

LB/TH/08/2026

TH6136

**INVESTIGATING THE EFFECT OF UNDER SLEEPER
PADS AND BALLAST MATS IN ATTENUATION OF
TRAIN INDUCED GROUND VIBRATIONS**

Marasinghe Mudiyansele Geeshan Chamodya Marasinghe

248075K

Master of Science (with Major Component of Research)

Department of Civil Engineering

Faculty of Engineering

University of Moratuwa

Sri Lanka

December 2025

INVESTIGATING THE EFFECT OF UNDER SLEEPER PADS AND BALLAST MATS IN ATTENUATION OF TRAIN INDUCED GROUND VIBRATIONS

Marasinghe Mudiyanseelage Geeshan Chamodya Marasinghe

248075K

Thesis submitted in partial fulfillment of the requirements for the degree
Master of Science (with Major Component of Research)

Department of Civil Engineering
Faculty of Engineering

University of Moratuwa
Sri Lanka

December 2025

DECLARATION

I declare that this is my own work and this thesis/dissertation does not incorporate without acknowledgement any material previously submitted for a degree or diploma in any other University or Institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text. I retain the right to use this content in whole or part in future works (such as articles or books).

Signature:

Date: 27/12/2025

The above candidate has carried out research for the PhD/MPhil/Masters thesis/dissertation under my supervision. I confirm that the declaration made above by the student is true and correct.

Name of Supervisors: Dr. H.G.H. Damruwan

Prof. C. S. Lewangamage

Signature of the Supervisors:

.....

Dr. H. G. H. Damruwan

Date: 29/12/2025

Prof. C. S. Lewangamage

Date: 29/12/2025

DEDICATION

This thesis is dedicated to all those who offered unwavering support and encouragement throughout my academic journey. This achievement is not mine alone, it is also a reflection of the compassion, guidance, and solidarity of the remarkable individuals who stood by me during this transformative period.

To my principal supervisor, Dr. H. G. H. Damruwan, I express my heartfelt gratitude. Your insightful guidance, generous mentorship, and continuous encouragement have been instrumental in shaping this research. I would also like to extend my sincere thanks to my co-supervisor, Prof. C. S. Lewangamage, for providing valuable guidance on various aspects throughout this study.

I lovingly dedicate this work to my family, whose constant encouragement and patience have been my greatest strength. I also dedicate this thesis to my friends and research colleagues for their motivation, support, and positivity throughout this endeavor. Finally, this work is for anyone who is passionate about learning and committed to the pursuit of knowledge through research.

Above all, I dedicate this achievement to the people of Sri Lanka, whose contributions sustain the nation's free education system and made my academic journey possible.

ACKNOWLEDGEMENT

First and foremost, I extend my heartfelt gratitude to my principal supervisor, Dr. H.G.H. Damruwan, from the Department of Civil Engineering at the University of Moratuwa, for his steadfast guidance, encouragement, and insightful mentorship throughout this research. His expertise, patience, and unwavering support have been pivotal in shaping the direction and quality of this thesis.

I also wish to sincerely thank my co-supervisor, Prof. C. S. Lewangamage, for providing valuable guidance on various aspects throughout this study. I am grateful to the technical officer and supporting staff at Structural Dynamics and Health Monitoring Lab for their assistance during the field studies.

I also wish to sincerely thank the Progress Review Committee, especially Dr. Lakshitha Fernando, for his critical feedback, thoughtful suggestions, and consistent support, which helped to enhance the depth and clarity of this work.

I am also thankful to the academic staff of the Department of Civil Engineering, University of Moratuwa, including Prof. H.R. Pasindu, the MSc Research Coordinator, and especially the academic staff members of the Structural Division, for their continued academic guidance and encouragement throughout the MSc program.

To my fellow researchers and colleagues, thank you for your collaboration, encouragement, and knowledge-sharing that enriched this journey and made it a truly collective effort. Finally, I am deeply grateful to my family and friends for their constant motivation, patience, and unwavering belief in me. Their love and emotional strength carried me through every challenge I faced along this path.

ABSTRACT

Increasing train speeds and axle loads in modern railway systems have intensified the problem of train-induced ground-borne vibration. Vibration generated at wheel–rail interface propagates through the track, ballast, and subgrade to nearby structures, causing potential discomfort, structural damage, and functional disruptions. Out of the available mitigation measures, under sleeper pads (USP) and ballast mats (BM) are very feasible methods that can be adapted in urban areas where the reserved area for track is limited. However, the complex interaction between these mitigation layers, the track system, and heterogeneous soil conditions remains insufficiently understood. This research addresses that gap through a comprehensive numerical investigation combining model validation and parametric analysis.

A three-dimensional finite element (FE) model of a coupled train–track–soil system was developed using the ABAQUS FE software to simulate time-domain vibration propagation. The model incorporated layered track components, rails, sleepers, ballast, sub-ballast, and subgrade, and implemented viscous boundaries to eliminate reflections from the truncated domain. A simplified three-car moving-load train model was adapted to accurately capture the vibration generation mechanisms. Model validation was performed against established field experiments from literature using several key metrics.

Following validation, a parametric study was designed to quantify the influence of USP and BM thickness and stiffness under two representative ground conditions, soft soil ($V_s = 200$ m/s) and stiff soil ($V_s = 400$ m/s). The chosen parameters, consistent with international standards and guidelines, included USP thicknesses of 8, 10, and 15 mm; stiffnesses of 0.15, 0.25, and 0.35 N/mm³; and BM thicknesses of 15, 23, and 30 mm with stiffnesses of 0.05, 0.09, and 0.22 N/mm³. Vibration attenuation was evaluated for near-field (0–10 m) and far-field (>10 m) receivers using Insertion Loss (IL) and frequency-dependent 1/3-octave band spectra.

Both USPs and BMs were found to be significantly more effective in stiff soil compared to soft soil, achieving stronger vibration reduction and extending attenuation further into the far field. In soft soil, soft or thick USPs can trigger near-field amplification due to soil–pad resonance, while stiff soil reliably delivers stable mitigation. Thicker and softer BMs provide the best performance, especially in mid-frequency bands (30–80 Hz) critical for human perception. Overall, vibration control is strongest near the track (within 10 m) and decreases with distance, with BMs offering superior and more consistent attenuation than USPs across all conditions.

The study provides clear guidance for choosing between USPs and BMs under different ground conditions, and show that simple, validated FE models can support reliable design decisions for urban rail vibration control.

Keywords: Train-induced ground vibration, Under sleeper pads, Ballast mats, Vibration attenuation

TABLE OF CONTENT

DECLARATION	I
DEDICATION	II
ACKNOWLEDGEMENT	III
ABSTRACT	IV
LIST OF FIGURES	IX
LIST OF TABLES	XII
LIST OF ABBREVIATIONS	XIII
CHAPTER 1	1
INTRODUCTION	1
1.1 Background of the Study	1
1.2 Research Problem	3
1.3 Aim and Objectives	5
1.4 Scope of the study.....	5
1.5 Significance of the study	6
1.6 Guide to thesis	6
CHAPTER 2	
LITERATURE REVIEW	8
2.1 Introduction.....	8
2.1.1 Growth of Railway Transport and High-Speed Rail.....	8
2.1.2 Ground-Borne Vibration and Noise: Mechanisms and Human/Environmental Impact	9
2.1.3 Prior Local Studies on Train Induced Ground Vibrations	9
2.2 Theoretical Background.....	11
2.2.1 Vibration Generation in Railways.....	11
2.2.2 Wave Propagation in Soil	11
2.2.3 Attenuation Mechanisms.....	14
2.2.4 Receiver Response	16
2.3 Review of Modelling Techniques.....	18
2.3.1 Analytical Approaches	18
2.3.2 Numerical Approaches.....	18
2.4 Vehicle Modelling Approaches	21
2.4.1 Constant axle load models	21
2.4.2 Randomly Varying axle load models.....	22

2.4.3	Detailed multi-body vehicle models	23
2.5	Vibration Mitigation Strategies	24
2.5.1	Vehicle-Based Measures	24
2.5.2	Track-Based Measures	27
2.5.3	Measures in Propagation Path	29
2.5.4	Special Focus: Elastic Track Elements	32
2.6	Chapter Summary	34
CHAPTER 3		
METHODOLOGY		35
2.1.	Introduction to the Chapter	35
2.2.	Overview and Research Design	35
2.3.	Track Modelling	36
2.3.3.	Geometric Model	36
2.3.4.	Material Model	38
2.3.5.	Layer Interactions	38
2.3.6.	Mesh Strategy	39
2.3.7.	Boundary Conditions	41
2.4.	Vehicle Modelling	43
2.4.3.	Lumped Mass Model	44
2.4.4.	Wheel Rail Coupling	45
2.4.5.	Implementation in the FEM software	46
2.5.	Validation Strategy and Datasets	47
2.5.3.	Reference Field Measurement Studies	47
2.5.4.	Validation Metrics	49
2.6.	Chapter Summary	50
CHAPTER 4		
FE MODEL VALIDATION AND DESIGN OF PARAMETRIC STUDY		51
4.1.	Introduction	51
4.2.	Validation of 3D FE Model	51
4.2.1.	Velocity Time Histories	51
4.2.2.	Frequency Content of Vertical Velocity	53
4.2.3.	Root Mean Square Velocity (Running)	55
4.2.4.	Vibration Attenuation	56
4.2.5.	Limitations of the FE Model	58

4.3. Design of Parametric Study	59
4.3.1. Optimizing Vehicle Model to Improve Simulation Efficiency	59
4.3.2. Selection of Soil Conditions and Baseline Response	60
4.3.3. Selection of Values for the Parametric Study	62
4.4. Chapter Summary	63
CHAPTER 5	
UNDER SLEEPER PAD: PARAMETRIC STUDY AND PERFORMANCE	
EVALUATION	64
5.1. Introduction	64
5.2. Effect of Under Sleeper Pad on Soft Soil	64
5.2.1. Effect on the Waveform	64
5.2.2. Peak Decay and Quantification of Loss Along the Free Field	66
5.2.3. Frequency-Specific Analysis of the Loss	69
5.3. Effect of Under Sleeper Pad on Stiff Soil	71
5.3.1. Effect on the Waveform	71
5.3.2. Peak Decay and Quantification of Loss Along the Free Field	72
5.3.3. Frequency-Specific Analysis of the Loss	75
5.4. Chapter Summary	76
CHAPTER 6	
BALLAST MATS: PARAMETRIC STUDY AND PERFORMANCE	
EVALUATION	77
6.1. Introduction	77
6.2. Effect of Ballast Mat on Soft Soil	77
6.2.1. Effect on the Waveform	77
6.2.2. Peak Decay and Quantification of Loss Along the Free Field	79
6.2.3. Frequency-Specific Analysis of the Loss	80
6.3. Effect of Ballast Mat on Stiff Soil	82
6.3.1. Effect on the Waveform	82
6.3.2. Peak Decay and Quantification of Loss Along the Free Field	83
6.3.3. Frequency-Specific Analysis of the Loss	86
6.4. Chapter Summary	87
CHAPTER 7	
CONCLUSIONS AND RECOMMENDATIONS	
7.1. Conclusion	88

7.2. Recommendations.....	90
7.3. Future Work.....	91
REFERENCES.....	92

LIST OF FIGURES

Figure 1.1:	Transmission of train induced ground vibrations from source to receiver (Ouakka et al., 2022).....	2
Figure 1.2:	Vibration measurement at the Koralawella site.....	3
Figure 1.3:	Measured vertical velocity of the house floor during passage of Train T9 at 2 seconds interval.....	4
Figure 2.1:	Historical maximum train speed timeline (UIC High Speed Department. High speed lines in the world; 2013).....	8
Figure 2.2:	Schematic and realistic mass/suspension vehicle system of a train.....	11
Figure 2.3:	Seismic wave propagation (Woods, 1968).....	12
Figure 2.4:	The three main wave forms in the ground association with the transmission (a) Rayleigh (b) Compressional (c) Shear (Peterie et al., 2014).....	14
Figure 2.5:	Exposure response curves following from a meta-analysis carried out in project, Cargovibes (Waddington et al., 2015).....	17
Figure 2.6:	A 2.5D approach.....	20
Figure 2.7:	A coupled BE/FE approach.....	20
Figure 2.8:	Track deflection due to the moving of X2000 HST (a) time history (b) frequency content (Kouroussis et al., 2014).....	22
Figure 2.9:	Simple dynamic model for multi body spring/damp system (a) 1/2 car body (b) 1/4 car body.....	23
Figure 2.10:	Different orders of wheel out-of-roundness (a) First-order (b) Second order (c) Third order (Ouakka et al., 2022).....	25
Figure 2.11:	Effects of changing wheel resilient material stiffness on ground vibration level (a) at 4 m distance (b) at 10 m distance (c) at 16 m distance (d) at 20 m distance (Kouroussis et al., 2012).....	26
Figure 2.12:	General frequency ranges of different excitation mechanisms (Ouakka et al., 2022).....	27
Figure 2.13:	Embedded railway system.....	28
Figure 2.14:	Resilient fastening system (a) high-resilience fastening system and (b) resilient pre-load fastening (Panodral, 2020).....	28
Figure 2.15:	Numerical visualization of peak particle velocity of ground during a HST passage for track at grade (left) and track on embankment (right).....	30
Figure 2.16:	Components of Under Sleeper Pads.....	32
Figure 2.17:	Illustration of BM and its installation.....	33
Figure 3.1:	Framework for the development of 3D FE model.....	36
Figure 3.2:	Basic methodological framework used in the study.....	36

Figure 3.3:	Half-track soil profile used in FE model with layer thickness....	37
Figure 3.4:	Basic layout of the track components.....	37
Figure 3.5:	Track-soil layer interactions.....	39
Figure 3.6:	Assigned connector elements for rail pad.....	39
Figure 3.7:	Comparison of wave transmission in a model with different element sizes.....	40
Figure 3.8:	Complete overview of the boundary conditions of the FE model	41
Figure 3.9:	Viscous dashpot for absorbing energy of S-waves (Cs) and P-waves (Cp).....	42
Figure 3.10:	Configuration of Thalys HST.....	43
Figure 3.11:	Lumped Mass Model.....	44
Figure 3.12:	Measured PPV as a function of distance away from track (Degrande & Schillemans, 2001).....	47
Figure 3.13:	Vertical velocity time histories at (a) 4m (b) 8m from track and frequency content of vertical velocity at (c) 4m (d) 8m during passage of Thalys HST at 314 kmph (Degrande & Schillemans, 2001).....	48
Figure 3.14:	Vertical velocity time histories at 16m from track center during passage of InterCity at 156kmph (Lombaert & Degrande, 2009b).....	48
Figure 3.15:	Vertical velocity time histories at 16m from track centre during passage of Thalys HST at 218 kmph (Lombaert & Degrande, 2009b).....	49
Figure 4.1:	Vertical velocity time histories at receivers at (a) 4 m (b) 8 m (c) 12 m (d) 16 m.....	52
Figure 4.2:	Frequency content relevant to vertical velocity at receivers (a) 4 m (b) 8 m (c) 12 m (d) 16 m.....	54
Figure 4.3:	Comparison of running V_{rms} of response from FE model with field data at receivers (a) 4m (b) 8m (c) 12m (d) 16m.....	55
Figure 5.1:	Velocity time histories of varying USP thickness on soft soil at (a) 4 m receiver and (b) 16 m receiver.....	65
Figure 5.2:	Velocity time histories of varying USP stiffness on soft soil at (a) 4 m receiver and (b) 16 m receiver.....	66
Figure 5.3:	PPV attenuation along the free field of soft soil with varying (a) USP thickness and (b) USP stiffness.....	67
Figure 5.4:	One-third octave spectrum of the insertion loss of vertical velocity across USP thickness on soft soil at (a) 4 m receiver and (b) 16 m receiver.....	69
Figure 5.5:	One-third octave spectrum of the insertion loss of vertical velocity across USP stiffness on soft soil at (a) 4 m receiver and (b) 16 m receiver.....	70

Figure 5.6:	Velocity time histories of varying USP thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver.....	71
Figure 5.7:	Velocity time histories of varying USP stiffness on stiff soil at (a) 4 m receiver and (b) 16 m receiver.....	72
Figure 5.8:	PPV attenuation along the free field of stiff soil with varying (a) USP thickness and (b) USP stiffness.....	73
Figure 5.9:	One-third octave spectrum of the insertion loss of vertical velocity across USP thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver.....	75
Figure 5.10:	One-third octave spectrum of the insertion loss of vertical velocity across USP stiffness on stiff soil at (a) 4m receiver and (b) 16m receiver.....	76
Figure 6.1:	Velocity time histories of varying BM thickness on soft soil at (a) 4 m receiver and (b) 16 m receiver (right).....	78
Figure 6.2:	Velocity time histories of varying BM stiffness on soft soil at (a) 4 m receiver and (b) 16 m receiver.....	78
Figure 6.3:	PPV attenuation along the free field of soft soil with varying (a) BM thickness and (b) BM stiffness.....	79
Figure 6.4:	One-third octave spectrum of the insertion loss of vertical velocity across BM thickness on soft soil at (a) 4m receiver and (b) 16m receiver.....	81
Figure 6.5:	One-third octave spectrum of the insertion loss of vertical velocity across BM thickness on soft soil at (a) 4m receiver and (b) 16m receiver.....	81
Figure 6.6:	PPV attenuation along the free field of stiff soil with varying (a) BM thickness and (b) BM stiffness.....	84
Figure 6.7:	One-third octave spectrum of the insertion loss of vertical velocity across BM thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver.....	86
Figure 6.8:	One-third octave spectrum of the insertion loss of vertical velocity across BM thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver.....	86

LIST OF TABLES

Table 1.1:	Interim vibration standards for Type 3 structures subjected to vibration due to vehicle movements (CEA, 2008).....	3
Table 1.2:	Recorded PPV on the floor of the structure during the passage of trains.....	4
Table 2.1:	Summary of the prior local studies.....	10
Table 2.2:	Material damping coefficients for different soil types (Bahrekazemi, 2004).....	16
Table 2.3:	Perception of vibration according to DIN, after (Smith et al., 2015).....	17
Table 2.4:	Different types of wave barriers and their effectiveness in attenuating ground vibrations.....	31
Table 2.5:	Classification of Under Sleeper Pads based on their stiffness (UIC).....	32
Table 3.1:	Summary of material properties of the track elements (Degrande & Schillemans, 2001).....	38
Table 3.2:	Element types and sizes of different parts of the FE model.....	40
Table 3.3:	Dashpot coefficients based on their location.....	43
Table 3.4:	Thalys HST dynamic properties (Kouroussis et al., 2011).....	43
Table 3.5:	InterCity train mass properties (Lombaert & Degrande, 2009b)....	44
Table 4.1:	Percentage variation of PPV and Vrms between FE model response and field data.....	58
Table 4.2:	Material properties for considered soft and stiff soil conditions (Bongini et al., 2012).....	61
Table 4.3:	Selected values for the parametric study.....	63
Table 5.1:	Quantified vibration reduction of varying USP thickness in soft soil.....	68
Table 5.2:	Quantified vibration reduction of varying USP stiffness in soft soil.....	68
Table 5.3:	Quantified vibration reduction of varying USP thickness in stiff soil.....	74
Table 5.4:	Quantified vibration reduction of varying USP stiffness in stiff soil.....	74
Table 6.1:	Quantified vibration reduction of varying BM thickness in soft soil.....	80
Table 6.2:	Quantified vibration reduction of varying BM stiffness in soft soil	80
Table 6.3:	Quantified vibration reduction of varying BM thickness in stiff soil.....	85
Table 6.4:	Quantified vibration reduction of varying BM stiffness in stiff soil	85

LIST OF ABBREVIATIONS

BM	Ballast Mat
CEA	Central Environmental Authority
FE	Finite Element
FEM	Finite Element Method
HST	High-Speed Train
IL	Insertion Loss
ISO	International Organization for Standardization
P-wave	Primary (compressional) seismic wave
PPV	Peak Particle Velocity
S-wave	Secondary (shear) seismic wave
USP	Under Sleeper Pad
UIC	International Union of Railways
V_{rms}	Root-Mean-Square Velocity
V_s	Shear-wave velocity
V_p	Compressional-wave velocity

CHAPTER 1

INTRODUCTION

1.1 Background of the Study

Over the years, global transportation trends have taken different shapes. The constant rise in demand for both passenger and freight railway carriage is a key highlight. One way to satisfy this continuous rise in demand for increased capacity has been to introduce high-speed passenger routes. This approach facilitates freeing up capacity on the existing networks for classic rail and also provides extra capacity for freight trains. On the other hand, increased capacity has led to higher vehicle load on the track. This, in combination with the increased speed of railway carriage, directly contributes to railway vibration – a growing engineering challenge, especially in densely populated urban environments (D. P. Connolly et al., 2015a). This vibration generated at the rail track can be transmitted through various track components and soil for considerable distances, eventually reaching humans and buildings in proximity. This leads to nuisance among the public, structural damages on buildings, and possibly causing disruptions to primary functions of buildings that rely on sensitive equipment (e.g. hospitals, manufacturing plants) (Kouroussis et al., 2014).

The interactions among trains, track, and subsoil generate ground-borne vibration before transmitting through the ground, which may reach the foundation of the building. Then the building elements start responding to the vibration of the foundations, and it may be observed either with oscillation of floors and walls or may be sensed by building occupants as perceivable vibrations and audible noise, as illustrated in Figure 1.1. Ground-borne vibration is defined by ISO 14837-1 as “vibration generated from the pass-by of the vehicle on the rail, propagated through the ground or structure into a receiving building” and this is typically associated with a frequency range varying between 1 and 100 Hz (Paul de Vos, 2017).

While this ground-borne vibration prevails as a key concern in rapidly developing high-speed rail transportation, it is essential that continuous efforts are taken to expand the understanding of track structural design and soil mechanics disciplines. To achieve this, specialised knowledge of track behaviour and soil response under varying train speeds is necessary (Li et al., 2018). There are numerous methods adopted to understand this track behaviour and soil response in previous studies. These include laboratory experiments, field investigations, analytical solutions, boundary element method, 2D and 3D finite element method (FEM). Each of these method carry its own advantages and drawbacks, but 3D FEM is capable of simulating the non-linear behaviour of complex geometries subjected to dynamic train loads with reasonable accuracy despite being computationally expensive (D. P. Connolly et al., 2015a; Kouroussis et al., 2014; Li et al., 2018).

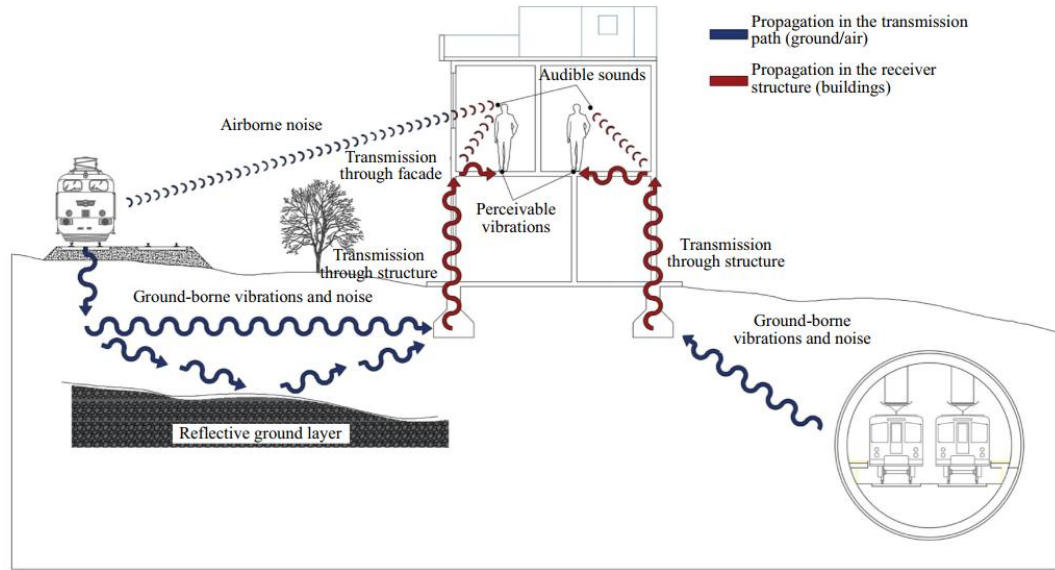


Figure 1.1: Transmission of train induced ground vibrations from source to receiver (Ouakka et al., 2022)

Understanding track behaviour and soil response allows practitioners to adopt different techniques to mitigate vibration propagation in vibration-sensitive regions. These techniques are typically in two categories: active and passive isolation. Isolation of vibration within the location of origination or within track structure generally refers to active isolation, which includes methods like adapting floating slab track, rail pads or resilient wheels. Passive isolation includes vibration screening measures that are adopted in the vibration propagation path, mostly at locations that are close to vibration-sensitive regions (D. Connolly, Giannopoulos, Fan, et al., 2013). Vibration mitigation methods that are to be adopted vary significantly based on various conditions like vibration intensity, track geometry, surrounding area, etc. The majority of passive isolation techniques often require considerable space to be adopted, such as open trenches and soil stiffening in propagation paths. So, they may not be feasible to be used, especially in urban areas. On the other hand, they have been widely studied in previous literature. Under sleeper pads (UPS) and ballast mats (BM) are feasible solutions that can be adopted in such conditions, and they are capable of reducing the propagation of vibrations generated at the rail and the sleeper during the passage of the train. Additionally, these measures have not been widely studied and require further investigations to understand their effectiveness in reducing train-induced ground vibration under different field conditions.

Therefore, this study is focused on developing a comprehensive finite element (FE) model that can effectively simulate propagation of vibrations in a soil domain during passage of trains. Using the validated model, effect of different parameters of UPSs and BMs on attenuation of ground vibrations is studied to contribute to the existing knowledge of these mitigation measures.

1.2 Research Problem

With the advancement of the rail transport system, freight and passenger trains have improved to a level that they are capable of carrying more capacity at a higher speed. This has led to environmental concerns regarding noise and vibrations, which are generally more perceivable, and the community and authorities are more concerned about. Ground-borne vibration caused by train passage can lead to discomfort and interference with human activities of neighbouring communities when they exceed vibration velocity and acceleration values above the specified threshold levels by authorities. Apart from the residential community, commercial and industrial communities that are related to vibration-sensitive equipment and activities are also severely affected by these vibrations (Avillez et al., 2012).

