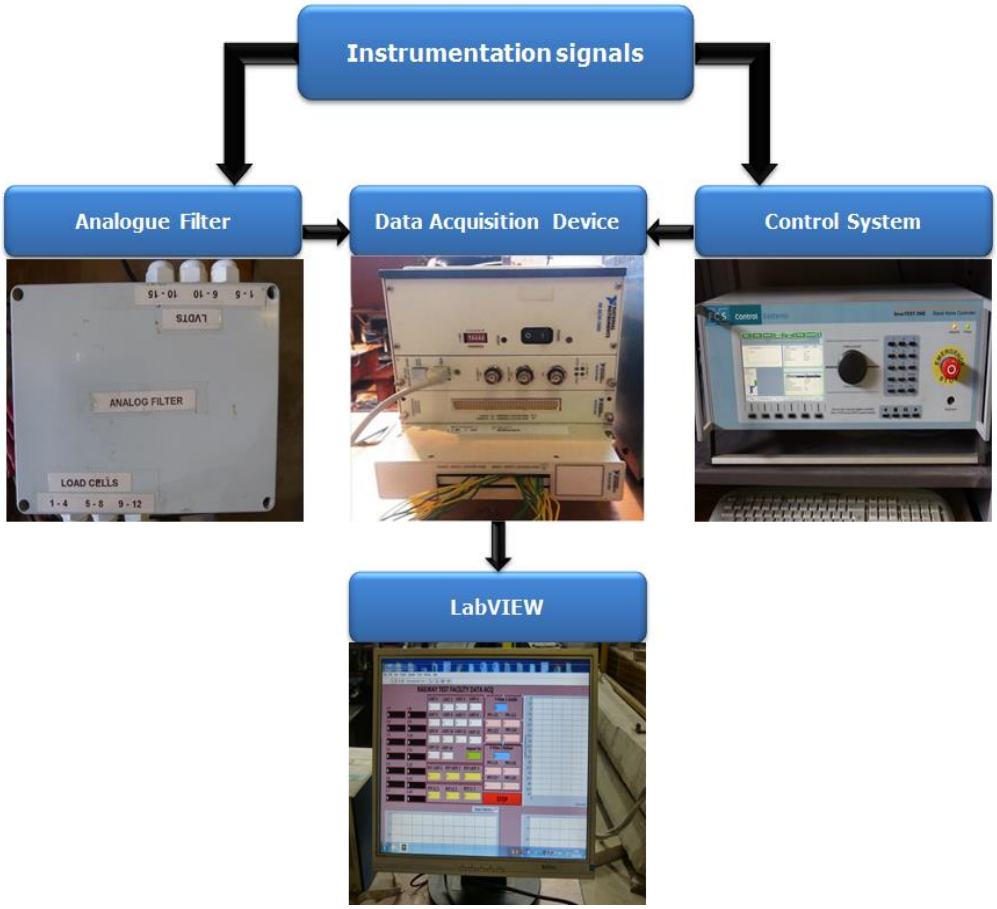


Figure 6-20 Data acquisition setup

6.7 MATERIALS

The ballast source and specification for the RTF tests were the same as the full-size ballast previously highlighted for the triaxial, composite element test, and box test experiments. The sleepers used for the RTF tests were full sized sleepers of the materials, weights, and dimensions indicated in Table 6-2.

Table 6-2 Summary of sleeper types

Sleeper Type	Sleeper Base Dimensions (mm)			Weight (Kg)
	Length	Width	Height	
Concrete Mono-block (G44)	2500	285	210	300
Concrete Twin-block	2400	300	210 -250*	250
Hardwood Timber	2600	290	170	166
Steel-reinforced plastic composite	2600	250	150	100
Steel (560H) Hook-in	2400	335	120 - 170**	84

* Inner to outer face of a single block parallel to running rail
** Sleeper middle to sleeper end

Figure 6-21 to Figure 6-25 show the sleepers that were used in this study. The photos were taken after they were tested. As can be seen in the photos the concrete mono-block and twin-

block sleepers are pre-fitted with integrated rail cradles. However the rest of the sleepers had to be fitted with detachable custom made cradles to allow the installation of the spreader beam roller supports. The aforementioned sleepers are varied in terms of material make up and geometry and considered as representative of the sleepers being widely used in the UK as well as Europe. Further details about each type of sleeper, their application, advantages, and disadvantages can be found in section 2-6.



Figure 6-21 Concrete mono-Block Sleeper



Figure 6-22 Concrete twin-Block Sleeper



Figure 6-23 Timber sleeper



Figure 6-24 Plastic Sleeper



Figure 6-25 Steel Sleeper

Similar to the box test and composite element test experiments (preliminary tests), in the RTF, performance of Under Sleeper Pad (USP) type1 and type2 was investigated. All the properties of the USPs used in the RTF were the same as in the preliminary tests.

6.8 GENERAL PREPARATION OF SAMPLE TRACKBED

For the five types of sleepers tested, preparation of trackbed was identical in all cases except in the case of the steel sleeper which will be explained later in this section. Below are the general steps to prepare the trackbed before starting a test:

1. Following installation of pressure plates and the overlying layers of rubber mats (Figure 6-5), 300-mm thick ballast was placed to achieve an even surface using wooden boards with footprints equivalent to a full size sleeper and levelled with a spirit level. This was repeated where necessary and ballast was added /removed when required. According to NR/L2/TRK/2102 (NetworkRail, 2016), the minimum ballast depth for mainline track is 300-mm. The top surface was also further levelled using an electric compactor to ensure a flat surface for good contact with the sleeper and repeatability of tests (Figure 6-26).



Figure 6-26 placed 300-mm ballast

2. Three sleepers were placed carefully on top of ballast by means of a bridge crane (Figure 6-27) with sleeper spacing of 650-mm, conforming to NR/L2/TRK/2102 (NetworkRail, 2016).



(a)



(b)

Figure 6-27 The RTF sleeper spacing (a) front view (b) top view

3. Once the sleepers had been placed, the vacant crib ballast was then placed and a ballast shoulder length of 375-mm (conforming to NR/L2/TRK/2102) was created at both end of sleepers (Figure 6-28).



Figure 6-28 Placed crib Ballast

4. Once shoulder ballast was in place, a slope of 45^{o} was manually formed to line up with guide markings on the concrete wall (Figure 6-29).

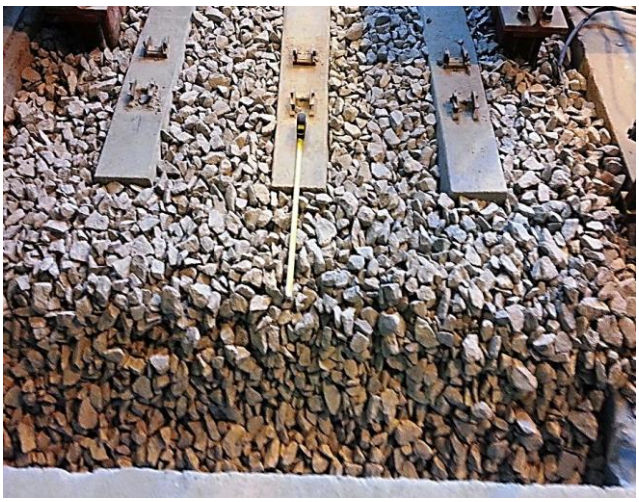


Figure 6-29 Placed shoulder ballast

In the case of the steel sleepers, two methods of installation were employed. The first method involved driving the steel sleepers (driven method) in turn (sleeper one – sleeper two – sleeper three) with an incremental static load of 20kN till a maximum load of 100kN was achieved. Prior to the application of the load increments, all three sleepers were subjected to a seating load of 5kN. Imposing an initial seating load and using a staged loading sequence is considered prudent to ensure that the support conditions for each sleeper remain as independent as practicable. Furthermore, the initial seating load ensured that the subsequent staged loading

sequence did not result in sleeper tilt or uplift. When the maximum 100kN load was reached, a visual inspection through the inspection holes revealed some evidence of voiding in the underside of the sleepers, the pattern of which was observed to be consistent for all three sleepers. At this stage it was not possible to push the sleeper into the ballast any further due to the limited load capacity of the rig and the resistance at spade ends of sleepers near the rail seat. Although the application of a 100kN preload during the installation of the steel sleepers using the driven method is likely to have resulted in early stiffening of the ballast ahead of cyclic loading, it is in keeping with the standard industry practice of installing steel sleepers with the assistance of a ballast train (Figure 2-25) with a bespoke machine (Figure 2-26) to drive them into the ballast bed before being tamped.

The second method of steel sleeper installation was known as "Wished-In-Place" (WIP). The WIP method is not actually used in track renewals; however, it was devised in this investigation to simulate the filling effects of frequent tamping during installation of steel sleepers in practice. For the WIP method, three domes (mounds), each consistent with the internal dimensions of a standard sized steel sleeper were created at the appropriate sleeper spacing. The three steel sleepers were placed over the created domes such that contact was achieved between the ballast particles and the underside of the sleepers at the ballast–sleeper interface. In areas where full contact could not be achieved, the process was assisted by manually pushing the ballast particles from the adjacent cribs into the underside of the sleeper. In contrast, for the driven steel sleepers the driving force was limited by the load capacity of the rig so areas of voiding could not be rectified with extra driving force. Furthermore, unseating the sleeper from its driven position to manually ensure full contact with the ballast at the sleeper interfaces, would likely have interfered with the lateral support or the side friction that had been mobilised during the driving process.

Consistent with the box and composite element tests, the sleeper types for the tests including Under Sleeper Pads (USPs) used an epoxy resin which was allowed to cure for a minimum of 12 hours as a means of bonding the two elements.

6.9 TEST PROCEDURE AND PROGRAMME

Once the set-up of the trackbed and the associated instrumentation were completed, the facility was assembled as shown in Figure 6-30. The actuator strokes and the LVTDs were assessed to ensure they had sufficient travel to accommodate the expected settlement. A 5kN seating load

was applied to ensure and maintain a contact between the load cell and the spreader beam. Once the seating load was applied, initial readings from all LVDTs, displacement transducers and load cells were recorded (prior to start of testing). A million cycles of dynamic load was then applied to each sleeper sequentially. The dynamic load applied was in the form of a sine wave with amplitude of 46kN, a frequency of 3Hz and a mean load of 51kN. All the instrumentation outputs were sampled at a rate of 60Hz corresponding to twenty samples per cycle.

In total, fourteen unique tests were performed in the RTF. In the first six tests, the performance of five different types of sleeper was tested. However, the performance of steel sleepers was investigated using two different installations, namely the driven and Wished-In-Place (WIP) methods. In the remaining eight tests, the performance of the sleeper types (with the exception of the steel sleeper) was investigated with the Type1 and Type2 USPs. A list of the full-scale tests conducted using the RTF is presented in Table 6-3. To verify the replicability of the conducted tests, concrete mono-block and twin-block sleeper tests were repeated over 500,000 cycles.



Figure 6-30 The RTF test set-up

Experiment Testing ID	Experimental Factors / Variables								
	Sleeper Type/Material						Ballast Interventions		Ballast Grading
	Concrete Mono-block	Concrete Twin-block	Timber	Plastic	Steel Driven	Steel Wished In Placed	Under Sleeper Pad Type I	Under Sleeper Pad Type II	Network Rail
Experiment 1	Y	N	N	N	N	N	N	N	Y
Experiment 2	N	Y	N	N	N	N	N	N	Y
Experiment 3	N	N	Y	N	N	N	N	N	Y
Experiment 4	N	N	N	Y	N	N	N	N	Y
Experiment 5	N	N	N	N	Y	N	N	N	Y
Experiment 6	N	N	N	N	N	Y	N	N	Y
Experiment 7	Y	N	N	N	N	N	Y	N	Y
Experiment 8	N	Y	N	N	N	N	Y	N	Y
Experiment 8	N	N	Y	N	N	N	Y	N	Y
Experiment 10	N	N	N	Y	N	N	Y	N	Y
Experiment 11	Y	N	N	N	N	N	N	Y	Y
Experiment 12	N	Y	N	N	N	N	N	Y	Y
Experiment 13	N	N	Y	N	N	N	N	Y	Y
Experiment 14	N	N	N	Y	N	N	N	Y	Y

Table 6-3 Test Programme

6.10 RESULTS

The key parameters of comparison from the RTF tests can be categorised as follows:

- Settlements of sleepers
- Stiffness of trackbed,
- Pressure on trackbed subgrade (foundation)
- Deflections of sleepers
- Bending stiffness of sleepers
- Coefficient of friction at ballast-sleeper interface

Settlement results were recorded for all three sleepers; however, the result from the middle sleeper is the main focus of the presentation in this section, since it best represented the in-situ track conditions. The outer sleepers provided the required boundary conditions for the middle

sleeper and according to the previous RTF studies, were expected to settle more. In section 6.10.6, recorded settlement for the outer sleepers will be presented and compared against the middle sleeper in the cases of different sleeper types with and without inclusion of USPs. Trackbed stiffness was measured using the heavy-light load method explained in section 4.7.2. Except where mentioned otherwise all stiffnesses and settlement results presented in this chapter were calculated using the average value of the 4 LVDTs positioned at the corners near the ends of the sleepers.

Similar to the box and CET tests, the settlement results for the first 10 loading cycles were ignored due to the non-reproducibility of the ballast behaviour at this stage. This uncertain behaviour of ballast can be explained as a result of the small number of ballast particles in contact initially with the underside of the sleeper and bedding into the ballast of the sleeper. Therefore, the sleeper settlement results were zeroed after 10 cycles to minimise the aforementioned uncertainty and to make the comparisons clearer.

When sleepers are newly placed or tamped, the contact area at the ballast-sleeper interface is idealised to be mainly below the rail seat (Raymond, 1978, Esveld, 2001). By packing ballast underneath the rail seats, the highest level of support would be provided directly underneath the rail. This pressure can be, however, varied after a number of loading cycles. For instance in the case where ballast is not well supported, it can lead to development of a gap underneath the end of a concrete mono-block sleeper. This condition according to Raymond (1978) is known as centre binding. In the case of centre binding, the sleeper is mainly supported at the centre and the rail loads are mainly transferred to the ballast in contact with the central section of the sleeper. The bending stiffness of a sleeper affects the tendency of a sleeper toward centre binding. Therefore, bending stiffness and deflections of sleeper were calculated to study the tendency toward centre binding and to investigate the impact of it on the maximum pressure on the trackbed subgrade. A series of static tests were performed on sleepers to measure their bending stiffness, details of which will be presented in the following section. The sleeper deflections were also calculated from differences in the settlements between the middle and the rail seats of a sleeper over a number of cycles under maximum load. These results were then explained with reference to related pressure graphs.

6.10.1 SLEEPER BENDING STIFFNESS

As presented in Figure 6-31, static tests were conducted on steel, timber, plastic, and mono-block concrete sleepers to compare the bending stiffnesses of the sleepers. The setup for the

test was arranged in a way that the sleepers were rested on two supports symmetrically and were subjected to a defined vertical load applied by a hydraulic jack. The vertical load was applied at a rate of 5kN/min until it reached 100kN (hydraulic jack limit), or the sleeper failed, or it was not safe to continue. The vertical deflection of the sleeper during the test was recorded by two LVDTs at the sleeper midspan, capable of measuring deflections up to 100-mm. In the case of the twin-block sleepers, the surface of the rod connecting the blocks was not flat (Figure 6-32) and it was not possible to apply the load on top of it in a similar way to the rest of the sleepers tested. Therefore, a steel plate was placed between the rod and the loading points (Figure 6-32) to achieve a flat surface to apply the load. Adding the steel plate, it was still not possible to complete the test, since the sleeper tilted as soon as the test was started and it was not safe to continue. After a few failed attempts to conduct the test and laboratory safety officer advice, the bending stiffness test for the twin-block sleeper was discarded.

Figure 6-33 shows the sleeper deflections against the applied loads for the tests conducted. In the case of concrete mono-block, the test was continued up to sleeper failure (Figure 6-33 (d)). For the timber sleeper, the sleeper behaviour was elastic and it was continued up to 100kN static load and as can be seen in Figure 6-33 (b), the permanent deflection of the timber sleeper after unloading was almost zero during the test. On the other hand, the deflections of the plastic sleeper during the test were large and the test was continued up to the reliable limit of the LVDTs (90% of LVDT's limits) used to measure the sleeper midspan deflections.

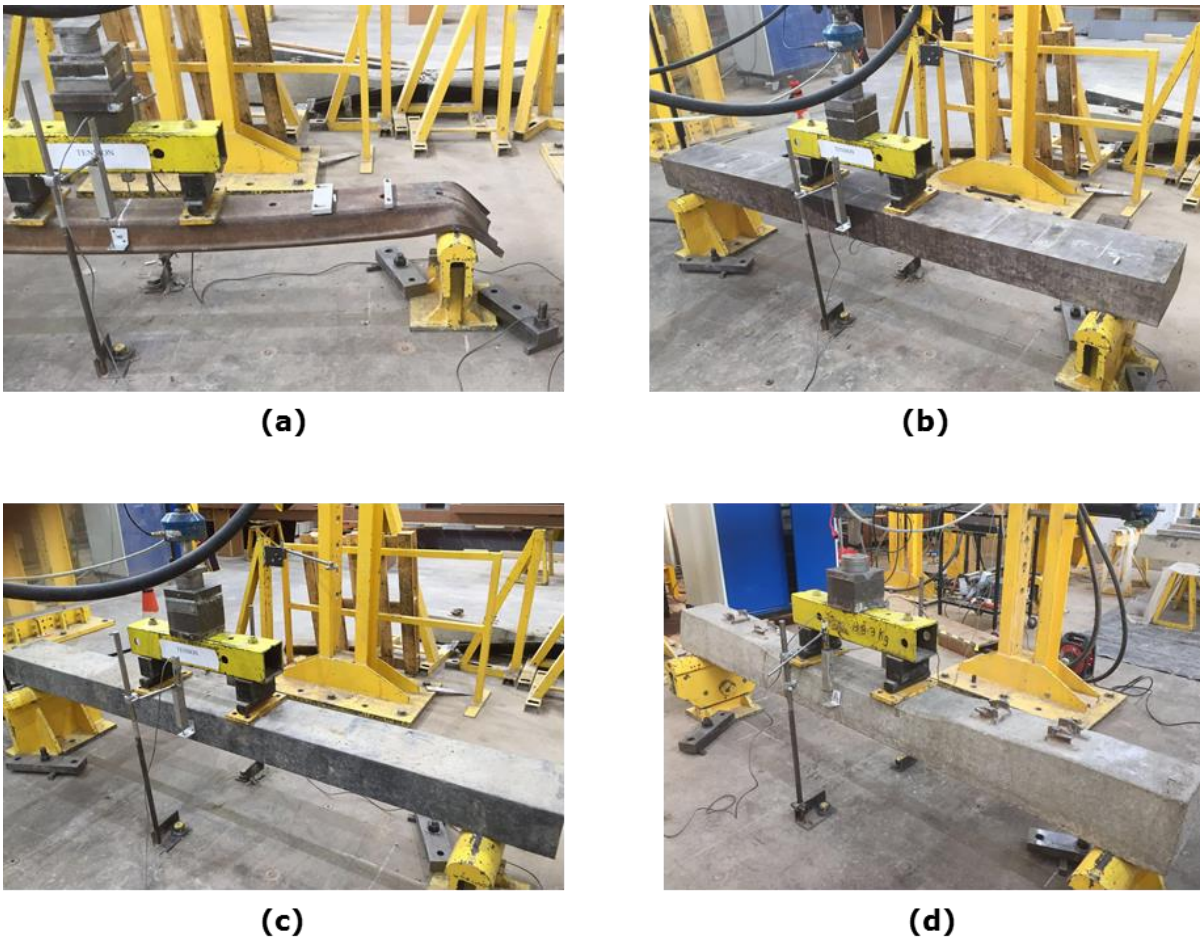


Figure 6-31 Bending stiffness test for (a) steel sleeper (b) timber sleeper (c)plastic sleeper (d) concrete mono-block



Figure 6-32 Bending stiffness setup for twin-block sleeper

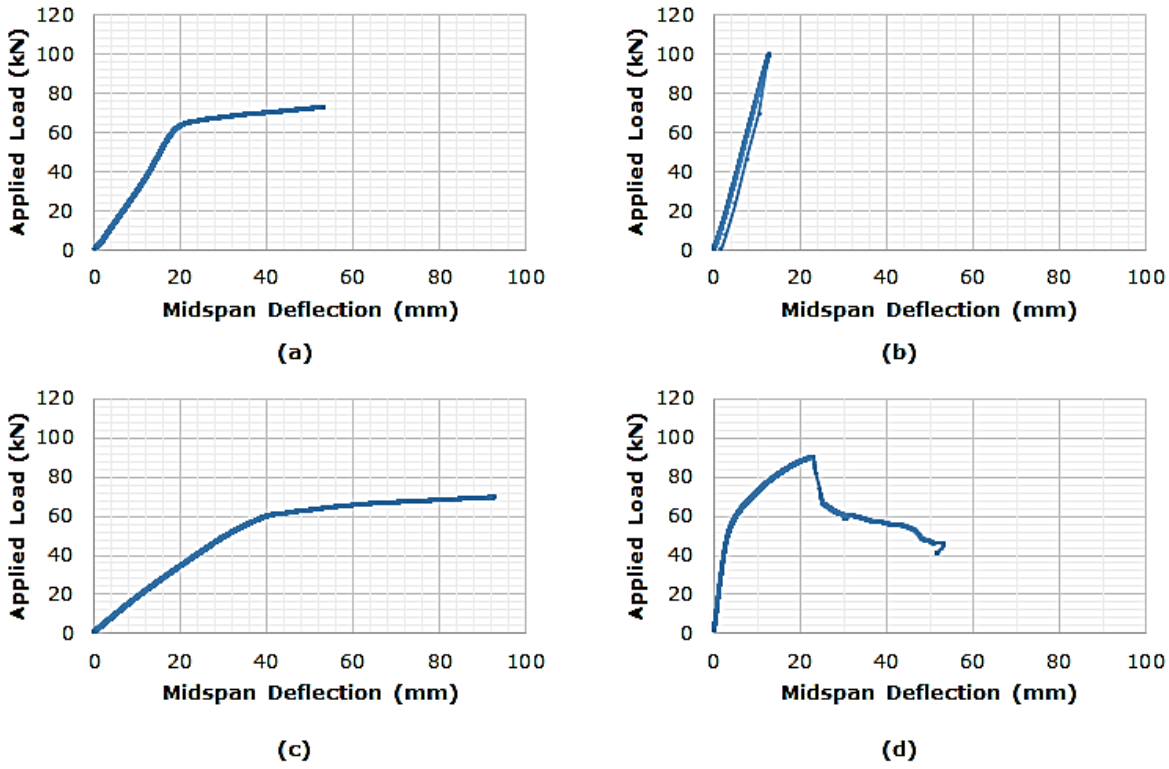


Figure 6-33 Applied load versus sleeper midspan deflection for (a) steel sleeper (b) timber sleeper (c)plastic sleeper (d) concrete mono-block

As shown in Figure 6-34, the tests conducted represented a simply supported beam carrying two symmetrically concentrated loads and therefore the bending stiffness of a sleeper (EI) can be calculated using Equation 6-4 according to the fundamental mechanics of materials during elastic (linear) behaviour. The applied load (P) and the midspan deflection of the sleeper (δ_{max}) were measured during the test. The distance between the simple supports (L) and the distance between the support and applied load (a) were fixed in all the tests and were 2.5m and 1.075m respectively.