In order to understand the context of this problem, a field measurement of vibrations was conducted inside temporary dwellings located in close proximity to the railway line. The permissible vibration limits for such structures are specified in Table 1.1 as per CEA (2008). These dwellings fall under the Type 3 structural category, as they are built using lightweight construction materials and are not designed to withstand seismic activity (ISO 4966:1990E). The field measurements were carried out near Koralawella railway station, situated between Moratuwa and Panadura stations, where the selected residential dwelling is positioned just 3 metres from the centre line of the railway track (Figure 1.2). Measurements were taken during the peak evening hours, from 5:00 PM to 6:00 PM, to coincide with frequent train passages.

Table 1.1: Interim vibration standards for Type 3 structures subjected to vibration due to vehicle movements (CEA, 2008)

Vibration Frequency (Hz)	Vibration in PPV (mm/s)
0 – 10	2.0
10 – 50	4.0
Over 50	8.0



Figure 1.2: Vibration measurement at the Koralawella site

Vibration data were recorded during the passage of nine trains, and the corresponding PPVs are presented in Table 1.2. The observed train speeds ranged from 29 km/h to 60 km/h. Since most of the trains (except Train T3) did not stop at the Koralawella Railway Station, they were travelling at speeds of approximately 45–60 km/h when passing the experimental location. Notably, six out of the nine events recorded vertical PPV values exceeding 12 mm/s, which is 50% higher than the maximum limit set by the CEA. It was also observed that the relationship between train speed and vibration intensity did not follow a strictly linear trend. For example, Train IDs T2, T6, and T8 recorded PPV values of 4.76, 5.40, and 5.05 mm/s at speeds of 50, 51, and 50 km/h, respectively. Although train speed is a primary factor influencing vibration levels, other variables, such as the weight of the engine and passenger cars, as well as the number of passengers, likely contribute to these deviations.

Figure 1.3 illustrates the detailed variation of PPV measured at 2-second intervals inside the temporary dwelling during the passage of Train T9. It is evident that during most of the train's passage, the measured vibration levels exceeded the CEA's permissible limits. These findings clearly indicate that train-induced vibrations pose a significant concern and must be carefully evaluated and mitigated in future railway development to minimise impacts on nearby communities and the environment.

Table 1.2: Recorded PPV on the floor of the structure during the passage of trains

Train ID	Train speed (km/h)	Vertical PPV (mm/s)
T1	46	15.38
T2	50	4.76
T3	29	12.81
T4	55	12.56
T5	60	12.96
T6	51	5.40
T7	48	12.91
T8	50	5.05
T9	58	22.45

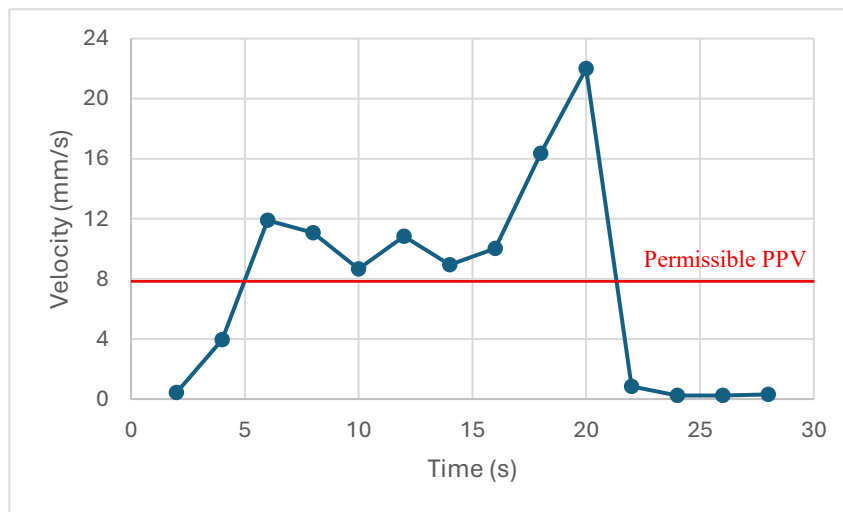


Figure 1.3: Measured vertical velocity of the house floor during passage of Train T9 at 2 seconds interval

Another concern is that while the urban built-up areas are mostly affected by train-induced vibrations, vibration mitigation measures that can be adopted in such areas are limited. Either active isolation techniques need to be used or else the track structure needs to be improved such that vibrations get attenuated within the track structure. While active isolation techniques like resilient wheels, floating slab tracks being very expensive and take considerable time to implement, measures like USPs and BMs can be effectively used to attenuate vibrations generated within the track structure.

To implement USP and BM, their behaviour and effectiveness need to be understood under different soil conditions and train characteristics. Previous studies have focused more on the variation of vibration and load transmission within the track structure, while its effect on near field and far field of the track surrounding is not fully explored.

1.3 Aim and Objectives

Based on the field measurements and the identified gaps in current knowledge, this study aims to develop and validate a 3D FE model to investigate the propagation of train-induced ground-borne vibrations, and to evaluate the attenuation achieved by USPs and BMs

The aim of the study is achieved by achieving the following objectives.

1. Develop and validate a 3D FE model of the coupled vehicle-track-soil system capable of simulating moving train loads.
2. Investigate the effect of USPs, under different parameter combinations on vibration control.
3. Investigate the effect BMs, under different parameter combinations on vibration control.

1.4 Scope of the study

This study will revolve around developing a FE model coupling train-track-soil system using ABAQUS software package. Developed model is validated against several field experiments conducted globally on different train configurations at several speeds. For the validation metrics, velocity time-histories, peak particle velocity (PPV) along soil domain, and frequency content of the excitation were used.

The validated model is then used to study the effect of varying parameters of USPs and BMs. For both USPs and BMs, thickness, stiffness, and damping were considered for the parametric study, and they were varied between practical values based on previous literature. Furthermore, a parametric study will be conducted on two soil conditions, namely, soft and stiff, by changing soil properties, and the vibration attenuation effect will be studied on the near-field (0-10m) and far-field (>10m) of the soil domain.

The developed FE model includes a realistic track geometry consisting of rails, sleepers, ballast, sub-ballast and subgrade, efficiently demonstrating a real track.

Special consideration was given in modelling the boundary surfaces, as the soil needs to be modelled as infinitely spanning elements. For this, viscous boundaries were utilised on all truncated faces of each ground layer. Considering the highly dynamic nature of the simulation, the Explicit solver of the ABAQUS software package was used to maintain computational efficiency while maintaining reasonable accuracy.

As for the assumptions and limitations of this study, soil elements were modelled as small-strain, linear elastic with Rayleigh-type damping. Additionally, rail irregularity was treated parametrically, and ballast degradation and plasticity were not modelled.

1.5 Significance of the study

This study provides a validated FE model capable of simulating vibration propagation in the soil domain during the passage of trains at different speeds. Since the validation is carried out for several scenarios, it can be effectively applied for different ground conditions with a few adjustments to the model.

Additionally, it fills a knowledge gap about the effect of varying parameters of USPs and BMs in attenuating train-induced ground-borne vibrations. Since the parametric study is conducted on soft and stiff soil conditions, practitioners can effectively utilize the results of this study in selecting vibration mitigation measures based on encountered soil conditions.

The results of this study are given in validated metrics such as velocity time histories, PPV decay with distance, V_{rms} , and frequency content (including third-octave bands), aligning with global guidelines.

This study can be effectively extended to include soil-structure interaction, more realistic roughness spectra, and transition zones, facilitating a deep dive into the subject area without the need to rebuild the core model.

1.6 Guide to thesis

This thesis mainly comprises 5 sections. Chapter 1: Introduction includes basic background of the study area along with the underlying research problem and motivation for this study. It also explains the aim and objectives of the study and its scope of significance. Chapter 2: the literature review provide a comprehensive understanding of the study topic. It includes every detail of the theories, approaches, and methods used during the study and what has been explored and studied under this subject area globally until now, and finally highlights existing knowledge gaps. Chapter 3: methodology provides an overall understanding of the data and measurements used in the study, both performed by the author and those which have been taken from previous studies. Additionally, it will give a comprehensive understanding of the approach used by the author in solving the problem, such that it can be replicated by another party from reading it. Chapter 4: FE model validation and design of parametric study provide details on validation of the developed FE model using several metrics and how the parametric study is designed. Chapter 5 and Chapter

6 provide results and findings of the parametric study on USPs and BMs respectively. Chapter 5: conclusion will summarise the broad study briefly and the conclusions that can be derived from the study findings. It will also discuss the recommendations that can be provided based on the study and future work related to this research problem.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter introduces the upcoming sections of the thesis. It starts by explaining how ground-borne railway vibrations are generated and travel through the track into the surrounding soil, helping to clarify the mechanisms addressed in later models. Next, the chapter reviews the main modelling techniques used in earlier research, from simple analytical methods to modern numerical approaches, and explains why a 3D time-domain FE method was chosen for this study. It then classifies mitigation strategies based on where they are applied (vehicle, track, ground, receiver) and highlights the roles of resilient elements, USPs, and BMs within the broader range of available tools. Additionally, the chapter discusses practical issues like boundary treatment and soil characterization, which are important for comparing different studies. The aim is not to provide every detail but to give a clear overview of what to focus on in the methods and results of the following chapters.

2.1.1 Growth of Railway Transport and High-Speed Rail

Railway transport has been the key mode of inland transport in freight and passenger transport, considering its efficiency, time conservation, and low cost. Hence, passenger and freight railway transport demand has increased over the last 50 years. To account for this, creating high-speed passenger routes has emerged as a proven way of freeing up capacity on existing rail networks and providing extra capacity for freight trains (D. P. Connolly et al., 2015b). The evolution of train speeds since the 1960s is shown in Figure 2.1.

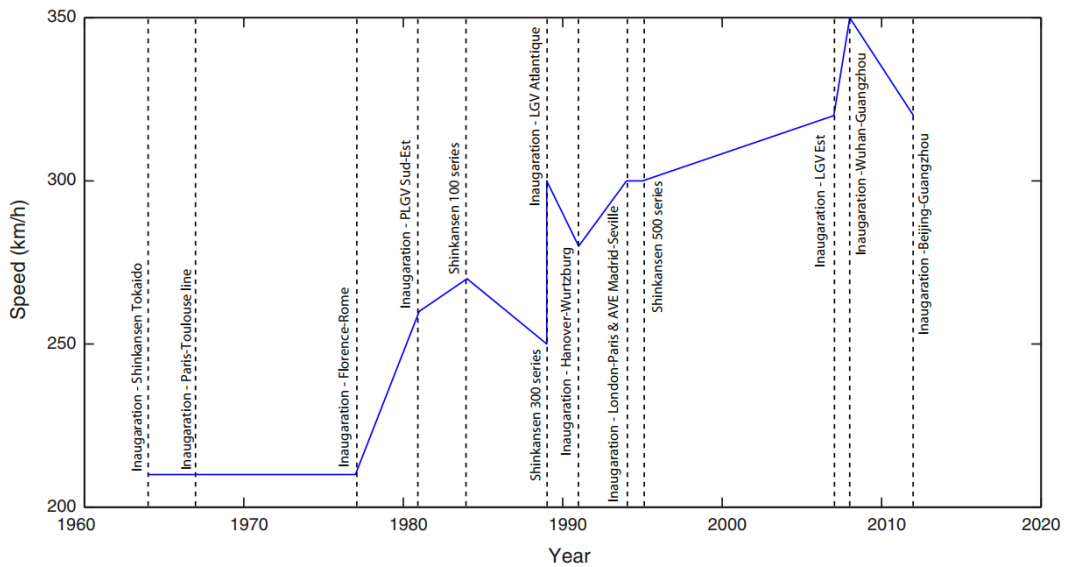


Figure 2.1: Historical maximum train speed timeline (UITC High Speed Department. High speed lines in the world; 2013)

With the growth of capacity and the speed of trains, railway-induced vibrations have now become an emerging engineering challenge, especially in densely populated urban areas. The higher speeds and heavier loads eventually affect the vibration levels within both train and track, affecting passenger comfort as well as the surrounding environment, particularly in urbanised areas. (D. P. Connolly et al., 2015b).

2.1.2 Ground-Borne Vibration and Noise: Mechanisms and Human/Environmental Impact

Interactions between trains, track and subsoil generate ground-borne vibration. It transmits through the ground and eventually reaches the building foundation, and the building responds to it. Vibration is then transmitted through the building structure and finally observed as a perceivable vibration of the floor by its occupants. Ground-borne vibration is typically associated with a frequency range from 1 to 100 Hz. As per ISO 14837-1, Ground-borne railway noise is defined as “noise generated inside a building by ground-borne vibration generated from the pass-by of a vehicle on rail” (Paul de Vos, 2017).

The annoyance caused by these ground-borne vibrations and also air-borne vibrations in the daily lives of exposed people has been a serious concern in public. Recent studies have highlighted that apart from the annoyance, there is a possibility of leaving permanent health effects on exposed people. A study done by Croy et al. (2013) Demonstrated that passage of trains can accelerate the heart rate up to 3bpm during sleep under some circumstances, and in the long run, this may affect cardiovascular functioning of exposed people.

2.1.3 Prior Local Studies on Train Induced Ground Vibrations

There are several fundamental studies conducted at the Department of Civil Engineering, University of Moratuwa, Sri Lanka that directly contribute to understanding vibration mitigation mechanisms and measurement methodologies in the Sri Lankan context. These studies, conducted over the past years (2021–2023), have developed practical instrumentation, experimental frameworks, and parametric analyses that provide an essential bridge between global knowledge and local applications. The findings from these investigations have informed the experimental design, material characterization, and boundary condition assumptions employed in this study as well. Table 2.1 provides an overview of those studies.

Table 2.1: Summary of the prior local studies

Author	Primary scope and method	Key findings
Thadsanamoorthey (2023)	Designed, tested and calibrated a portable wireless vibration monitoring gadget for railway environments. FEM study on effect of soil types and train speed in vibration generation	Developed a low-cost, battery-powered sensor unit capable of real-time vibration recording and wireless data transfer; validated measurements against reference accelerometers, demonstrating sufficient precision for engineering assessment of train-induced vibrations. Results from validated FE model using field measurements highlighted that vibration intensity significantly vary with the soil types and higher train speeds lead to high vibration generation with resonance effect. This resonance frequency is influenced by the elastic modulus of the soil
Sankeeth (2022)	Assess the effect of wooden and concrete sleepers on train induced ground vibrations and role of rail pad in vibration isolation using a 3D FE model	The PPV at sleeper has reduced by half with the insertion of rail pad and in the free field at 1 m from the track, PPV reduction is recorded as approximately 3 times. The PPV in the concrete sleeper is slightly higher than that of the wooden sleeper. In the free-field, vibration levels shown to be high where the wooden sleepers have been used.
Abiram (2021)	Effect of embedded wave barriers in screening of train induced ground vibrations is studied using validated 3D FE model. Performance of the barrier with varying properties of the infill material is investigated.	Overall embedded wave barrier isolation provides superior vibration screening Infill material of the trench should have lowest elastic modulus possible to get higher vibration screening For each depth of embedded barrier layer, there is an optimum unit weight for the material, that provides maximum vibration screening Void ratio of infill material plays a major role in overall performance of the barrier where higher void ratio provides higher screening.

Arachchi (2023)	Identifying the effect of rail pad and its properties on vibration control using validated 3D FE model	Use of high-density polyethylene as rail pad material leads to substantial reduction of vibration propagated to the track. Thickness of the rail pad does not have a considerable impact on vibration control.
-----------------	--	--

2.2 Theoretical Background

2.2.1 Vibration Generation in Railways

When a train moves over a rail, contact point between train wheel and rail generates railway vibration and noise. Forces generate at this point can be classified for as quasi-static and dynamic components (Lombaert & Degrande, 2009a). Quasi-static forces typically arise due to the weight of the train and are independent of train speed. Track response and near field up to a distance of one quarter of wavelength is dominated by these forces (D. Thompson, 2009). Source of vibration generation, the vehicle can be schematically shown as a multi body mass/suspension system as in Figure 2.2. This mainly comprise of 3 masses: car body, bogie, and wheel and each those masses connected together with spring-damper suspension system. On the other hand, dynamic excitation is directly time dependent and generated from factors like changes in stiffness due to sleeper placement, irregularities at the wheel/rail interface and the soil support conditions (D. P. Connolly et al., 2015b)

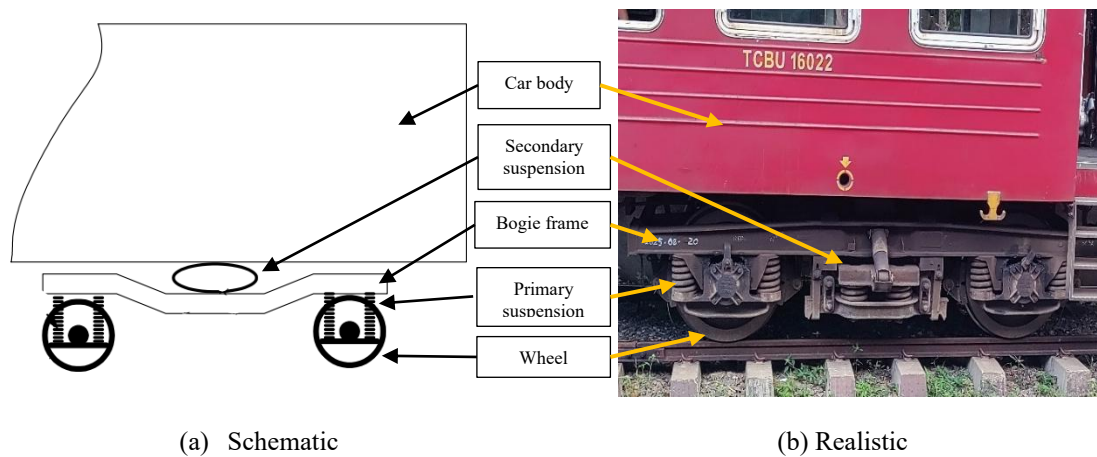


Figure 2.2: Schematic and realistic mass/suspension vehicle system of a train

2.2.2 Wave Propagation in Soil

Once vibration is generated at the wheel/rail interface, it transfers through the track into the free field in the form of waves. These waves are categorized as either body waves or surface waves. Surface waves propagate along a soil surface, while body

waves propagate beneath the soil surface. Figure 2.3 illustrates the wave propagation characteristics more clearly (D. P. Connolly et al., 2015b).

Out of all waves, compressional waves (P-waves) are the fastest and it propagates in a longitudinal direction. Shear waves (S-waves) propagate in a transverse direction and travel slower than P-waves but faster than Rayleigh waves. Rayleigh waves are the slowest type of seismic waves. P and S wave speeds can be calculated using equation 2.1 and equation 2.2, while Rayleigh wave speed can be calculated using several formulas. Although there can be other theoretically possible wave types associated with vibration propagation, in this study only Compressional waves, Shear waves, and Rayleigh waves are focused since they are responsible for the transmission of majority of excitation energy (Rayleigh waves transmit approximately two third of total excitation energy) (Woods, 1968).

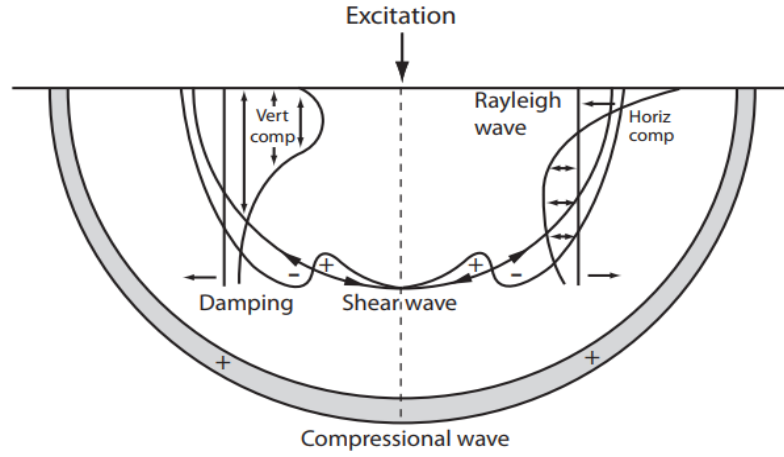


Figure 2.3: Seismic wave propagation (Woods, 1968)

$$C_p = \sqrt{\frac{\lambda + 2\mu}{\rho}} \quad (2.1)$$

$$C_s = \sqrt{\frac{\mu}{\rho}} \quad (2.2)$$

Where ρ is density, λ is bulk modulus and μ is the shear modulus. Nelson and Saurenman (1983) presented Equation 2.3 to understand the attenuation of wave intensity in linear elastic half space

$$v = v_0 \left(\frac{r}{r_0} \right)^{-n} \cdot e^{-\alpha (r-r_0)} \quad (2.3)$$

Where v_0 is particle velocity at the source, r_0 is the distance from the source to the reference point on the ground, r is the distance from the source to receiver, n is the power of geometric attenuation and α is the factor for material damping.

In the analysis of soil behaviour numerous parameters are utilized and each of those parameters are used under different circumstances. Even though soil characteristics are measured and presented as moisture content, particle size distribution, liquid and plastic limits and so on, numerical analysis of wave propagation through soil can be explained using few key soil properties, namely, density, Poisson's ratio, Young's modulus and damping. In fact, the effect of other soil characteristics are often accounted by changing the values of those 4 properties. (D. P. Connolly et al., 2015b)

Rayleigh waves

In investigation of train-induced ground vibrations, Rayleigh waves can be considered as the most significant type of waves. Propagation of these waves occur along the ground surface where both horizontal and vertical motion of particles occur within the plane of wave propagation (Figure 2.4(a)). Apart from the elliptical motion of particles, Rayleigh waves are characterised by exponential decrease of wave amplitude with the soil depth (Eitzenberger, 2008).

Under soft soil conditions, considering the efficiency of Rayleigh waves in energy transmission along the surface, they become paramount of important. The effect of these waves dramatically increases when the speed of the train equals or exceed the Rayleigh wave velocity of the supporting soil (Auersch, 2021). This phenomenon is called 'approaching the critical velocity'. Bahrekazemi, (2004) showed that when the train speed exceeds the critical velocity of the ground, the vibration levels of ground soil can be increased dramatically. This condition can be critical for instances where the track is supported by soft soil, where the Rayleigh wave velocities can get reduced to about 50-150 m/s.

The Rayleigh waves in train-induced ground vibrations are of the frequency content ranging from 1-80 Hz generally. Considering the frequency dependent attenuation in soil, excitation with lower frequencies typically become prominent at greater distance from the track. Studies have shown that in soft soils, the significant frequency range for ground vibration extends from 0-10 Hz for cohesive soils, with higher frequencies observed in granular materials (UIC, 2007).

Body Waves

The other main type of waves found in train induced ground vibrations are: compressional (P-waves) and shear (S-waves) components. These waves propagate through the soil medium and has a critical role in transmission of excitation.

Compressional waves can be distinguished by its direction of oscillation, which occur parallel to the direction of wave propagation (Figure 2.4(b)), resulting in compression and rarefaction of the soil medium. Out of all waves associated with seismic wave propagation, these waves possess the highest speed. This propagation velocity depends on the density of the ground, typical velocity ranging from 800 m/s to 1500 m/s for soils ranging from average stiff soil and water saturated soil (Paul de Vos, 2017). P-

waves are significant in the near field region of railway applications since the vertical vibration response primarily controlled by them (Bahrekazemi, 2004).

Shear waves can be distinguished by their particle motion which is perpendicular to the direction of propagation (Figure 2.4(c)) and exhibit lower velocities compared to

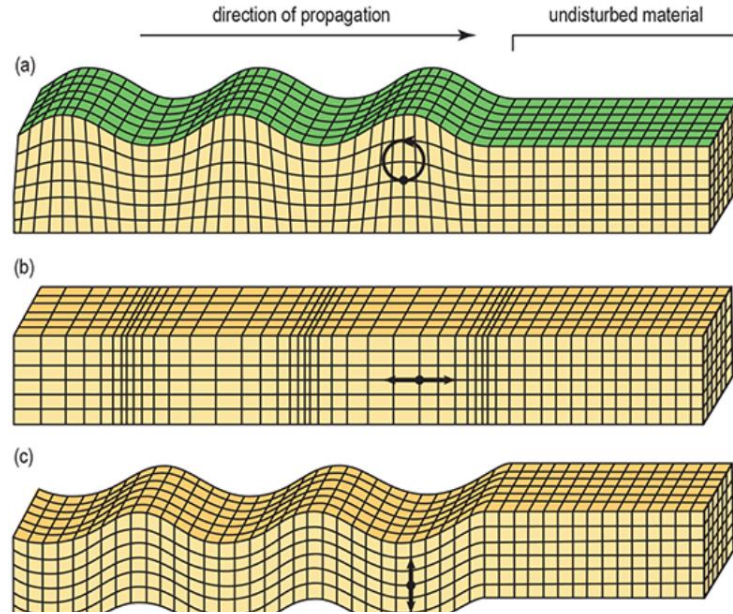


Figure 2.4: The three main wave forms in the ground associated with the transmission (a) Rayleigh (b) Compressional (c) Shear (Peterie et al., 2014)

P-waves. Velocities typically range from 30-500 m/s and are considerably dispersive (Paul de Vos, 2017). These waves are crucial for transmitting horizontal ground motions and contribute significantly to structural vibrations. The importance of shear waves increases with distance from the source, as they maintain their energy more effectively than compressional waves in certain soil conditions (D. P. Connolly et al., 2015b).

The interaction between P-waves and S-waves in layered soil systems can result in complex wave interference patterns, leading to amplification or attenuation of vibrations depending on the soil stratification. Body waves are subject to reflection and refraction phenomena at soil layer interfaces, which can significantly alter the vibration transmission characteristics (Svinkin, 2008).

2.2.3 Attenuation Mechanisms

Geometric Damping

Geometric damping results from reduction of the wave amplitude as it propagates through a specific medium away from the source. This attenuation behaviour occurs completely due to the geometry and is independent of material properties. Geometric damping constitutes of mainly 2 components; spherical and cylindrical spreading.

Spherical spreading occurs when the waves generated from a point source and propagate in three-dimensional space. The amplitude reduction for this scenario is proportional to $1/r$, where r is the distance from the source. This mechanism is particularly relevant for body waves emanating from discrete sources such as wheel-rail contact points (Massarsch & Fellenius, 2008).

Cylindrical spreading phenomena is more relevant for the field of train-induced ground vibrations, where the linear nature of the track leads to line sources. For this case, reduction of wave amplitude is proportional to $1/\sqrt{r}$. Cylindrical wave propagation is particularly relevant for surface waves generated by moving trains, as the continuous nature of the loading creates an effective line source along the track alignment (Chai et al., 2020).

The transition between different geometric spreading patterns depends on the source characteristics and the dominant wave type. At near distances, spherical spreading may dominate for body waves, while at greater distances, cylindrical spreading becomes more important for surface waves. Woods (1968) established that Rayleigh waves dominate at distances greater than approximately 20 m from the track, where cylindrical spreading becomes the primary geometric damping mechanism (Bahrekazemi, 2004; Chen et al., 2019).