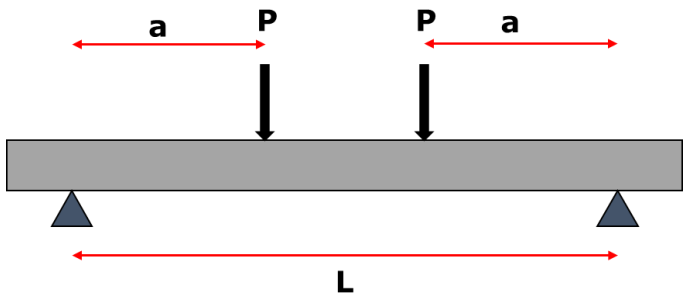


Figure 6-34 Simply supported beam carrying two symmetrically concentrated loads

$$\delta_{max} = \frac{Pa}{24EI}(3L^2 - 4a^2) \qquad \textbf{Equation 6-4}$$

The calculated bending stiffness for the sleeper types are presented in Table 6-4. The results showed the largest bending stiffness for the concrete mono-block and the lowest in the case of the plastic sleeper.

Table 6-4 Stiffness bending of different sleepers

Sleeper types	Bending stiffness (EI) (kN.m^2)
Concrete mono-block	5300
Plastic	500
Timber	2200
Steel	900

6.10.2 TEST REPEATABILITY

To verify the repeatability of results for the conducted tests, concrete mono-block and twin-block sleeper tests were repeated over 500,000 cycles; the settlement results of which are presented in Figure 6-35. As shown in the figures the settlement results of the repeated tests for both concrete mono-block and twin-block are comparable with the first tests and the differences are within acceptable level.

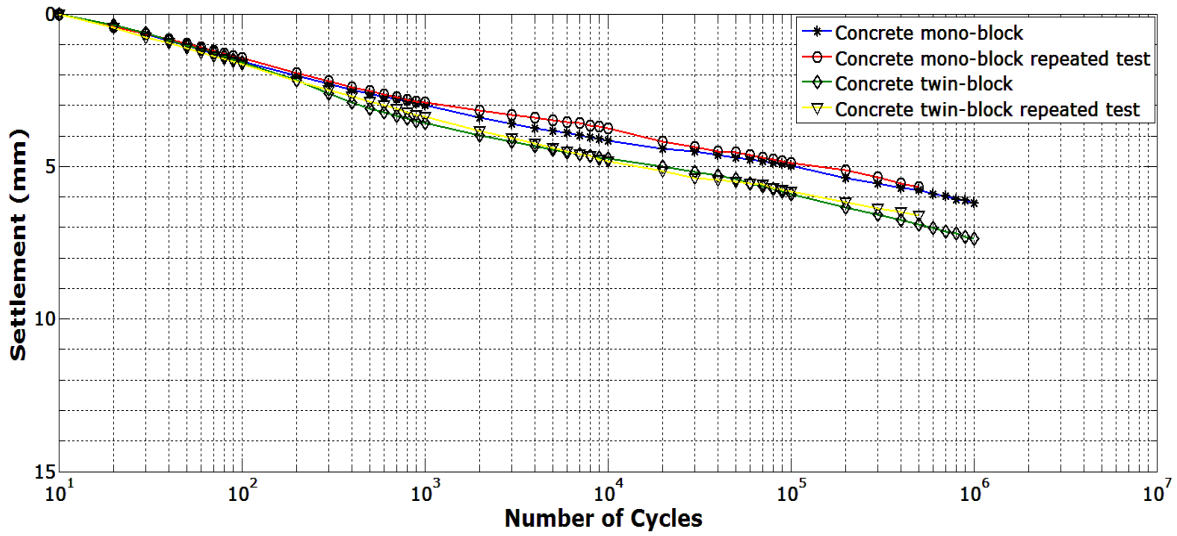


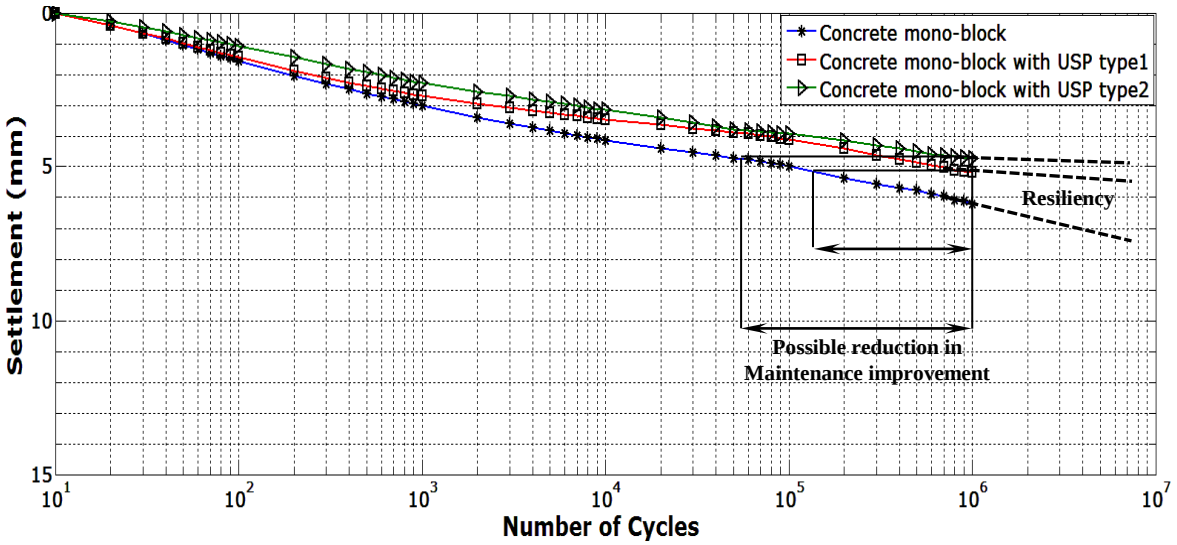
Figure 6-35 Settlement performance repeated test for concrete mono-blcok and twin-block

6.10.3 EFFECT OF USP INCLUSION

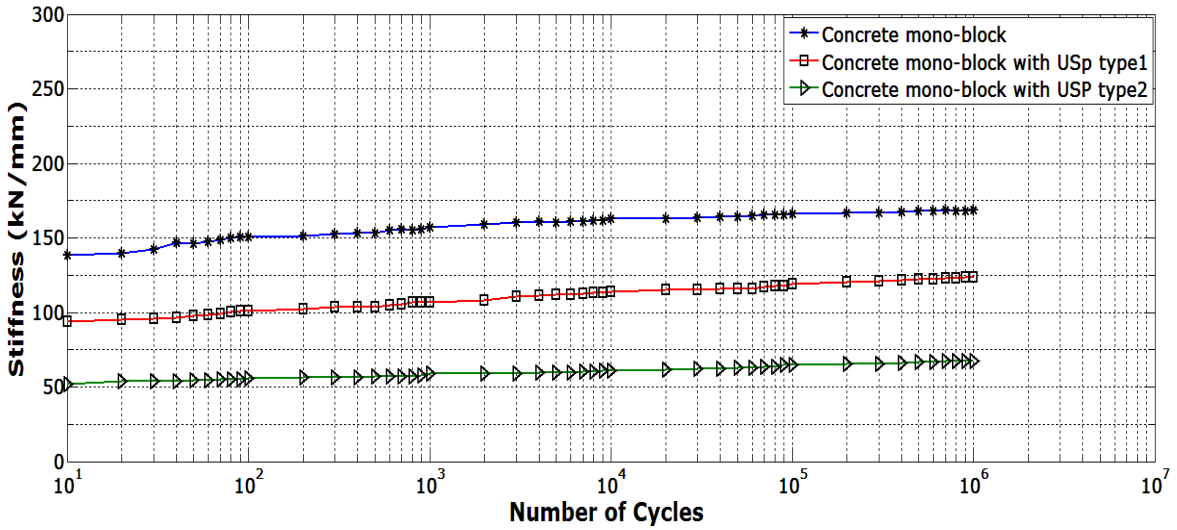
Concrete Mono-Block Sleeper

In the case of the concrete mono-block (Figure 6-36 (a) and (b)), it can be seen that use of USP type1 and type2 (similar to the concrete section in box and CET tests) reduced the settlement of the sleeper. As illustrated in Figure 6-36 (a), permanent settlement of the mono-block sleeper in the case of 'no USP' after approximately 55k and 150k cycles was equal to one million cycles with USP type2 and type1 respectively. This can be considered as an indication of the possible

reduction in maintenance related to settlement using USP for mono-block concrete sleepers. Greater trackbed resiliency was also observed at the end of one million cycles when USPs were used (especially in the case of USP type2) compared to 'no USP'; this meant that in the case of 'no USP' larger permanent deformation would be expected for the sleeper if the test was continued over 1 million cycles compared to when the USPs were used (Figure 6-36 (a)). With regards to the trackbed stiffness (Figure 6-36 (b)), inclusion of USPs resulted in a reduction in trackbed stiffness, which was more evident in the case of USP type2.



(a)



(b)

Figure 6-36 Effect of USP inclusion on concrete mono-block sleeper performance (a) settlement (b) trackbed stiffness

With regard to concrete mono-block sleeper deflections (differences between deflections at sleeper centre and rail seats – Figure 6-37), slight hogging was observed at the middle of the sleeper with and without a USP which was consistent with the high bending stiffness of concrete

mono-block (Table 6-4). Slight hogging at the middle of sleeper was also consistent with higher subgrade pressure observed at the rail seat compared to the middle of the concrete mono-block sleeper (Figure 6-38). However, the subgrade pressure at the rail seat reduced over time and increased at the middle of sleeper. This showed the greater contribution of the middle of the sleeper to carrying the load (centre binding), which can happen over time by migration of ballast away from the ends of sleeper towards the shoulder. This consequently reduced the ballast support at sleeper rail seats. As can be observed in Figure 6-38, this is evident after 100k cycles in the case of 'no USP'; however this happened at higher numbers of cycles when USPs were used (irrespective of type). This can logically be due to the higher coefficient of friction at the sleeper-ballast interface in the case of a USP which causes higher resistance to migration of ballast compared to the 'no USP' scenario. The other possible reason is that the inclusion of USPs (regardless of type) reduced hogging at the middle of sleeper compared to 'no USP'; which can be as a result of larger contact between sleeper base and ballast at the ballast-sleeper interface. This gave the sleeper more uniform support throughout the sleeper base and therefore reduced deflections (hogging) at the middle of the sleeper.

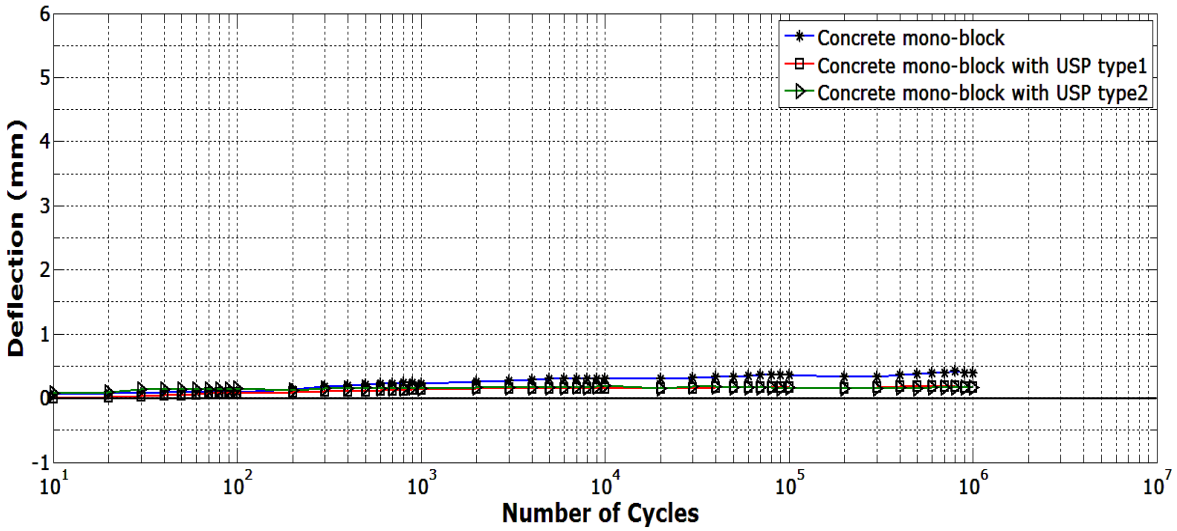
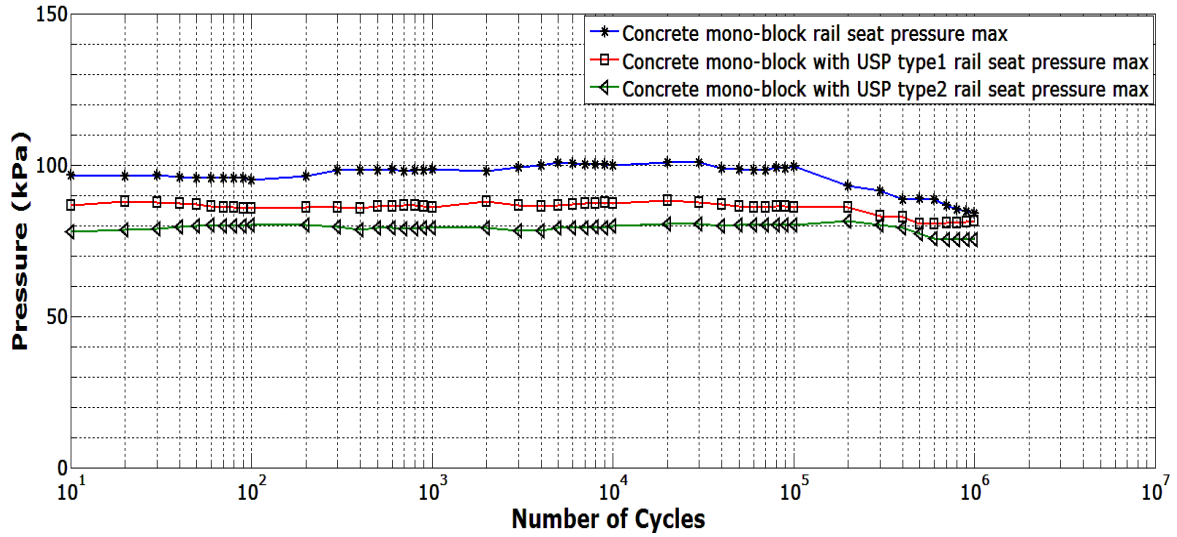
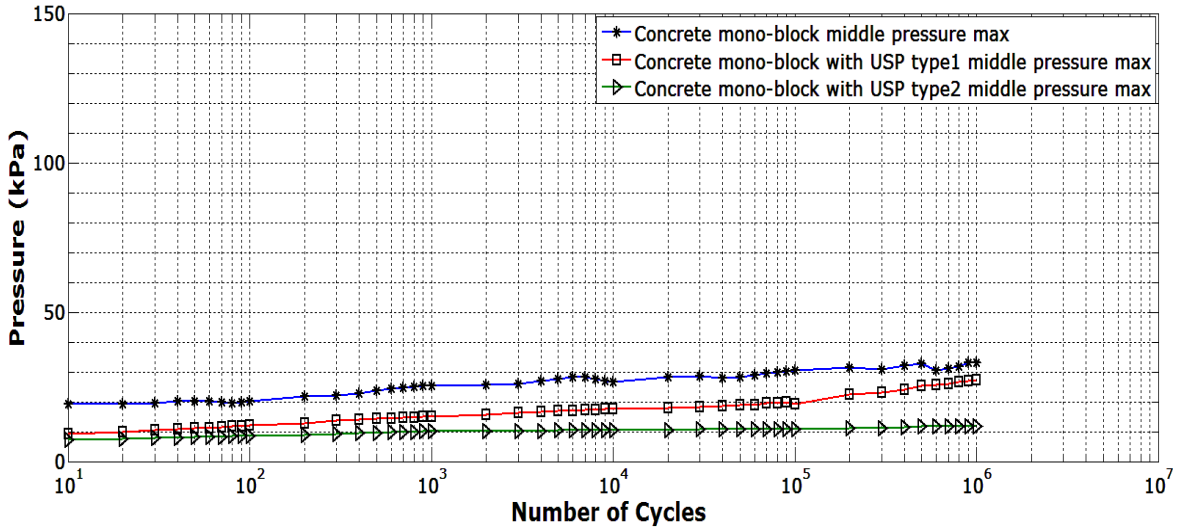


Figure 6-37 Effect of USP inclusion on concrete mono-block sleeper deflection performance



(a)



(b)

Figure 6-38 Effect of USP inclusion on subgrade pressure for concrete mono-block sleeper (a) rail seat (b) middle of sleeper

Concrete Twin-Block Sleeper

The middle of the sleeper in the case of concrete twin-block sleepers was made of a steel bar which was covered by crib ballast when the sleeper setup was completed. The different geometry of twin-block did not allow mounting of the LVDTs at the middle of the sleeper as for the rest of the sleepers. Therefore, the LVDTs for the middle of the sleeper were located at the edge of each block near to the connecting steel bar as presented in Figure 6-39.

For concrete twin-block sleepers, inclusion of USP type1 and type2 comparably improved the performance of the sleeper; however, in terms of resiliency slight improvements were observed when USPs were used. With regard to maintenance reduction possibilities, the twin-block

settlement performance without USPs at 1 million cycles was equal to USP type1 and USP type2 at 55k and 40k accordingly; these ratios are better than for USPs and mono-block concrete.

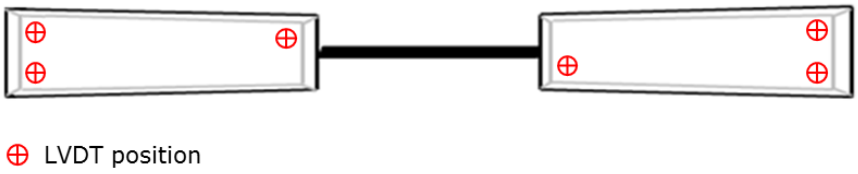
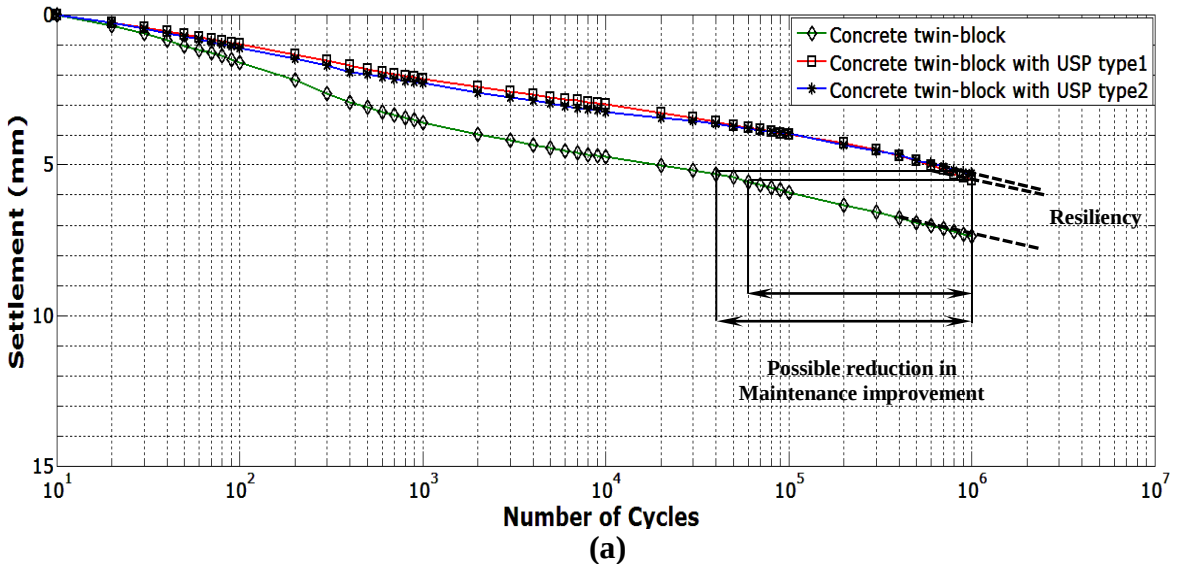
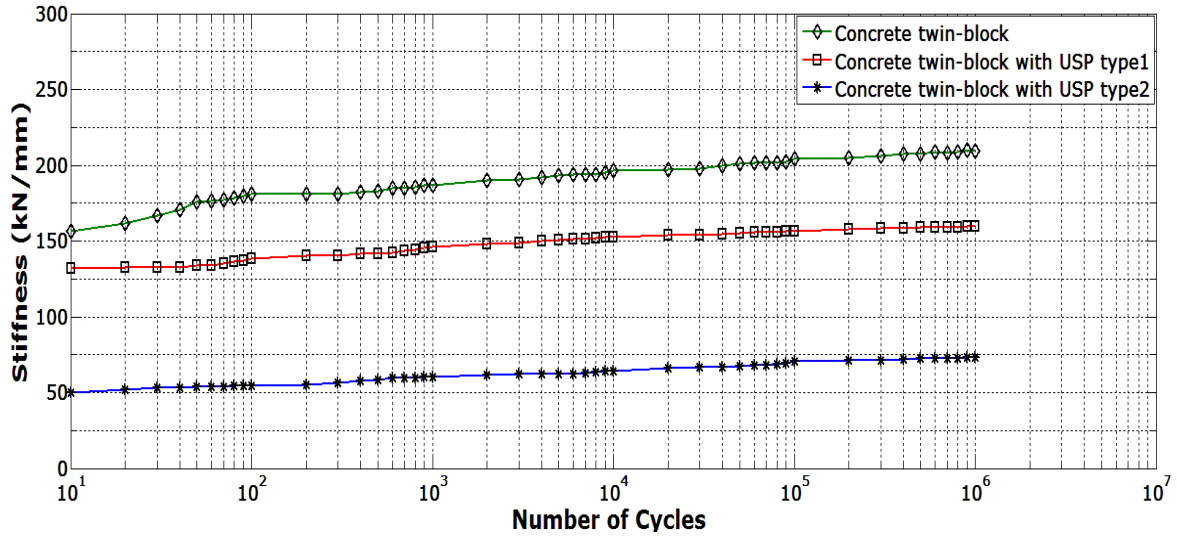


Figure 6-39 Location of LVDTs for concrete twin-block



(a)



(b)

Figure 6-40 Effect of USP inclusion on concrete twin-block sleeper performance (a) settlement (b) trackbed stiffness

Deflections for concrete twin-block sleepers are presented in Figure 6-41. As can be seen in the figure, in the absence of USPs, the sleeper hogging at the middle of sleeper was evident; however this was significantly reduced when USPs were used (regardless of type). The higher sleeper deflections for twin-block compared to those of mono-block can suggest lower bending

stiffness (EI) and higher load damping capacity for the twin-block compared to the mono-block. Therefore, with regard to subgrade pressure (Figure 6-42) the pressure recorded in case of twin-block was less than those recorded in the case of mono-block sleeper. This can also be attributed to the individual blocks (of the sleeper) possibly twisting under load that increase the load damping capacity of the twin-block sleeper compare to the mono-block and consequently resulted in less pressure being transmitted to the formation. In addition, inclusion of USPs reduced the pressure at both the middle of the sleeper and the rail seat. A possible reason can be the load damping effect of USPs and consequently in case of the USP type2 (soft pad) pressure was less compared to the USP type1, as the USP type2 is softer and thicker and has more load damping capability.