Material Damping

In contrast of geometric damping, material damping represents energy dissipation that take place within soil materials caused by internal friction and deformation during wave propagation. This phenomenon depends on the frequency and also change considerably with soil type, moisture content, and stress level.

For clayey soil, material damping is comparably higher than granular soils based on the higher water content and cohesive nature of the soil. Damping factor for clayey soil can vary from 0.02-0.08, with softer clays showing higher values. Clay soil is more effective in attenuating higher frequency components of seismic wave considering its frequency-dependent nature, ultimately leading the vibration spectrum to shift towards lower frequency range with increasing distance from the source (Chen et al., 2019; UIC, 2007).

Granular soils present lower material damping compared to cohesive soils. Intergranular friction and rearrangement of soil grains during cyclic loading facilitate the damping in granular materials. Compared to fin-grained soil, coarse-grained soils show comparably higher near-field attenuation considering their higher porosity and low density, which increase energy dissipation through internal friction (Sandqvist & Milicevic, 2020).

The material damping coefficient (α) in the classical attenuation equation varies with soil properties, vibration frequency, and source energy. Experimental data indicates that the coefficient depends on the dominant frequency of propagated waves, distance from the source, and soil stratification at the site. Small strain material damping in

soils typically ranges from 2-7%, though this can be significantly exceeded by radiation damping effects (Svinkin, 2008). In accordance with the Equation 2.3, Typical values for material damping coefficient for few soil types are given in Table 2.2.

Table 2.2: Material damping coefficients for different soil types (Bahrekazemi, 2004)

Soil type	Material damping coefficient , α (m^{-1})
Water-saturated clay	0.04 – 0.12
Loess and loessal soil	0.10
Sand and silt	0.04

2.2.4 Receiver Response

Starting from generation of train induced vibrations at the track, it propagates through soil media and eventually reach the foundation of nearby buildings. Vibration travels through the structural elements of the building affecting elements itself and also sensitive equipment attached to the structure or felt by the people in the building as perceivable vibration and audible noise.

Damage cause by train-induced ground vibration on buildings is not a highlighted topic since most of them are architectural damages and instance where structural damages were identified are quite rare (Ouakka et al., 2022). The vibration levels recorded in instances where architectural damage has occurred are generally higher than 5mm/s. this value exceeds the vibration intensities caused during train passage (approximately between 0.1-0.6 mm/s) by far (Paul de Vos, 2017). A state-of-the-art review by Nelson & Saurenman (1983) stated that the probability of a building being subjected to structural damage due to particle velocities less than 50mm/s is 5% and there are no reported incidents of the occurrence of structural damage due to vibration velocities less than 25mm/s. ISO 4866 (2010) provides detailed guidelines for vibration measurements and their impact on structures. As per this standard, there are numerous parameters that need to be considered in evaluating its effect on structures, namely, duration of dynamic excitation force, frequency and range of vibration intensity. Apart from that, structure-related factors such as type and condition of the structure, natural frequencies, damping, and the soil at the site.

Train-induced ground vibrations can interfere with the performance of sensitive equipment used in facilities such as laboratories, hospitals, and industrial buildings. Therefore, it is important that the vibrations near these types of facilities remain within specified limits. Generally, for all sensitive equipment, the manufacturer provides necessary details about the maximum vibration levels that the equipment can operate under. ISO 10811-1 (2000) and ISO 10811-2 (2000) provide guidelines on

measurement, evaluation and classification of vibration and shock in buildings with sensitive equipment.

On the contrary, human annoyance is the most significant impact of ground-borne vibrations caused during train passage. Based on ISO 2631-12:2003, root mean square of the signal (RMS value) is used to express the threshold of perception. RMS value of 1 mm/s at 1 Hz and 0.1 mm/s at 10 Hz and higher are considered as limits, although there are possible differences between individuals in their perception based on their sensitivity (Ouakka et al., 2022). The absolute threshold of weighted vertical vibration perception is around 0.015m/s^2 as per ISO 2631-1:1997. Apart from the perceivable vibration, vibration itself generates an audible noise, for which the specified limits can be derived from legal limits for environmental noise, which are 35-40 dB during the day and 25-30 dB during nighttime (Croy et al., 2013b). People are likely to be more annoyed when they are exposed to both noise and vibration rather than only being exposed to vibration. Figure 2.5 shows exposure–response curves and their confidence intervals from a meta-analysis (using a total of 4129 exposure and response data) conducted during a project called Cargovibes (Waddington et al., 2015). In the left-hand curve at $v_{\text{dir, max}} = 0.4 \text{ mm/s}$, which is higher range of realistic levels for rail traffic, about 0-5% is highly annoyed while up to 15% is annoyed.

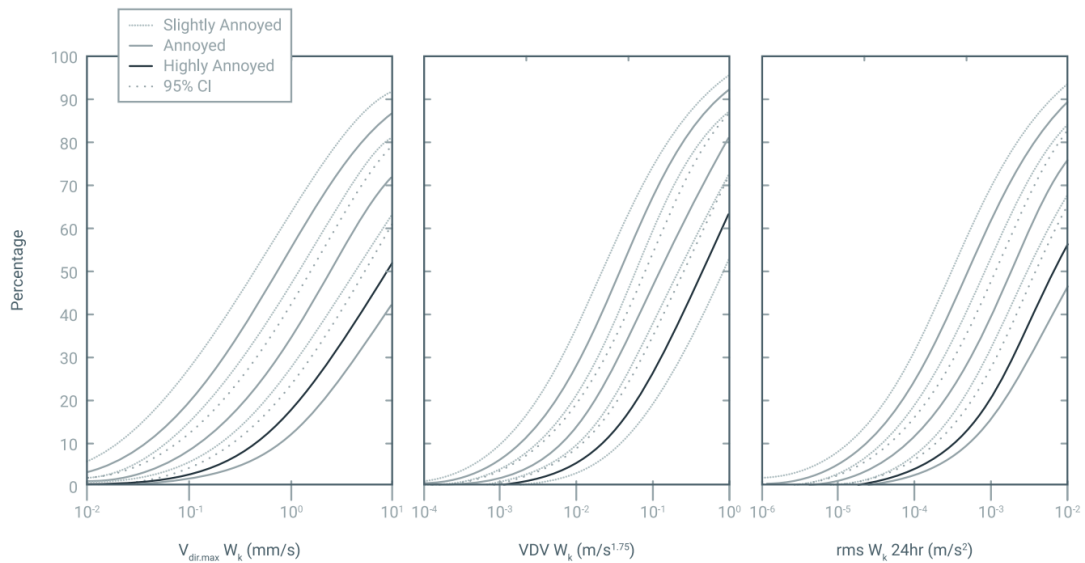


Figure 2.5: Exposure response curves following from a meta-analysis carried out in project, Cargovibes (Waddington et al., 2015)

A representation of the response to vibration with different values is given in Table 2.3 (Smith et al., 2015). The values in the column mentioning KB values could be interpreted as equal to a rms mm/s value.

Table 2.3: Perception of vibration according to DIN, after (Smith et al., 2015)

Weighted maximum vibration velocity (KB values)	Perception
0.1	Threshold of perception, just noticeable
0.2	Weakly noticeable

0.4	Noticeable
0.8	Awakening threshold, clearly noticeable
1.6	Strongly noticeable
6.3	Very strongly noticeable

2.3 Review of Modelling Techniques

Over the years, numerous models have been developed in the context of predicting train-induced ground vibrations. A common characteristic among most of those models is that they have been developed by coupling sub-models for train, track and ground. With the requirement of capturing the excitation mechanism of the train and the dynamic behaviour of the track and soil, models developed for this scenario are comparatively large and computationally expensive. (D. J. Thompson et al., 2019).

2.3.1 Analytical Approaches

In the context of modelling electrodynamic waveform, it was first analysed by Lamb (1904) to develop analytical expressions to capture wave propagation within a homogeneous half-space due to a stationary point load. Although adapting these expressions in validating complex wave propagation models proved unsuccessful, it is hard to adapt such an analytical approach for a train/track model representing a railway system. However, researchers have modified and extended Lamb's work to facilitate moving load simulation, tailoring it to railway systems (D. P. Connolly et al., 2015b). Among the 3d analytical models developed to predict the behaviour of railways at grade, initial focus was only on predicting the quasi-static component of the response for a homogenous half space (Dieterman & Metrikine, 1997) and layered half space (Madshus & Kaynia, 2000). Later, the implementation of dynamic excitation on the track enabled evaluating the complete vibrational response due to train passage (Auersch, 2005). However, when modelling a train-track system analytically, model complexity is reduced through many assumptions. Therefore, researchers have moved to adapt numerical solutions to account for the complexities mentioned above.

2.3.2 Numerical Approaches

Finite Difference Time Domain (FDTD)

Finite difference time domain method (FDTD) (Graves, 1996) is a numerical modelling tool which is based on the approximation of a strong formulation of the seismic wave using a central differencing integration scheme. This approach domain is generally detached into velocity and stress components that are staggered in two-dimensional (2D)/three-dimensional (3D) space. The main advantage of the FDTD method is that it is computationally efficient, as the well-structured nature of velocity/stress discretisation allows the workload to be divided among several computer processors. Additionally, compared to other numerical methods, the implementation of absorbing boundary conditions is quite straightforward within FDTD, which is vital in studying wave propagation (Giannopoulos et al., 2012).

Despite the above advantages, unsatisfactory performance of the FDTD method was observed in modelling complex geometries and free surfaces, which are essential in simulating the train-track system (D. P. Connolly et al., 2015b). However, Katou et al. (2008) adapted this approach to model a railway track and embankment where experimentally collected time history loading from a train is used as input force. Supporting this, Thornely-Taylor (2004) Utilised the FDTD technique with an unpublished approach to reduce dispersion within the complex track geometries to model the railway track explicitly.

A semi-analytical method was proposed by Sheng et al. (1999b) in place of challenges encountered with absorbing boundaries by the FDTD method. In this case, governing equations for the vehicle were computed in the frequency domain, while the frequency-wavenumber domain was used to solve equations related to track and soil. A single transfer function was possible for the overall system using this approach, which resulted in it being computationally efficient compared to other numerical methods (Sheng et al., 1999a). However, in this approach, many assumptions needed to be made to formulate analytical expressions that were used to calculate transfer functions. Hence, difficulties could be encountered in changing the geometry and calculating structural vibration. Additionally, in-situ experiments were required to identify stress distribution between sub-models as the coupling between the track and ground was based on it (Sheng, 2001).

Finite Element Method (FEM)

Finite element method (FEM) emerged as an alternative for FDTD method highlighting its key strength in modelling of complex geometries (Petyt, 2010). Additionally, availability of graphical user interfaces associated with commercial software such as ABAQUS, LS-DYNA, ANSYS have made FEM quite popular among researchers. For the context of modelling a railway system, their advantage comes with the ability to model the track components and connections between them explicitly (D. Connolly, Giannopoulos, & Forde, 2013). In this case also, for infinitely spanning domains like soil, absorbing boundary conditions (ABC) need to be adapted at boundary surfaces to prevent reflection of waves back to the domain (D. P. Connolly et al., 2015b).

For time domain FE modelling, solutions such as infinite elements (Bettess & Zienkiewicz, 1977), the combined FE-thin layer method (Barbosa et al., 2012), and the scaled boundary FE method (Ekevid & Wiberg, 2002) have been proposed. Perfectly matched layers (PML) is considered as a more efficient approach compared to previous ABC approaches that can be used to dampen the wave amplitudes reaching boundary surface. It uses a series of layers that are similar to material properties in the modelling domain, which act to stretch both the real and imaginary coordinate space, and it has been used more recently due to its superior performance (Barbosa et al., 2012; Berenger, 1994).

Time domain simulation for 3D wave propagation carries a major drawback, being computationally expensive due to the requirement of computing the solution at each

timestep. As a solution, analysing in the frequency domain is adapted, where for each interested frequency, the eigenvalue problem is solved, ultimately leading to a reduced computational requirement. However, most of the ABCs in the time domain cannot be used in the frequency domain FE approach making it hard to prevent wave reflecting from boundary surface (D. P. Connolly et al., 2015b).

Coupled FE-BE and 2.5D Approaches

Coupling of finite element with boundary element method (BEM) is frequently adapted as a solution for the above limitation (Çelebi et al., 2006). The BEM only requires the boundary of the domain to be meshed and uses greens functions to model wave propagation. This allows modelling of vibrations at large offsets from the source, which is important for railway applications where far-field excitations need to be investigated. As in previous cases, BEM possess the disadvantage of modelling irregular geometries within the domain. To fill this gap, FEM is used in modelling complex geometries of the track, combining the strengths of both approaches as shown in Figure 2.6 (D. P. Connolly et al., 2015b). Considering the reduced computational time, frequency domain approach (Sheng et al., 2006) is more frequently used compared to time domain formulation (Galvín & Romero, 2014). Additionally, modelling of damping is quite straightforward in formulating BE equations in frequency domain, whereas in time domain, simulating camping can be challenging.

Jones et al. (2002) developed a 2D coupled FE-BE model, which was applied by Garcia-Bennett et al. (2012) to study the effect of trenches adjacent to railway lines in attenuating ground vibrations. Comparison of 3D FE-BE models with 2D models in Andersen & Jones (2006) revealed that although 2D models are capable of predicting the response, 3D models are required to predict the response up to an absolute level.

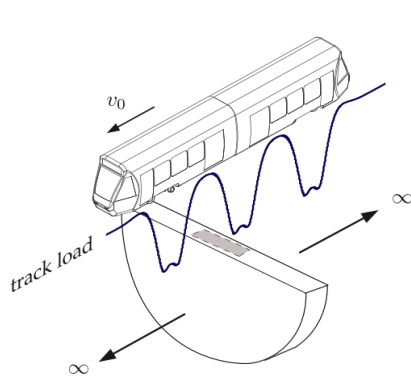


Figure 2.6: A 2.5D approach

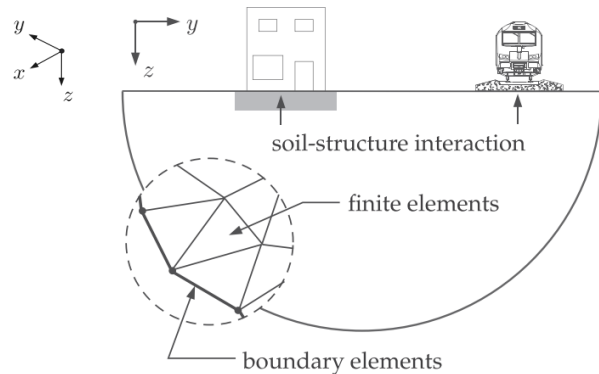


Figure 2.7: A coupled BE/FE approach

The limitation of the BE/FE approach is that computing the solution can be computationally expensive since the matrix formulation for the FE region is well structured and uniform, while for the BE regions, matrices become sparse. Use of the 2.5D approach has been a promising solution in minimising the computational demand of previously mentioned large 3D models, where the geometry and material properties are uniform along the track direction (François et al., 2010). In this approach, a 2D FE and/or BE mesh is used, and the third dimension is represented by a series of

wavenumbers as in Figure 2.7. Later, the inverse Fourier transformation over wavenumber can be applied to obtain the complete 3D solution. This approach was first adapted by Yang et al. (2003). In a finite/infinite element method. Jean et al. (2004) presented a 2.5D BE method while Sheng et al. (2005) presented a 2.5D FE-BE model to predict the response of trains in tunnels and on embankment.

Fully 3D models

Although 2.5D approach is more efficient, the effect of train induced ground vibrations reaching structures cannot be accurately modelled. Fully 3D models can be effectively adapted for such cases which also have the ability to account the inhomogeneities in the track (D. J. Thompson et al., 2019). Galvín et al. (2010a) developed a time domain fully coupled 3D multi-body-FE-BE model to predict vibration due to train passages. The BE model used full-space Green's functions (i.e. fundamental solutions) so that the ground surface had to be meshed. D. Connolly, Giannopoulos, & Forde (2013) developed a 3D FE model that is solved in the time domain to study the ground response to a moving vehicle, and many authors have adapted this model to study response under numerous conditions.

The common challenge of numerical techniques like FE and BE is that the discretization of the element should be small so that it is capable of representing wave phenomena. Some authors suggest that there should be at least six elements per wavelength (Dominguez, 1993), while some suggest that it need to be 10 or more. However, this mainly depends on the frequency of interest, and with the frequency number of elements increase considerably (D. J. Thompson et al., 2019).

2.4 Vehicle Modelling Approaches

2.4.1 Constant axle load models

The simplest way to define a moving vehicle load is a sequence of moving axle loads. This approach can provide clear information on track deflection and responses related to critical velocities. A plain track/foundation model is sufficient to analyse track deflection due to the moving weight of a vehicle. As shown by Grassie et al. (1982) to capture the low-frequency excitation, a track resting on a simple elastic support is satisfactory. An analytical method was proposed by Krylov & Ferguson (1994) to determine the deflection $w(x,t)$ caused by a constant load, P , moving at a constant speed of v_0 using equation 2.4. Here, the track is considered as a Euler-Bernoulli elastic beam with Young's modulus E_r , cross-sectional momentum I_r resting on a Winkler foundation, which was defined by its stiffness of K_f per unit length

$$E_r I_r \frac{\partial^4 w}{\partial x^4} + K_f w = P \delta(x - v_0 t) \quad (2.4)$$

Quasi-static deflections and their frequency of excitation are given in Figure 2.8.

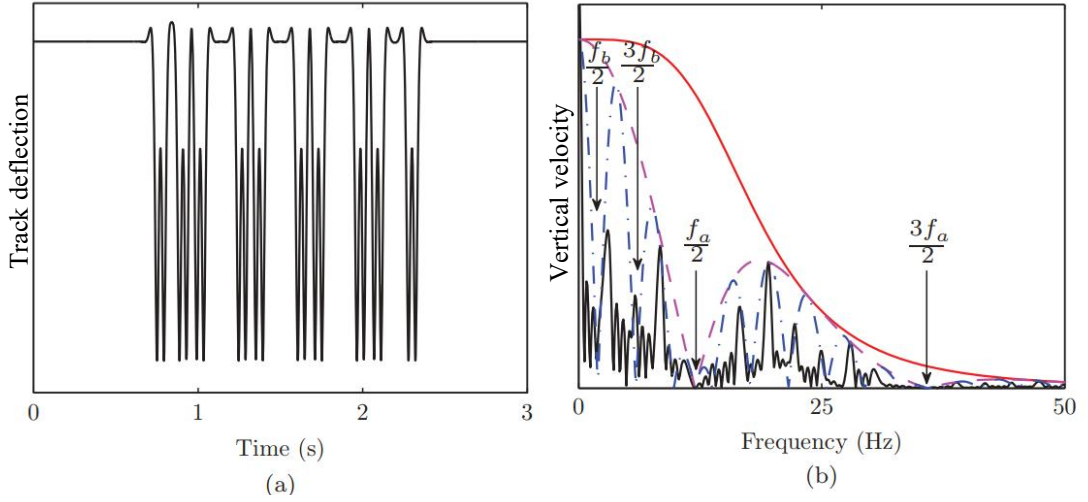


Figure 2.8: Track deflection due to the moving of X2000 HST (a) time history (b) frequency content (Kouroussis et al., 2014)

2.4.2 Randomly Varying axle load models

One key assumption of the previous approach in vehicle modelling is that the contact between the rail and the wheel is considered perfectly smooth. However, imperfection between wheel and rail surfaces play a key role in determining the wheel/rail forces that are important in examining the dynamic excitation of the track. To account for wheel/rail unevenness in the random dynamic response of track deflection, several significant enhancements need to be made in the sequence of constant axle load, as in equation 2.5 (Kouroussis et al., 2014).

$$w(t) = w_{st}(t) + w_{dyn}(t) \quad (2.5)$$

Where $W_{st}(t)$ is the track deflection due to axle load and additional parameter of $W_{dyn}(t)$ represents the track displacement induced due to the contact between irregular rail or wheel surface.

Being an unpopular approach to simulate moving train load, Lu et al. (2006) used it to present an algorithm to be used in coupled vehicle/track systems to simulate random moving axle loads. Lombaert & Degrande (2009a) successfully applied this method in a numerical model that was developed to predict train-induced vibrations. The author critically evaluates the contribution of quasi-static excitation and dynamic excitation, and eventually, it was adopted in the calculation of domestic and High-speed trains in Belgium. However, this method possesses the key challenge of the inability to distinguish between the unevenness of the track and the wheel

2.4.3 Detailed multi-body vehicle models

Although low-frequency vibration is considered to have the greatest critical impact on humans and structures, detailed vehicle modelling can show unaccounted aspects of vibration generation. For this purpose, a basic model can be derived that accounts for the vehicle's unsprung, semi-sprung, and sprung masses, which are interconnected by spring and damper elements representing the primary and secondary suspension systems of train coaches. Figure 2.9 shows such a simple dynamic model that can be adapted.

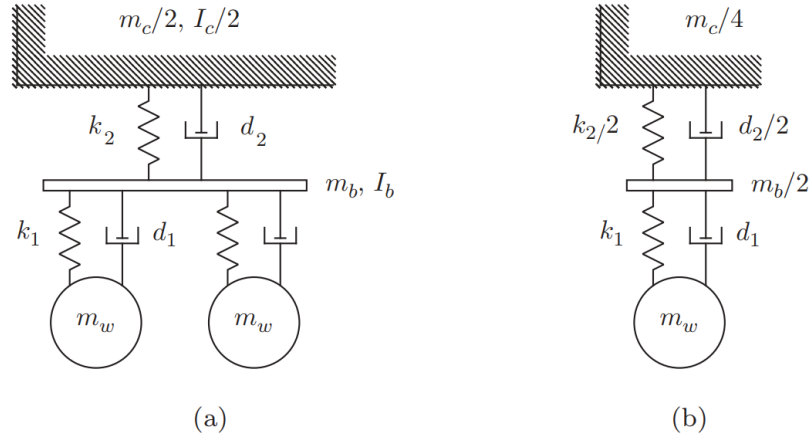


Figure 2.9: Simple dynamic model for multi body spring/damp system (a) 1/2 car body
(b) 1/4 car body

Limitations of this approach include difficulties obtaining dynamic parameters of the train from manufacturers and challenges coupling the detailed model with the track/soil system. Based on Figure 2.12(b), a linear system of equations of motion can be defined as in Equation 2.6

$$[M_v]\{\ddot{q}_v\} + [C_v]\{\dot{q}_v\} + [K_v]\{q_v\} = \{f_v\} \quad (2.6)$$

Where M_v , C_v , and K_v are the mass, damping, and stiffness matrices, respectively. In vector f_v external forces (gravity forces) are accounted (Kouroussis et al., 2014). In coupling the vehicle model with the track, configuration parameters q_v are used through a Hertzian contact (Johnson, 1985) defined by equation 2.7.

$$N = K_{Hz}d^{3/2} \quad (2.7)$$

Here, N is the imposed load, d is the material deformation, and K_{Hz} is the Hertzian constant, which depends on the radii of arch between the wheel and rail, and the elasticity of material for both bodies. In most cases, a value of 1GN/m is used for K_{Hz} (Kouroussis et al., 2014).

Based on the nature of Hertzian contact, lumped mass models can be derived in 2 forms as a linear model in frequency domain (Galvín et al., 2010a; Romero et al., 2010) and non-linear model in time domain (D. Connolly, Giannopoulos, Fan, et al., 2013; D. Connolly, Giannopoulos, & Forde, 2013) simulations.

Adapting a detailed vehicle model is essential in studying ground vibrations, since targeted mitigation can be implemented on the vehicle itself. In order to achieve that, prior knowledge of the coupled dynamics of the vehicle, track, and soil is required. In Kouroussis et al. (2013), modal analysis showed that the vehicle's bounce and pitch natural frequencies vary significantly depending on whether the track is flexible or rigid. Mirza et al. (2012) found numerically that vibrations increase when the primary suspension is made stiffer and when the unsprung mass is larger. Based on results from simulations, Costa et al. (2012) suggested including the unsprung masses and the semi-sprung (bogie) masses in prediction models to meaningfully improve accuracy.

2.5 Vibration Mitigation Strategies

Numerous mitigation measures can be adapted to keep train-induced ground vibrations under a specific limit aiming to reduce its impact on humans and structures. These measures have been tested and evaluated in over the years. The selection of mitigation measures needs to be carefully analysed and assessed to ensure optimum results are obtained since suitable mitigation measures depend on numerous aspects such as type of trains (HST, passenger, freight), track condition, geological conditions, surrounding environment and community, etc. This section will provide an overall understanding of the available mitigation measures and their suitability for different contexts, as well as their performance in attenuating ground-borne vibrations.

Mitigation measures can be applied to the basic parts of a railway system: vehicle, track, and propagation path. Apart from these basic parts, it is also possible to reduce the vibrations reaching receiver (humans or buildings) by means of available seismic isolation solutions. In fact, these are commonly applied to reduce the effect of seismic activities and are not specific to vibrations caused by train passages. Through full-scale and scaled models, these mitigation measures are tested and evaluated. However, integrating their results in computer simulations allows the ability to evaluate the effectiveness of those measures under varying circumstances.

2.5.1 Vehicle-Based Measures

The vehicle system has a significant impact on generation of train-induced vibrations. At present railway coaches are manufactured using bogie system. Wheels are connected to the bogie frame using the primary suspension system, and the bogie frame is connected to the coach body using the secondary suspension system. A carefully designed bogie system has the ability to significantly reduce the vibration levels generated by the train. Normally, vehicles with soft primary suspension produce lower levels of vibrations compared to those with stiff primary suspensions (Wilson et al., 1983). Although vehicle design parameters play a crucial role in reducing ground-borne vibrations, manufacturers are more focused on enhancing the stability and ride comfort of passengers. There are a few key aspects that manufacturers can focus to control ground-borne vibrations (Ouakka et al., 2022). They are,

- Improving wheel roundness

- Reduction of unsprung mass
- Reduction of speed
- Resilient wheels

Improving wheel roundness

Since the vibration and wheel/rail interface are generated, the wheel itself is quite significant in this case. Due to faults in the manufacturing process, repeated loading at high frequencies, wheels tend to change their original round shape (Nielsen & Johansson, 2000), while rapid deceleration and braking can lead to the formation of wheel flats. Whenever the wheel flats struck on the rail during the rotation, high frequency impact forces are generated resulting vibrations in the track (Ouakka et al., 2022). A few orders of wheel being out of roundness are shown in Figure 2.10 as first-, second and third-order, while it can be increased up to 23rd-order and even more as per previous research (Wu et al., 2017).