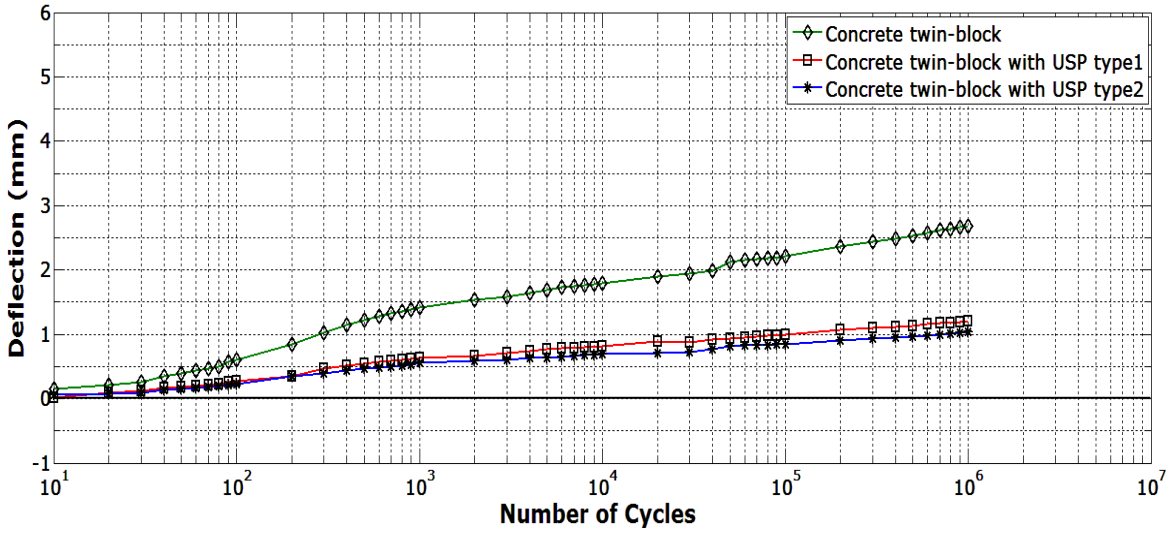


Figure 6-41 Effect of USP inclusion on concrete twin-block sleeper deflection performance

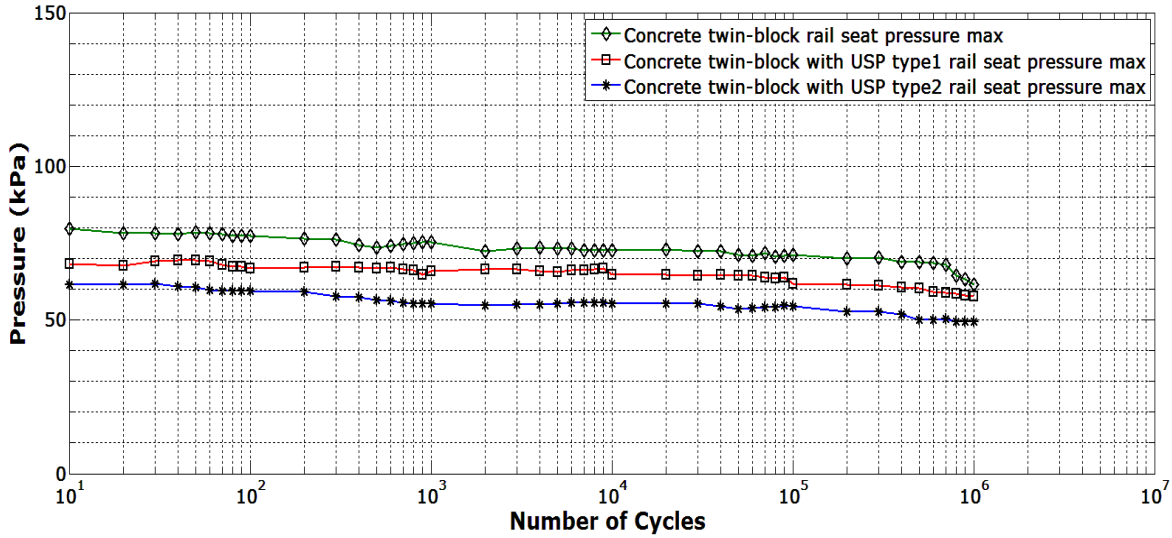
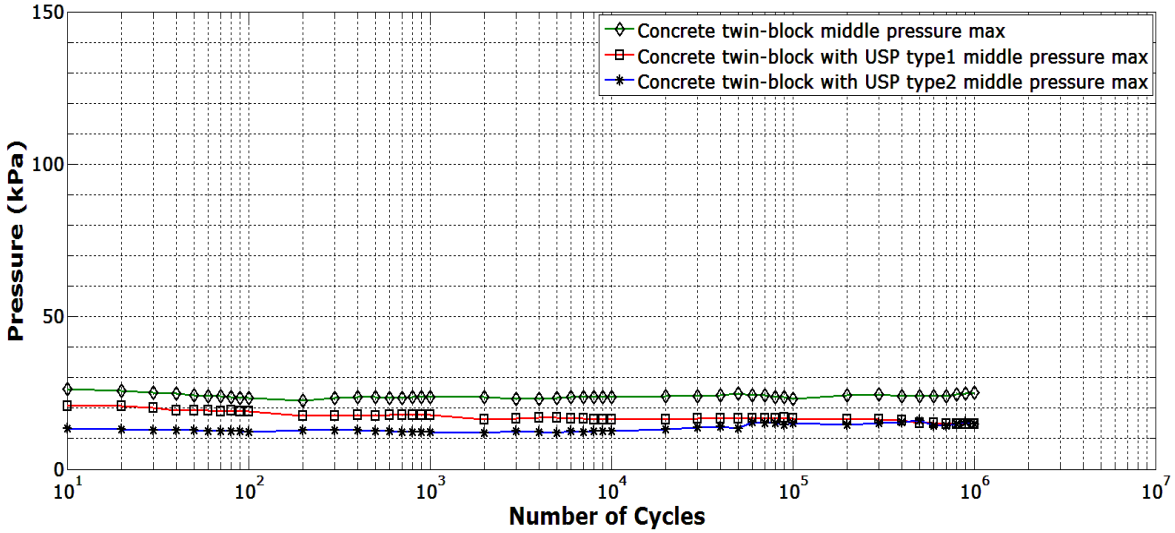


Figure 6-42 (a) Effect of USP inclusion on subgrade pressure for concrete twin-block sleeper rail seat



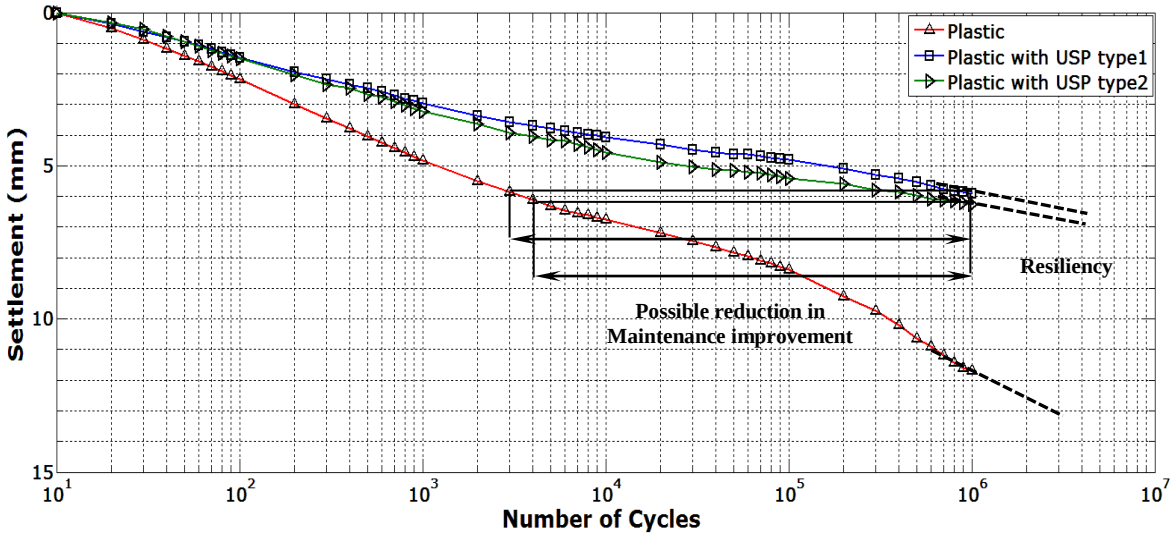
(b)

Figure 6-42 Effect of USP inclusion on subgrade pressure for concrete twin-block sleeper (a) rail seat (b) middle of sleeper

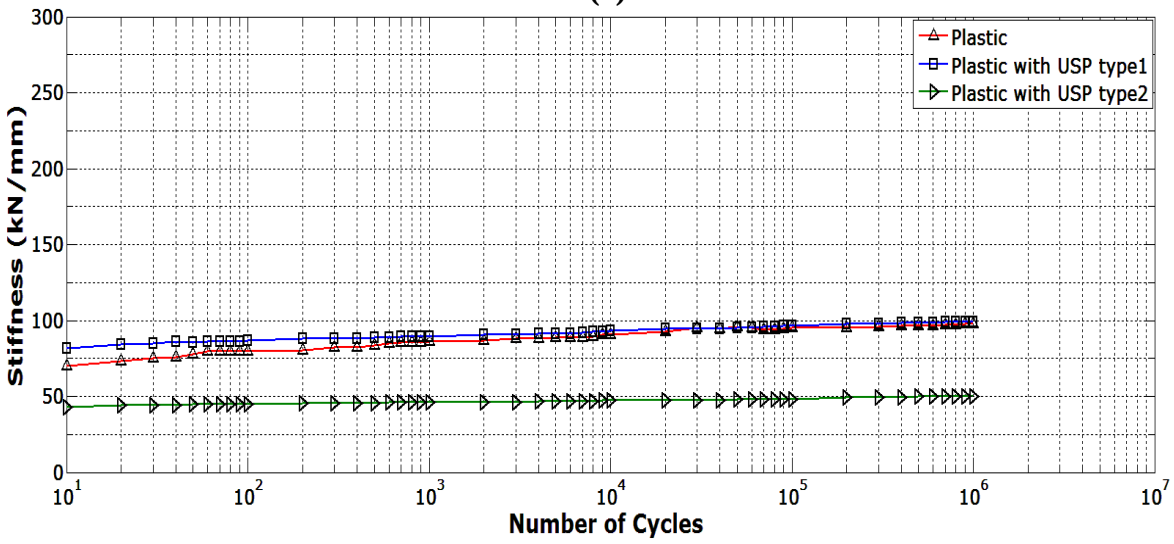
Plastic Sleeper

For the plastic sleeper (Figure 6-24 (a) and (b)), the inclusion of USPs (regardless of type) reduced the sleeper settlement significantly. However, in contrast to the performance of the rest of sleepers with USPs, use of USP type1 improved the settlement slightly more than USP type2. This possibly shows that the optimal stiffness range of USP inclusion for plastic sleepers was closer to the range of the USP type1 stiffness. The significant improvement in settlement performance of the plastic sleeper in the case USP inclusion (regardless of type) compared to the 'no USP' case can be logically explained by the great difference in coefficient of friction between the plastic sleeper and the USPs at the ballast-sleeper interface (coefficient of friction for USPs are approximately twice that of the plastic sleeper (Table 4-4)).

With regard to the maintenance work reduction related to settlement, as can be seen in Figure 6-24 (a), the settlement result for USP type1 and type2 after approximately 3k and 4k cycles (respectively) was equal to that of the plastic sleeper without USP after 1 million cycles. Furthermore, greater trackbed resiliency was also observed at the end of 1 million cycles when USPs (regardless of type) were used than without USP. Therefore in the case of the plastic sleeper, inclusion of USP type1 is strongly recommended, considering the comparable trackbed stiffnesses in the case of 'no USP' and USP type1 cases (Figure 6-24 (b)).



(a)



(b)

Figure 6-43 Effect of USP inclusion plastic sleeper performance (a) settlement (b) trackbed stiffness

The deflections of the plastic sleeper (Figure 6-44) showed hogging at the middle of the sleeper and confirmed the low bending stiffness of the sleeper compared to the other sleeper types (Table 6-4), especially in the case of 'no USP'. However, the deflections of the sleeper were reduced by introduction of USPs. As far as pressure at subgrade is concerned for the plastic sleeper (Figure 6-45), lower pressure at the rail seat of the sleeper was observed compared to concrete mono-block and twin block (which had high bending stiffness). The low bending stiffness and consequent high deflections of the plastic sleeper possibly led to high load damping effects that reduced the transmitted pressure to the subgrade compared to the mono-block and twin-block. Pressure at the middle of the plastic sleeper in the case of 'no USP' started increasing from the beginning of the test. This was possibly as a result early migration of ballast at the rail seat and large sleeper deflections from the beginning of the test which

increased the contribution of the middle of the sleeper in carrying the load, and consequently increased the load transmitted to the subgrade at the middle of the sleeper. This did not happen in the early stages of the test when USPs used (regardless of type). The higher coefficient of friction at the ballast-sleeper interface in the case of USPs compared to the plastic sleeper without USP can be the possible reason (Table 4-4). According to Lampo et al. (2003), plastic sleepers are too smooth (or friction resistant) for any noticeable mechanical locking into ballast to occur, which is consistent with the low measured coefficient of friction for the plastic sleeper in section 4.6.

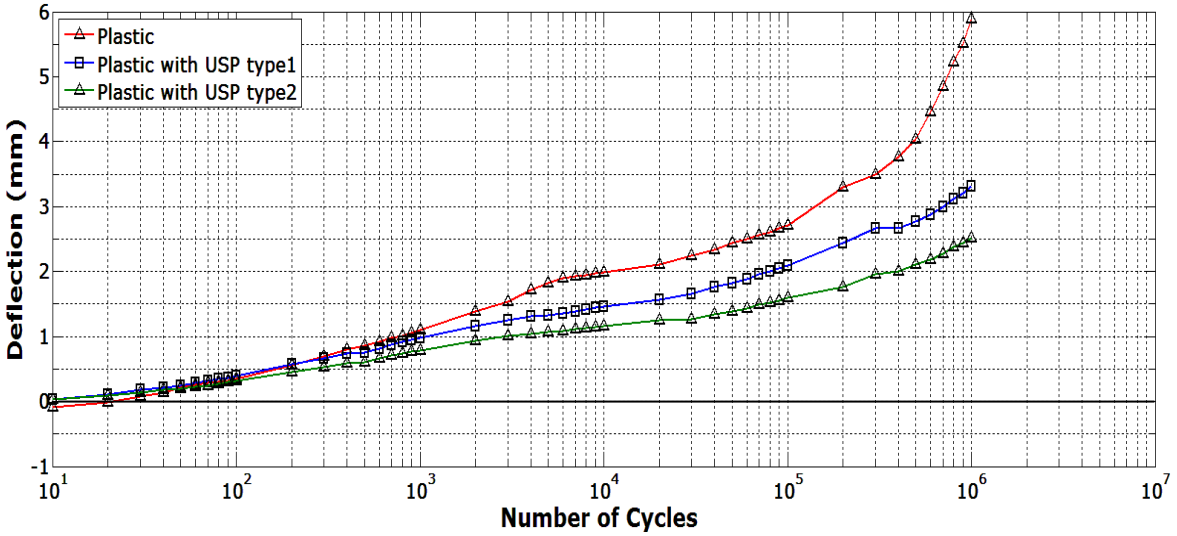


Figure 6-44 Effect of USP inclusion on plastic sleeper deflection performance

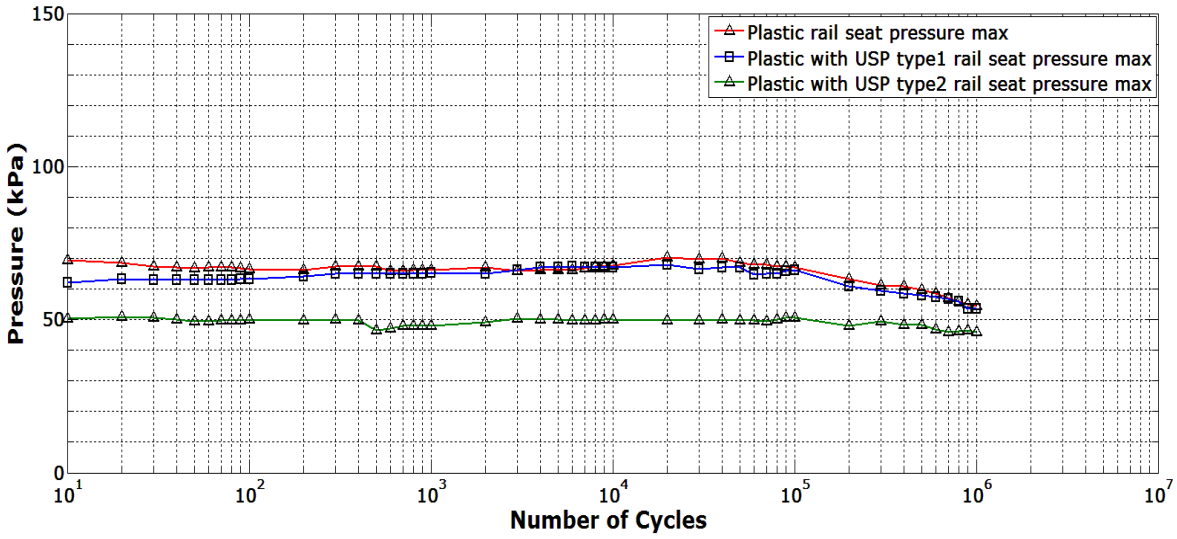
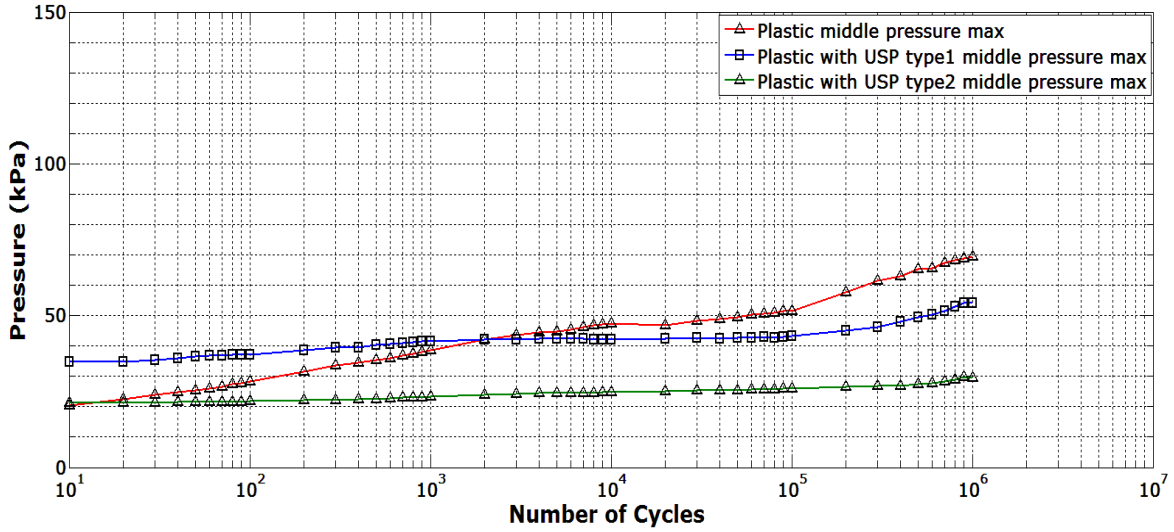


Figure 6-45 (a) Effect of USP inclusion on subgrade pressure for plastic sleeper rail seat



(b)

Figure 6-45 Effect of USP inclusion on subgrade pressure for plastic sleeper (a) rail seat (b) middle of sleeper

Timber Sleeper

Data on inclusion of USPs in the case of timber sleepers is presented in Figure 6-46. As can be seen in Figure 6-46 (a), the settlement results showed less settlement, higher resiliency, and significant possibility of reduction in maintenance works when USPs were used (irrespective of type). Settlement of the timber sleeper with USP type1 and USP type2 at 1 million cycles was comparable with timber without USP at approximately 15k and 6k respectively.