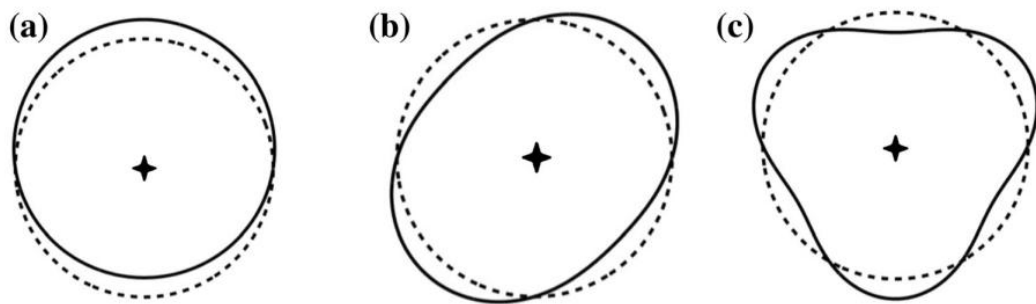


Figure 2.10: Different orders of wheel out-of-roundness (a) First-order (b) Second order (c) Third order (Ouakka et al., 2022)

This can be addressed with proper maintenance of wheel system, improvement of sliding protection and use of high-quality steel in wheel manufacturing, eventually resulting in 5-10 dB reduction of noise levels (Ouakka et al., 2022).

Reduction of unsprung mass

While the suspension system reduces the effect of train car and bogie mass, unsprung mass directly impacts the rail. Therefore, the reduction of unsprung mass helps reduce track damage as well as reduce vibration levels. However, reducing unsprung mass below a specified level is problematic due to safety concerns and vehicle dynamics design (Ouakka et al., 2022). The mass of a train wheel set typically ranges from 700 to 3500kg, and its reduction can be achieved either by optimising the cross-sectional area of the wheel or by material. This reduction is limited between 5-10%, which results in reducing vibration levels of 2-4 dB (UIC, 2011). However, as discussed earlier, reducing the wheel mass can reduce wheel quality, which eventually results in increased vibration levels due to wheel defects.

Influence of speed

The speed of the train has a significant impact on the amount of ground-borne noise and vibration produced. Specifically, if the train speed is cut in half, the vibration

levels can decrease by around 6 dB (Hanson et al., 2006). For instance, in the early 1990s, during the high-speed train (HST) project in Ledsgard, Sweden (Holm et al., 2002), several measures were implemented to reduce vibrations. The HST operations began in Spring 1997 at a speed of 200 km/h. Soon after, unusually high vibrations were noticed at certain locations. These vibrations were about ten times stronger than those recorded from heavy train traffic under soft soil conditions, which were already considered problematic. To address this, the speed of the trains was reduced to 160 km/h and then to 130 km/h. By lowering the operational speed from 190 to 130 km/h, it was possible to lower the impact level (IL) to 26 dB at a distance of 12 m from the source. Generally, as the train speed approaches the Rayleigh wave speed of the ground, there can be a considerable increase in track vibrations and, consequently, greater ground-borne effects. (D. P. Connolly & Costa, 2020; Dong et al., 2019).

Resilient wheels

Resilient wheels can be effectively used to reduce wheel squeal on tight turns, and a noise reduction of 10–20 dB can be achieved (Hanson et al., 2006). The effect of changing the stiffness of resilient wheel equipping trams to reduce the transmitted ground vibrations is shown in Figure 2.11, in terms of Insertion Loss. Here, the standard wheel stiffness of 145 MN/m is reduced to 13 and 18 MN/m (Kouroussis et al., 2012).

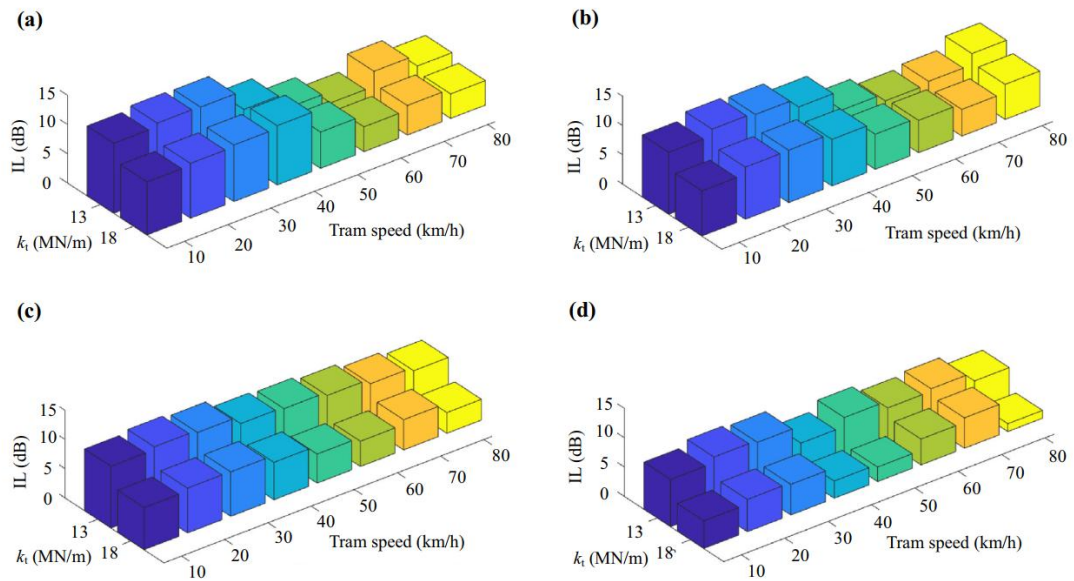


Figure 2.11: Effects of changing wheel resilient material stiffness on ground vibration level (a) at 4 m distance (b) at 10 m distance (c) at 16 m distance (d) at 20 m distance (Kouroussis et al., 2012)

2.5.2 Track-Based Measures

In urban areas where issues related to railway vibrations are more prominent, it makes sense to focus on mitigation measures at the source, such as the vehicle and wheeltrack, rather than addressing them along the path (ground) or at the receiver (nearby residents and buildings). As shown in Fig. 2.12, among the various dynamic forces that contribute to railway vibrations, those caused by the vehicles occur at lower frequencies, while the forces linked to the wheel, rail, and track happen at higher frequencies. (Ouakka et al., 2022).

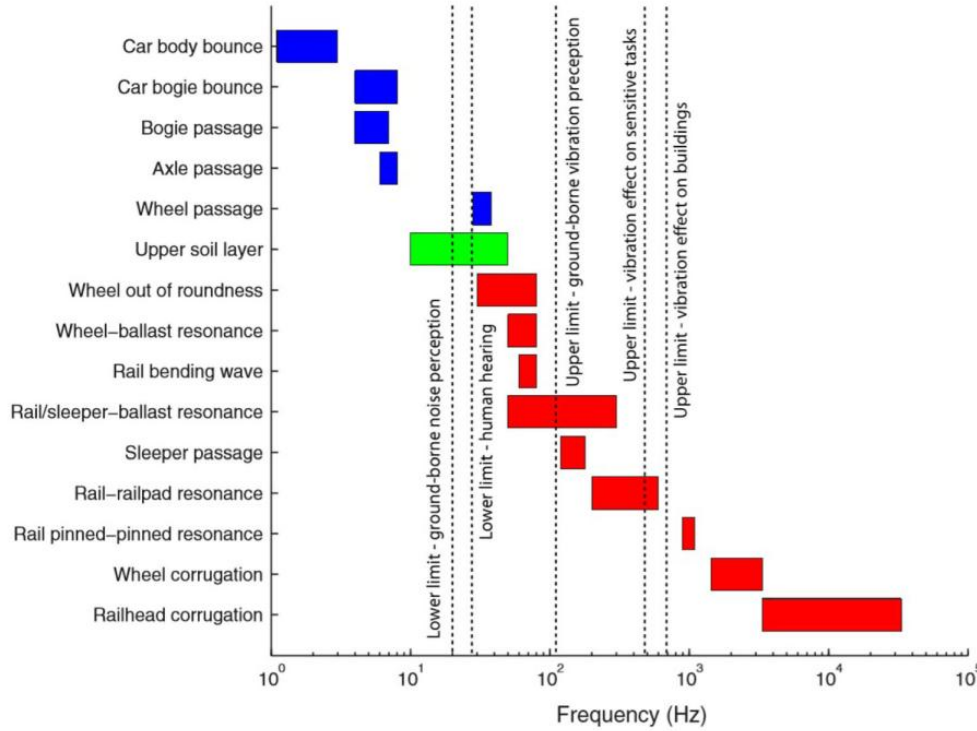


Figure 2.12: General frequency ranges of different excitation mechanisms (Ouakka et al., 2022)

One key factor affecting ground-borne effects is the overall stiffness of the track. If the stiffness is too low, it leads to greater soil deformation and ballast deformation. Conversely, if the stiffness is too high, it makes it easier for corrugation to develop (Pita et al., 2004). Stiffness values vary across countries depending on the types of trains anticipated, such as freight or passenger services. Therefore, it is challenging to establish a single ideal stiffness value, as many rail lines accommodate different types of trains. Proper selection of components such as rail, fastenings, sleepers, and ballast is crucial for reducing the formation and spread of vibrations. Increasing the flexibility of these superstructure components enhances their ability to dampen (or dissipate) vibrations that occur at the wheel-rail interface (Lakusic' & Ahak, 2012).

Rail improvements

The irregularities in the railway track and ballast can significantly contribute to vibrations. In fact, having a well-aligned track can reduce up to 10 dB in ground-borne noise when trains are moving at speeds of 320 km/h. This highlights the importance of regular maintenance for the rails in reducing vibrations. Additionally, embedded rail systems (Figure 2.13) serve as an alternative solution, particularly for urban tram lines. Research by Lakusic & Ahak (2012) indicated that after continuously embedded rails were installed, vibration reductions ranging from 12.9 to 18.6 dB were observed on a tramline, depending on the type of tram vehicle used.

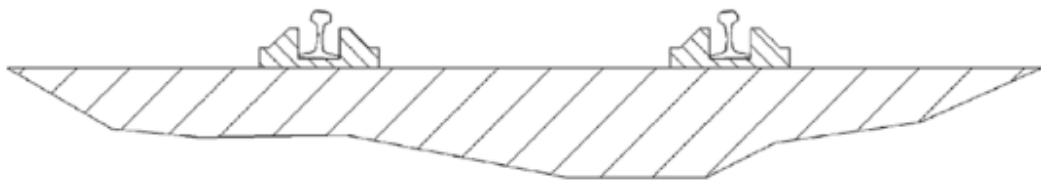


Figure 2.13: Embedded railway system

Rail fasteners

Rail fasteners have been commonly used for a long time, even though improvements have been made over the years. One important type is called resilient rail fasteners. Recently, new designs have been introduced, such as those used in the Thameslink project in London, which support the rail with resilient blocks. This innovation has led to a reduction of 13 dB in ground-borne noise (Bewes et al., 2015). Further studies have shown that high resilience fastening systems (Figure 2.14), in which elastic elements support the rail at the web and prevent direct contact between the rail foot and sleeper, allow for significantly greater vertical deflection of the rails during operation. This results in a lower vertical stiffness for the entire system, which can reduce vibrations by 5 to 10 dB at higher frequencies (Hanson et al., 2006). Additionally, when the resilient pad is pre-compressed with a load equivalent to 80% of the standard static load during vehicle passage, vibration isolation of 20 dB can be achieved within a frequency range of 25 to 20 Hz (Ouakka et al., 2022).

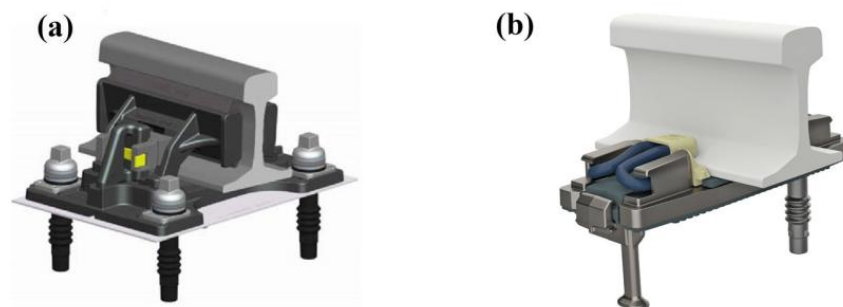


Figure 2.14: Resilient fastening system (a) high-resilience fastening system and (b) resilient pre-load fastening (Panodral, 2020)

Sleepers and ballast improvements

Sleepers can be made of either concrete or wood. Among these, concrete sleepers are more commonly used due to several economic benefits, including easier installation, longer lifespan, and lower maintenance and operating costs. On the other hand, wooden sleepers have a better capacity for damping vibrations (Ferro et al., 2020; Mateus et al., 2008). Research conducted by Hanson et al. (2006) indicated that wooden sleepers can reduce vibrations by 5 dB. Furthermore, placing elastomeric pads between the sleepers and the ballast bed can also help with vibration reduction. These elastomeric pads typically consist of two layers: the top layer is made from viscoelastic rubber which effectively dampens vibrations, while the bottom layer is a coarse geotextile that helps protect the upper layer from damage caused by the crushed ballast material (Sol-Sánchez et al., 2015). According to UIC, using pads under the sleepers can achieve noise reduction levels ranging from 8 to 20 dB. One of the benefits of these pads is that they are easy to install during the renewal of sleepers, as they come pre-attached to the sleeper's bottom (Bongini et al., 2012). In contrast, ballast mats, which can be used in both surface and underground settings, provide vibration reductions between 3 and 15 dB. However, if a ballast mat is excessively soft, it may weaken the ballast layer under the impact of train vibrations. This can negatively affect ride quality unless consistent maintenance is carried out, which would increase cost (Sol-Sánchez et al., 2015).

Alternative track technologies

Alongside the traditional ballasted track, various other track design methods aim to reduce ground-borne noise and vibration. When it comes to vibration damping, track structures with a ballast bed perform better than those built on specially reinforced concrete slabs, often referred to as slab tracks, showing improvements of up to 3–5 dB (Wheel/Rail Noise, 1997). Additionally, increasing the height of the ballast bed can further diminish vibration transmission. Research by the German Railways (Wheel/Rail Noise, 1997) indicates that raising the ballast bed height from the standard 30 cm to 75 cm can reduce vibrations for frequencies below 10 Hz by 6 dB.

An alternative to ballasted tracks is floating-slab tracks. In this system, the track is placed on a thick concrete slab supported by rubber bearings. Floating-slab tracks are mainly utilized to control vibrations and ground-borne noise from underground trains, especially when significant reduction is needed (Grootenhuis, 1977). The use of floating slabs and other mass-spring systems can achieve a vibration reduction of 10 dB for frequencies above 16 Hz and 25 dB for frequencies at 125 Hz (Hemsworth, 2000).

2.5.3 Measures in Propagation Path

In the context of train-induced ground-borne vibrations, the propagation path (soil) medium plays a key role. There are numerous mitigation measures that can be adapted in the propagation path, which have been widely studied in literature. These are

generally applied in the surface train applications, where they act between the source and the receiver as a barrier, diverter or damper reducing the intensity of vibrations on the receiver. The most straightforward solution that can be adapted in propagation path is increasing the distance between source and the receiver. However, this solution is feasible only if there is adequate free land to be reserved for the track and also the cost of the land is cheaper than other mitigation measures (Ouakka et al., 2022).

Embankments

Embankments are originally used to maintain the horizontality of the track, allowing it to span at an acceptable level over low-lying ground. In addition to that, they have a considerable effect on attenuating the vibrations generated at the rail before propagating to subsoil and this was confirmed by D. Connolly et al. (2013) through a prediction model. With an embankment at least 3 m high, a noise reduction of up to 5 dB(A) at the point of emission can be achieved (Lakus'ic' & Ahak, 2012). Olivier et al. (2016) showed the effect of subgrade configuration in vibration transmission where embankments with specific materials were capable of trapping the wave energy within the embankment acting as a wave guide as illustrated in Figure 2.15.

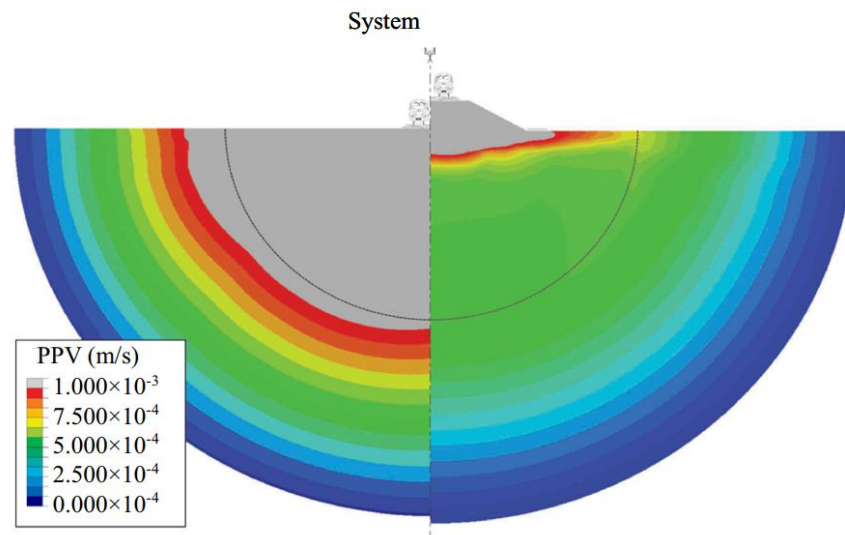


Figure 2.15: Numerical visualization of peak particle velocity of ground during a HST passage for track at grade (left) and track on embankment (right)

Wave barriers

One of the most common measures that is adopted in the propagation path is the use of wave barriers. These can be either open trenches or trenches with in-filled materials, providing different levels of wave damping based on the characteristics of the infill material. Characteristics and performance of several wave barriers are presented in Table 2.4.

Table 2.4: Different types of wave barriers and their effectiveness in attenuating ground vibrations

Type of wave barrier	Description
Open trenches	For the reduction of vibrations at a 30 Hz frequency, a minimum depth of 4.5 m is required (Hanson et al., 2006). For optimum performance, trench depth should span at least half of the Rayleigh wavelength (D. J. Thompson et al., 2016). The level of the water table and stability concerns can limit the depth of open trenches at a certain level.
Trenches with in-fill materials	Gas cushions can reduce 6-10 dB when the depth is equivalent to the wavelength (Massarsch & Fellenius, 2008) Rubber chips can achieve a reduction exceeding 12 dB (Hassoun et al., 2019) Polystyrene is capable of reducing up to 12 dB at 20 Hz and 6 dB at 40 Hz (François et al., 2012)
Steel/concrete piles	Around 80-90% of reduction is achievable from open trenches, depending on the cross-section and the number of piles (Kattis et al., 1999)
Sheet pile wall	An insertion loss of about 3 dB could be achieved at 25 Hz (Coulier et al., 2015)

In urban regions where wave barriers in transmission path is not feasible, underground barriers beneath the rail track could be implemented. These barriers are implemented by mixing lime or cement into the existing soil (Ouakka et al., 2022).

Another approach that can be adapted in the propagation path is the placement of a heavy mass next to the rail track. This is generally achieved by a gabion wall composed of stone or concrete, and it facilitates attenuating vibrations at frequencies which are higher than the resonance frequency of the mass on the ground (Dijckmans et al., 2015).

Soil stiffening

By changing the geotechnical properties (improving the bearing capacity) of the soil on either side of the track, some reduction in ground vibrations could be achievable (Adam et al., 2007). Hemsworth (2000) demonstrated that a 14 dB reduction in vibration levels within the frequency range of 4-32 Hz could be achieved by reducing the coherence of the subsoil.

Gabion walls and mass-loading

Another approach that can be adapted in the propagation path is the placement of a heavy mass next to the rail track. This is generally achieved by a gabion wall composed of stone or concrete, and it facilitates attenuating vibrations at frequencies which are higher than the resonance frequency of the mass on the ground (Dijckmans et al., 2015).

2.5.4 Special Focus: Elastic Track Elements

Under Sleeper Pads

Under Sleeper Pads (USP) are resilient elastomeric layers attached to the bottom of railway sleepers with the aim of providing additional elasticity between the sleeper and the ballast. Since their introduction in Switzerland during 1980s, they have been widely used in Australia, Europe and Asia with the aim of improving track performance and to reduce transmission of track vibration on to the ballast layers (Kaewunruen et al., 2017; Lakusic et al., 2010). A Schematic diagram of USP is shown in Figure 2.16. They are manufactured using polyurethane elastomers, ethylene vinyl acetate, or rubber with encapsulated air voids, which facilitate the redistribution of loads and reduce ballast degradation (Cao et al., 2014). Additionally, USP isolate the vibration of sleepers so that the ballast and formation are uncoupled from the wheel/rail interaction that facilitates the reduction of ground vibrations (Kaewunruen et al., 2017).

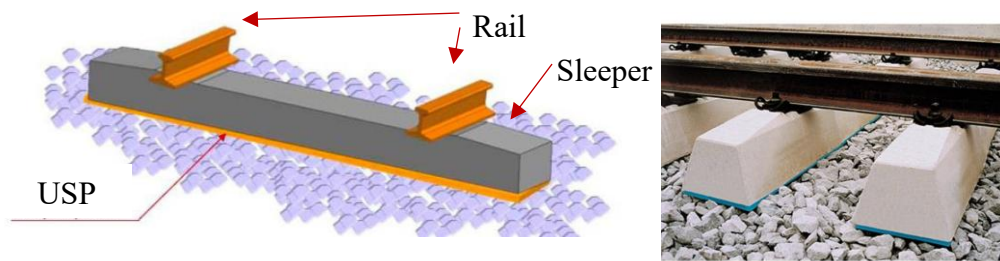


Figure 2.16: Components of Under Sleeper Pads

According to UIC, USPs can be classified based on their stiffness as in Table 2.5. Softer pads show increased vibration insulation, but at a cost of increased track deflection. Stiffer pads improve the overall track stability while vibration insulation shows poor performance (Cao et al., 2014).

Table 2.5: Classification of Under Sleeper Pads based on their stiffness (UIC)

Classification	Stiffness	Application
Stiff	$0.25 \text{ N/mm}^3 < C_{\text{stat}} \leq 0.35 \text{ N/mm}^3$	Heavy rail
Medium stiff	$0.15 \text{ N/mm}^3 < C_{\text{stat}} \leq 0.25 \text{ N/mm}^3$	Light rail
Soft	$0.10 \text{ N/mm}^3 < C_{\text{stat}} \leq 0.15 \text{ N/mm}^3$	Not commonly use
Very soft	$< C_{\text{stat}} \leq 0.10 \text{ N/mm}^3$	Not commonly use

Experimental studies on USP have confirmed the ability of USPs in vibration reduction but also highlighted some complex behaviour. Lakusic et al. (2010) showed that elastomeric USPs are capable of reducing vibration amplitudes up to 30% above frequencies of 40 Hz, making them more suitable for noise-sensitive areas, turnouts, and bridges. Field monitoring conducted by Kaewunruen et al. (2017) at rail joints demonstrated the effectiveness of USPs with moderate stiffness in attenuating transient impact waves, but also highlighted increased vibration amplitudes in sleepers and rails. This implies that while USPs protect the ballast and formation from high-frequency impulses, they may inadvertently amplify vibrations in superstructure components.

Several numerical models have explored the effect of USPs on train-induced vibrations. Insa et al. (2012) demonstrated that USPs improve load distribution, smooth stiffness transitions, and reduce ballast accelerations at high speeds. Cao et al. (2014) extended this by developing a vehicle–track–ground model with saturated poroelastic soil, showing that USPs reduce free-field ground vibrations by up to 20 dB at frequencies above 50 Hz at subcritical train speeds. However, near or above critical speed, their effectiveness decreases as quasi-static axle loads dominate. Importantly, softer USPs improved insulation effects but also amplified track deflections and dynamic responses

Ballast Mats (BMs)

Ballast mats are resilient layers that are installed under the ballast or sub-ballast and are usually made up of synthetic or natural rubber as shown in Figure 2.17. As USPs, basic functions of BMs are to reduce track stiffness and attenuate ground vibrations induced by train passages. Practically BMs can be placed directly beneath the ballast layer or under the sub-ballast layer. But studies have shown that placement of BM under the sub-ballast improves the track performance by reducing resonance amplification while maintaining the structural integrity (Alves Costa et al., 2012a). BMs are classified mainly by stiffness, which directly control their vibration isolation efficiency. Softer mats allow lower natural frequencies (20–50 Hz), making them effective for mitigating vibrations in the range of environmental concern. However, very soft mats can lead to excessive track deflections and lead to maintenance issues (Yuan et al., 2021).



Figure 2.17: Illustration of BM and its installation

Field experimental studies have confirmed that noticeable vibration attenuation can be achieved through the use of BMs. For instance, Auersch (2006) reported reductions up to 20 dB in tunnel conditions with mats of stiffness around 4×10^7 N/m³. A case study conducted by Yuan et al. (2021) on subways demonstrated that mats can reduce ground acceleration levels by ~10 dB within 30 m of the track centreline. Increasing mat thickness and static foundation modulus further enhanced this effect. Additionally, Alves Costa et al. (2012a) have used a 2.5D FEM–BEM approach to model train–track–ground interaction and concluded that the introduction of mats can considerably reduce the free-field vibrations, up to 20 dB in the frequency range above 40 Hz.

2.6 Chapter Summary

This review presents the main concepts necessary for understanding and reducing ground vibrations caused by trains. It differentiates between the effects of the source, the path, and the receiver, and emphasizes the significance of soil type and layering. It also points out the importance of Rayleigh waves that occur close to the ground surface. Among the different modelling methods, the 3D time-domain FE approach is recognized as effective for situations where it is essential to consider track specifics, vehicle interactions, and soil boundaries together. The chapter categorizes mitigation strategies by their level, emphasizing unique selling points and best practices as useful track-based solutions for areas with limited space. In summary, this chapter lays the foundation for exploring train-induced ground vibrations using FEM methods.

CHAPTER 3

METHODOLOGY

2.1. Introduction to the Chapter

This chapter aims to provide a detailed exploration of the methodology and approach utilized in the current study to achieve its objectives. Initially, a general overview of the methodology will be presented, followed by an in-depth discussion of its key components. The software package chosen for developing the finite element (FE) model and the explicit solver employed for running simulations will be outlined. Validation precedes any parametric work and uses published field measurements to ensure the model reproduces vibration levels and spectral features observed in practice.

The chapter is organised as follows: Section 3.2 outlines the overall research design and key assumptions. Section 3.3 details the track model geometry, material properties, mesh strategy, and boundary conditions. Section 3.4 describes the vehicle abstraction, wheel rail contact, and their implementation via VUFIELD/VDLOAD. Section 3.5 presents the validation datasets and the metrics used. Section 3.6 defines the parametric study on under-sleeper pads and ballast mats, including modelling approaches, parameter ranges, and response measures for near and far-field receivers.