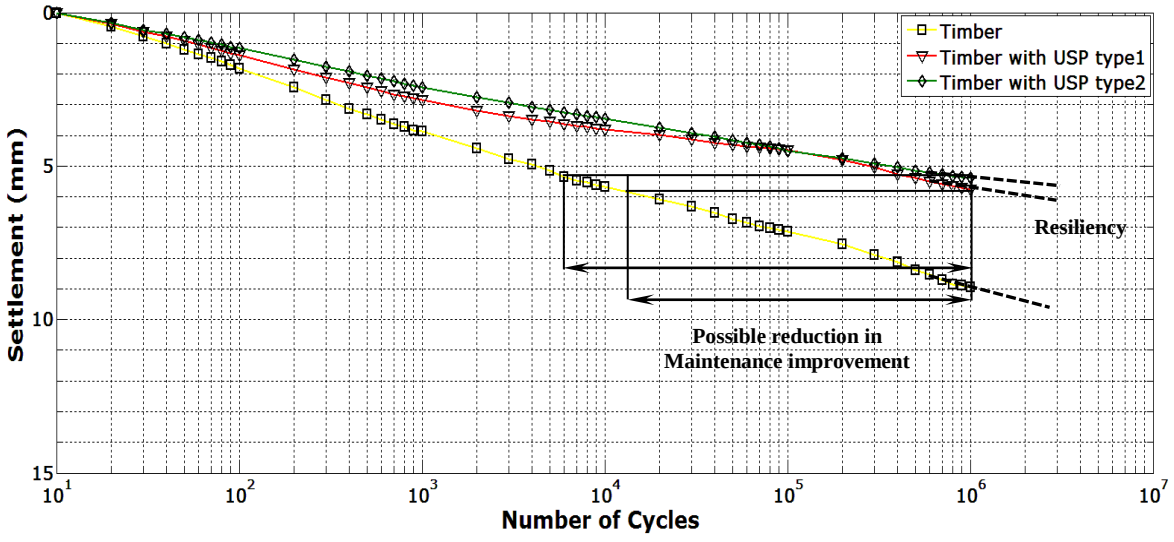
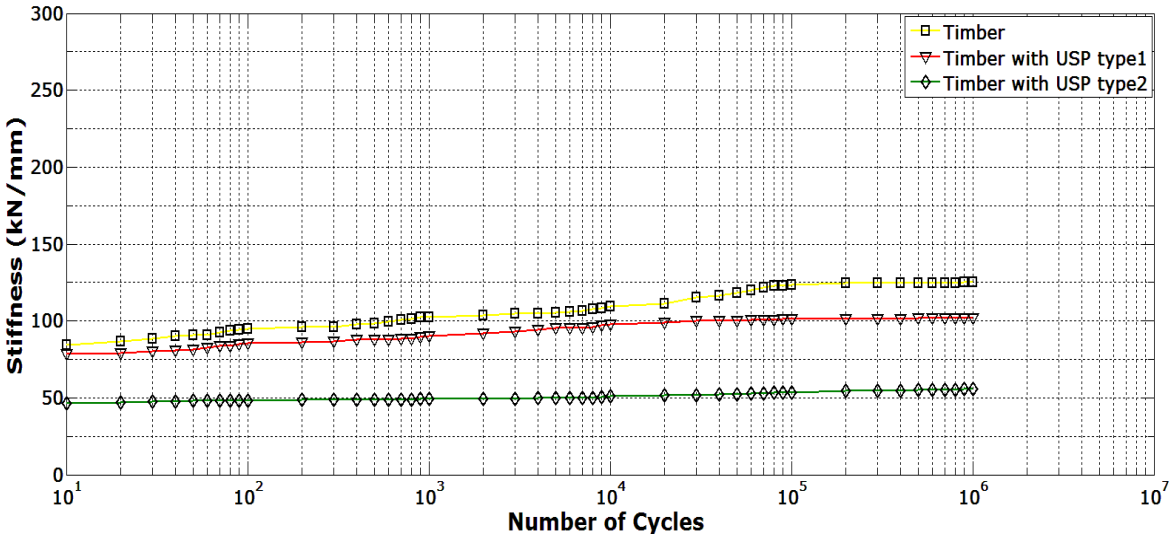


Figure 6-46 (a) Effect of USP inclusion timber sleeper performance settlement



(b)

Figure 6-46 (b) Effect of USP inclusion timber sleeper trackbed stiffness

Deflections for the timber sleeper are presented in Figure 6-47. The hogging at the middle of the sleeper in the case of 'no USP' was larger compared to the concrete mono-block and lower compared to the plastic sleeper, which was consistent with the bending stiffness measured for the various sleeper types which timber sleeper had greater lower bending stiffness compared to concrete mono-block and greater bending stiffness compared to plastic sleeper (Table 6-4). Similar to the other sleeper types, inclusion of USPs (regardless of type) reduced the sleeper deflections (hogging) at the middle of sleeper; however the reduction was larger in the case of USP type2.

With regard to the subgrade pressure for the timber sleeper (Figure 6-48), in the case of 'no USP' pressure reduction at the rail seat and gain at the middle of the sleeper (due to ballast migration at sleeper ends) began approximately after 100k, similar to the case of 'no USP' when concrete mono-mono block was used. However, in the case of concrete mono-block the magnitude of pressure at the rail seat was higher compared to the timber sleeper. The logical reason can be the greater width of the timber sleeper which reduced the stress at the ballast-sleeper interface and consequently transmitted less pressure to the subgrade. Additionally, the sleeper load damping effect in the case of timber sleepers can be higher due to the lower bending stiffness and the higher sleeper resiliency (Figure 6-34) of timber sleepers compared to concrete mono-block and consequently the transmitted pressure to the subgrade at the rail seat was lower. Similarly inclusion of USPs increased the load damping effect, therefore giving less pressure

on the subgrade at both the rail seat and the middle of the sleeper compared to 'no USP' (Figure 6-48).

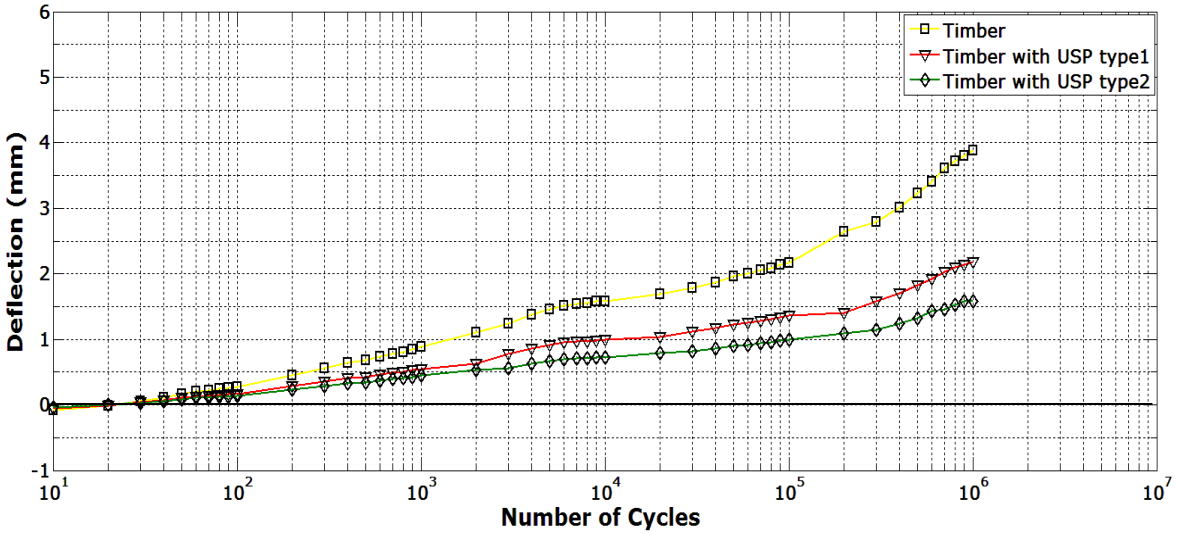
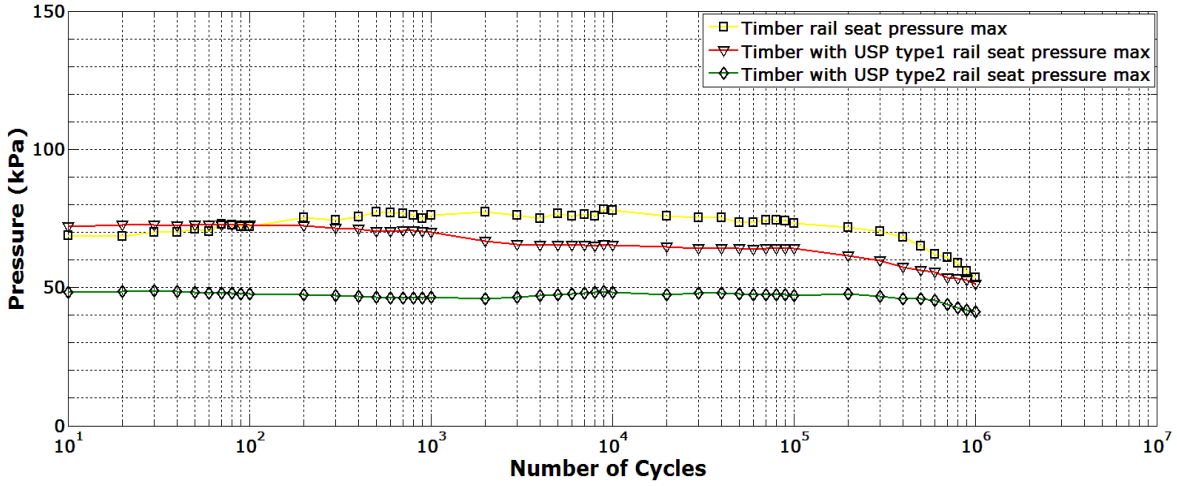
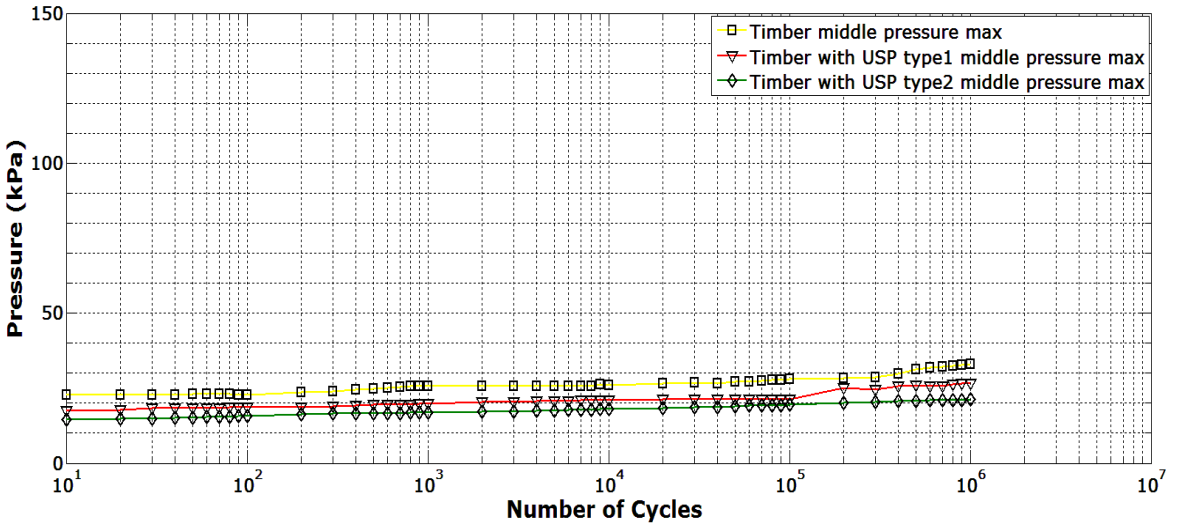


Figure 6-47 Effect of USP inclusion on timber sleeper deflection performance



(a)



(b)

Figure 6-48 Effect of USP inclusion on subgrade pressure for timber sleeper (a) rail seat (b) middle of sleeper

6.10.4 EFFECT OF SLEEPER TYPE

In Chapter 4 (box test) and Chapter 5 (CET), the effects of friction coefficient at the ballast-sleeper interface and sleeper geometry on sleeper settlement for different sleeper section types were discussed. In this section, in order to better understand performance of different real size sleeper types, in addition to the aforementioned factors, bending stiffness and deflections of sleeper were also measured.

The steel sleeper results are presented separately due to the fact that it was not possible to affix a USP to the base of the sleeper and different methods of installation were used for it compared to the rest of sleepers.

Evolution of sleeper type permanent settlement and trackbed stiffness for concrete mono-block, concrete twin-block, plastic and timber sleepers over a million of cycles is presented in Figure 6-49. With regard to settlement, the order of results for the sleepers was consistent with those of the box and CET.

Out of all sleeper types, the concrete mono-block sleeper gave the least settlement (6.2-mm) and the concrete twin-block sleeper gave 19%, timber sleeper 44%, and plastic sleeper 88% more settlement than the concrete mono-block. The sleepers were different in geometry, weight, coefficient of friction at ballast-sleeper interface, and sleeper bending stiffness.

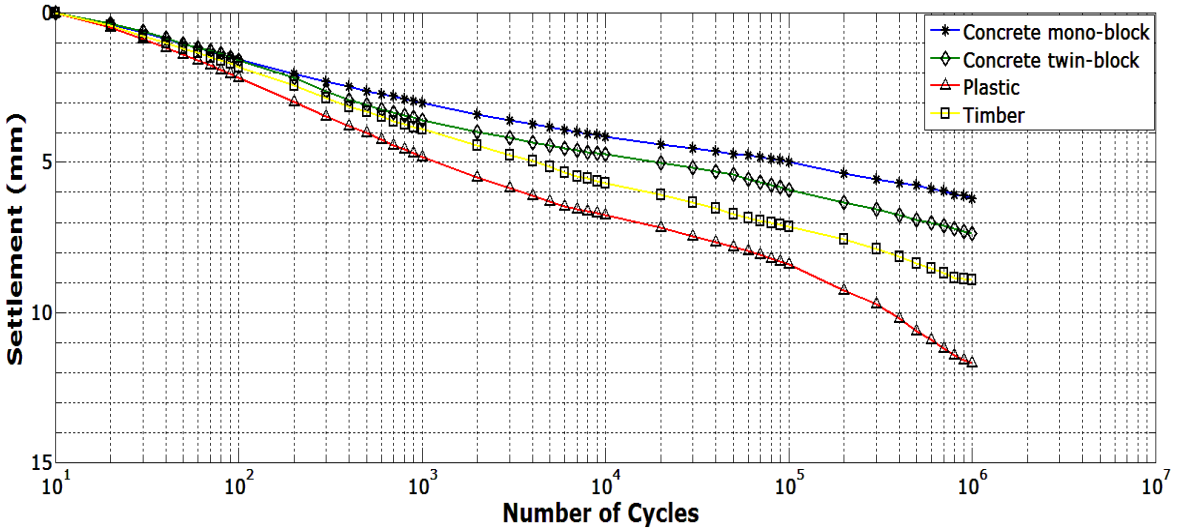
With respect to geometry, in general wider and deeper sleepers settled less compared to narrower and shallower sleepers. This can be explained due to the fact that a wider sleeper spreads load over a wider area compared to a narrower sleeper and therefore reduces the stress at the ballast-sleeper interface and the settlement of the sleeper. A deeper sleeper compared to a shallower sleeper provides higher friction with ballast particles on the vertical faces of the sleeper, which reduces the settlement of the sleeper. As shown in Table 6-2 , the concrete twin-block is slightly wider and deeper than the concrete mono-block sleeper; however, it settled slightly more (1.2-mm) than the concrete mono-block. This can logically be ascribed to the rod in the middle of the sleeper which reduced the soffit area of the twin-block compared to the mono-block and consequently increased the stress at the ballast-sleeper interface and settlement of the twin-block sleeper. The timber sleeper is 5-mm wider than the concrete mono-block; however, it is 30-mm shallower than the concrete mono-block and it therefore settled more than the concrete mono-block. Of all the sleeper types, the plastic sleeper was the narrowest and shallowest and therefore settled the most.

With regard to weight of sleeper, considering the maximum applied load of 97kN, the effect of the weight of each sleeper on settlement should be insignificant. However, it may influence the positional stability of the sleeper.

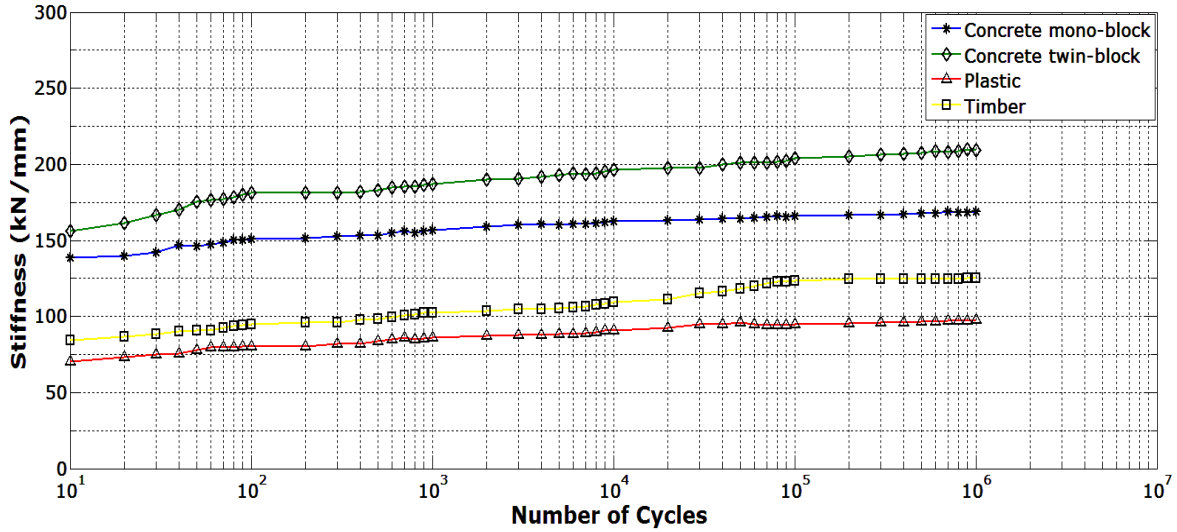
Concerning the coefficient of friction of each sleeper at the ballast-sleeper interface (table 4-4), the results showed that a higher coefficient of friction resulted in lower sleeper settlement, which was consistent with results from the box and CET tests. The logical reason for this is that a sleeper with a higher coefficient of friction should develop a better interlock with the ballast which can limit the sleeper displacement both horizontally and vertically. Therefore the concrete sleepers (regardless of type), which have higher coefficient of friction compared to the plastic and timber sleepers, settled less; the same applies between timber and plastic sleeper.

When bending stiffness of each sleeper is considered, it is observed that sleepers with a higher bending stiffness settled less. This may be due to the fact that a sleeper with lower bending stiffness would logically have higher sleeper deflection (Figure 6-50), which can affect (increase) ballast movement underneath a sleeper.

Trackbed stiffness for different sleeper types is presented in Figure 6-49 (b). Considering the same preparation method was used for each test, identical initial density of trackbed was assumed for all the test arrangements studied. Observation of trackbed stiffness for all sleeper types showed a general trend of initial increase in stiffness followed by an almost constant stiffness towards the end of tests. The initial increase in stiffness can be related to the particle rearrangement at the beginning of the test and the constant stiffness towards the end of test can be due to the approach of the trackbed to a state of pure resilience. The initial increase of stiffness in the case concrete twin-block was more evident compared to the other sleeper types. This may logically be due to the smaller soffit area of the twin-block, which increased the stress at the ballast-sleeper interface and consequently the pace of ballast rearrangement at the beginning of the test. Apart from the concrete twin-block, observations showed a higher average trackbed stiffness over a million cycles for sleepers with a larger bending stiffness compared to those with lower stiffness. This can be explained by the fact that a sleeper with a larger bending stiffness would give less sleeper deflection and consequently less ballast movement underneath sleeper.



(a)



(b)

Figure 6-49 (a) Evolution of sleeper types permanent settlement for sleepers without USPs (b) Evolution of trackbed stiffness for sleepers without USPs

The sleeper deflections for each sleeper for all sleeper types are presented in Figure 6-50. As can be seen in the figure all sleepers showed hogging at the middle of the sleeper, however, the largest effect was in the case of the plastic sleeper and the least in the case of the concrete mono-block; this was consistent with the lowest bending stiffness being that of the plastic sleeper and the largest bending stiffness that of the mono-block. A sleeper with higher bending stiffness (e.g. mono-block) compared to a sleeper with lower bending stiffness (e.g. plastic) gave lower tendency for hogging and eventually centre binding and therefore higher subgrade pressure at the rail seat (Figure 6-51 (a)).

The high sleeper hogging at the middle of sleeper for the plastic sleeper towards the end of the test was an indication of the sleeper being supported by ballast mainly at the middle of the sleeper and therefore increased the subgrade pressure at the middle of the sleeper (Figure 6-51 (b)).