Through these discussions, the chapter aims to provide a comprehensive understanding of the methodological framework that underpins this research.

2.2. Overview and Research Design

This study follows a basic framework that can be adapted to parametric studies incorporating FE modelling, as shown in Figure 3.1, with a few specific adjustments. Following the confirmation of the research problem and formulation of the aim and objectives of the study. A comprehensive literature review is conducted to identify the theoretical background, experimental studies and numerical modelling that have been adapted, which are explained extensively in Chapter 2. In reference to numerous techniques adapted in the literature, a baseline 3D FE model was developed to capture the propagation of train-induced ground vibrations effectively. The framework adapted in the development of the base FE model is outlined in Figure 3.2, and studies by previous researchers (Connolly, et al., 2013; Connolly et al., 2014; Degrande & Schillemans, 2001) are taken as the primary references for the development of the FE model.

The validated FE model is then integrated with USP and BM, and a study on key parameters like thickness, stiffness, and damping across practical ranges from literature under two soil archetypes: soft and stiff, is conducted to understand the response in the near-field and far-field of the soil domain.

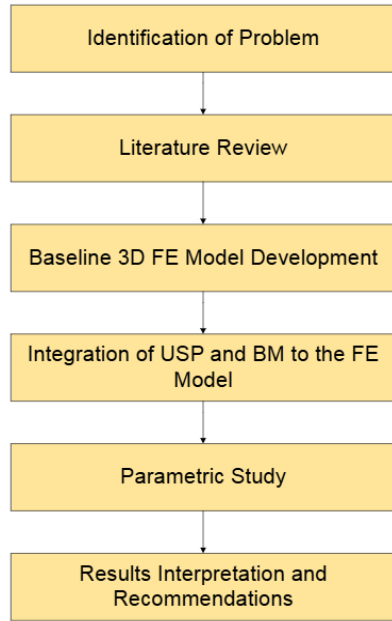


Figure 3.1: Basic methodological framework used in the study

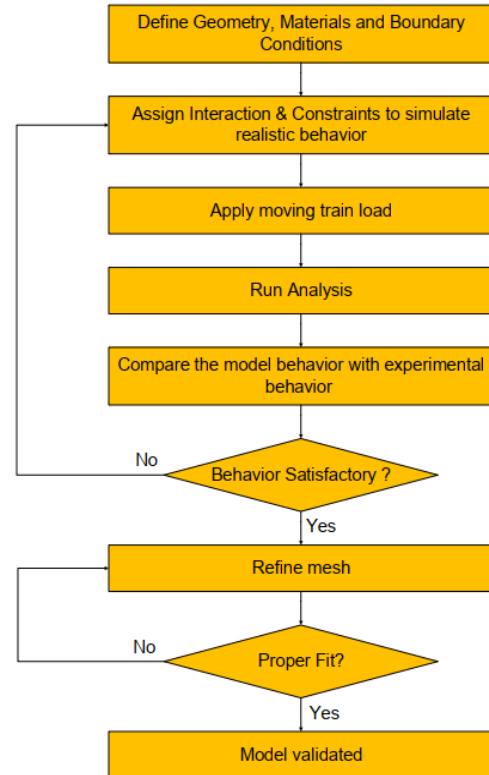


Figure 3.2: Framework for the development of 3D FE model

2.3. Track Modelling

2.3.3. Geometric Model

The basic 3D FE model for this study was developed based on the field experiment conducted by Degrande & Schillemans (2001). In this study, the authors have conducted measurements of free field vibrations during the homologation tests for the high-speed train track connecting Brussels and Paris. The FE model utilized in this analysis replicates the soil layers and dimensions of the track components to mirror the conditions present at the homologation testing site closely. The fundamental layout of the track is depicted in Figure 3.3. Given the symmetry of the track along the rail direction, only half of the track was modelled, as illustrated in the sectional and isometric views in Figure 3.4.

The total length of the FE model domain was established at 60 m, effectively representing the rail span over this distance. The lateral dimension of the soil domain, which extends perpendicularly to the rail span, was selected to be 20 m. This choice facilitates the capture of both near-field (0-10 m) and far-field (>10 m) responses while maintaining computational efficiency (Kouroussis et al., 2014). A total of 60 m of track was modelled, with rail being modelled as a continuously welded solid

rectangular section with dimensions $0.153 \text{ m} \times 0.078 \text{ m}$, laid at 1.435 m gauge. Timoshenko beam elements with 0.1 m length are selected to model the rail instead of Euler-Bernoulli elements since they have extra degrees of freedom. Therefore, the model can capture shear deformation and shear forces, which is useful in representation of transmission of high-frequency loads into the track structure more realistically. All the other track components apart from the rail is modelled using 8 noded solid cuboid elements.

Rail is supported by 92 sleepers positioned with a centre-to-centre spacing of 0.65 m . Each sleeper measured $0.242 \text{ m} \times 0.2 \text{ m} \times 2.42 \text{ m}$, resulting in a modelled length of 1.21 m for the half of the track considered.

The sleepers were supported by a ballast layer with a thickness of 0.3 m , a sub-ballast layer of 0.2 m , and a subgrade layer measuring 0.5 m in thickness. This entire track structure rested upon a subsoil extending to a depth of 15 m , as evidenced in Figure 3.4.

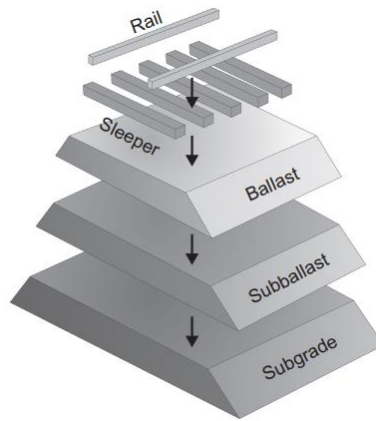


Figure 3.3: Basic layout of the track components

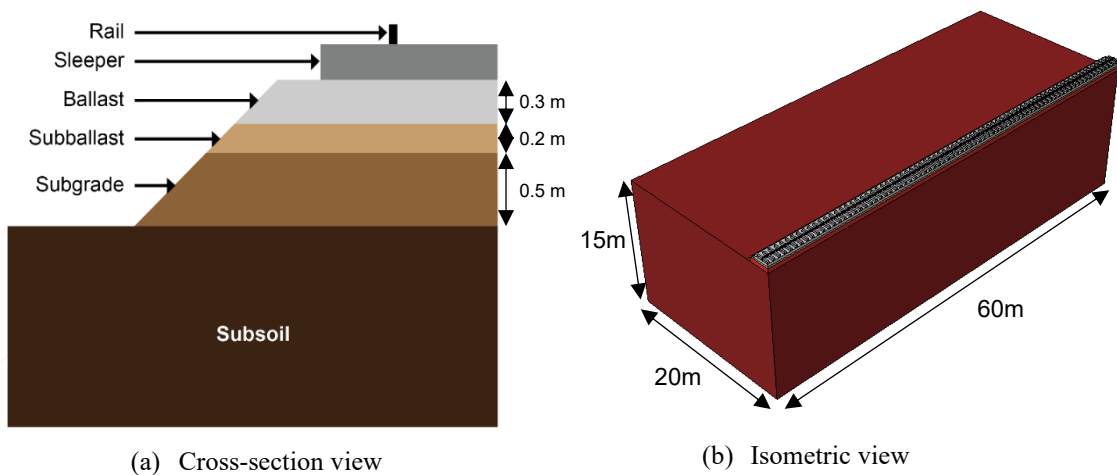


Figure 3.4: Half-track soil profile used in FE model with layer thickness

2.3.4. Material Model

The material model used in this FE model is linear elastic model with damping. Rails and sleepers were assigned linear elastic steel and concrete properties replicating standard conditions. Track ballast, subballast, and subgrade layers are modelled as linear elastic to capture dynamic responses at minor strains alongside the subsoil. A summary of the material properties used in each track element is explained in Table 3.1 in reference to the location where field measurements were conducted by Degrande & Schillemans (2001).

Table 3.1: Summary of material properties of the track elements (Degrande & Schillemans, 2001)

Track element	Young's modulus (MPa)	Poisson's ratio	Density (kg/m ³)
Rail	210,000	0.25	7900
Sleepers	30,000	0.40	2400
Ballast	100	0.35	1800
Sub ballast	300	0.35	2200
Subgrade	127	0.35	2100
Subsoil	129	0.30	1600

Degrande & Schillemans (2001) suggested damping values for the subsoil based on a spectral analysis of surface waves test conducted on site. They revealed that the damping properties of the subsoil are equivalent to a radiation damping coefficient of 2.1 and a material damping ratio of 0.03. Additionally, Kouroussis et al. (2011) suggested a uniform stiffness proportional Rayleigh damping value of $\beta = 0.0004$ s for all track layers and soil for similar site conditions to model energy dissipation in soil layers accurately.

2.3.5. Layer Interactions

Layer interfaces were modelled to reflect how forces transfer through the track to the ground while allowing realistic separations where they can occur.

Interfaces between soil horizons and track layers that are not expected to separate (e.g., subgrade–sub-ballast, sub-ballast–ballast footprint outside the sleeper seat) were tied using Abaqus TIE constraints as in Figure 3.5. This enforces compatible displacements across the interface and avoids artificial slip in the layered continuum.

Under each sleeper the ballast surface and the sleeper underside were defined as surface-to-surface contact with separation allowed as shown in Figure 3.5. In compression the contact transfers load, in uplift the interface can open, so no tensile

force is carried. This captures the nonlinearity at the sleeper seat and prevents unrealistically stiff behaviour under unloading.

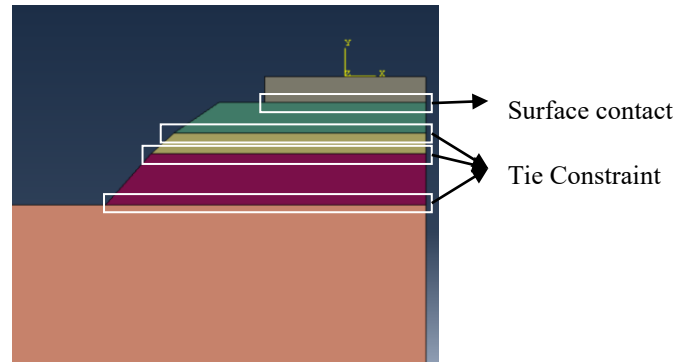


Figure 3.1: Track-soil layer interactions

The rails were connected to the sleepers through rail pads represented by connector elements placed at each rail seat as illustrated in Figure 3.6. The pad properties followed the validation reference with nominal thickness of 10 mm and vertical stiffness of 100 MN/m (Degrande & Schillemans, 2001). Using connectors here isolates the pad compliance from the sleeper and rail, gives a clean load path for the moving load, and keeps the local contact well-conditioned numerically.

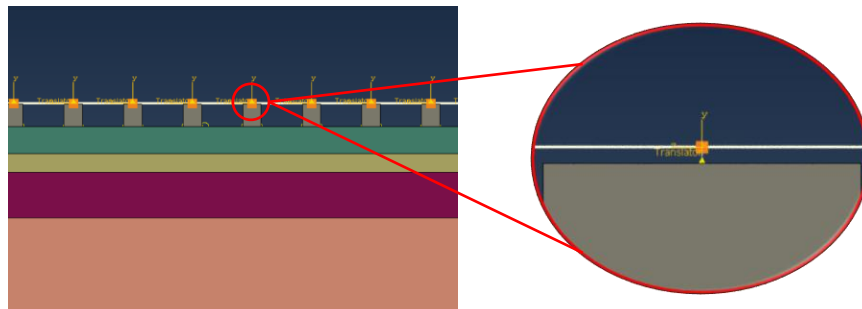


Figure 3.2: Assigned connector elements for rail pad

These choices keep the load transfer realistic in the vertical direction (where vibration is most relevant for the free-field receivers) while avoiding artificial locking or permanent attachment at interfaces that can physically open.

2.3.6. Mesh Strategy

A combination of beam and solid elements was utilised for the complete FE model. In beam elements, assumptions are made to reduce the problem to one dimension mathematically. It can be used to model elements in which the length is considerably larger than the cross-section of the element, a condition similar to a rail. Out of the available formulation types for beam elements, Euler-Bernoulli theory and Timoshenko theory, Timoshenko beam elements (B31 in ABAQUS) were assigned for rail elements, considering their additional degrees of freedom. This is specifically helpful in modelling shear forces, which are vital in accurately predicting the transmission of high-frequency forces into the track structure (D. Connolly, Giannopoulos, & Forde, 2013).

All the other track components are modelled using 8-noded solid cuboid elements (C3D8R in ABAQUS) with an element size of approximately 0.2 m.

The selection of mesh size for finite elements needs to be carefully conducted, as it needs to be small enough to accurately capture the wave propagation in the soil medium while maintaining computing time at a feasible level. This is determined by the motion with the highest frequency of interest, which must be accurately conveyed through the FE model. The finite elements should be sufficiently small to represent these high-frequency motions with short wavelengths accurately. The shortest wavelength of the system is governed by Equation 3.1 (Hall, 2000).

$$\lambda = \frac{C_s}{f_{max}} \quad (3.1)$$

Where C_s is the Shear wave velocity and f_{max} is the highest frequency of interest. Since compressional waves travel faster and possess longer wavelengths, capturing propagation of shear waves is focused here. Lysmer (1978) suggested that providing eight elements per wavelength will result in more than 90% accuracy on wave amplitudes in the high frequency range. This can be illustrated using Figure 3.7, which compares a model with 5 elements per wavelength and 10 elements per wavelength.

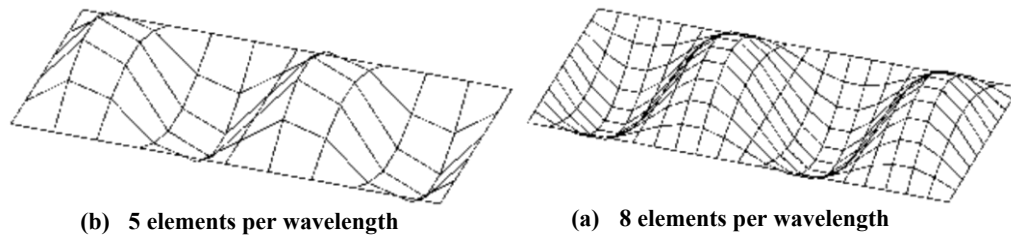


Figure 3.3: Comparison of wave transmission in a model with different element sizes

The shear wave velocity associated with the subsoil is 177 m/s (Kouroussis et al., 2011) and the highest frequency of interest can be considered as 100 Hz in relation to the vehicle modelling approach adopted. Based on that, the equivalent shortest wavelength can be calculated as 1.77 m. An element size of approximately 0.22 m must be considered to simulate this wavelength accurately. Based on the above considerations, the element size of the FE model is selected as described in Table 3.2.

Table 3.2: Element types and sizes of different parts of the FE model

Element	Element type	Element size
Rail	Timoshenko beam elements (B31)	0.1 m
Sleeper	8-noded linear brick elements (C3DR)	0.1 m
Ballast	8-noded linear brick elements (C3DR)	0.2 m
Sub ballast	8-noded linear brick elements (C3DR)	0.2 m
Subgrade	8-noded linear brick elements (C3DR)	0.2 m
Subsoil	8-noded linear brick elements (C3DR)	0.2 m

2.3.7. Boundary Conditions

As discussed before, keeping the number of elements in a FE analysis as low as possible is important to make the computational process more efficient. Yet, because the size of these elements depends on how waves travel and the highest-frequency motions we are interested in, reducing the number of elements actually means we need to minimize the volume included in the analysis. Therefore, deciding where to place the boundaries of the soil volume is a key aspect of FE modelling.

Realistically, soil is an infinitely spanning component, where bedrock can be considered a natural boundary condition. Other than that, boundary conditions in the FE model need to be carefully placed such that the region of interest is sufficiently far away from those boundaries. For the context of this study, three main boundary conditions are considered: symmetric boundary conditions, fixed boundary conditions, and absorbing boundary conditions. Figure 3.8 illustrates the assignment of each boundary condition on different surfaces of the FE domain.

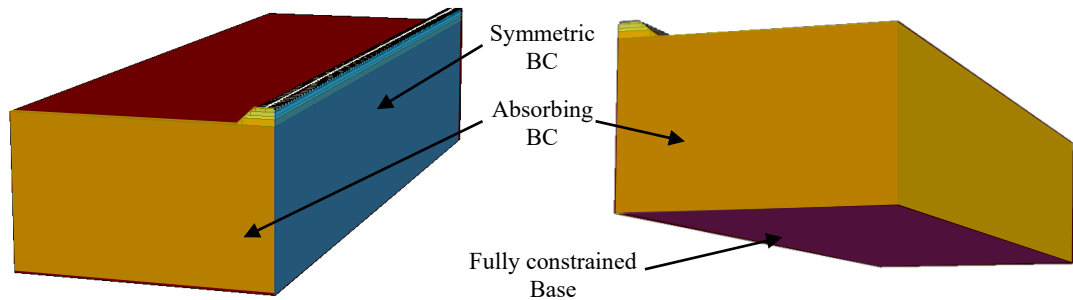


Figure 3.4: Complete overview of the boundary conditions of the FE model

To simulate the bedrock, the bottom surface of the FE domain is constrained with a fixed boundary condition, where translation along the x, y, and z axes and rotations around these axes are restrained.

As previously described, considering the symmetry of the railway track, only half of the track is considered in the FE model. Say the rail is spanning along the Z axis, and the Y axis depicts the direction of gravity. In the ZY plane, a symmetric surface (perpendicular to the X axis) exists, as shown in the blue colour in Figure 3.6. Therefore, the X axis needs to be assigned for this surface symmetric boundary condition. XSYMM can be used to build this built-in boundary condition in ABAQUS. This Boundary condition restrains translations along the X axis ($U1=0$) and rotations around the Y and Z axes ($UR2=UR3=0$).

The most significant part in defining boundary conditions in problems associated with soil domains is avoiding waves reflecting back to the soil domain. This may lead to inaccurate results. Therefore, for boundary surfaces in an FE model where no natural boundaries exist realistically, absorbing boundaries need to be assigned. In the ABAQUS software, there are two approaches to assigning absorbing boundaries. One assigns a layer of infinite elements at the boundary surfaces, and the other assigns a viscous boundary using dashpots. However, the infinite elements relevant to the study

scenario (CIN3D8) do not support the ABAQUS Explicit solver we use to run the simulations. A viscous boundary condition using dashpots must be assigned on surfaces where reflection must be avoided.

Viscous boundaries are made up of viscous dashpots with fixed properties placed on every node in the boundary surface. The amount of energy these viscous materials can absorb is influenced by the angle at which the waves hit. Since waves can approach the boundary from various angles, some energy will always be reflected. To reduce the impact of this reflection from viscous boundaries, one can increase the distance between the boundary and the area of interest. Figure 3.9(a) illustrates how the placement of dashpots needs to be done to absorb the energy for S-waves and P-waves. A dashpot placed perpendicular to the boundary surface will absorb its energy since the P-wave occurs in a direction parallel to the wave propagation. Since S-waves happen perpendicular to the wave propagation, dashpots must be placed in a direction parallel to the boundary surface. Since there are two directions parallel to the boundary surface, two dashpots must be assigned along both those directions, as illustrated in Figure 3.9(b), which shows the assignment of dashpots on boundary surfaces in the developed FE model.

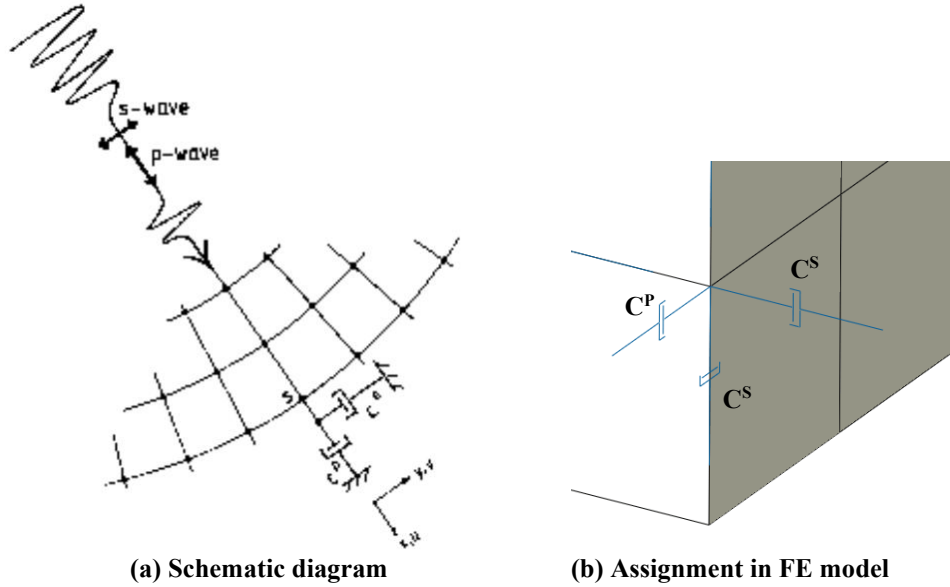


Figure 3.5: Viscous dashpot for absorbing energy of S-waves (C_s) and P-waves (C_p)

Dashpot coefficient for S-waves and P-waves can be calculated following Equations 3.2 and 3.3 respectively (Erlingsson, 1993).

$$C^S = \rho A C_S \quad (3.2)$$

$$C^P = \rho A C_P \quad (3.3)$$

Where, ρ is the density of soil, C is the speed of wave propagation, and A represents the absorbing area of the dashpot, which depends on the location of the dashpot. Different dashpot coefficients used for the subsoil, based on their location, are shown in Table 3.3.

Table 3.3: Dashpot coefficients based on their location

Dashpot	At centre node		At edge node		At corner node	
	Area (m ²)	Coefficient value (N.s/m)	Area (m ²)	Coefficient value (N.s/m)	Area (m ²)	Coefficient value (N.s/m)
C ^S	0.04	11,328	0.02	5,664	0.01	2,832
C ^P	0.04	21,120	0.02	10,560	0.01	5,280

2.4. Vehicle Modelling

Since the FE Model development and validation were based on the experimental investigations conducted by Degrande & Schillemans (2001) and Lombaert & Degrande (2009b), vehicle properties associated with trains relevant to the above studies are used in modelling vehicles. The main train incorporated in this study is the Thalys High Speed Train for validation and parametric study. In addition, the passage of the InterCity train between Brussels and Koln is also considered in validating the model for lower train speeds.

Train and axle configuration of Thalys HST is shown in Figure 3.10, and dynamic vehicle properties of it are explained in Table 3.4 as given by Kouroussis et al. (2011). Thalys HST consists of two locomotives (traction cars), one at the front pulling the other carriages and one at the back supporting by pushing. Locomotives are supported by two bogies, where two axles support each bogie. The carriage next to the locomotive is supported by one and a half bogies, where the second bogie is shared with the carriage next to the sidecar. All other central carriages, depicted as central cars in Figure 3.10, share both bogies with neighbouring carriages. In summary, Thalys HST has two locomotives, two side cars and six central cars, leading to 13 bogies and 26 axles.

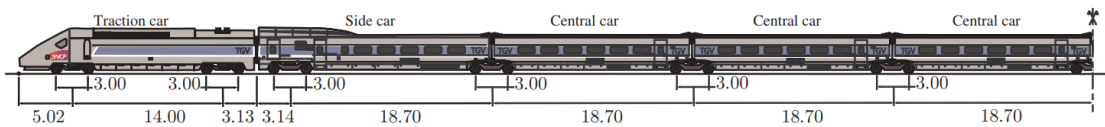


Figure 3.6: Configuration of Thalys HST

Table 3.4: Thalys HST dynamic properties (Kouroussis et al., 2011)

	m_c (kg)	I_c (kg.m ²)	m_b (kg)	I_b (kg.m ²)	m_w (kg)
Traction car	53,442	1.15×10^6	3,261	2.87×10^3	2009
Side car	34,676	1.49×10^3	8,156	7.19×10^3	2009
Central car	14,250	0.61×10^3	1,400	1.23×10^3	2050
	k_1 (MN/m)	c_1 (kNs/m)	k_2 (MN/m)	c_2 (kNs/m)	
Traction car	2.09	40	2.45	40	

Side car	2.09	40	2.45	40
Central car	1.63	40	0.93	40

m_c : car-body mass; m_b : bogie mass; m_w : wheel set mass; I_c : car-body pitch inertia; I_b : bogie pitch inertia; k_1 : primary suspension stiffness; k_2 : secondary suspension stiffness; c_1 : primary suspension damping; c_2 : secondary suspension damping

The InterCity train consists of one locomotive, seven standard central coaches, and one back coach. The carriage length L_t , the distance L_b between bogies, the axle distance L_a , the total axle mass M_t , the sprung axle mass M_s and the unsprung axle mass M_u of all carriages are summarized in Table 3.5.

Table 3.5: InterCity train mass properties (Lombaert & Degrande, 2009b)

	Axles	L_t (m)	L_b (m)	L_a (m)	M_t (kg)	M_s (kg)	M_u (kg)
Traction car	4	19.11	10.40	3.00	22,500	19,677	2,823
Side car	4	26.40	18.40	2.56	11,610	10,100	1,500
Central car	4	26.40	18.40	2.56	11,830	10,286	1,544

2.4.3. Lumped Mass Model

A similar approach can be adapted in vehicle modelling as the track is modelled considering the symmetry. Since the train coach is also symmetrical in the direction of train movement, it can be divided into two separate spring-damper systems replicating half of the coach. This approach facilitates reduced computational demand while producing results similar to those of modelling the full coach as a single body (Galvín et al., 2010b).

The mass components discussed in Tables 3.4 and 3.5, namely, unsprung (wheel) mass, (semisprung) bogie mass, and sprung (car-body) mass, are interconnected by spring and damper elements equivalent to the train's primary and secondary suspension characteristics. The lumped mass model can be developed, as shown in Figure 3.11.

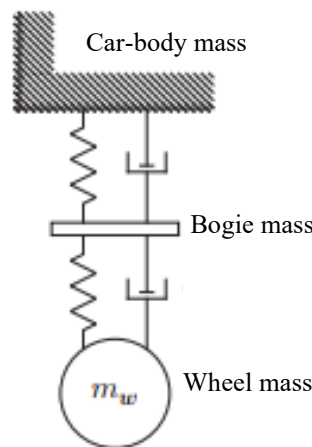


Figure 3.7: Lumped Mass Model

Considering the equal distribution of car-body and bogie mass among the available wheels in the carriage, the lumped mass model can be further simplified. For the case of a traction car, the body mass relevant to a single lumped mass model can be taken as $m_c/8$, and the mass relevant to the bogie can be taken as $m_b/4$. Similarly, considering the number of axles supported by different carriage types, the mass relevant to each component of the lumped mass model can be derived.