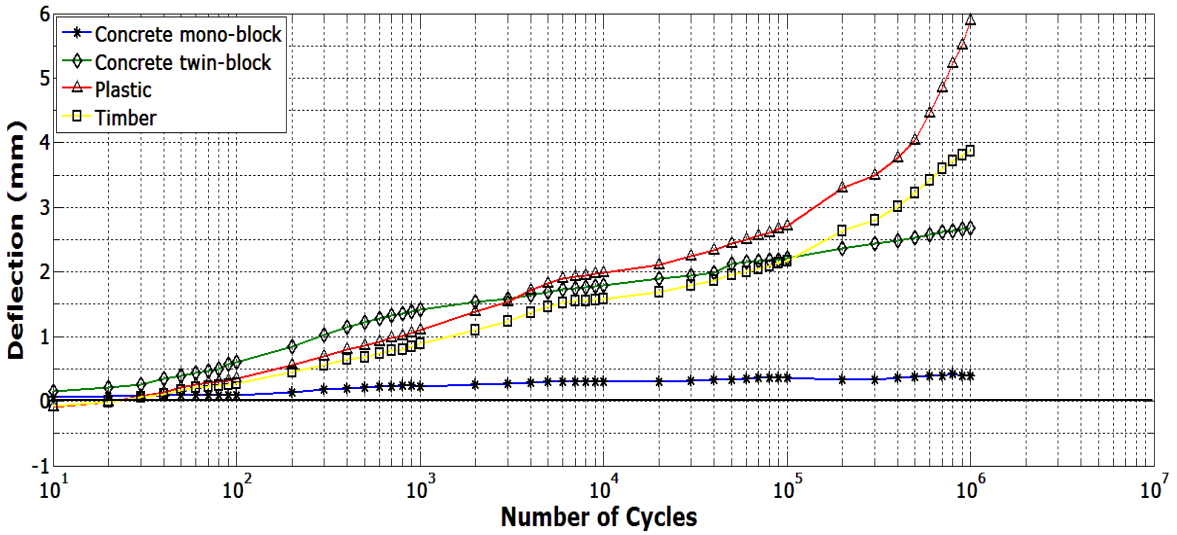


Figure 6-50 Evolution of sleeper deflections of sleeper types for sleepers without USPs

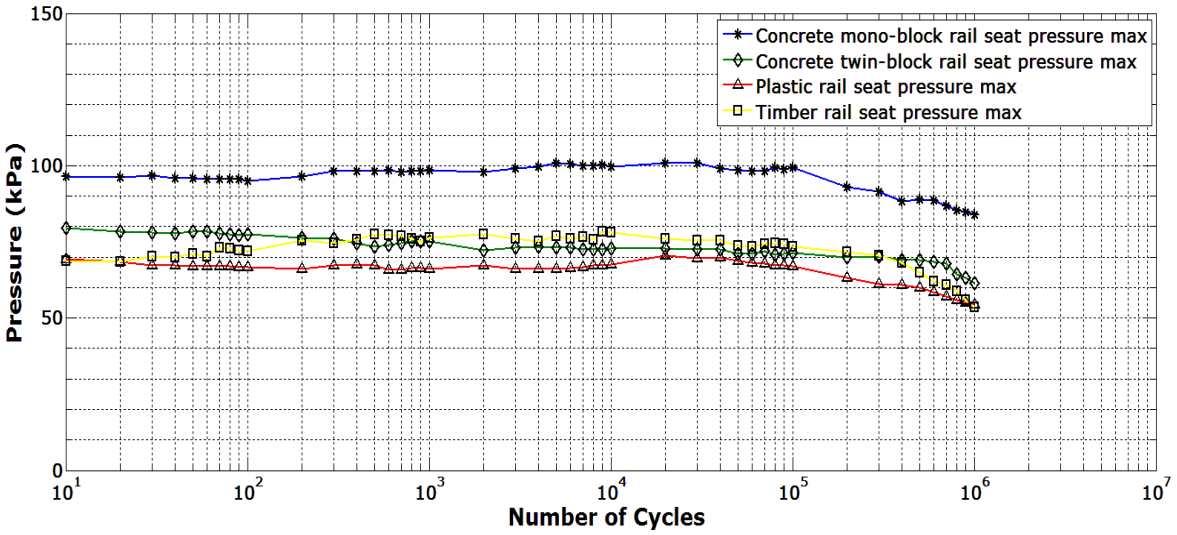
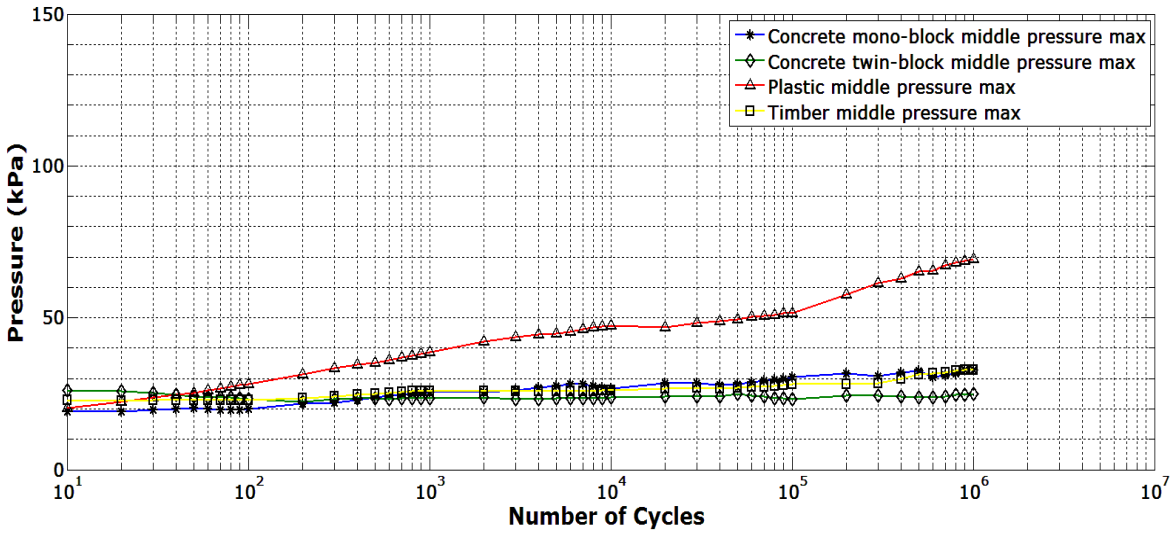


Figure 6-51 (a) Evolution of subgrade pressure for sleepers without USPs rail seat



(b)

Figure 6-51 Evolution of subgrade pressure for sleepers without USPs (a) rail seat (b) middle of sleeper

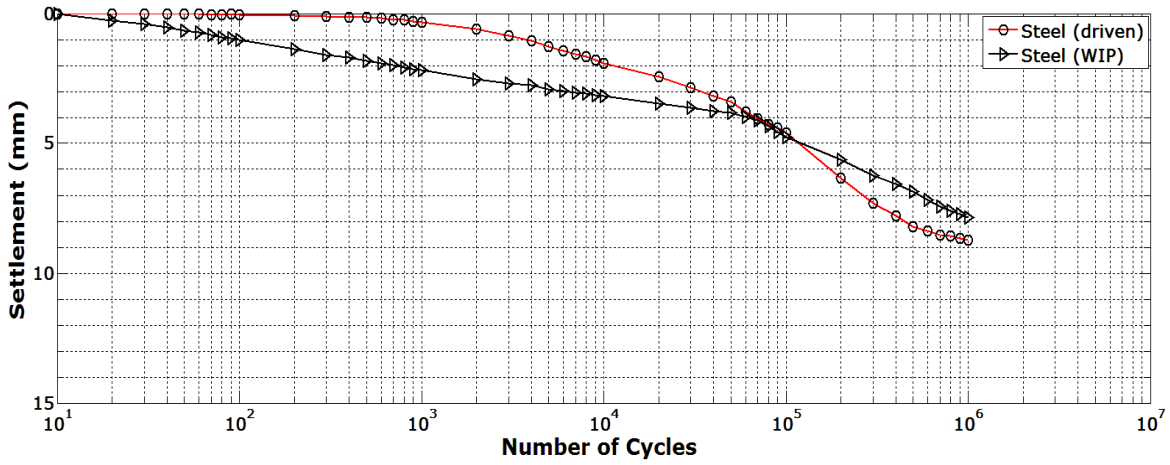
6.10.5 STEEL SLEEPER (DRIVEN VERSUS WISHED-IN-PLACE)

As explained in section 6.8, two installation methods were used in the case of the steel sleeper, namely the driven method and Wished-In-Place (WIP). The sleeper settlement and trackbed stiffness for the aforementioned methods are presented in Figure 6-52. As may be seen from the settlement results in Figure 6-52 (a), up to 100k cycles the driven steel sleeper performed better compared to the WIP steel sleeper. However, over the full duration of 1 million cycles, permanent deformation of the ballast resulted in the WIP sleeper outperforming the driven sleeper. A possible reason is the initial static load applied on the driven sleeper used to push the sleeper into the ballast. The applied static load possibly gave the ballast underneath the sleeper more initial compaction compared to the WIP case and, hence, less settlement at the beginning of the test; however, it also possibly broke or caused cracks in the ballast particles underneath the sleeper which in the long term (higher number of cycles) made the ballast weaker and consequently caused higher settlement in the case of driven steel sleepers compared to the WIP sleepers.

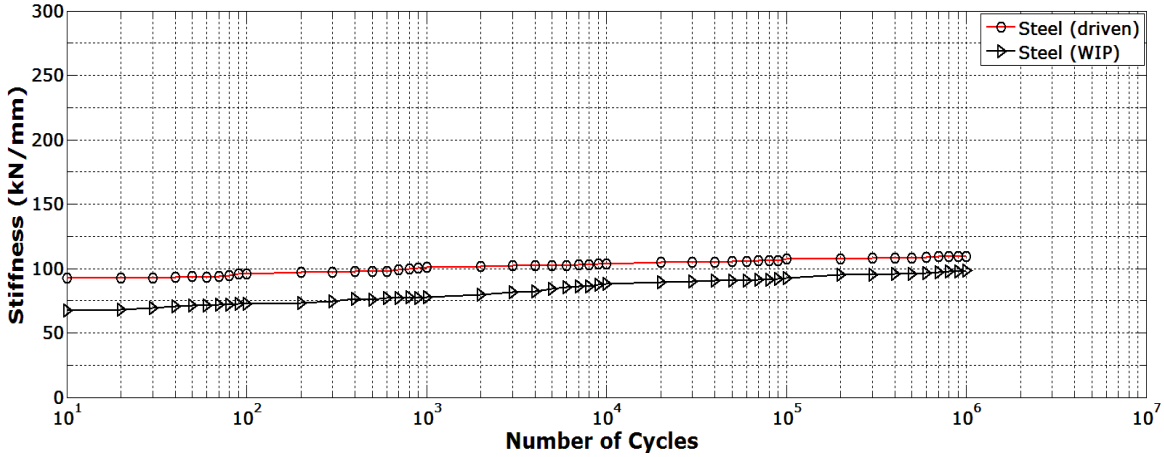
For the first 30k cycles or so, the settlement observed for driven steel was in agreement with the box test results, which showed the driven U-section (representing a steel sleeper) settling less than the concrete sleeper.

With regard to the trackbed stiffness (Figure 6-52 (b), as can be seen at the beginning of the test the driven steel sleeper had slightly higher stiffness compared to the WIP steel. However, as

the test went on the trackbed stiffness in both driven and WIP became comparable. This may logically be due to the initial applied static load in the case of the driven steel sleeper.



(a)



(b)

Figure 6-52 Driven steel sleeper versus WIP steel sleeper
(a) settlement (b) trackbed stiffnes

Sleeper deflections for driven and WIP steel sleepers are presented in Figure 6-53. As can be seen in the figure, hogging at the middle of the sleeper was observed in the case of the WIP sleeper and in contrast sagging in the case of the driven sleeper. This may logically be due to the different installation methods used for the steel sleeper which gave better support at the middle of the sleeper in the case of WIP. The initial static applied load on rail seats in the case of the driven sleeper was possibly less effective at the middle of the sleeper in pushing the sleeper into the ballast bed and this therefore caused gaps (voids) underneath the sleeper. This is consistent with higher subgrade pressure under the driven sleeper at the rail seat compared to the WIP sleeper and higher subgrade pressure of the WIP at the middle of the sleeper compared to the driven sleeper (Figure 6-54).

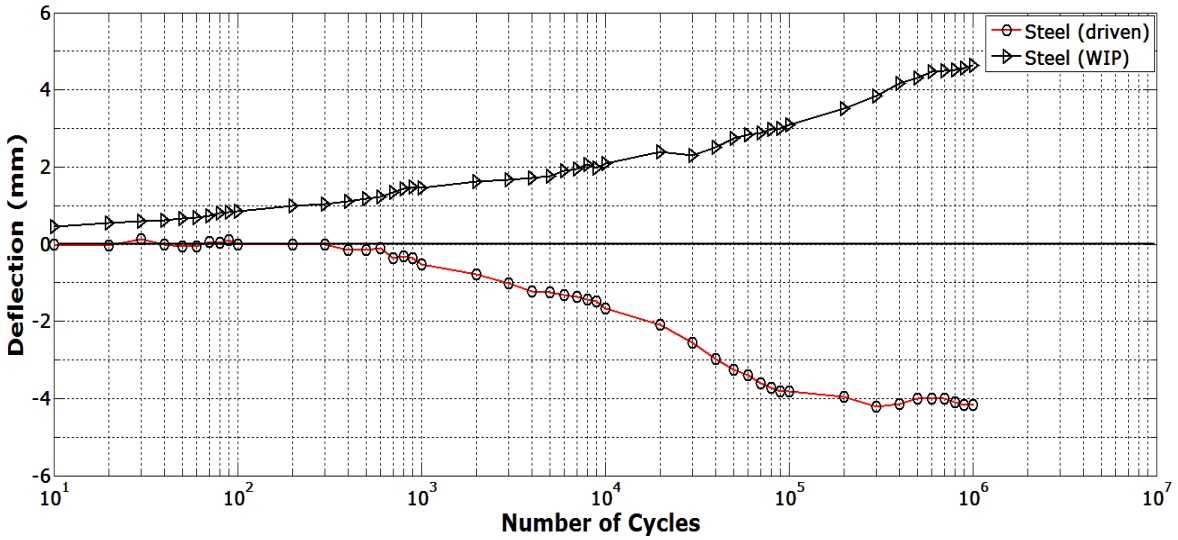
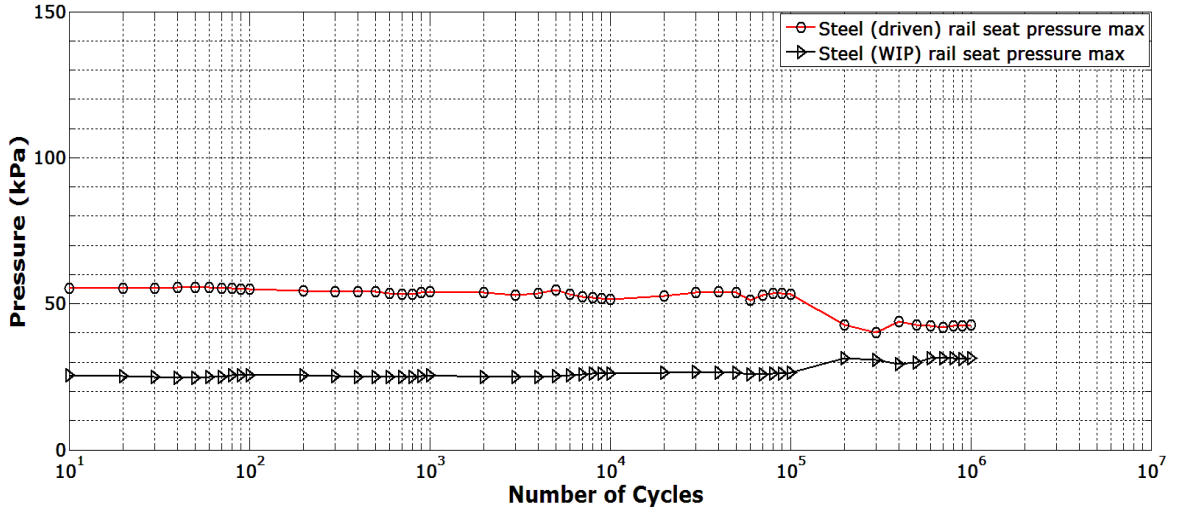
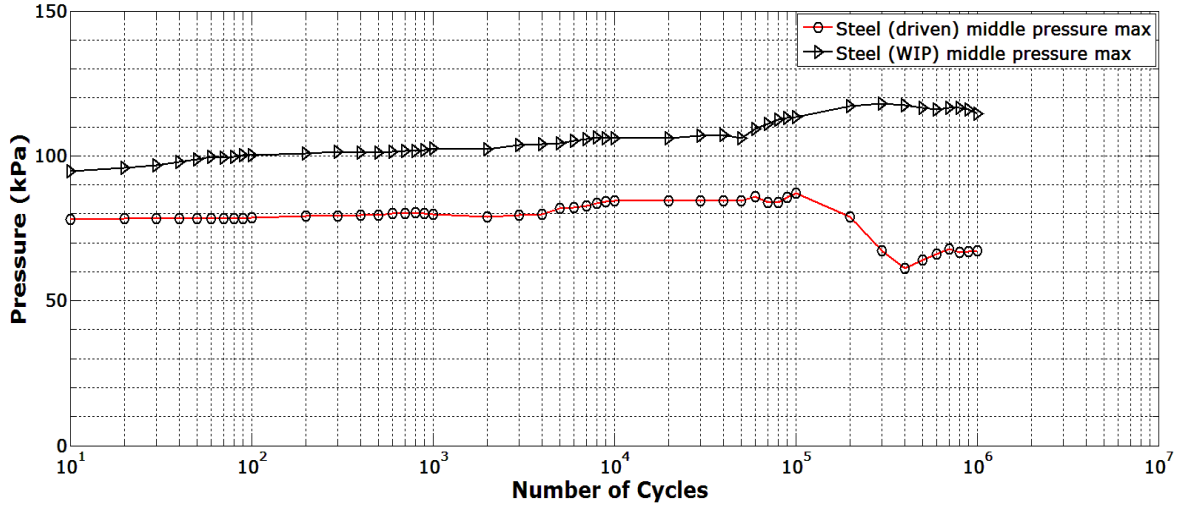


Figure 6-53 Sleeper deflections for driven steel sleeper versus WIP steel sleeper



(a)



(b)

Figure 6-54 Subgrade pressure for driven steel sleeper versus WIP steel sleeper (a) rail seat (b) middle of sleeper

6.10.6 MIDDLE VERSUS OUTER SLEEPER

Settlement (at a million cycles) and trackbed stiffness (averages over a million cycles) for the middle sleeper (sleeper 2) and outer sleepers (sleepers 1 and 3) are presented in Table 6-5. As expected and discussed in section 6.4 the outer sleeper settled more than the middle sleeper, which was consistent with all previous studies using the RTF. The outer sleepers provided realistic boundary conditions for the middle sleeper and therefore the middle sleeper was the closest representative of a real life railway sleeper.

The outer sleepers gave comparable results with respect to settlement and trackbed stiffness. However, sleeper 1 generally gave slightly higher settlement and lower stiffness compared to sleeper 3. This may logically be due to the direction of travel, that sleeper 1 was loaded first and the sleeper 3 last; this made the initial compaction level for the ballast underneath slightly higher in the case of sleeper 3 compared to sleeper 1 and consequently resulted in slightly lower settlement and higher trackbed stiffness in the case of sleeper 3.

As can be seen in the table the difference between the settlement of the outer sleepers and the middle sleeper in the case of plastic sleepers was the lowest compared to the other sleeper types. This may possibly be due to the fact that the plastic sleeper was the narrowest and the plastic outer sleeper had the greatest distance from the pit wall and the RTF frame, and consequently received least influence from the boundary conditions.

Observation also showed that inclusion of USPs (regardless of type) gave closer results in settlement and trackbed stiffness for the middle and outer sleepers. This may possibly be due to more uniform contact between ballast and sleeper at the ballast sleeper interface for all three sleepers in the case of USP inclusion compared to the ‘no USP scenario’.

Table 6-5 Settlement (at a million cycle) and trackbed stiffness (averages over a million cycles) for middle sleeper and outre sleepers

Test	Permanent settlement (mm)			Trackbed stiffness (kN/mm)		
	Sleeper 1	Sleeper 2	Sleeper 3	Sleeper 1	Sleeper 2	Sleeper 3
Concrete mono-block	12.5	6.2	11.9	126.9	158.5	131.5
Concrete mono-block + USP type1	7.5	5.2	7.0	89.3	110.4	93.6
Concrete mono-block + USP type2	6.6	4.7	6.3	51.8	60.2	53.1
Concrete twin-block	13.8	7.4	13.0	165.8	191.2	160.9
Concrete twin-block + USP type1	7.1	5.5	6.5	127.5	148.2	134.2
Concrete twin-block + USP type2	6.4	5.3	6.0	49.7	62.8	54.2
Plastic	13.3	11.7	13.0	71.2	88.0	68.8
Plastic + USP type1	7.1	5.9	6.8	61.3	91.8	70.9
Plastic + USP type2	7.7	6.2	7.1	40.9	46.9	43.0
Timber	12.3	8.9	11.9	80.7	107.6	84.9
Timber + USP type1	8.4	5.7	7.9	67.1	80.3	74.7
Timber + USP type2	7.3	5.4	6.8	40.1	50.6	42.4
Steel (driven)	15.8	8.7	15.3	91.02	101.7	94.7
Steel (WIP)	14.10	7.8	13.04	68	83.6	75.1

6.10.7 EFFECT OF USP TYPE1

Consistent with the results using the box and CET apparatus, inclusion of USP type1 using the RTF generally showed less variability in settlement for different sleeper types (Figure 6-55 (a)). This may logically be due to the fact that inclusion of USP type1 gave comparable coefficient of friction at the ballast-sleeper interface for all sleeper types.

With regard to the trackbed stiffness, while the inclusion of USP type1 gave lower stiffnesses, the stiffness graph also showed smoother behaviour through the loading cycles compared to 'no USP' (Figure 6-55 (b)). Possibly, the attachment of USP type1 maintained a more uniform stiffness with loading cycles by creating a more homogenous interface with the ballast, consequently minimising the potential for settlement.