Considering the lumped mass model, equation of motion for the system can be written as Equation 3.2.

$$\begin{aligned} \begin{bmatrix} k_2 & -k_2 & 0 \\ -k_2 & k_1 + K_2 & -k_1 \\ 0 & -k_1 & k_1 \end{bmatrix} \begin{bmatrix} u_c \\ u_b \\ u_w \end{bmatrix} + \begin{bmatrix} c_2 & -c_2 & 0 \\ -c_2 & c_1 + c_2 & -c_1 \\ 0 & -c_1 & c_1 \end{bmatrix} \begin{bmatrix} \dot{u}_c \\ \dot{u}_b \\ \dot{u}_w \end{bmatrix} \\ + \begin{bmatrix} m_c & 0 & 0 \\ -k_2 & m_b & 0 \\ 0 & 0 & m_w \end{bmatrix} \begin{bmatrix} \ddot{u}_c \\ \ddot{u}_b \\ \ddot{u}_w \end{bmatrix} = \begin{bmatrix} m_c g \\ m_b g \\ m_w g \end{bmatrix} \end{aligned} \quad (3.2)$$

2.4.4. Wheel Rail Coupling

The following primary step is to connect the lump mass model above with the track model developed in the FE domain. This is achieved by coupling the wheel and the rail using a non-linear Hertzian contact spring (Johnson, 1985). This allows the load exerted on the rail by the wheel to be a function of the wheel and rail displacement as given in Equation 3.3.

$$\begin{aligned} F_w &= k_H(u_w - u_r - r)^{1.5}, u_w - (u_r + r) < 0 \\ &= 0, u_w - (u_r + r) > 0 \end{aligned} \quad (3.3)$$

Here, F_w represents the interaction force between the wheel, k_H represents the Hertzian constant, and u_w and u_r represent wheel displacement and rail displacement at a given time step respectively. The r represents the irregularity at the rail where the wheel is located at a given time step. In this equation, the inequality $u_w - (u_r + r) < 0$ depicts the scenario where the wheel and rail are in contact, while the other inequality depicts the condition where the wheel and rail are separated, leading to a contact force of null. In most cases, a value of $1 \text{ GN/m}^{1.5}$ is assumed for the Hertzian constant (D. Connolly, Giannopoulos, & Forde, 2013).

When defining wheel-rail contact, it is important to consider the effect of irregularities present in the rail surface caused by various operational effects such as braking and track debris. Apart from the low frequencies associated with rail vibrations, which are caused by wheel passage over sleepers at constant spacing, high-frequency excitation can be introduced to the system by considering the rail irregularity.

Considering the unavailability of rail irregularity data of the experimental site, it can be assumed that irregularity is of a periodic nature as a function of the distance along the rail, as given in equation 3.4 (D. Connolly, Giannopoulos, & Forde, 2013).

$$r(x) = A \cdot \sin (w_r x) \quad (3.4)$$

Here, x is the distance along the rail from the initial position, A is the amplitude of the irregularity, and W_r is the circular spatial frequency of the irregularity ($A = 0.005$ m and $W_r = 62.8$ rad/s).

With the inclusion of wheel-rail contact force, the system's equation of motion can be rewritten as Equation 3.5.

$$\begin{aligned} \begin{bmatrix} k_2 & -k_2 & 0 \\ -k_2 & k_1 + K_2 & -k_1 \\ 0 & -k_1 & k_1 \end{bmatrix} \begin{bmatrix} u_c \\ u_b \\ u_w \end{bmatrix} + \begin{bmatrix} c_2 & -c_2 & 0 \\ -c_2 & c_1 + c_2 & -c_1 \\ 0 & -c_1 & c_1 \end{bmatrix} \begin{bmatrix} \dot{u}_c \\ \dot{u}_b \\ \dot{u}_w \end{bmatrix} \\ + \begin{bmatrix} m_c & 0 & 0 \\ -k_2 & m_b & 0 \\ 0 & 0 & m_w \end{bmatrix} \begin{bmatrix} \ddot{u}_c \\ \ddot{u}_b \\ \ddot{u}_w \end{bmatrix} = \begin{bmatrix} m_c g \\ m_b g \\ m_w g + F_w \end{bmatrix} \end{aligned} \quad (3.5)$$

2.4.5. Implementation in the FEM software

ABAQUS FEM software does not provide built-in features to apply moving loads and hence, it needs to be implemented through user subroutines. For this case, the VDLOAD subroutine can be used to apply loads varying with location and time. However, to apply Hertzian contact between wheel and rail, their displacements must be known, which VDLOAD fails to accomplish. It does not possess the ability to access the real-time nodal displacements of the rail directly. As a solution for this, a separate subroutine called VUFIELD can be effectively employed to capture and write rail displacements into field variables. VDLOAD can then read those field variables, solve the vehicle's lumped-mass equations of motion, evaluate the Hertzian contact force, and finally apply the resulting force on the rail

The VUFIELD subroutine runs every time increment and assigns displacement values to user field variables at rail nodes. Here, it is used to sample the rail vertical displacement at the current contact locations of each wheel and store these values so that VDLOAD can retrieve them in the same increment.

VDFIELD subroutine computes the time-dependent moving load at the rail's integration points. It also explicitly advances the vehicle DOFs (quarter-car per wheel) so that the contact force depends on the current wheel and rail positions.

For each time increment, the following step-by-step approach is repeated under the considered subroutine.

1. Based on the starting point and train speed, longitudinal positions of wheels are updated, and contact nodes of the rail under each wheel are identified.
2. For each wheel within the rail region (60 m), VUFIELD reads the vertical rail displacement, u_r , at the nearest contact nodes and writes them into the field variable

3. VDLOAD retrieves the rail displacement from the field variable and solves the vehicle's lumped-mass equation of motion using an explicit central-difference scheme.
4. For each wheel, the contact condition of the rail and wheel is evaluated based on the rail displacement, wheel displacement, and track irregularity at the considered location.
5. Based on the contact conditions, F_w is applied on the rail as a line load at the current contact integration points. To avoid spurious nodal spikes, the contact force is distributed to a short rail segment (2 beam elements)

2.5. Validation Strategy and Datasets

2.5.3. Reference Field Measurement Studies

This study faced a significant limitation in obtaining field measurements as the available instrument to monitor vibration levels had accuracy-related problems and could not capture measurements at a high frequency. Additionally, there were no available measuring instruments to capture particle vibration at several locations during the passage of a single train. Therefore, field experiment data published in the literature were used to validate the FE model. The studies conducted by Degrande & Schillemans (2001) and Lombaert & Degrande (2009b) are selected for this purpose.

Degrande & Schillemans (2001) conducted free-field vibration measurements during the passage of Thalys HST on Brussels-Paris line. Here, they have measured vibration levels on the sleeper and at multiple offsets (4-72 m) from the track centre during several passages of Thalys HST at speeds ranging from 223 to 314 kmph. The paper reports a PPV vs distance plot at varying train speeds as shown in Figure 3.12. This dataset is widely used to validate railway vibration models, particularly because it spans a long-offset range and multiple speeds. Additionally, detailed vertical velocity time histories and their frequency content during the passage of Thalys HST at 314 kmph, are recorded as shown in Figure 3.13.

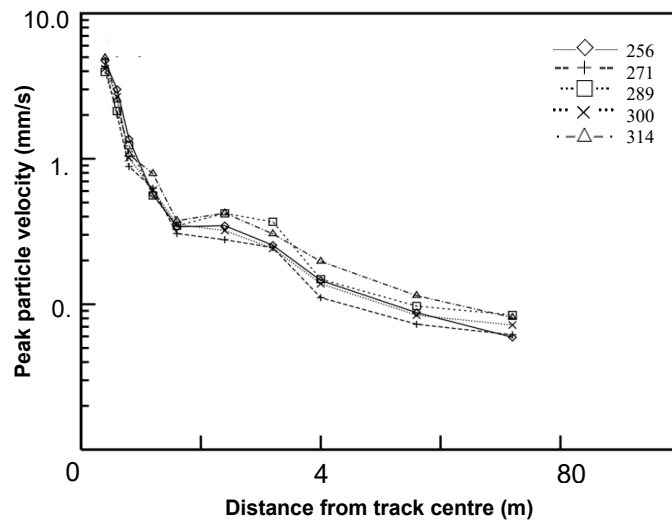


Figure 3.8: Measured PPV as a function of distance away from track (Degrande & Schillemans, 2001)

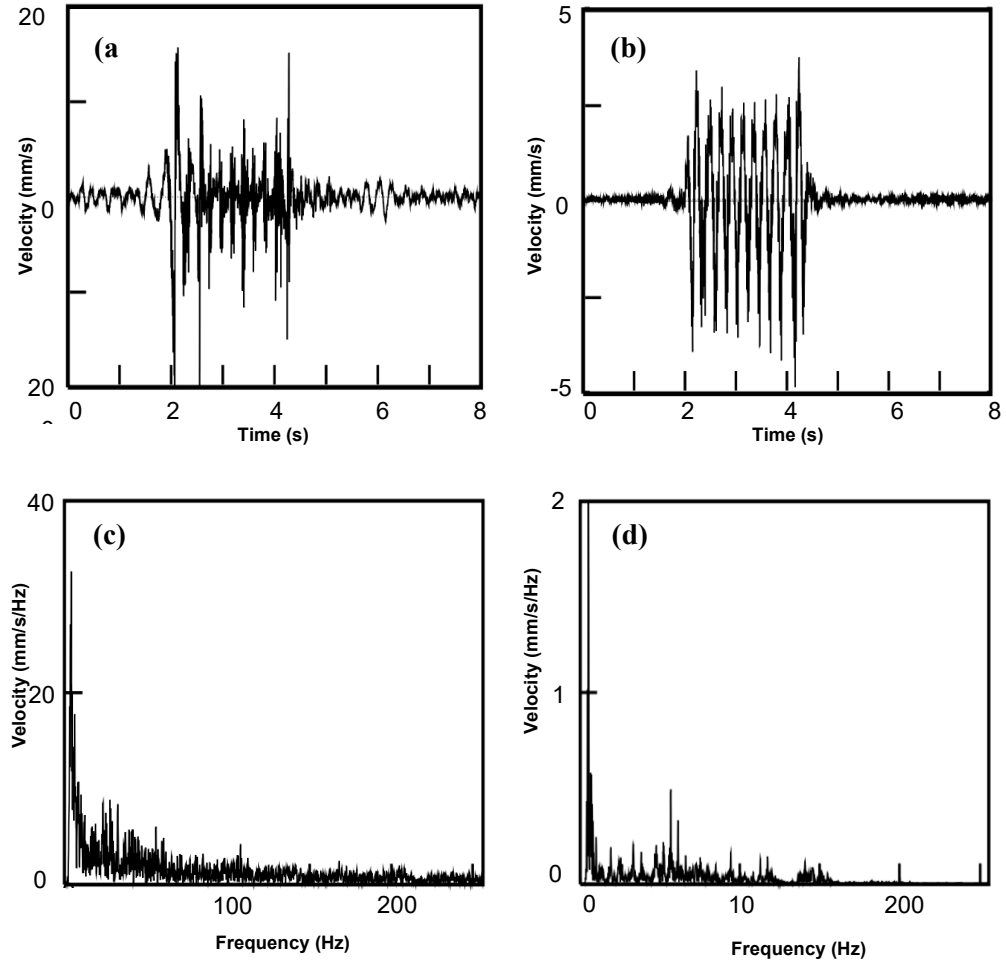


Figure 3.9: Vertical velocity time histories at (a) 4m (b) 8m from track and frequency content of vertical velocity at (c) 4m (d) 8m during passage of Thalys HST at 314 kmph (Degrande & Schillemans, 2001)

Since Degrande & Schillemans (2001) provide detailed vibration data only for the passage of Thalys HST at 314 kmph, field measurements given by Lombaert & Degrande (2009b) is used to extend the validation of the FE model for lower speeds as well. The vertical velocity time histories recorded at 16 m from the track centre during the passage of an InterCity train at 156 km/h and a Thalys HST at 218 km/h, as shown in Figures 3.14 and 3.15, respectively.

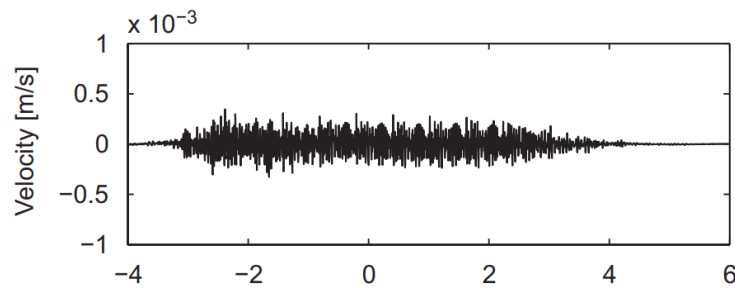


Figure 3.10: Vertical velocity time histories at 16m from track center during passage of InterCity at 156kmph (Lombaert & Degrande, 2009b)

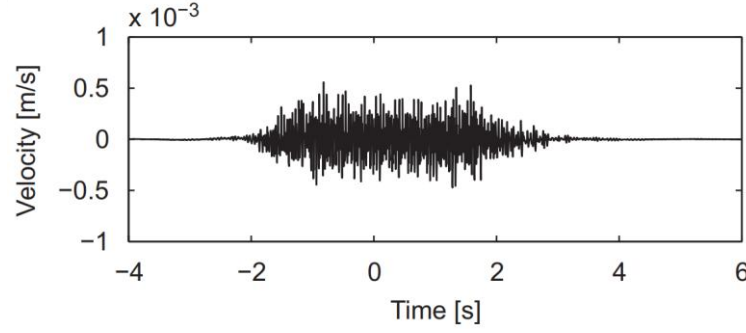


Figure 3.11: Vertical velocity time histories at 16m from track centre during passage of Thalys HST at 218 kmph (Lombaert & Degrande, 2009b)

2.5.4. Validation Metrics

Since the passage of a train is a highly dynamic event, the FE model was validated using four complementary metrics.

Vertical velocity time histories

The simulated vertical velocity variation ($v_z(t)$) of particles during the train passage was compared with the field measurements. For near-field response, offsets at 4-8 m were considered, while offsets at 12-19 m were considered for far-field response. Here, a few qualitative checks were focused on validation, namely, timing and envelope of the train passage, visibility of passage of the traction car with heavy axle load, and overall waveform shape consistent with field records.

Peak particle velocity attenuation with distance

For each receiver located 4–19 m from the track centre, the PPV was calculated using Equation 3.6, plotted against distance, and compared with the field trend. This allows the radiation and material damping in the FE model, along with the effectiveness of the absorbing boundary conditions, to be assessed.

$$PPV = \max|v_z(t)| \quad (3.6)$$

Frequency content (FFT)

A single-sided FFT was computed at each receiver on the window containing the train passage. FFT was derived by applying a Fourier transform to the detrended, windowed velocity time histories to obtain the amplitude vs frequency trend. Dominant frequencies were directly read from this spectrum. The key focus in this metric is to check the ability to simulate high-frequency excitation caused by rail irregularities.

Running RMS of Velocity

Running RMS of velocity can be effectively utilised to gauge time-varying energy during the passage, and it can be computed using Equation 3.7. For a time window, T_w , a value ranging from 0.125 to 1s can be used to balance time-frequency resolution. Since the instantaneous velocity varies rapidly, point-by-point comparison of time

histories is sensitive to phase and timing offsets. A running V_{rms} , computed over a short time window, provides a more stable signal envelope, reducing noise sensitivity. This makes validation more robust while capturing rise, peak and decay of vibration levels during the train passage.

$$V_{rms}(t) = \sqrt{\frac{1}{T_w} \int_{t-T_w}^t v_z^2(t) dt} \quad (3.7)$$

1/3-Octave Band Levels

Third-octave bands summarise the velocity signal into standard frequency ranges that match how vibration is assessed in practice. Here, velocity is passed through a set of filters centred at 4–80 Hz (e.g., 4, 5, 6.3, ..., 80 Hz) and the RMS in each band is taken over the train-passage window. This provides insight into the energy carried by each frequency while being less sensitive to small timing or spectral artefacts than raw FFT lines.

2.6. Chapter Summary

This chapter describes the complete method for modelling and analysing ground vibration caused by trains. A 3D FE model of the track and soil was created in Abaqus, using realistic layers and materials, and was meshed according to wavelength requirements. To minimize reflections and maintain a compact domain, symmetry, fixed, and viscous boundaries (like dashpots) were added. The train was modelled as a simple multi-body system, and the wheel-rail forces were calculated using nonlinear Hertzian contact that included periodic irregularities in the rail, following previous research. Since Abaqus does not have a built-in moving-load contact feature, rail displacements were recorded by VUFIELD at each time increment, while VDLOAD adjusted the vehicle's degrees of freedom and applied the resulting contact forces, creating a staggered, explicit link between the train and the rail. The model's accuracy was checked against actual field data using different metrics: velocity time histories, peak particle velocity (PPV) compared to distance, frequency content (FFT), running V_{rms} , and 1/3-octave bands, ensuring that waveform timing, attenuation, and spectral peaks were consistent for both near-field and far-field receivers. With the model validated, a parametric study was conducted to evaluate the impacts of thickness and stiffness, on soft and stiff soils at various distances.

CHAPTER 4

FE MODEL VALIDATION AND DESIGN OF PARAMETRIC STUDY

4.1. Introduction

This chapter details the validation of the 3D FE model developed for simulating train-induced ground vibration and outlines the framework for the subsequent parametric study. The chapter begins by benchmarking the FE model against published field measurements, using a range of metrics to ensure that the simulation accurately captures the dynamic behaviour of the train-track-soil system under realistic loading conditions. Following validation, the chapter presents the systematic design of a comprehensive parametric study aimed at evaluating the influence of key mitigation measures, USPs and BMs on ground vibration attenuation. Emphasis is placed on the rationale for selecting model parameters, baseline ground conditions, and the process for constructing the parametric matrix covering thickness and stiffness variations under both soft and stiff soil profiles. The responses are evaluated at near-field (0-10 m) and far-field (>10 m) receiver locations.

4.2. Validation of 3D FE Model

In this section, the validation results of the 3D FE model are presented in relation to the field measurements obtained from the homologation test conducted by Degrande & Schillemans (2001). Validation metrics include waveform and timing, energy envelope, peak response evaluation, energy decay law, and frequency content analysis.

4.2.1. Velocity Time Histories

Figure 4.1 compares field and simulation responses at four receivers from the track centreline. Near track response demonstrates high frequency modulation with sharp spikes. Out of all receivers, the onset of the train passage is the earliest at the 4 m receiver (Figure 4.1(a)), and the FE model and field traces rise almost simultaneously, while some slight local phase difference is visible on some spikes. However, the peak has occurred within the same time window for both responses. At the 8 m receiver (Figure 4.1(b)), the signal is comparatively smoother than at 4m. As expected from wave propagation, arrival is delayed relative to the 4m receiver, and the FE model and field responses demonstrate a similar envelope. The dominant peak appears between 2.4-2.6 s time window in the FE model and between 2.6-2.8 s time window in the field measurements, where the end locomotive of the train exits the track domain. Further, at 12 m from the track (Figure 4.1(c)), the waveform becomes visibly smoother and due to higher frequency content diminishing, envelope-domination can be observed. Considering the realistic behaviour, this is justifiable as the high frequency

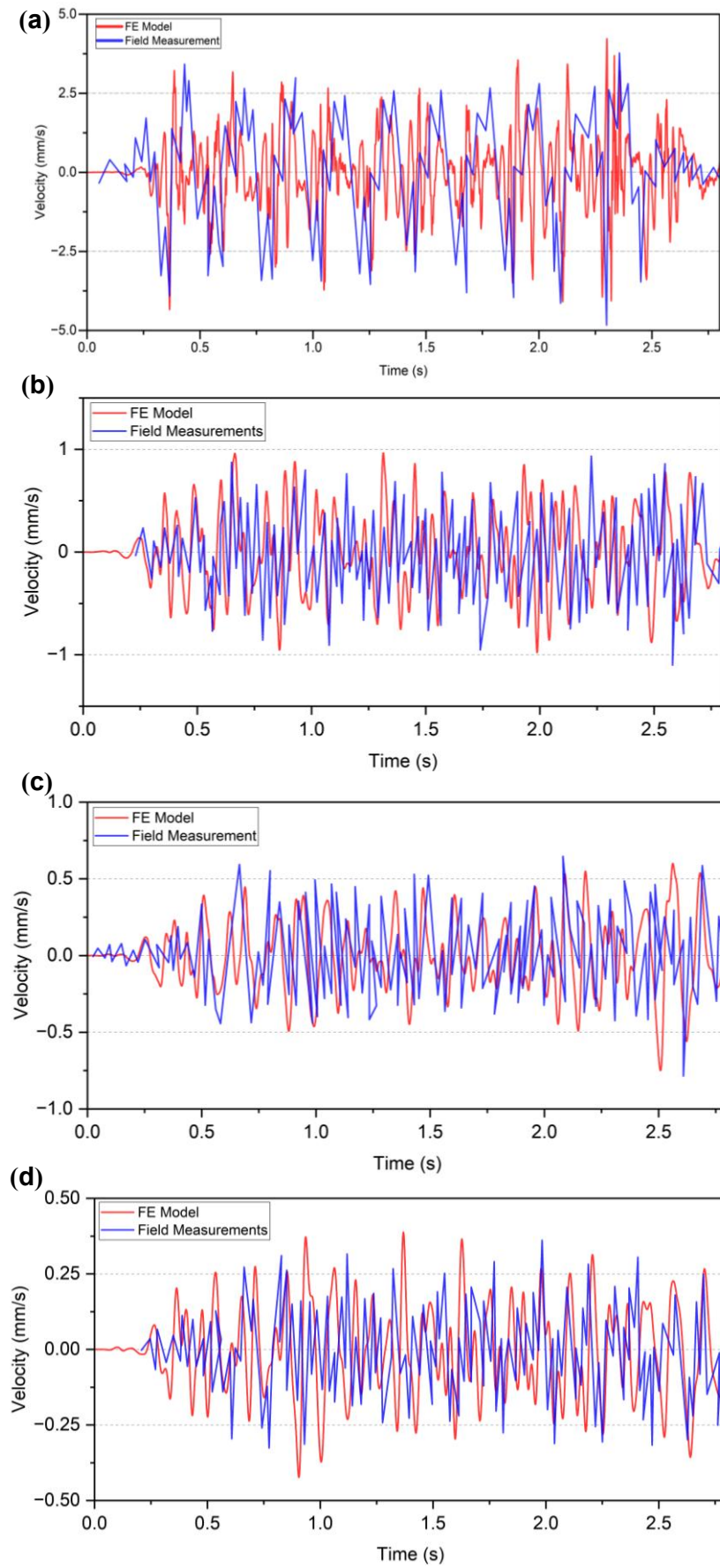


Figure 4.1: Vertical velocity time histories at receivers at (a) 4 m (b) 8 m (c) 12 m (d) 16 m

Excitation attenuates quickly within the near-field region, while the far-field excitation is governed by low frequency excitation. The absolute peak time in the model and field responses is recorded simultaneously, with a few minor shifts between peaks also observed. The 16m receiver (Figure 4.1(d)) possesses the smoothest signal of the four with a low-frequency envelope. Peaks at the start and the end of the signal have occurred simultaneously with the same magnitude. However, during the mid-passage of the train, peaks observed in the model response are slightly higher than the field response. The possible reason for this might be the efficiency of viscous boundaries in damping the incoming waves.

The data shows an apparent change in signal characteristics from 4 m to 16 m. Initially, the signals are highly modulated and full of spikes, but as the distance increases, they become smoother and more focused on the waveform's envelope. The signals' arrival and peak times are progressively delayed with distance. Overall, the peaks observed in the model align closely with the measured data, showing only slight differences in timing, which decrease as the distance increases. This pattern is consistent with what is expected in signal propagation and reinforces the accuracy of the FE model used in the study.

4.2.2. Frequency Content of Vertical Velocity

Figure 4.2 shows the PPV variation with the frequency at 4 different distances. At the 4 m receiver, the spectrum is broad and active, characterised by a strong low-frequency peak within the single-digit to low-teens Hz range, accompanied by numerous secondary lines that extend above 100 Hz. The FE model effectively replicates the significant low-frequency peak and the dense cluster of components observed up to several tens of hertz. However, compared to the field measurements, the model exhibits slightly more energy in the mid-frequency range (approximately 20 to 60 Hz). In contrast, the field data display a few distinct narrow-band peaks, particularly in the mid- and high-frequency ranges, typical of measurements close to the track. As the offset increases to 8 m, the spectrum shows a downward shift and appears less irregular. The model and the field data maintain the same prominent low-frequency peak, and the positions of the peaks in the 10 to 60 Hz range are generally aligned. Additionally, high-frequency content (greater than approximately 100 Hz) is significantly diminished in both traces; the model shows a slightly earlier roll-off, while the field spectrum still retains some faint lines up to 100 Hz.