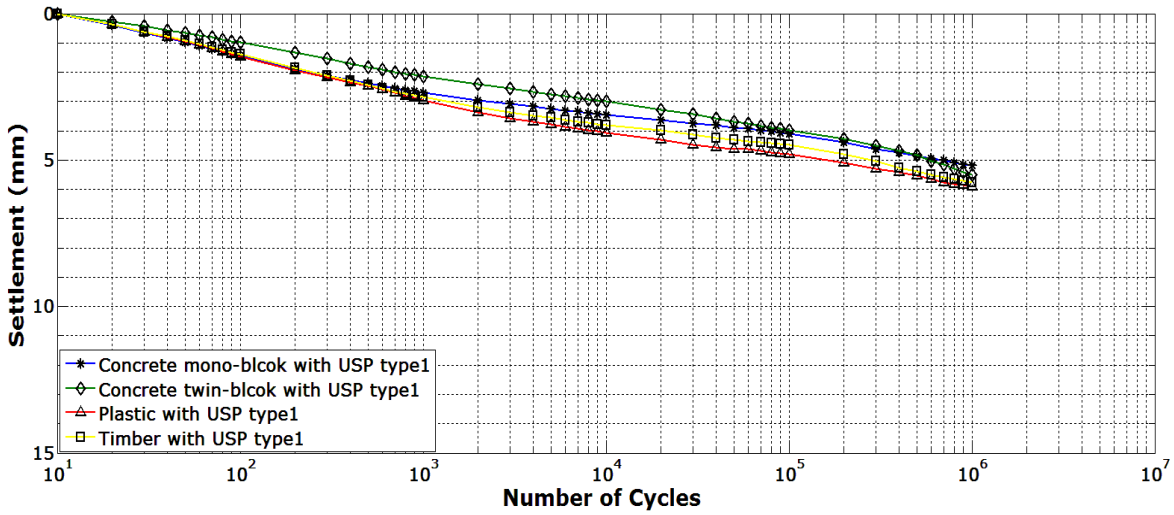
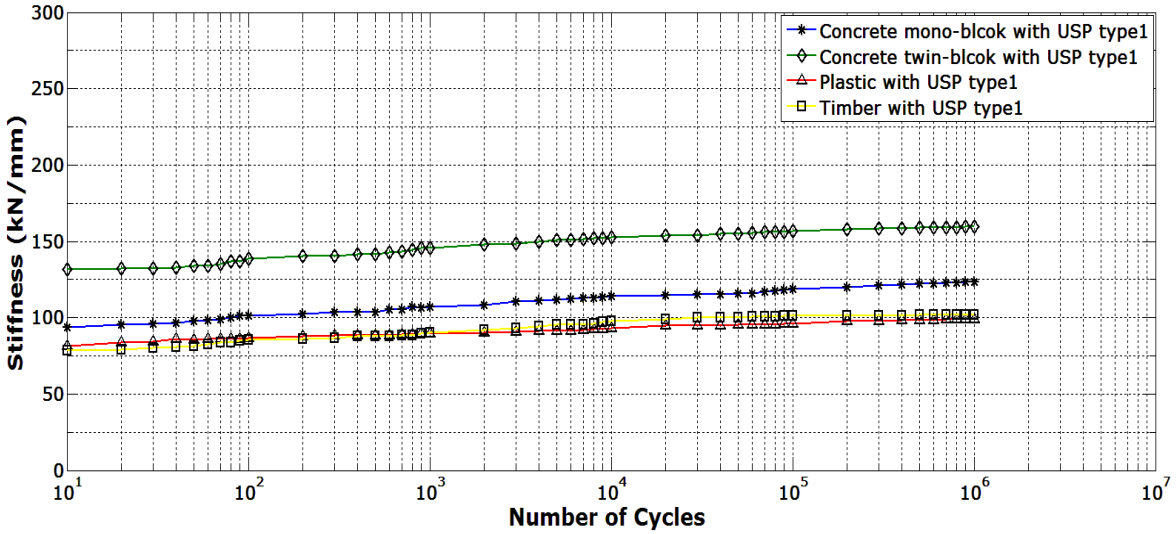


Figure 6-55 (a) Evolution of sleeper types permanet settlement for sleepers with USP type1



(b)

Figure 6-55 (a) Evolution of sleeper types permanet settlement for sleepers with USP type1 (b) Evolution of trackbed stiffness for sleepers with USP type1

With regard to sleeper deflections for sleepers with USP type1 (Figure 6-56), results showed that the magnitudes of hogging at the middle of the sleeper were reduced for all sleepers when USP type1 was used compared to the 'no USP' scenario.

The reduction of hogging was 58% for concrete mono-block, 55% for concrete twin-block, 30% for the plastic sleeper, and 43% for the timber sleeper, which indicates that the hogging at the middle sleeper was reduced more for the sleeper with higher bending stiffness compared to those with lower stiffness when USP type1 was used. This may be due to the fact that a sleeper with higher bending stiffness conforms less to the ballast surface at the ballast-sleeper interface compared to a sleeper with lower bending stiffness in the case of 'no USP'. Therefore inclusion of USP can reduce the sleeper hogging at the middle of the sleeper more for a sleeper with a higher bending stiffness compared to lower one.

With regard to subgrade pressure, in the case of USP type1 (Figure 6-57), inclusion of USP type1 generally reduced the subgrade pressure at both rail seat and middle of the sleeper for all sleepers tests compared to the 'no USP' scenario. However, the average percentage reduction at the middle of the sleeper was more than the reduction at the rail seat for all sleeper tests. This can be explained by the fact that inclusion of USP type1 reduced the hogging at the middle of the sleeper.

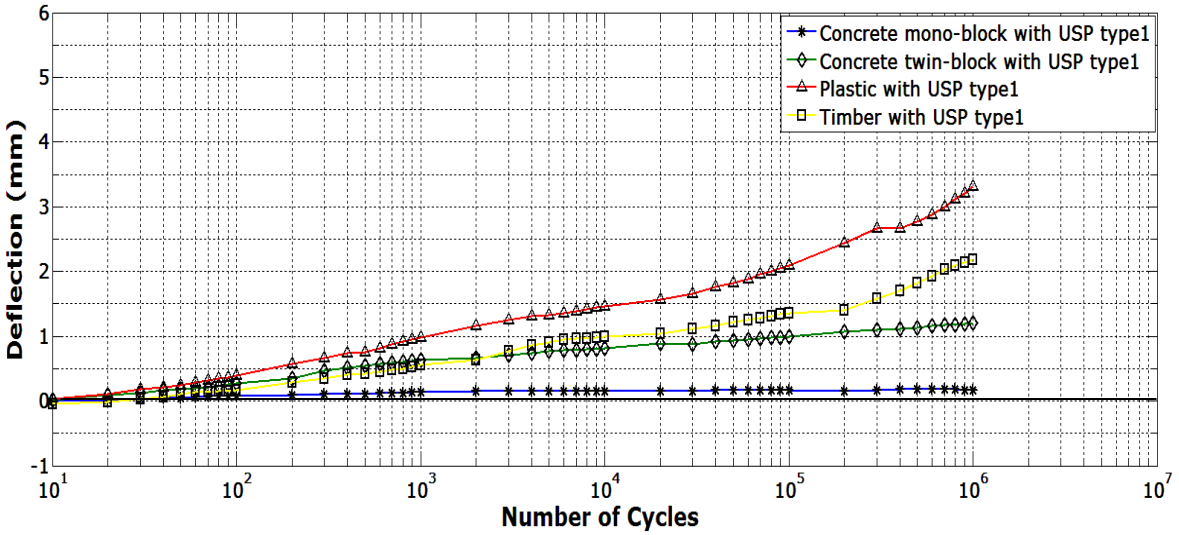
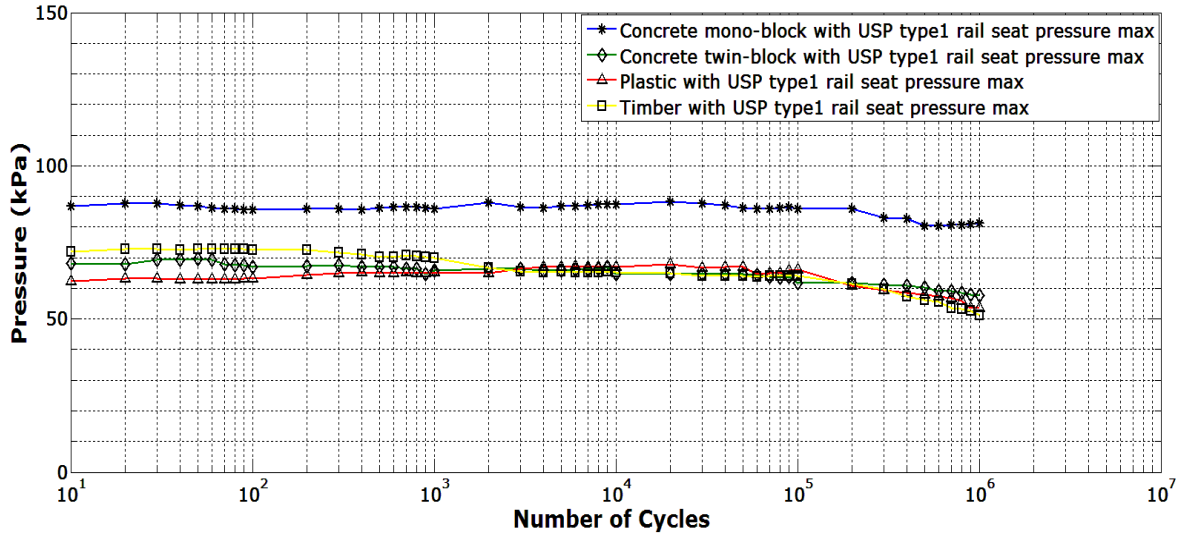
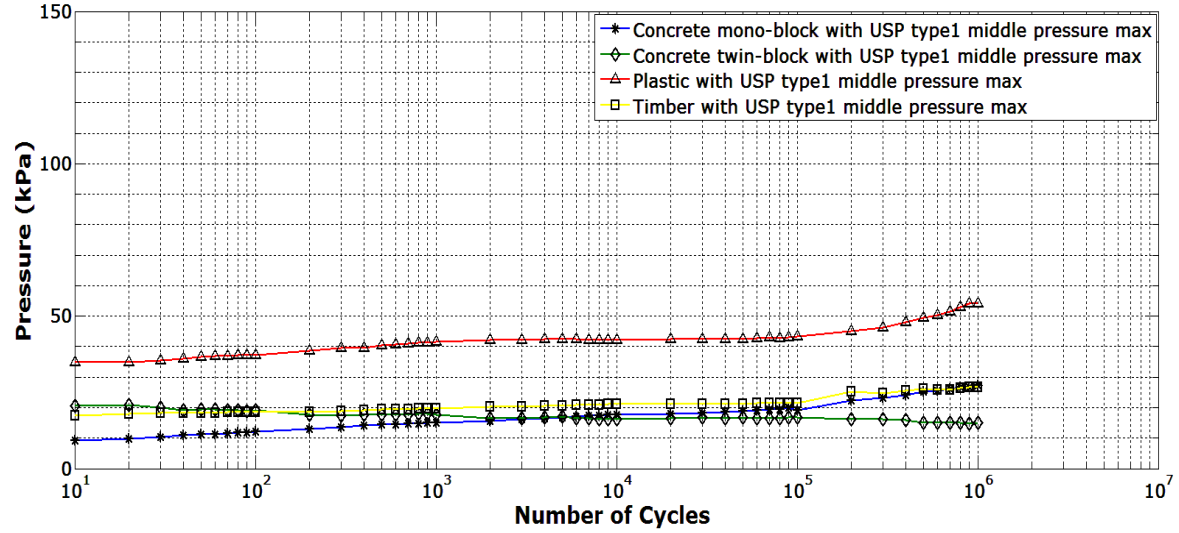


Figure 6-56 Evolution of sleeper deflections of sleeper types for sleepers with USP type1



(a)



(b)

Figure 6-57 Evolution of subgrade pressure for sleepers with USP type1 (a) rail seat (b) middle of sleeper

6.10.8 EFFECT OF USP TYPE2

When USP type2 was used (Figure 6-58 (a)), similar to inclusion of USP type1, a small variability in settlement for different sleeper types was observed, which was consistent with results using the box and CET apparatus. Similarly, this can be due to the fact that inclusion of USP type2 gave comparable coefficients of friction at the ballast-sleeper interface for all sleeper types.

With regard to trackbed stiffness (Figure 6-58 (a)), USP type2 led to a reduction in variability; the difference in average trackbed stiffness over a million cycles between the largest case (concrete twin-block sleeper) and lowest case (plastic sleeper) was approximately 15 kN/mm. This was approximately 56 kN/mm when USP type1 was used and 103 kN/mm in the case of 'no USP'. The attachment of USP type2 increased the contact area between sleeper base and ballast at the ballast-sleeper interface compared to when USP type1 was used (Figure 4-12) and therefore maintained a more uniform stiffness with loading cycles by creating a more homogenous interface with the ballast.

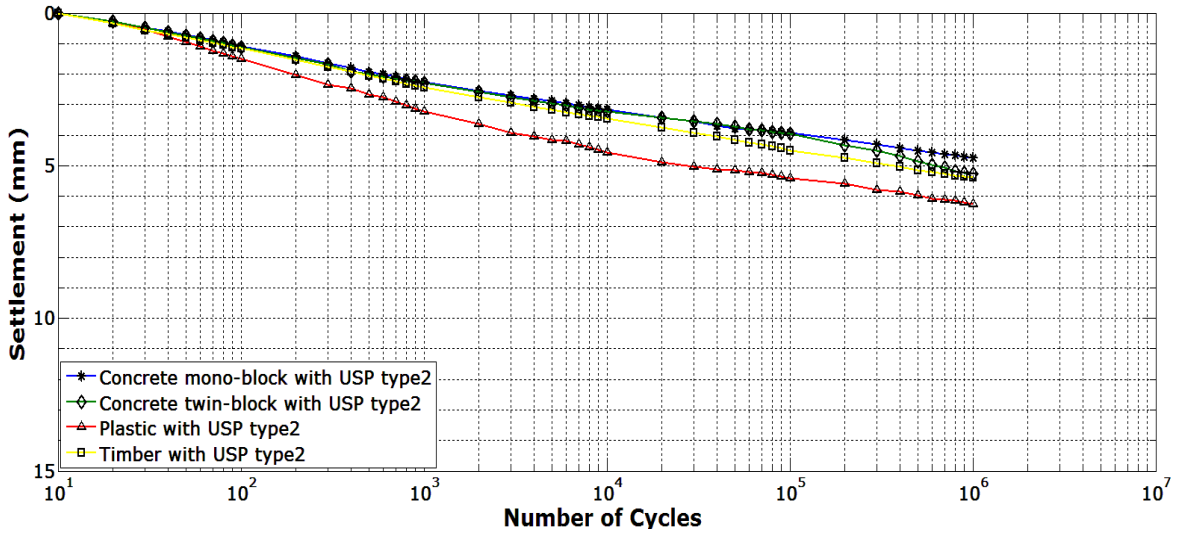
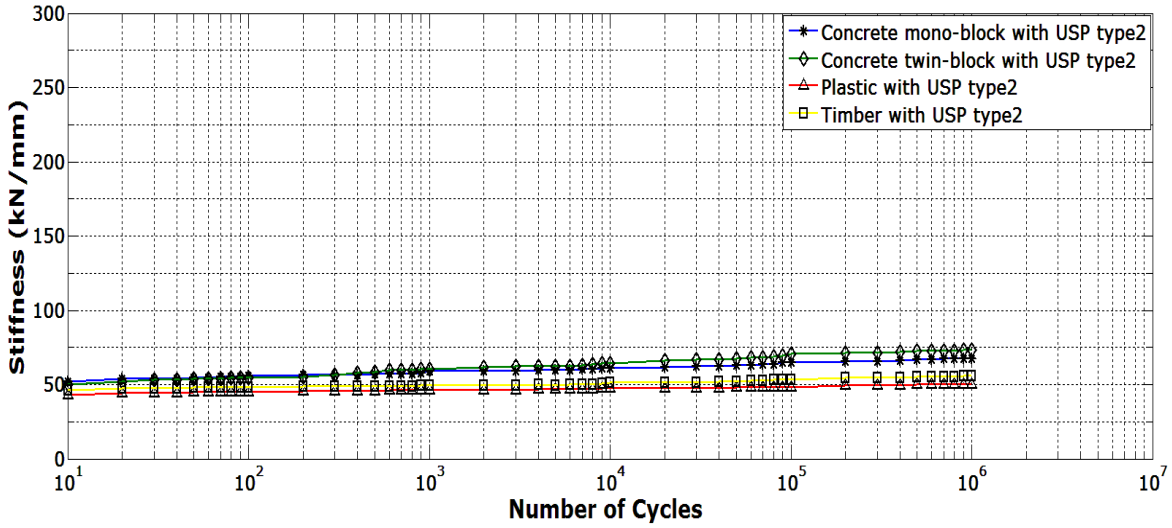


Figure 6-58 (a) Evolution of sleeper types permanet settlement for sleepers with USP type2



(b)

Figure 6-58 (a) Evolution of sleeper types permanet settlement for sleepers with USP type2 (b) Evolution of trackbed stiffness for sleepers with USP type2

Sleeper deflections in the case USP type2 are presented in Figure 6-59. Observation showed that generally the magnitude of reduction in hogging at the middle of the sleeper was more compared to when USP type1 was used for all sleeper tests; when USP type1 and USP type2 sleeper tests were compared to the 'no USP' scenario. The reductions in hogging at the middle of the sleeper in the case of USP type2 were 61% for concrete mono-block, 60% for concrete twin-block, 59% for timber, and 57% for the plastic sleeper. There was a smaller variability in percentage of reduction of hogging at the middle of the sleeper for all sleeper tests when USP type2 was used compared to the reductions in the case of USP type1. However, similar to when USP type1 was used, the hogging at the middle of the sleeper was reduced most for sleepers with high bending stiffness.

Subgrade pressure in the case of USP type2 at the rail seat and middle of sleeper for all sleeper tests is presented Figure 6-60. Similar to when USP type1 was used, in the case of USP type2 observation showed a general reduction in the subgrade pressure at both rail seat and middle of sleeper for all sleepers tests compare to 'no USP' (with greater average percentage reduction at the middle of the sleeper). However these reductions in subgrade pressure were more evident in the case of USP type2 compared to USP type1 and as can been seen in Figure 6-60 (a) and Figure 6-60 (b) gave smaller variability between different sleeper types (especially at the middle of the sleeper, Figure 6-60 (b)). This can be explained by the fact that hogging at the middle of sleeper in all cases when USP type2 was used was less than when USP type1 was used.

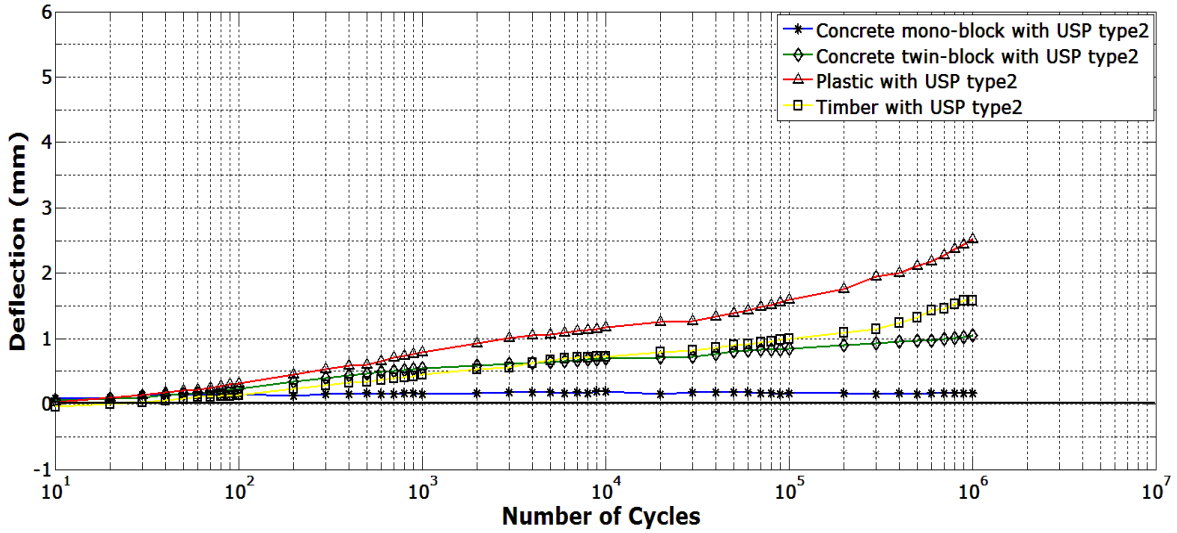
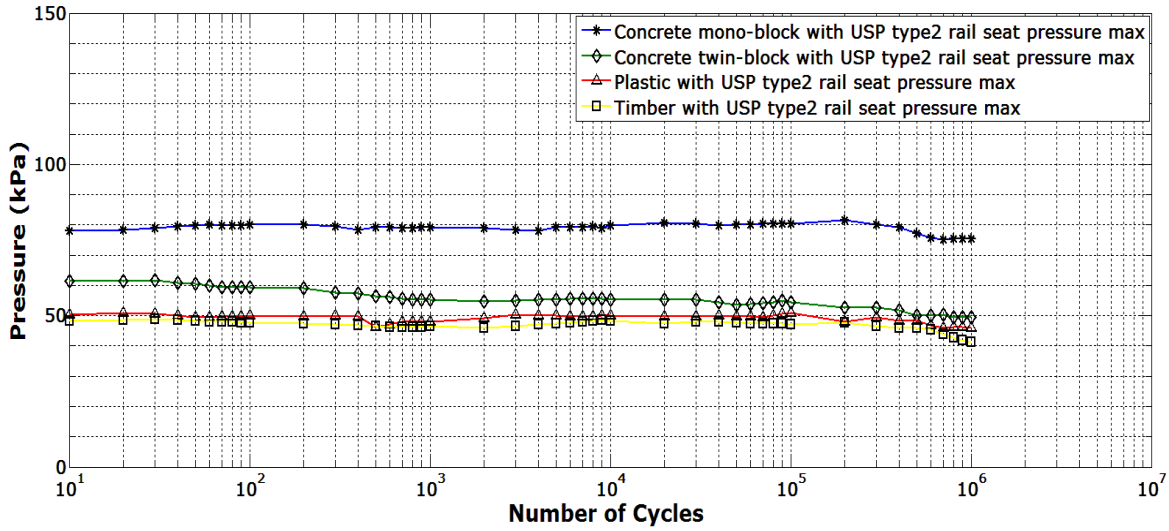
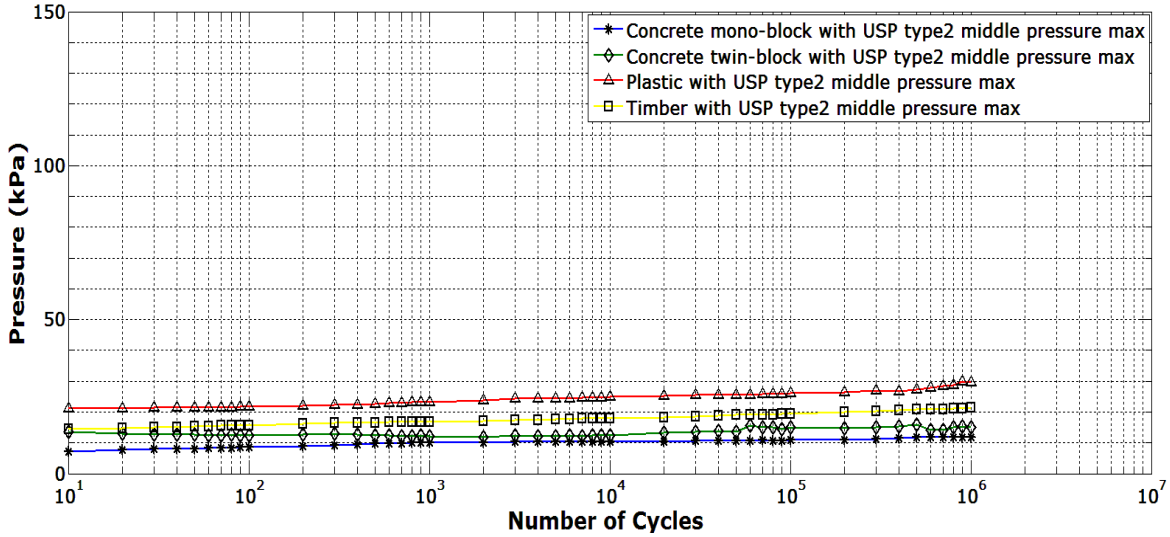


Figure 6-59 Evolution of sleeper deflections of sleeper types for sleepers with USP type2



(a)



(b)

Figure 6-60 Evolution of subgrade pressure for sleepers with USP type2 (a) rail seat (b) middle of sleeper

6.10.9 SUMMARY OF DIFFERENT SLEEPERS TESTING

Summaries of all tests conducted with respect to sleeper settlement, trackbed stiffness, and sleeper deflection are presented in Figure 6-61, Figure 6-62, and Figure 6-63 respectively. In addition, a summary of maximum subgrade pressure at the middle of the sleeper and the rail seat for all test configurations is also presented in Table 6-6.

Network Rail in standard NR/L2/TRK/4239 (NetworkRail, 2015) provided the required trackbed stiffness (dynamic sleeper support stiffness (DSSS)) for different lines (Table 2-5). In Table 6-7, the average trackbed stiffness for all the tests is compared with those NR requirement and suitability of them for different lines is shown. As can be seen in the table, out of all sleep types tests in the case of 'no USP', trackbed stiffness when plastic sleeper was used was not enough to be used for new lines with speed above 100mph. When USPs were used, the trackbed stiffness in the case USP type2 for all sleeper types was not enough to bed used for new lines with speed above 100mph.

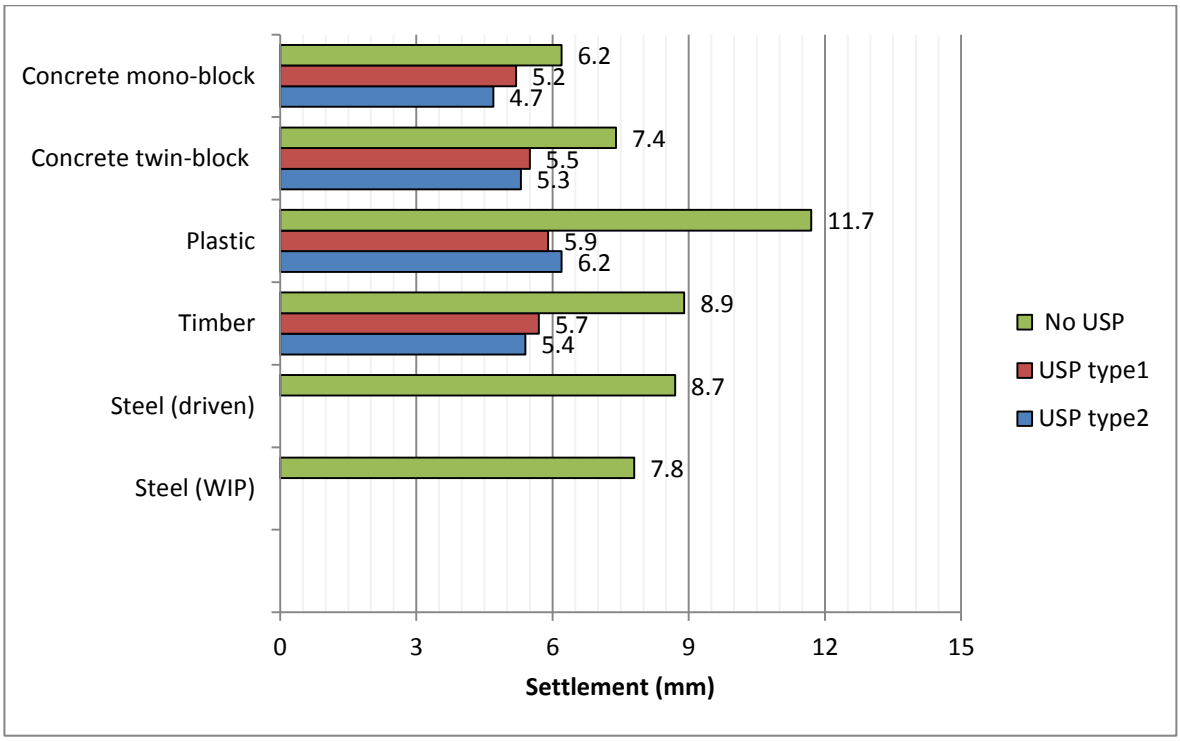


Figure 6-61 Sleeper settlement at one million cycles

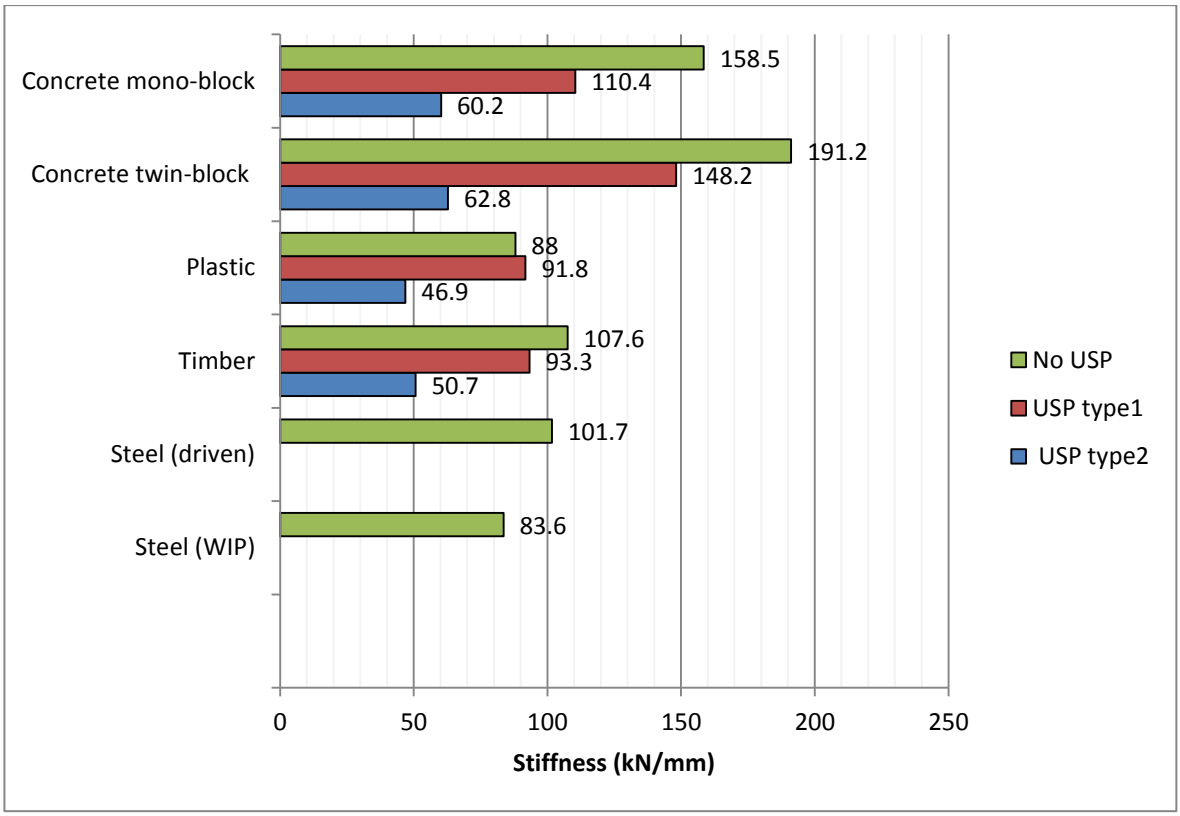


Figure 6-62 Average trackbed stiffness over one million cycles

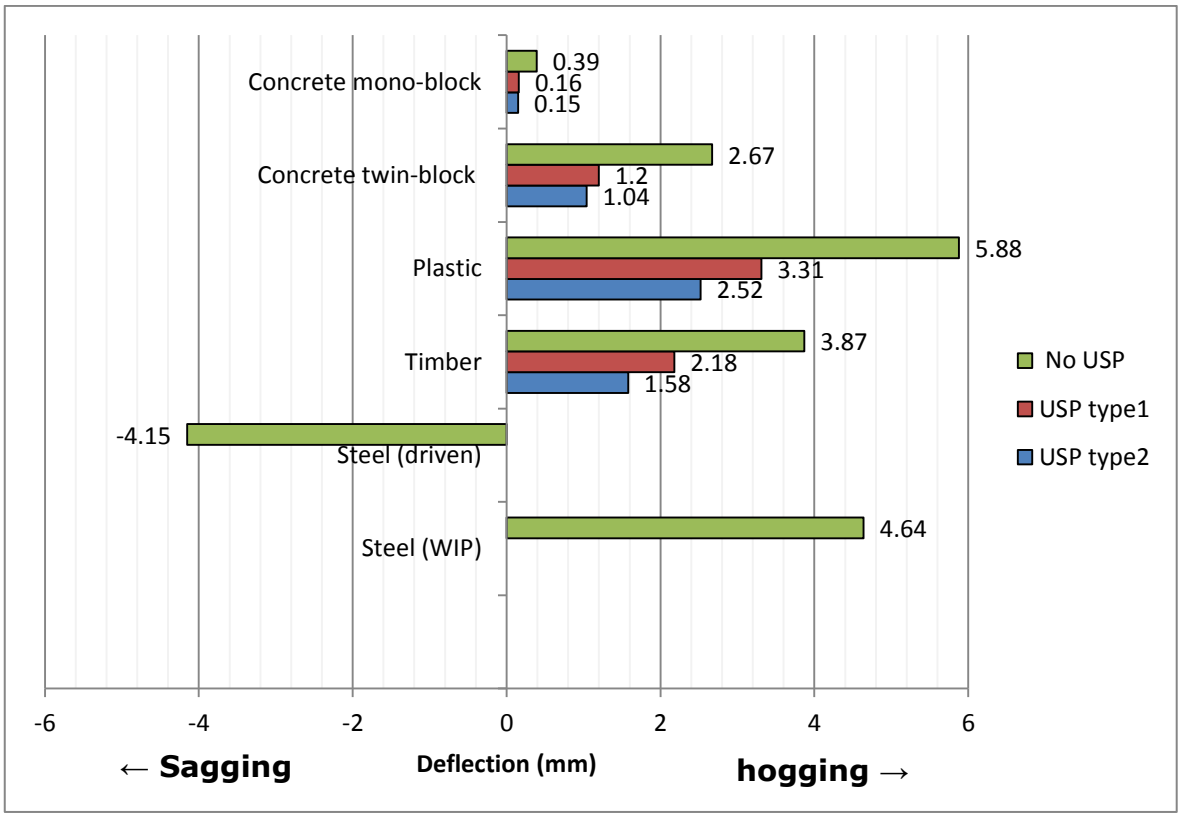


Figure 6-63 Sleeper deflections at one million cycles

Table 6-6 Maximum subgrade pressure at the middle of sleeper and the rail seat

	Maximum subgrade pressures (kPa)											
	Sleeper Rail Seat						Middle of sleeper					
Test / No. of cycles	10th	100th	1000th	10000th	100000th	1000000th	10th	100th	1000th	10000th	100000th	1000000th
Concrete mono-block	96.5	95.0	98.5	99.7	99.4	84.0	19.1	20.1	25.5	26.6	30.4	33.0
Concrete mono-block + USP type1	86.7	85.5	86.0	87.3	85.9	81.3	9.1	12.0	15.2	17.6	19.2	27.3
Concrete mono-block + USP type2	77.9	80.1	79.1	79.9	80.3	75.5	7.3	8.6	10.1	10.5	10.8	11.8
Concrete twin-block	79.7	77.4	75.2	72.7	71.3	61.4	26.3	23.3	23.6	23.7	23.2	25.0
Concrete twin-block + USP type1	68.2	66.9	65.9	64.7	61.8	57.7	20.6	19.0	17.8	16.2	16.7	15.0
Concrete twin-block + USP type2	61.6	59.4	55.4	55.4	54.5	49.4	13.5	12.3	12.2	12.3	15.0	14.9
Plastic	69.3	66.6	66.2	67.7	67.0	54.6	20.3	28.2	38.6	47.5	51.6	69.3
Plastic + USP type1	62.2	63.3	65.3	67.1	66.1	53.5	34.8	37.1	41.6	42.1	43.3	54.3
Plastic + USP type2	50.5	49.9	48.0	50.1	50.8	45.8	21.2	21.8	23.3	24.8	26.1	29.6
Timber	68.8	72.0	76.3	78.0	73.4	53.5	22.8	22.9	25.8	26.0	28.1	32.9
Timber + USP type1	72.0	72.5	70.0	65.2	64.0	51.2	17.4	18.6	19.7	21.1	21.4	26.6
Timber + USP type2	48.2	47.6	46.4	48.3	47.3	41.2	14.5	15.7	16.8	18.0	19.5	21.4
Steel (driven)	55.3	55.0	54.1	51.6	53.2	42.9	78.1	78.5	79.8	84.4	87.1	67.4
Steel (WIP)	25.4	25.7	25.4	26.1	26.3	31.3	94.8	100.3	102.6	106.0	113.4	114.5

Table 6-7 Suitability of trackbed stiffness for different lines

	Existing Lines			New track	
Test / Line speed	<50mph	50 to 125mph with geogrid reinforcement	50 to 125mph without geogrid reinforcement	Up to 100mph	Above 100mph
Concrete mono-block	✓	✓	✓	✓	✓
Concrete mono-block + USP type1	✓	✓	✓	✓	✓
Concrete mono-block + USP type2	✓	✓	✓	✓	x
Concrete twin-block	✓	✓	✓	✓	✓
Concrete twin-block + USP type1	✓	✓	✓	✓	✓
Concrete twin-block + USP type2	✓	✓	✓	✓	x
Plastic	✓	✓	✓	✓	x
Plastic + USP type1	✓	✓	✓	✓	x
Plastic + USP type2	✓	✓	x	x	x
Timber	✓	✓	✓	✓	✓
Timber + USP type1	✓	✓	✓	✓	x
Timber + USP type2	✓	✓	x	x	x
Steel (driven)	✓	✓	✓	✓	✓
Steel (WIP)	✓	✓	✓	✓	x

6.10.10 SLEEPER SELECTION GUIDANCE

Tests carried out using the box, CET, and RTF apparatus resulted in consistent performance ranking for sleeper types with respect to sleeper settlement and trackbed stiffness. However, as discussed in the CET chapter (chapter 5), the magnitudes of measured sleeper settlement and trackbed stiffness results were not comparable due to the differences in test condition such as load, frequency of loading, level of confinement, size of sleeper, etc. The confirmation of consistency of results of the box and CET with those of the RTF allowed a relatively large number of simplified tests to be carried out in both confined (box) and semi-confined (CET) conditions within a reasonable cost and time frame.

The ranking performance of sleepers identified and validated by using three different apparatus can help in developing sleeper selection guidance based on sleeper settlement and trackbed stiffness. The purpose of this guidance is to assist in logical selection of sleeper type and it should be used together with consideration of other track components and the conditions of the site (e.g. acceptable subgrade pressure (Table 6-6)); and suitability trackbed stiffness (Table 6-7). Table 6-8 and Table 6-9 present a qualitative performance ranking of full-size railway sleepers investigated using the RTF in respect to sleeper settlement and trackbed stiffness respectively. It should be noted that for sleeper selection in the case of USPs, more credits should be given to trackbed stiffness than sleeper settlement as the settlement performance of sleepers with inclusion of USPs are very close; this is specially important in the case of high speed lines.

Table 6-8 Sleeper selection guidance based on settlement

	No USP	USP Type1	USP Type2
↓ Increasing settlement	Mono-block	Mono-block	Mono-block
	Twin-block	Twin-block	Twin-block
	Steel (driven)	Plastic	Plastic
	Steel (WIP)	Timber	Timber
	Timber		
	Plastic		

Table 6-9 Sleeper selection guidance based on average trackbed stiffness

	No USP	USP Type1	USP Type2
↓ Increasing average stiffness	Twin-block	Twin-block	Twin-block
	Mono-block	Mono-block	Mono-block
	Timber	Timber	Timber
	Steel (driven)	Plastic	Plastic
	Plastic		
	Steel (WIP)		

6.11 SUMMARY

Performance of five different sleeper types namely concrete mono-block, concrete twin block, plastic, timber, and steel sleeper was studied with respect to sleeper settlement, trackbed stiffness and subgrade pressure using the laboratory full-scale Railway Test Facility (RTF) at the University of Nottingham. In addition, the effect of inclusion of two different Under Sleeper Pads (USP) types namely USP type1 (stiff) and USP type2 (soft) for all the sleepers apart from the steel sleeper was investigated. All the tests were carried out for 1 million cycles simulating the passing of a 20 Tonne axle load.

Linearly Variable Differential Transformers (LVDTs) were used to measure the settlement at the middle of the sleeper and sleeper ends. Bending stiffness of sleepers (apart from concrete twin-block) was also measured through a series of static tests on sleepers. Additionally, subgrade pressure was also monitored using two pressure plates at the ballast-subgrade interface corresponding to the middle of the sleeper and the sleeper rail seat.

The test set-up processes for all sleeper types were identical apart from the steel sleeper which required different installation methods. The repeatability of results was investigated by repeating the concrete mono-block and twin block tests. Apart from repeated tests, 14 unique tests were conducted.

As far as sleeper settlements are concerned, the best performance was observed for the concrete mono-block sleeper and the worst for the plastic sleeper. Wider and deeper sleepers performed better with respect to sleeper settlement compared to narrower and shallower sleepers. Increased coefficient of friction at the base of the sleeper reduced sleeper settlement and extended the ballast support at the sleeper rail seat for a longer time period as did sleepers with higher bending stiffness. Observation of subgrade pressure where a drop in pressure at the rail seat and an increase at the middle of the sleeper also showed the extent of support at the sleeper rail seats. The bending stiffness results of individual sleepers were also consistent with sleeper deflections calculated from the settlement differences between the middle of the sleeper and the rail seats. The concrete mono-block sleeper (with the largest bending stiffness) gave the least deflection over one million cycles and the plastic sleeper (with the lowest bending stiffness) gave the largest deflection. With regard to the trackbed stiffness, the concrete sleepers resulted in the stiffest trackbed followed by the timber sleeper and then the plastic sleeper. Steel sleeper deflections at the middle of the sleeper were different depending on the method of installation;

the Wished In Placed (WIP) sleeper showed hogging whereas the driven sleeper resulted in sagging. The extent of steel sleeper deflections for both methods were comparable, although the directions of the deflections were opposite. The driven method may be the preferred option considering it is ideal for a sleeper to be mainly supported at the rail seats.

Inclusion of USPs (regardless of type) were beneficial in the following ways compared to the 'no USP' scenario:

- Increased trackbed resiliency
- Reduced maintenance works related to settlement
- Reduced sleeper deflections (hogging) at the middle of sleeper
- Reduced rate of deterioration of sleeper support at rail seat
- Reduced differences in the performance of different sleeper types
- Reduced tendency towards centre binding
- Reduced transmitted pressure on subgrade

Out of the two USP types tested, the USP type2 in the majority of test configurations performed slightly better with respect to the aforementioned benefits of USP inclusion. The least settlement was observed in the case of concrete mono-block and USP type2. With respect to improvement in settlement, the highest settlement improvement was observed in the case of the plastic sleeper with USP type1 (47%); use of plastic sleeper therefore is strongly discouraged unless there is a USP. Inclusion of USPs (regardless of type) reduced the trackbed stiffness except in the case of plastic with USP type1; the extent of reduction was larger in the case of USP type2 and therefore use of USP type2 for new high seeped lines is discouraged. Considering use of USP type1 and type2 showed comparable settlement performance but different trackbed stiffness, use of them can be specially beneficial at places such as transition zones where gradual changes in trackbed stiffness are required.