At a distance of 12 m from the receiver, a notable spectral smoothing occurs, with energy primarily concentrated in two distinct regions: a low-frequency cluster below approximately 20–25 Hz and a mid-frequency feature around 50–60 Hz. The FE model effectively captures this division. While the field data exhibits a more pronounced narrow peak in the mid-frequency range, the model shows a slightly higher energy level in the low-frequency cluster. Beyond 100 Hz, both spectra converge to similar levels at the noise floor. At 16 m, the response predominantly features low frequencies, with most energy observed below 30–40 Hz. Both the model and field data align in their primary peak location and overall roll-off characteristics. The FE model

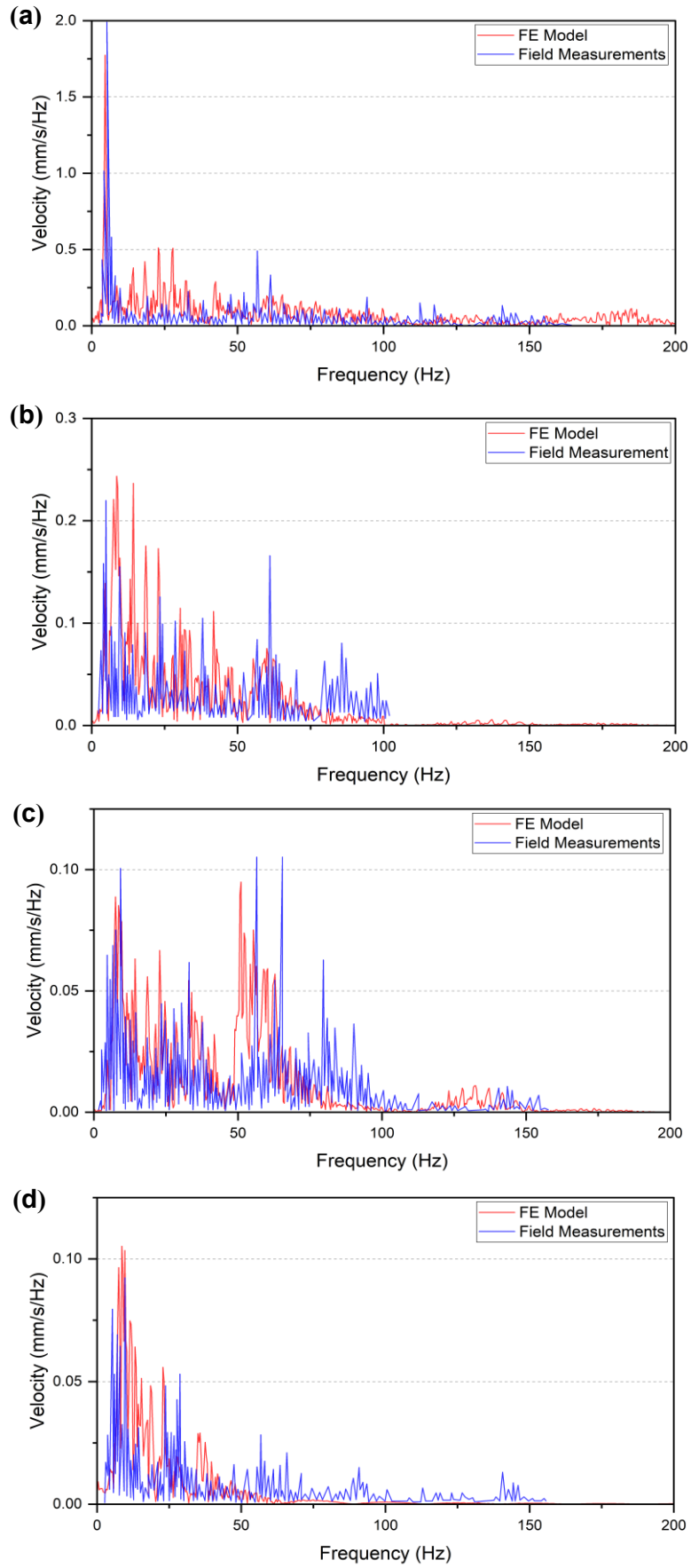


Figure 4.2: Frequency content relevant to vertical velocity at receivers (a) 4 m (b) 8 m (c) 12 m (d) 16 m

presents almost no energy above 60 Hz. In contrast, the field recordings indicate only isolated, low-level signals extending into higher frequencies, which is consistent with the limitations of measurement noise in far-field conditions.

Across 4–16 m, the positions of the prominent peaks and the overall spectral shapes in the model match the measurements. Differences are minor and mainly in the height of secondary features, not where the energy sits. This consistency by frequency supports the validity of the FE model for predicting vibration content across distance.

4.2.3. Root Mean Square Velocity (Running)

Since the instantaneous velocity in time histories varies rapidly, point-by-point comparisons are sensitive to small timing differences. For a more static comparison of responses, running V_{rms} can be used. It converts fluctuations in time histories into an energy envelope, giving a stable basis. Figure 4.3 illustrates the running V_{rms} at each receiver discussed earlier for the 2.8 s passage of Thalys HST at 314 kmph.

In the near-field receivers (4 m and 8 m), the field data represent peak V_{rms} at the end of passage, which is relevant to the case where the rear traction car moves away from the receiver. The FE model response also captures this phenomenon, where peaks can be observed where the rear traction car leaves the model domain. In contrast, peaks were observed in the far-field receivers (12 m and 16 m) when the front traction car moved towards the receivers and entered the modal domain in the field data and FE model response, respectively.

At the 4 m receiver (Figure 4.3(a)), the shape and timing of the envelope are closely matched for both responses. At the end of the passage, the peak V_{rms} is recorded as

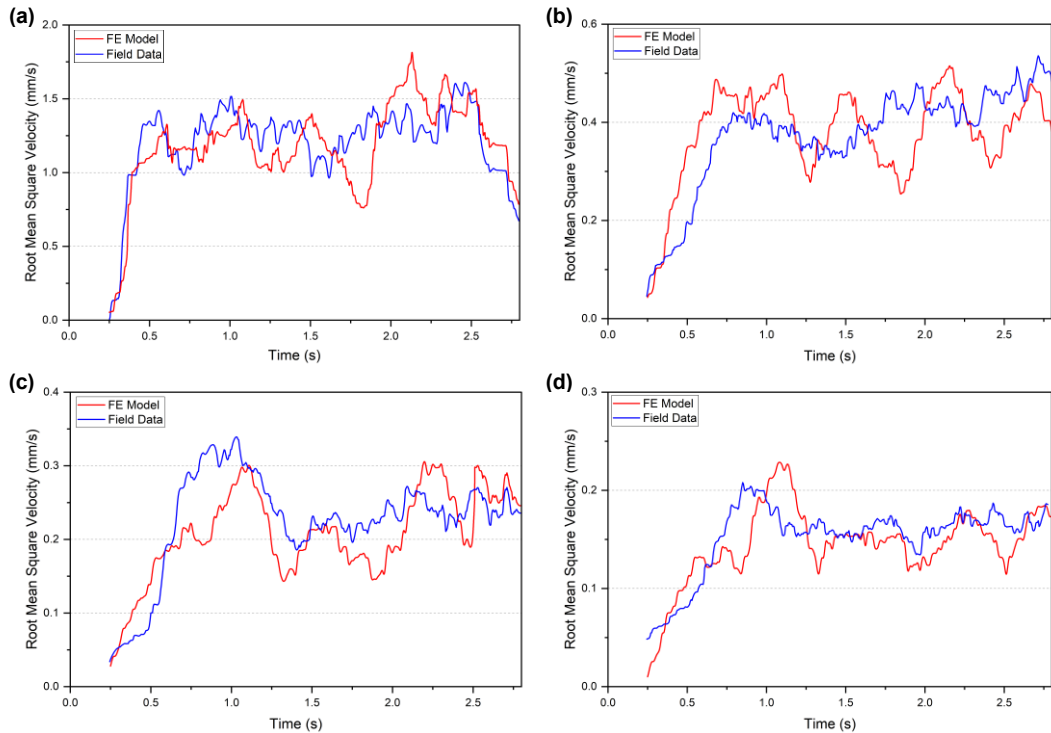


Figure 4.3: Comparison of running V_{rms} of response from FE model with field data at receivers (a) 4m (b) 8m (c) 12m (d) 16m

1.75 mm/s for the FE model compared to 1.6 mm/s peak of the field data. At the 8 m receiver (Figure 4.3(b)), several overestimated peaks and lows are seen in the mid-passage; however, the overall agreement is acceptable. At the far-field receivers (Figure 4.3(c) and Figure 4.3(d)), the envelope shows a gradual rise to a shallower plateau. Field data and FE model responses remain consistent throughout the passage, with minor phase differences and peak variations being around 0.04 mm/s.

Overall, the V_{rms} results confirm that the FE model reproduces the time-varying energy of the pass-by, with similar onset times, comparable plateau or peak levels, and consistent envelope shapes.

4.2.4. Vibration Attenuation

In addition to previously discussed metrics, attenuation of the intensity of vertical vibration in the FE model is compared with the field measurements in the reference study. Through this behaviour, the soil domain of the FE model can be validated in relation to the actual behaviour of the soil. In addition, the effectiveness of the viscous boundary condition used in the FE model is compared in simulating the infinitely spanning soil domain. Vertical velocities of particles observed at 4, 6, 8, 12, 16, and 24 m in the field experiment are compared with vertical velocities obtained from the FE model at similar receiver locations. Since the FE soil domain spans only 20 m in the lateral direction, the furthest receiver is considered at 19 m, as the 20 m receiver is located at the boundary surface, and the connection of boundary surface nodes with dashpots mimicking viscous boundaries can produce inaccurate results. In the reference field experiment, velocity measurements at 19 m or 20 m are not available. Therefore, velocity measurements at 16 and 24 m were considered in approximating the velocity level at 19 m through linear interpolation. Attenuation behaviour is compared using PPV attenuation (Figure 4.4), which validates the model's ability to reproduce the peak amplitude decay of ground vibration, and V_{rms} attenuation (Figure 4.5), which validates the energy-based decay of vibration over the whole passage. Since V_{rms} is an average response over time, it is less sensitive to brief spikes or small timing offsets and therefore complements PPV by checking the overall radiated energy.

Both PPV and V_{rms} decrease rapidly within the near-field (4-8 m), while the reduction becomes much gentler in the far-field (12-19 m). This agrees with the realistic behaviour of the vibration attenuation since both low-frequency and high-frequency excitations support the near-field vibration levels. In contrast, the far-field vibration levels are governed by the low-frequency excitations. Vibration intensities of the FE model follow the same attenuation gradient as the field measurements in both metrics. This indicates that the model's soil damping and viscous boundary are behaving realistically. Figure 4.4 depicts that the PPV of the FE model is slightly lower than the field measurements in the near-field; however, it tracks closely at far-field from 12-19m. Regarding the V_{rms} , the FE model is consistently lower compared to field measurements in both near- and far-field. Additionally, the model response at the 19m

receiver, all on the same attenuation curve, supports the soil representation and the boundary treatment used.

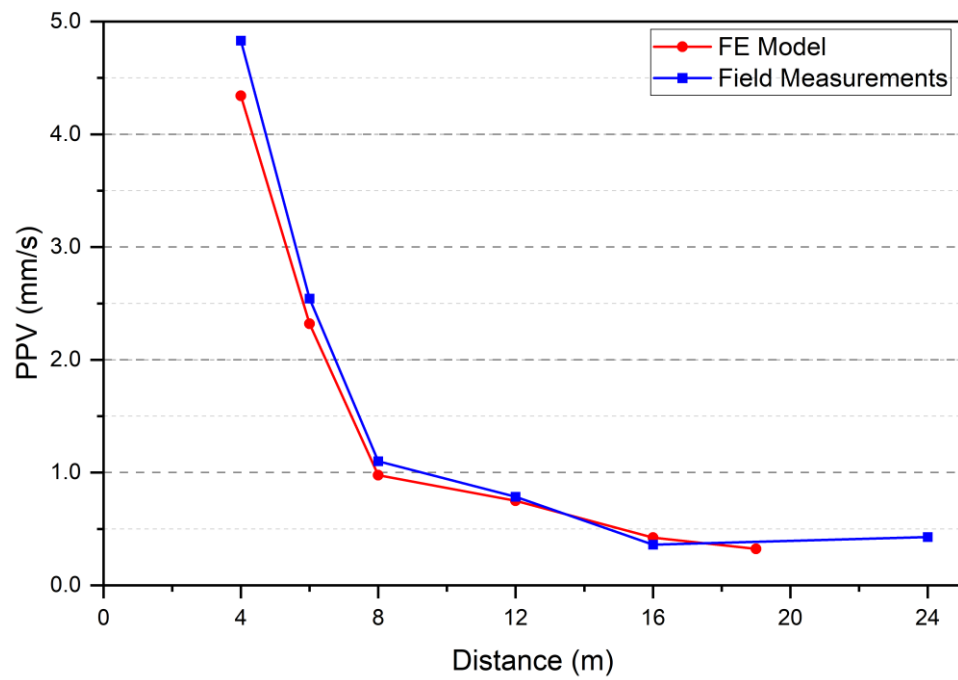


Figure 4.4: Attenuation of PPV with the distance from track centre

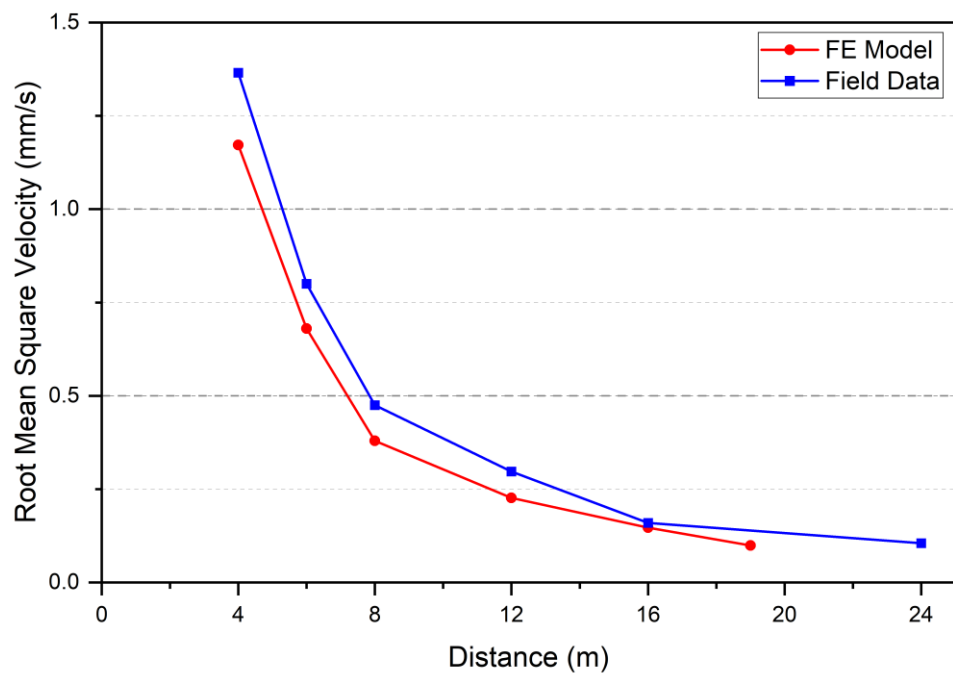


Figure 4.5: Attenuation of V_{rms} with the distance from track centre

Table 4.1 provides a further numerical comparison of PPV and V_{rms} between Model response and field measurements, along with percentage variation.

Table 4.1: Percentage variation of PPV and V_{rms} between FE model response and field data

Receiver	PPV			V_{rms}		
	FE Model (mm/s)	Field Data (mm/s)	Variation	FE Model (mm/s)	Field Data (mm/s)	Variation
4 m	4.341	4.831	10.14%	1.172	1.365	14.18%
6 m	2.320	2.543	8.75%	0.680	0.800	15.00%
8 m	0.978	1.100	11.11%	0.379	0.475	20.15%
12 m	0.750	0.786	4.60%	0.227	0.297	23.69%
16 m	0.423	0.361	-17.07%	0.147	0.160	8.20%
19 m	0.324	0.386	16.27%	0.099	0.139	28.78%

4.2.5. Limitations of the FE Model

Although the accuracy of the FE model predictions is checked under 4 different parameters, it still possesses following limitations.

1. The roughness profile was assumed to be periodic. This helps to control the excitation in a simple way, but it does not capture the full variability seen on real tracks. Using measured roughness spectra from site surveys would improve the realism, especially for mid–high frequency content.
2. Model validation was aligned with the Brussels–Paris HST track and soil conditions, following the field data by Degrande & Schillemans (2001). While agreement was good for that case, applicability to other soils and track forms still needs to be tested. Further validation on different sites would strengthen confidence in wider use.
3. Only homogeneous soil profile was modelled. Many corridors have layered or partially saturated ground. Layering can change dispersion, cause waveguide effects, and shift dominant bands. The accuracy of the model under layered profiles should be checked in future work.
4. The validation focused on high-speed passages. Prediction accuracy at lower speeds (about 50–100 km/h, common in Sri Lanka) has not been demonstrated here. In addition, most simulations are sub-critical; near-critical/super-critical regimes (close to Rayleigh speed) were not explored, so potential resonance-type amplification at very high speeds lies outside the present scope.
5. Viscous dashpots were used at absorbing boundaries. They work best for normal incidence. For oblique/grazing incidence, absorption is imperfect and small reflections can return into the domain. This likely explains the slight PPV rise at 19 m compared to 16 m. Larger domains or thin absorbing layers could reduce this effect.

6. Rails, sleepers, ballast and sub-ballast were modelled as linear elastic with small damping. Ballast degradation, sleeper–ballast slip, and settlement under repeated loading were not included. The model therefore targets short-term vibration transmission, not long-term maintenance or geometry evolution.
7. Receivers were placed in the free field. Soil-structure interaction was not modelled, so the results should not be read as direct predictions of indoor floor vibration.

Overall, the model is suitable for relative comparisons under the representative soil and validated speed range. Site-specific prediction should include measured roughness, layered soils, extended speed studies, enhanced boundaries, and, where needed, building interaction.

4.3. Design of Parametric Study

The design of the parametric study involved developing a systematic framework to evaluate how the mechanical properties of USPs and BMs influence vibration attenuation performance under different subgrade conditions. The study employed a validated three-car train model to ensure computational efficiency and realistic dynamic excitation. The vibration reduction efficiency of USPs and BMs was quantified using Insertion Loss (IL), a widely accepted metric defined as Equation 4.1 (Hunt et al., 2003). A positive IL indicates effective vibration reduction, whereas a negative value signifies amplification relative to the reference (no mitigation) case. This study framework ensures an objective evaluation of vibration mitigation performance across all test scenarios while maintaining consistency between soil conditions, parameter variations, and measurement metrics.

$$IL = 20 \log_{10} \left(\frac{\text{Vibration Intensity with mitigation measure}}{\text{Vibration Intensity without mitigation measure}} \right) \quad (4.1)$$

4.3.1. Optimizing Vehicle Model to Improve Simulation Efficiency

In the previous chapter, it was discussed how the developed 3D FE model accurately captures the realistic behaviour of vibration generation and propagation. Using the full train model for validation in the parametric study is not time-efficient, as a single simulation takes approximately 13 hours to complete. Since our focus is on understanding the behaviour of vibration attenuation in this study, it is unnecessary to model the entire train.

To identify a more efficient train model for the parametric analysis, we compared four different models:

- Full train

- Traction car only
- Two-car model (traction and side cars)
- Three-car model (traction, side, and central cars)

The responses of Peak Particle Velocity (PPV) attenuation obtained from the FE models for these four configurations are illustrated in Figure 4.6.

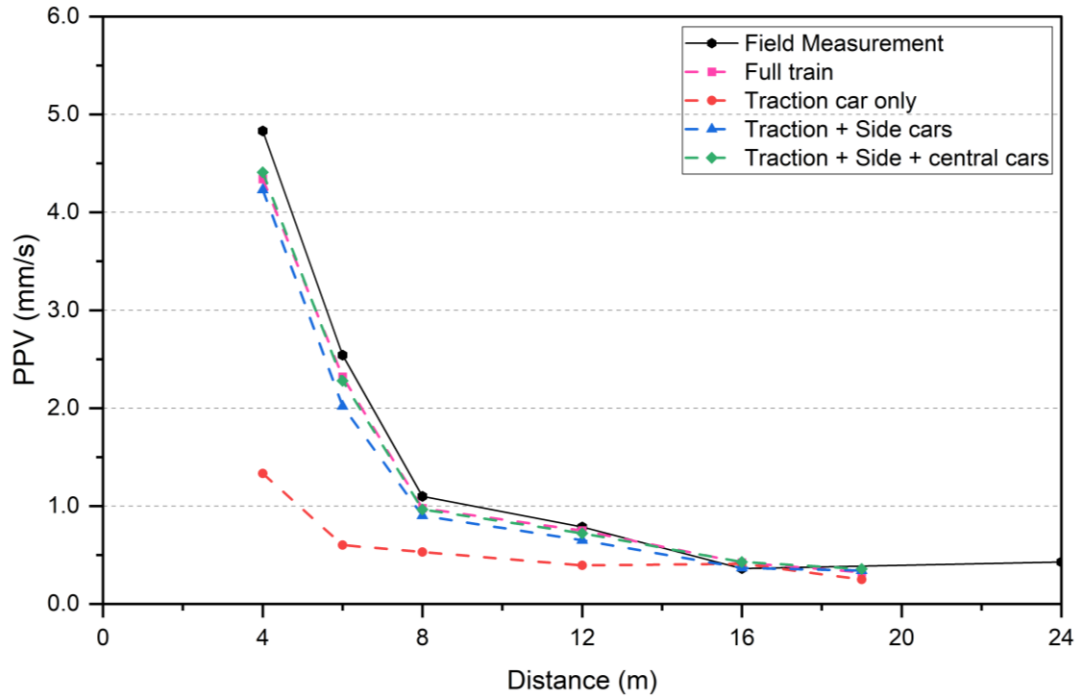


Figure 4.6: PPV attenuation along free field for different train models

Both the two-car and three-car models were able to reproduce responses similar to those of the full train model. However, the three-car model was more successful in closely tracing the overall response of the full train. The responses in both near-field and far-field points were found to be comparable to the full train response, as well as in agreement with field measurement data. Conversely, the traction car only model tended to underestimate vibration levels in most regions of the free field.

The three-car model benefits from a comprehensive representation of dynamic vehicle properties, as noted in Table 3.4. Additionally, it covers the entire 60 m soil domain, allowing the model to effectively account for the wheel loads acting along the complete rail length. This improved modelling efficiency reduced the simulation time by around 50% compared to the full train model. Therefore, it was decided to use the three-car model for all subsequent parametric simulations.

4.3.2. Selection of Soil Conditions and Baseline Response

Since the focus of this study is to evaluate the parameters of USP and BM, variation of soil conditions is not extensively studied. However, there is a possibility that the contribution of USP and BM may vary depending on the subsoil condition where the

track is supported. To address this consideration, two distinct soil conditions, soft soil and stiff soil, were examined based on the shear wave velocity of the subsoil. Table 4.2 illustrates the material properties used in the FE model for the considered soil conditions (Bongini et al., 2012).

Table 4.2: Material properties for considered soft and stiff soil conditions (Bongini et al., 2012)

	Soft soil	Stiff soil
Shear velocity (m/s)	200	400
Young's modulus (MPa)	140	720
Density (kg/m³)	1400	2000
Poisson's ratio	0.3	0.3

Small material damping was applied consistently in both cases. The damping values were kept the same in the parametric runs so that changes in response are due to the mitigation measures only.

Before moving on to the effect of USP and BM, the response of the two soil conditions was checked under the three-car train model introduced earlier. This gives a clear reference to compare the performance of mitigation measures. In this section, the in-depth behaviour of several metrics will be discussed, focusing on the near-field and far-field. For a detailed analysis of near-field behaviour, the response of the receiver at 4 m from the track centre is considered, while the receiver at 16 m is considered to understand the detailed far-field behaviour.

Figure 4.7 illustrates the time history of vertical velocity during the passage of a three-car train model in two soil conditions without mitigation measures. Figure 4.7 (a) shows that soft soil exhibits larger peaks and a more complex waveform, indicating that the excitation caused by the wheel at the rail is transmitted to the ground surface with minimal damping. In contrast, the stiff soil trace has smaller peaks and a smoother waveform, demonstrating its greater damping capacity compared to soft soil. Both signals are noticeably smoother and of lower magnitude at the far-field receiver shown

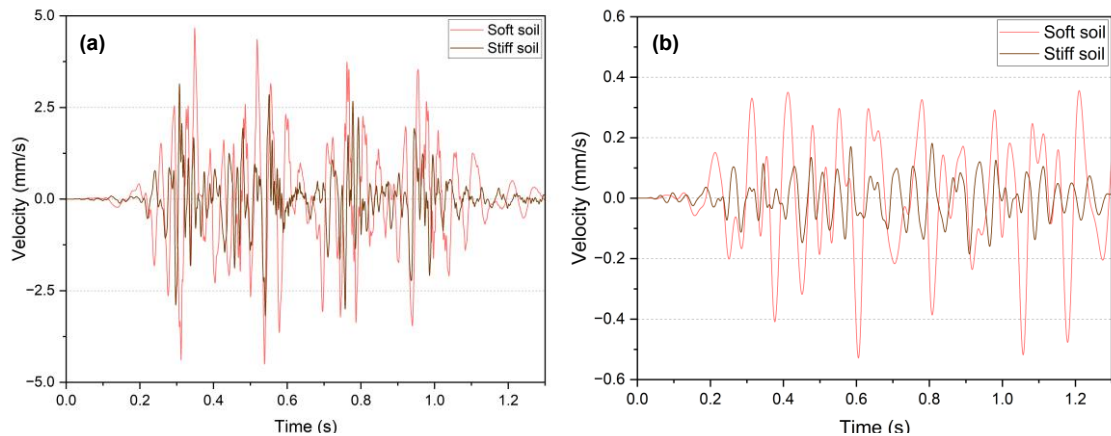


Figure 4.7: time history of vertical velocity at (a) receiver at 4 m (b) receiver at 16 m

in Figure 4.7 (b). The soft soil still exhibits higher peaks than the stiff soil and retains slightly more low-frequency content. Vibration in stiff soil decays more rapidly.

Figure 4.8 illustrates the decay of vibration velocities as the distance from the track center increases. PPV decreases at similar rates for both soils, while the soft soils maintain higher values in both near-field and far-field regions compared to stiff soil, indicating their lower damping ability. Vibration levels in soft soil continue to decline in the far-field, but in stiff soil, very low excitations (close to 0 mm/s) remain constant beyond 16 m without dampening. The RMS of the velocity reflects the total excitation energy during the entire train passage. This also shows the same trend as PPV, with the soft soil carrying more energy at every distance.

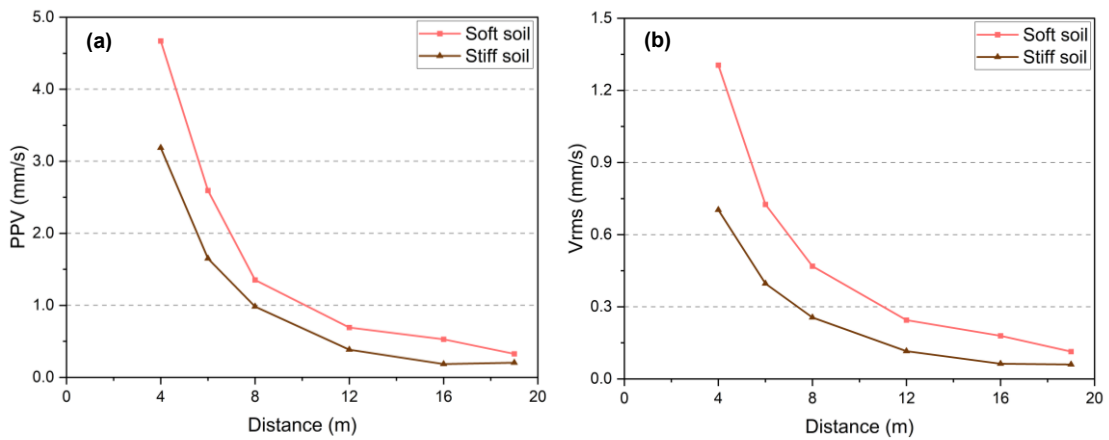


Figure 4.8: (a) PPV Attenuation and (b) V_{rms} attenuation along the free field

These results define the baseline for both ground conditions. All mitigation cases (USPs and BMs) will be evaluated relative to this baseline: insertion loss at each receiver and in each 1/3-octave band is calculated as the difference between the mitigated case and the corresponding no-mitigation case on the same soil.