7. CONCLUSIONS AND FURTHER RESEARCH

7.1 CONCLUSIONS

It was mentioned in Chapter 1 that the aim of this study was to develop an improved understanding of the impact of sleeper, ballast and sleeper/ballast interface modifications on the performance of conventional ballasted track, with a particular focus on: 1) the effect of sleeper and sleeper/ballast interface types; 2) the effect of ballast grading, layering system, bonding and reinforcement. The aim was achieved through addressing the objectives set out in Chapter 1 and highlighted below in turn:

Phase 1, all tests in this phase were conducted using all three apparatus, first the box and CET and then the RTF to investigate the effect of:

- ***Sleeper Type:***
  - ☑ ***Using box and CET, by testing concrete, timber, plastic, hollow rectangular, and U sections using the current standard NR ballast grading.***
  - ☑ ***Using RTF, by testing concrete mono-block, concrete twin-block, timber, plastic, and steel sleepers using the current standard NR ballast grading.***

Test results from using box, CET (preliminary tests), and RTF (full-scale tests) apparatus gave consistent performance ranking for sleeper types with respect to sleeper settlement and trackbed stiffness. Nevertheless, the magnitudes of measured sleeper settlement and trackbed stiffness results were not comparable due to the differences in test conditions such as load, frequency of loading, level of confinement, size of sleeper, etc.

It was found that the type of sleeper has an influence on the performance of a railway track. The investigated sleepers were different in geometry, friction coefficient at the base, and bending stiffness. With respect to settlement performance of a railway track, sleepers with higher depth and width, coefficient of friction, and bending stiffness performed better. Out of all investigated sleeper types, the concrete mono-block gave the least settlement and the plastic sleeper gave the most. Although the settlement magnitudes of sleeper types over one million cycles were different, similar patterns of settlement were observed; a rapid rate of settlement during initial loading cycles followed by gradual reduction in the rate of settlement in succeeding loading cycles.

With regard to the average trackbed stiffness, a similar pattern was followed by all sleeper types; a rapid increase in stiffness during initial loading cycles followed by gradual reduction

in the rate of stiffness increase in succeeding loading cycles. However, a higher average value of stiffness was found over a million cycles for sleepers with a larger bending stiffness compared to those with lower stiffness.

With regard to subgrade pressure at the rail seat, the same pattern was found in all tests. In all cases the pressure was almost constant with increasing number of loading cycles up to the point that ballast support at the sleeper rail seat began reducing due to migration of ballast away from the ends of the sleeper towards the shoulder. At this point, the pressure at the rail seat then began decreasing while it increased at the middle of the sleeper. It was also found that a sleeper with higher bending stiffness (e.g. mono-block) compared to a sleeper with lower bending stiffness (e.g. plastic) gave lower tendency for hogging and eventually centre binding and therefore higher subgrade pressure at the rail seat.

The concrete mono-block sleeper gave the highest bending stiffness during the static test compared to the rest of the sleeper types and as a result gave the smallest deflections (hogging) at the middle of the sleeper with increasing number of loading cycles. However, the pressure on the ballast-subgrade interface at the rail seat was the largest, which showed the low load damping capability of the concrete mono-block as a result of high bending stiffness. Concrete also showed the largest coefficient of friction at the ballast-sleeper interface in the friction test. Furthermore, the average trackbed stiffness in the case of mono-block was the second highest after concrete twin-block, high enough to be used for existing and new high speed lines.

Concrete twin-block sleeper settlement performance was close to that of the concrete mono-block (twin-block settled 19% more over one million loading cycles). However, the deflections (hogging) at the middle of the sleeper were higher for twin-block, which made it less stable compared to mono-block. The pressure on the ballast-subgrade interface at the rail seat was lower than for mono-block, which showed the higher load damping capability of twin-block compared to mono-block.

The timber sleeper during the static loading test showed a very high level of elasticity with almost no permanent deflection up to 100kN applied static load. This can reduce the chance of failure for timber sleepers the most compared to the rest the investigated sleeper types. The fiction coefficient at the base of timber sleeper was less than for concrete sleepers (both mono-block and twin-block). The settlement of the timber sleeper was more than mono-block (44%). The average trackbed stiffness in the case of the timber sleeper was less than for concrete mono-

block and twin-block and it was not acceptable to be used for new lines with speed above 100mph.

Plastic sleeper settlement performance was the worst out of the investigated sleeper types (settled 88% more than mono-block). The plastic sleeper showed the lowest bending stiffness during the static loading test, the largest deflections (hogging) at the middle of the sleeper over one million loading cycles, and the smallest friction coefficient at the base in the friction test. The pressure on the ballast-subgrade interface at the rail seat was the lowest due to low bending stiffness and deflections of the plastic sleeper. This consequently increased the load damping capacity of the sleeper the most compared to the rest of the investigated sleepers. The trackbed stiffness in the case of the plastic sleeper was not enough to be used for new lines with speed above 100mph.

The steel sleeper performed differently depending on the method of installation: driven or Wished-In-Place (WIP). The settlement performance of the steel sleeper during initial loading cycles (up to 100k) was better in the driven case, however over one million loading cycles the WIP performed better. Steel sleeper deflections at the middle of the sleeper showed hogging behaviour in the case of the WIP and in contrast showed sagging behaviour in the driven case. As a result higher subgrade pressure was observed for the driven sleeper at the rail seat compared to the WIP case and higher subgrade pressure for the WIP at the middle of the sleeper compared to the driven sleeper. The trackbed stiffness in the case of the driven sleeper was more compared to the WIP case and was acceptable to be used for both existing and new high speed lines. However, for the WIP sleeper, the trackbed stiffness was not enough to be used for new lines with speed above 100mph.

Considering all aforementioned factors, although each sleeper can be useful in different scenarios depending on the conditions of the site, out of all tested sleepers, the concrete mono-block performed best, closely followed by the concrete twin-block.

- ***USPs:***
  - ***Using box and CET, by testing two different types of USP on all sleeper sections except U section.***
  - ***Using RTF, by testing two different types of USP on all sleeper types except steel sleeper.***

Testing inclusion of USPs affixed to the base of all sleepers for different sleeper types showed consistent performance in respect to settlement and trackbed stiffness in the box, CET and RTF apparatus.

Use of a USP (regardless of type) improved the settlement performance of sleepers for all the investigated sleeper types compared to the 'no USP' case as well as reducing the maintenance works related to settlement. However, USP type2 (soft USP) in the majority of test configurations showed slightly better performance. Application of a USP reduced the differences in the settlement performance of sleepers for different sleeper types. The USPs gave the highest friction coefficient at the base of a sleeper in the friction test compared to all the investigated sleepers. One of the reasons for the comparability of settlement performance of different sleeper types when USPs were used, can be the comparability of friction coefficient at the base of the sleepers. Use of USPs generally conferred greater resilience to the trackbed over one million loading cycles

Inclusion of USPs in the majority of sleeper tests reduced the trackbed stiffness. However the amount of reduction was more when USP type2 was used.

Deflections (hogging) at the middle of the sleeper, and as a result the tendency of the sleeper towards centre binding, were reduced when USPs were used. Attachment of USPs to the base of sleepers improved the load damping capacity of the sleepers and as a result pressure on the subgrade at both middle of sleeper and sleeper rail seat decreased.

Out of all sleeper tests with USPs, the least settlement was observed in the case concrete mono-block with USP type2. The largest improvement in settlement was observed in the case of the plastic sleeper with USP type1 compared to the plastic sleeper without USP (47%).

Attachment of USPs to the base of the concrete mono-block sleeper significantly improved the track resiliency after one million loading cycles. The settlement performances of the mono-block with USP type1 and type2 were comparable. However, the difference in average trackbed stiffness reduction when USPs were used was significant. The stiffness in the case of mono-block with USP type1 was acceptable to be used for both existing and new high speed lines. However, in the case of mono-block with USP type2, it was enough to be used for new lines with speed up to 100mph. Inclusion of USPs for concrete mono-block had the most significant effect in reduction of deflections (hogging) at the middle of the sleeper and consequently decreasing the rate of deterioration of sleeper support at the rail seat compared to the rest of the investigated sleepers.

Use of USPs (regardless of type) for concrete twin-block had less effect on trackbed resiliency compared to those of the concrete mono-block sleeper. Similar to the mono-block sleeper,

inclusion of USP type1 and USP type2 comparably improved the settlement performance of the twin-block sleeper, but the effect of USPs on average trackbed stiffness reduction was significantly different when different USP types were used. In the case of USP type1 the average stiffness was still large enough to be used for both existing and new high speed lines. However, when USP type2 was used the stiffness in the case of new lines was acceptable only to be used for speed up to 100mph.

Attachment of USPs (irrespective of type) to the base of the timber sleeper improved the settlement performance of the sleeper more than those of the concrete sleepers (both mono- and twin-block). The effect of USP inclusion was also significant on resiliency of trackbed after one million loading cycles. Average trackbed stiffness reduction was only significant in the case of USP type2. The stiffness in the case of USP type1 was acceptable to be used for existing lines with speed up to 125 mph and new lines with speed up to 100mph. However, in the case of USP type2 the trackbed stiffness was only acceptable to be used for existing lines with speed of less than 50mph.

Inclusion of USPs (regardless of type) for the plastic sleeper improved the settlement performance of the sleeper the most compared to the rest of the investigated sleepers. This can be partially ascribed to the fact that the friction coefficient at the base of the sleeper significantly increased compared to the 'no USP' case. Use of USPs also had considerable effect on improvement of resiliency of trackbed after one million loading cycles and reduction of maintenance work related to settlement. The plastic sleeper was the only sleeper that had slightly better settlement performance with USP type1 compared to USP type2. Use of USP type1 also slightly increased the average trackbed stiffness compared to the 'no USP' case. Average trackbed stiffness of the plastic sleeper with USP type1 was acceptable to be used for existing lines with speed up to 125 mph and new lines with speed up to 100mph. However, when USP type2 was used the average trackbed stiffness was only enough to be used for existing lines with speed of less than 50mph.

Considering all factors above, use of USP type 1 with concrete mono-block and concrete twin block is recommended for the high speed lines. Application of USP type2 for high speed lines is not suggested, especially in the case of plastic and timber sleepers. Use of plastic sleepers is strongly discouraged unless with USP type1.

The following have been found to be beneficial in the case of inclusion of USPs (regardless of type) compared to the 'no USP' scenario:

- Increased trackbed resiliency
- Reduced maintenance works related to settlement
- Reduced sleeper deflections (hogging) at the middle of the sleeper
- Reduced rate of deterioration of sleeper support at rail seat
- Reduced differences in the performance of different sleeper types
- Reduced tendency towards centre binding
- Reduced transmitted pressure on subgrade
- Made more the trackbed stiffness more uniform

Phase 2, the tests in this phase were carried out using both/one of the box and CET apparatus to investigate the effect of:

- ***Ballast grading (box & CET):***
  - ☑ ***By testing the strength of the modified current standard NR ballast grading using smaller ballast grading (2/3 scale) by conducting monotonic triaxial tests with large-scale triaxial apparatus.***
  - ☑ ***By testing the 2/3 scale ballast on concrete, timber, and plastic sections with and without two different USP types.***

Using the large-scale triaxial apparatus to conduct monotonic triaxial tests with confining pressures of 15kPa, 30kPa and 60kPa, it was found that the strength and angle of friction for 2/3 scale ballast was generally comparable to or slightly higher than those of the current standard NR ballast grading sample.

Testing the 2/3 scale ballast with both box and CET apparatus showed consistent performance withrespect to settlement and trackbed stiffness. It was found that use of smaller ballast grading compared to the current standard NR ballast grading reduced the settlement and increased the trackbed stiffness in all tests with and without USPs. However, the performance ranking of sleeper sections with respect to settlement and trackbed stiffness when 2/3 scale ballast was used was similar to that of the full-size ballast (current standard NR ballast grading). The lowest settlement resulted in the case of a concrete section with USP type2 and 2/3 scale ballast comparable to that of the timber sleeper section with USP type2 and 2/3 scale ballast. The plastic sleeper section showed the largest improvement in settlement performance when 2/3 scale ballast was used.

Regardless of USP type, less differences in settlement performance of sleeper sections in all tests were observed when 2/3 scale ballast was used compared to full-size ballast; use of USPs or 2/3 scale ballast both increased the number of contacts and load spreading ability at the sleeper-ballast interface.

- ***USP stiffness (box & CET):***
  - ☑ ***By testing USPs with a greater range of stiffness made from extra layers of USPs employing concrete sleeper section and the current standard NR ballast grading.***

Tests using the box and CET apparatus on the effect of stiffness of USPs showed consistent results in terms of settlement performance ranking of USPs attached to a concrete sleeper section. When multi-layer USPs were used, the double-layer USPs (regardless of type) showed the best settlement performance compared to single-layer and triple-layer USPs. The USP stiffness range of double-layer USPs, therefore, can be considered as an indication of the optimal stiffness range to improve trackbed settlement. Use of multi-layer USP (regardless of type) reduced the trackbed stiffness, however, the reduction in the case of multi-layer USP type2 was significant compared to multi-layer USP type1.

Considering both settlement and trackbed stiffness performance of multi-layer USPs, double-layer USP type1 is recommended as the best option to improve the settlement performance of trackbed compared to the rest of the multi-layer USPs.

- ***Two-layered ballast system (box):***
  - ☑ ***By testing two different systems; sample 1 made from the current standard NR ballast grading on top and 2/3 scale ballast at the base, sample 2 made from 2/3 scale ballast on top and the current standard NR ballast grading at the base.***

Modification of the trackbed by introducing a two-layered ballast system showed better settlement performance for sample 2 compared to sample 1. When two-layered ballast samples were compared to the single-layered ballast samples (full-size and 2/3 size), sample 2 performed better than both single-layered ballast samples. However, the settlement performance of sample 1 was only better than the full-size single-layered sample. The average trackbed stiffness of sample 2 was also larger than both of the single-layered ballast samples. However, in the case of sample 1 the average trackbed stiffness was larger than that of the full-size single-layered sample but smaller than the 2/3 size single-layered sample.

- ***Elastic elements inclusion (box)***
  - ☑ ***By testing inclusion of end-of-life protective packaging polystyrene in two different sizes separately with different inclusion percentages.***

Reinforcement of ballast using standard end-of-life protective packaging polystyrene (regardless of particle size), did not improved the settlement performance of the trackbed in the majority of inclusion percentages and therefore is not recommended. Out of 1%, 2.5%, 5%, and 10% polystyrene inclusions using small and large size polystyrene elements separately, only inclusion of 1% small size polystyrene slightly improved settlement and increased average trackbed stiffness compared to the basic test with no polystyrene inclusion. Of the large and small size polystyrene inclusions, the small-sized particles resulted in lower ballast layer settlements and higher stiffness values compared to the large inclusions when 1% and 2.5% polystyrene inclusions were used. However, in the case of 5% and 10% polystyrene inclusions, the large size led to lower ballast layer settlements and greater stiffness values compared to the small size.

- ***Frictional Sleeper and frictional USP (box):***
  - ☑ ***By testing a frictional concrete sleeper section and a frictional USP made by modifying the underside of a concrete sleeper section and USP respectively.***

Modification of both concrete sleeper section base and USP type2 to obtain a frictional sleeper and USP produced less settlement and higher trackbed stiffness compared to the normal concrete sleeper section and USP type2. However, a higher reduction in settlement and increase in stiffness was observed when a frictional USP was used compared to the frictional sleeper. Therefore, use of a frictional sleeper not only improved the lateral resistance of a ballasted railway track (as demonstrated in past research), but also it was found that it improved the settlement performance as well.

- ***Ballast bonding/gluing (CET)***
  - ☑ ***By testing a low cost glue made from end-of-life protective packaging polystyrene and pure d-Limonene on the current standard NR ballast grading***

Application of a low cost glue, made from end-of-service packing polystyrene and pure d-Limonene, to the ballast bed reduced the settlement of the trackbed (37%) and increased the average stiffness of trackbed (23%) when a concrete sleeper section used. Application of the glue to the ballast bed was also tested in combination with use of USP type2 on the base of the concrete sleeper section and 54% improvement in settlement and comparable trackbed stiffness was observed compared to the basic test. The glue therefore improved significantly the low

trackbed stiffness in the case of USP type2. In the case of USP type2 and the concrete section, glued ballast performed 38% better than non-glued ballast in terms of settlement and it increased the trackbed stiffness by 78%.

7.2 RELEVANCE OF RESEARCH FOR PRACTICE

A railway track in service deteriorates over time and suffers from loss of as-built geometry with respect to line and level. This deterioration is mainly due to the settlement of ballast under repeated loading cycles. To return the track to the as-built/acceptable geometry, maintenance is required. To avoid interruption to service and cost of track maintenance, it is important to improve the performance of a railway track in such a way that it can last longer with no/less maintenance.

In this study potential alterations of ballast, sleeper, and ballast-sleeper interface to improve the performance of a ballasted railway track was investigated. The main implications of this study for practice are as follows:

- Geometry, friction coefficient at base, and bending stiffness of a sleeper have influence on the performance of a railway track and in terms of settlement performance of a track, sleepers with higher depth and width, coefficient friction, and bending stiffness perform better. These were demonstrated by the settlement performance of the concrete mono-block sleeper compared to the rest of investigated sleeper types. Use of a sleeper with a low coefficient friction at the base (e.g. plastic) is discouraged. The performance of a sleeper can also be influenced by the method of installation (e.g. steel sleeper).
- Considering the average trackbed stiffness for all of the investigated sleepers types, the concrete sleepers (regardless of type) are recommended as appropriate options to be used for high-speed lines.
- The base of sleepers at the ballast-sleeper interface can be modified to a grid shape with an appropriate size corresponding to the biggest ballast particle size placed underneath. This can improve the lateral resistance of railway track as well as its settlement and stiffness. This can be done during the production of the railway sleepers. This can especially improve the performance of plastic sleepers by increasing the low coefficient friction at its base.
- Modification of the base of a sleeper by attaching USPs can improve the performance of a track not only in terms of reducing settlement and making the trackbed stiffness more uniform, but also reducing the transmitted pressure on the formation, which can reduce the amount of formation treatment works. The test results of inclusion of USPs for all different

sleeper types provide useful information on the actual response of USPs on real track in addition to the properties provided by the manufacturer prior to decisions of USPs application. USP inclusion can be done during replacement of sleepers and can be especially useful for short but critical parts of railway tracks such as transition zones, turnouts, and crossovers.

- Use of USPs is beneficial in performance improvement of a railway track, however, the stiffness and underside shape of USPs in contact with ballast can be modified to optimise and increase the benefits of USPs with respect to the lateral resistance, settlement, and trackbed stiffness of a railway track.
- The performance of the current standard NR ballast grading can be improved by using smaller ballast grading. Choosing an appropriate smaller ballast grading size also would increase the stiffness of the trackbed without compromising the strength and acceptable level of drainage.
- Use of a two-layered ballast system made from ballast with small grading on top and ballast with the current standard NR grading at the base can perform better as a ballast bed compared to a single-layered ballast system made from the current standard NR ballast grading, in terms of both settlement and trackbed stiffness. By using a two-layered ballast system, the need for ballast tamping can also be avoided or minimised; as demonstrated in the literature review, ballast tamping can cause ballast particle breakage and it is not an effective long-term solution and requires repeating after a relatively short time. Maintenance of a two-layered ballast system (if required) can be done by stoneblowing, as in this method, ballast with small grading is blown underneath the sleeper and it causes less damage to the trackbed.
- A low cost glue can be produced from end-of-life protective packaging polystyrene and pure d-Limonene that not only improves the settlement and trackbed stiffness performance of a railway track, but also helps recycling the discarded polystyrene which does not bio-degrade for years. Application of the glue to the ballast bed in combination with use of USPs can compensate the trackbed stiffness reduction due to the inclusion of a USP as well as giving a better settlement performance compared to use of USP only.

7.3 RECOMMENDATION FOR FUTURE STUDY

The following are suggested for future research based on the results presented in this study:

- The effect of different sleeper types on performance of a railway track has been investigated in this research. The focus of this study was mainly to cover the most common existing sleeper types. However, many other types may be available. In addition to the sleeper performance data presented in this study, testing of new types of sleepers can contribute to guidance for selection of sleeper type. However prior to testing of a new sleeper, the installation method for the sleeper should be carefully considered in ordered to simulate the closest possible condition to the real track.
- Although in this study performance data on attachment of USPs to the base of sleepers for different sleeper types using a full-scale railway facility have demonstrated several benefits, use of Discrete Element Modelling (DEM), to simulate the ballast-USP interface in ballasted track can provide more detailed understanding on how USP works.
- The effect of 2/3 scale NR ballast on performance improvement of different sleepers types was varied, e.g. plastic had the largest improvement. Therefore, this grading can be optimised in the case of each sleeper types to achieve the optimal performance; the optimised ballast grading then can be tested along with the corresponding sleeper on the top layer of a two-layered ballast system.
- Inclusion of end-of-service packing polystyrene as an elastic element to ballast sample did not improved the performance of the ballasted track. However, more studies can be done in this area to find out the optimal mechanical properties of an elastic element to be added to the ballast in order to improve the performance of ballasted track; this can be done through both numerical and experimental studies. The outcome can facilitate finding more materials that can be used as an elastic element especially with regard to recycling/reusing of materials that last for years and have negative impact on the environment.
- Although the low cost glue made from end-of-life protective packaging polystyrene and pure d-Limonene in this study worked well and improved the performance of the ballasted track, more studies can be done on chemical and physical properties of this glue to find the best fit for the track conditions.
- More combination tests (e.g. application of glue in combine with USPs) can be conducted to find out the collective influence of the modifications on performance of the railway track. In the case of the combination tests conducted in this study, e.g. glue and USP or the 2/3 scaled ballast and USP, the combination of modifications together supported each other to improve the performance of the railway track. However, it is suspected that there may also be modifications that together would not support each other to enhance the performance of the railway track.

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