4.3.3. Selection of Values for the Parametric Study

Since the USPs and BMs are thin rubber pads, their influential parameters for the vibration damping, thickness and stiffness are considered in this parametric study. Table 4.3 lists the values used in the parametric study. Thickness values are taken from sizes commonly offered by manufacturers for conventional ballasted tracks referring to their brochures. The mid values of thickness (10 mm for USP and 23 mm for BM) are the common baseline that are typically used, and the other values are selected to represent the upper end and lower end. Stiffness values are selected based on standard classes used in international guidelines. Use of values specific for standard classes instead of arbitrary numbers allows the results to be easily compared with practice.

Mid values of each parameter are considered as reference when varying other parameters. For example, in USP, when varying the stiffness over 3 ranges, thickness

is kept constant as 10 mm for all stiffness cases. Similarly, when varying the thickness over 3 values, stiffness is kept constant at medium level (0.25 N/mm³).

Table 4.3: Selected values for the parametric study

Parameter	Values	Reference
USP		
Thickness	8 mm	Getzner (2021), Pandoral (2020)
	10 mm	
	15 mm	
Stiffness	0.15 N/mm ³ (Soft)	Kaewunruen et al. (2017)
	0.25 N/mm ³ (Medium)	
	0.35 N/mm ³ (Stiff)	
BM		
Thickness	15 mm	Sehner et al. (2023), UIC (2022)
	23 mm	
	30 mm	
Stiffness	0.05 N/mm ³ (Soft)	DIN 45673-5 (2010), Alves Costa et al. (2012b)
	0.09 N/mm ³ (Medium)	
	2.22 N/mm ³ (Stiff)	

4.4. Chapter Summary

This chapter confirms that the 3D FE train–track–soil model is reliable and then explain the plan for the parametric study. The model is checked against published field data using complementary measures (time histories, PPV and V_{rms} with distance, and frequency content) and shows good overall agreement. To keep simulations efficient, a calibrated three-car train abstraction is selected because it reproduces the full-train response closely. Finally, the chapter defines the study matrix: two representative soils (soft and stiff) and practical ranges of USP/BM thickness and stiffness, with performance reported mainly as insertion loss and banded levels at standard receiver offsets.

CHAPTER 5

UNDER SLEEPER PAD: PARAMETRIC STUDY AND PERFORMANCE EVALUATION

5.1. Introduction

This chapter represents a parametric investigation to show the effects of Under Sleeper Pad (USP) for mitigating train-induced ground vibration. The study evaluates USP performance under two ground conditions: soft and stiff soil. For each soil setting, the dynamic response of the track is systematically analysed by varying USP thickness and stiffness, capturing both near-field and far-field effects. The analyses include (i) changes in the vertical velocity waveforms at receivers situated at near-field and far-field, (ii) quantification of the attenuation achieved along the free-field, and (iii) one-third octave band analysis to reveal frequency-specific mitigation characteristics. Special focus is placed on how ground conditions, USP geometry, and material properties interact to affect vibration reduction across the trackside and into the surrounding environment.

5.2. Effect of Under Sleeper Pad on Soft Soil

5.2.1. Effect on the Waveform

Figure 5.1 shows a comparison of vertical velocity time histories at different thickness levels (8 mm, 10 mm and 15 mm) in both near field (4 m) and far field (16 m). Based on Figure 5.1 (a), in the near-field, all USP configurations display a noticeable reduction in amplitude compared to the baseline conditions. This reduction becomes more significant as the USP thickness increases, with the 15 mm USP producing the greatest damping of peak velocity, followed by the 10 mm and 8 mm pads. The waveform's temporal structure is similar across all USP thicknesses, indicating that the pads mainly reduce amplitude rather than changing the frequency content or the overall waveform pattern. In the far field response, shown by Figure 5.1 (b), the waveforms for all thicknesses nearly overlap, with minimal differences in peak velocities. The damping effect observed near the track is less pronounced at this distance, suggesting that the attenuation caused by the USP decreases quickly as the distance from the track increases, due to the soil's geometric and material damping dominating.

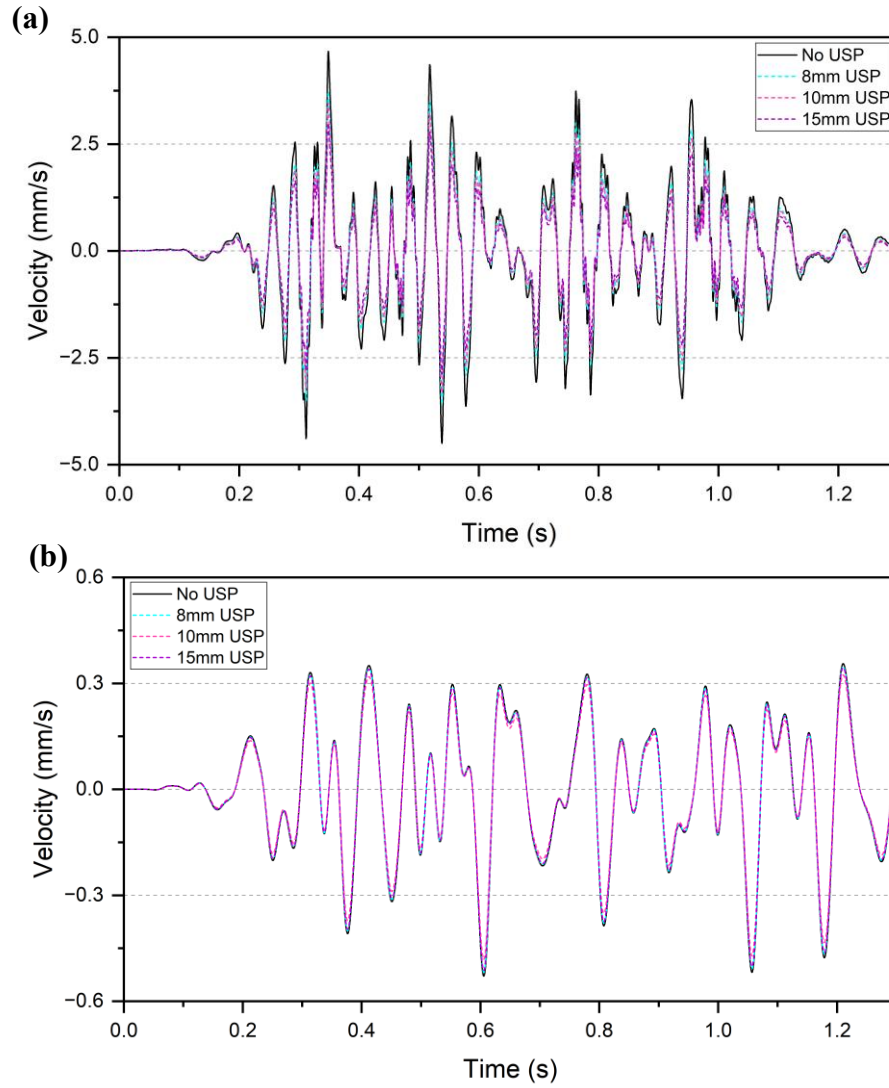


Figure 5.1: Velocity time histories of varying USP thickness on soft soil at (a) 4 m receiver and (b) 16 m receiver

Figure 5.2 shows the vertical velocity time history at near-field and far-field receivers across different USP stiffness levels. A typical behavior across the stiffness variation is that as the pad becomes less stiff, the attenuation ability of the USP increases. However, in soft soil, the waveform of soft USP closely follows the waveform of the no USP condition in the near-field, as shown in Figure 5.2 (a). This indicates that a soft pad does not significantly change the dynamic transmission from sleeper to soil under soft ground conditions. Other stiffness cases show reduced amplitudes, with medium USP achieving the greatest reduction. According to Figure 5.2 (b), the variation of the far-field response is similar to the previous case, with a negligible reduction in velocity. The overall waveform across different USP stiffnesses remains consistent with the baseline condition, confirming that USP stiffness primarily influences amplitude rather than the shape of the time history.

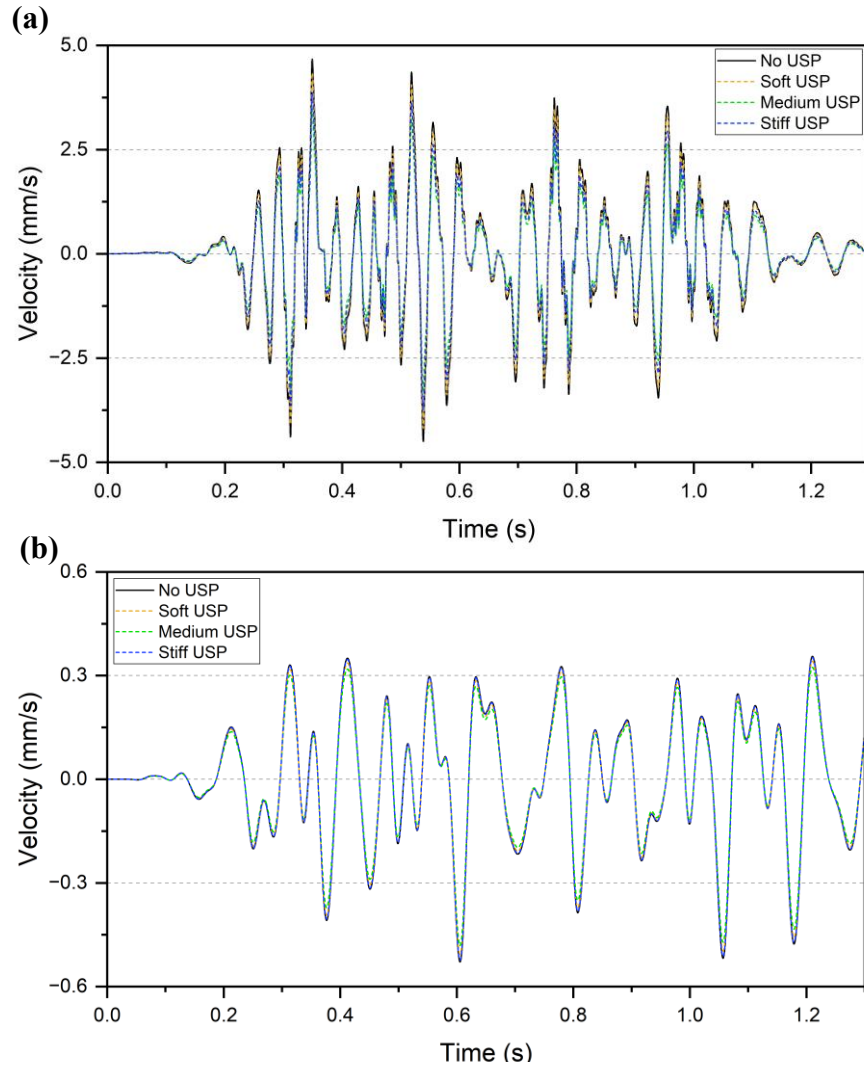


Figure 5.2: Velocity time histories of varying USP stiffness on soft soil at (a) 4 m receiver and (b) 16 m receiver

5.2.2. Peak Decay and Quantification of Loss Along the Free Field

Figure 5.3 visualizes the PPV attenuation with varying USP thickness (Figure 5.3(a)) and stiffness (Figure 5.3(b)) as a function of distance from the track centre. The key observation here is that all USP configurations show reduced PPV compared to the unmitigated baseline at all distances. In Figure 5.3 (a), increased USP thickness results in enhanced vibration reduction. Beyond 8 m, all USP thickness cases converge to the baseline and then barely outperform the no USP condition. Further away from the 16m receiver, the effect of USP in vibration attenuation becomes almost negligible. The near-field amplification observed previously is detailed further in Figure 5.3 (b). The soft USP shows almost no improvement over the baseline in PPV up to 8 m, while beyond this, attenuation aligns closely with medium and stiff USPs. Both the medium and stiff pads provide modest reductions in PPV, especially in the 4-8 m range, with the medium USP generally performing best. At all distances beyond 12 m, all curves approach similar PPV values, suggesting that surface soil damping and geometric spreading dominate far-field vibration behaviour, regardless of USP configuration.

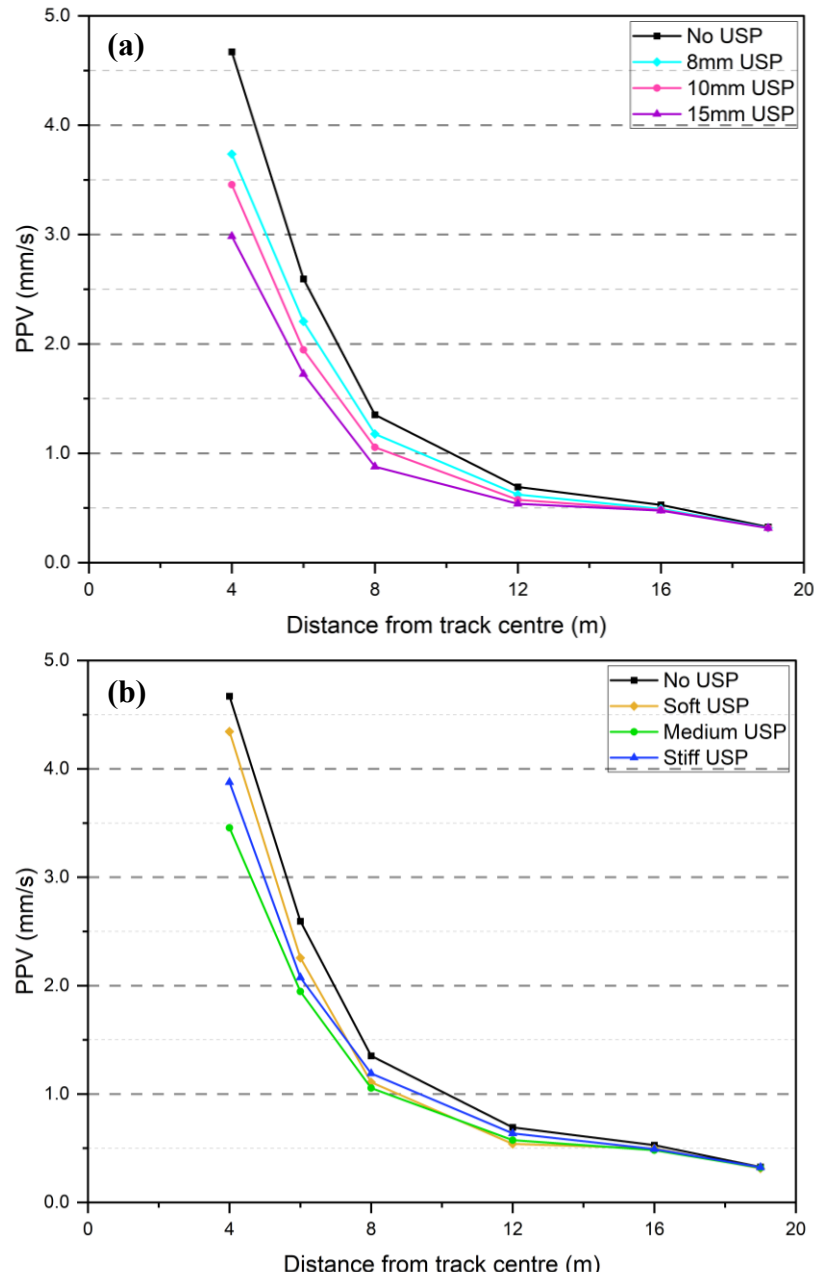


Figure 5.3: PPV attenuation along the free field of soft soil with varying (a) USP thickness and (b) USP stiffness

The waveform and PPV attenuation behaviour can be quantified using IL, which facilitates a more comprehensive comparison of USP performance across receiver locations under different parameters. Table 5.1 shows IL and percentage reduction for thickness variations along distance in the free field. The highest ILs are consistently associated with thicker pads. At 4 m, the 15 mm USP achieves a 3.89 dB IL, almost double that of the 8 mm pad (1.94 dB). This trend continues through 6 m and 8 m, confirming that increased pad thickness improves near-field vibration attenuation. Beyond 16 m, IL drops below 1 dB for all thicknesses, with only slight differences between configurations, confirming the behaviour observed in PPV attenuation plots above. Overall, increased USP thickness outperforms thinner pads at near receivers,

but the marginal benefit decreases with distance, highlighting the importance of optimizing thickness for local rather than far-field control.

Table 5.1: Quantified vibration reduction of varying USP thickness in soft soil

Receiver location (m)	8mm		10mm		15mm	
	IL (dB)	% reduction	IL (dB)	% reduction	IL (dB)	% reduction
4	1.94	20%	2.62	26%	3.89	36%
6	1.41	15%	2.50	25%	3.54	34%
8	1.21	13%	2.16	22%	3.76	35%
12	0.92	10%	1.62	17%	2.17	22%
16	0.63	7%	0.82	9%	0.92	10%
19	0.20	2%	0.28	3%	0.26	3%

Table 5.2 shows the quantified attenuation across different stiffness levels. The medium stiffness USP offers the best overall performance at most receiver locations, achieving 2.16–2.62 dB between 4–8 m. The IL for soft USPs ranges from 0.63 to 1.21 dB between 4–6 m, which is even less than the IL achieved by stiff USP (1.62–1.94 dB). This poor performance of soft USP aligns with earlier observations of near-field amplification. However, at 8 m, soft USP (1.72 dB) managed to surpass the attenuation of stiff USP (1.11 dB) and even achieved the highest IL (2.16 dB) at the 12 m receiver, surpassing both medium and stiff USPs. Additionally, the vibration reduction drops significantly in the far-field, with IL falling below 0.35 dB (equivalent to a 4% reduction) for all stiffness cases beyond 19 m. Overall, the medium USP configuration consistently provides the best balance between effective near-field attenuation and the prevention of resonance amplification.

Table 5.2: Quantified vibration reduction of varying USP stiffness in soft soil

Receiver location (m)	Soft		Medium		Stiff	
	IL (dB)	% reduction	IL (dB)	% reduction	IL (dB)	% reduction
4	0.63	7%	2.62	26%	1.62	17%
6	1.21	13%	2.50	25%	1.94	20%
8	1.72	18%	2.16	22%	1.11	12%
12	2.16	22%	1.62	17%	0.72	8%
16	0.54	6%	0.82	9%	0.63	7%
19	0.35	4%	0.28	3%	0.09	1%

The observed near-field velocity amplification under soft USP can be explained as a local resonance phenomenon. A soft soil base combined with a soft USP can yield a coupled system whose natural frequency coincides with dominant excitation frequencies (typically 25–40 Hz), causing "insertion gain" (negative insertion loss) within several meters of the track. This effect is well documented in the literature and parametric studies, which caution against excessive compliance in soft soil settings (Çati et al., 2020; Hildebrand, 2001).

5.2.3. Frequency-Specific Analysis of the Loss

Analysing the insertion loss across frequency bands is essential because railway-induced ground vibrations are highly dependent on frequency, with human perception and structural response often most sensitive within specific bandwidths (typically 8–80 Hz). Examining insertion loss in one-third octave bands helps identify frequency ranges where mitigation measures are most or least effective, revealing not only how well they work but also potential risks of amplification or resonance.

Figure 5.4 illustrates the one-third octave band spectra of insertion loss (IL) for vertical velocity as a function of USP thickness. In the near field, as shown in Figure 5.4 (a), negative IL is observed until 45 Hz, where 10 mm USP reaches a maximum negative IL of -2.5 dB, while 15 mm shows the least amplification. Beyond 45 Hz, all curves sharply transition to positive IL, indicating a solid shift from resonance amplification to effective vibration isolation. Between 56 and 71 Hz, all thickness variations display a maximum positive IL of 6, 7.4, and 8.95 dB in order of increasing thickness. This demonstrates that thicker USPs are superior for mid-frequency vibration reduction, which is crucial for human perception and structure-borne noise mitigation.

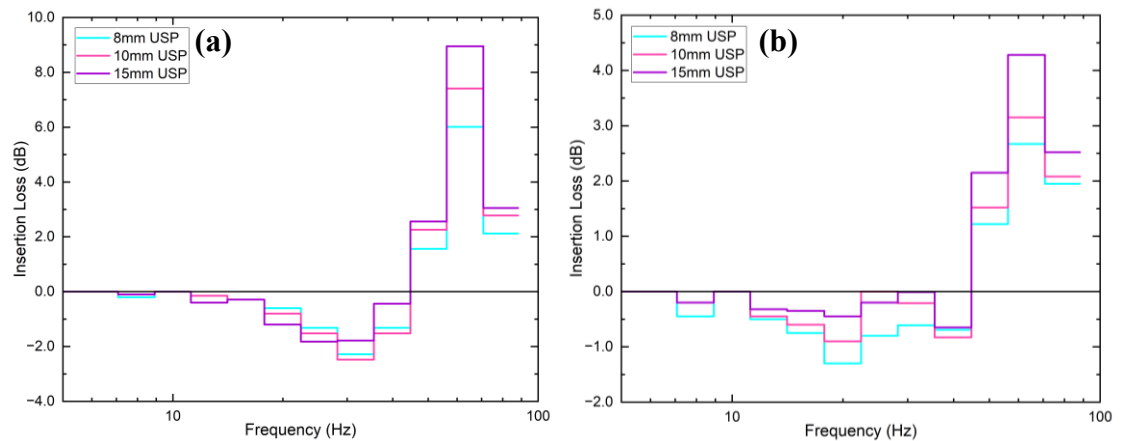


Figure 5.4: One-third octave spectrum of the insertion loss of vertical velocity across USP thickness on soft soil at (a) 4 m receiver and (b) 16 m receiver

Based on the far-field behaviour shown in Figure 5.4 (b), the magnitude of IL is significantly lower across all frequency bands, and the differences between thicknesses are smaller. Amplification is weaker but remains below 45 Hz for all thicknesses, whereas for 10 mm and 15 mm USP, IL drops to nearly zero within the 22–36 Hz range. Similar to the near-field, a peak IL occurs in the 56–71 Hz band with a

magnitude of 4.3 dB for 15 mm USP and 2.67 dB for 8 mm USP. These observations clearly show that the additional benefit of increased thickness quickly diminishes with distance, indicating that soil filtering reduces both vibration and mitigation benefits as the wavefront propagates.

Figure 5.5 illustrates the one-third octave spectrum of IL for vertical velocity, comparing soft, medium, and stiff USP in the near-field (4 m) and far-field (16 m). According to Figure 5.5 (a), the effect of USP is observed for frequencies above 7 Hz and up to 45 Hz, where negative IL is present across all stiffness levels. Soft USP shows the highest amplification with IL reaching up to 2.88 dB. In the near-field, IL transitions from negative to positive at 45 Hz, reaching a peak IL between 56 – 71 Hz, with medium USP achieving 7.4 dB IL, and soft and stiff USP showing a peak of around 4 dB. However, the soft USP exhibits the least IL within the 45-88 Hz frequency band, which, combined with the higher negative ILs at lower frequencies, results in a very small reduction in overall vibration amplitude in the near-field, as discussed earlier.

Based on Figure 5.5 (b), far-field IL is consistently lower than near-field across all frequencies and USP stiffness variations. The highest IL is 3.82 dB with soft USP, followed by medium (3.15 dB) and stiff (2.4 dB) USPs in the 56–71 Hz frequency range. Differences among pad stiffnesses are less noticeable, although soft and medium USPs provide slightly higher IL in the mid-frequency range. The stiff pad remains less effective across all bands. The transition from negative to positive IL occurs at 45 Hz for medium and stiff USPs. For the soft USP, IL is negative until 22.5 Hz, becomes zero between 22–28 Hz, and then increases, reaching its peak.

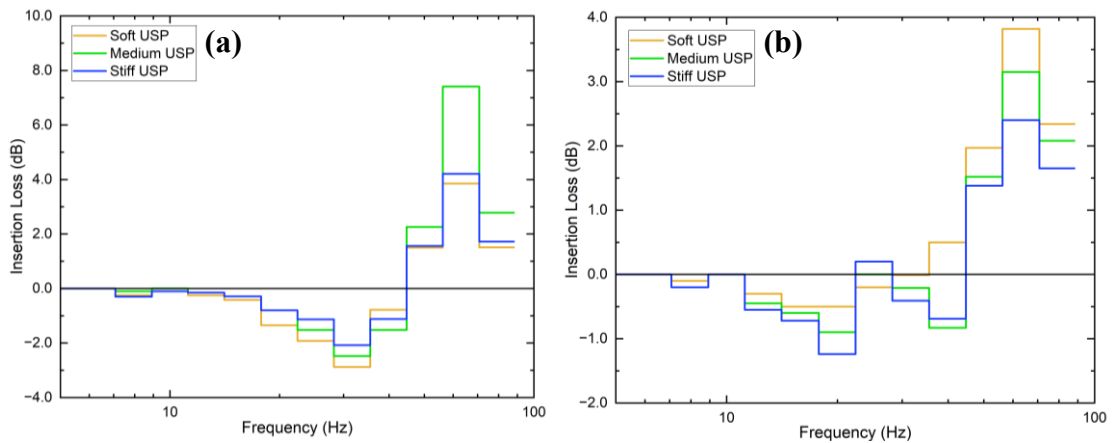


Figure 5.5: One-third octave spectrum of the insertion loss of vertical velocity across USP stiffness on soft soil at (a) 4 m receiver and (b) 16 m receiver

5.3. Effect of Under Sleeper Pad on Stiff Soil

5.3.1. Effect on the Waveform

Figure 5.6 shows the vertical ground velocity time histories at near-field and far-field receivers for stiff soil conditions, comparing the effect of thickness. The introduction of USP has caused noticeable attenuation in the near-field, which becomes more pronounced as the thickness increases from 8 mm to 15 mm, as shown in Figure 5.6 (a). The insertion of USP has not changed the basic time-domain structure of the vibration waveform. While velocity amplitudes are generally much smaller in the far field, unlike in soft soil conditions, a slight attenuation of velocity amplitudes is observed in Figure 5.6 (b). The 15 mm USP results in the greatest peak suppression, followed by 10 mm and 8 mm. However, the differences between the cases are less significant, indicating that geometric spreading and soil damping reduce the impact of USPs in the far field.

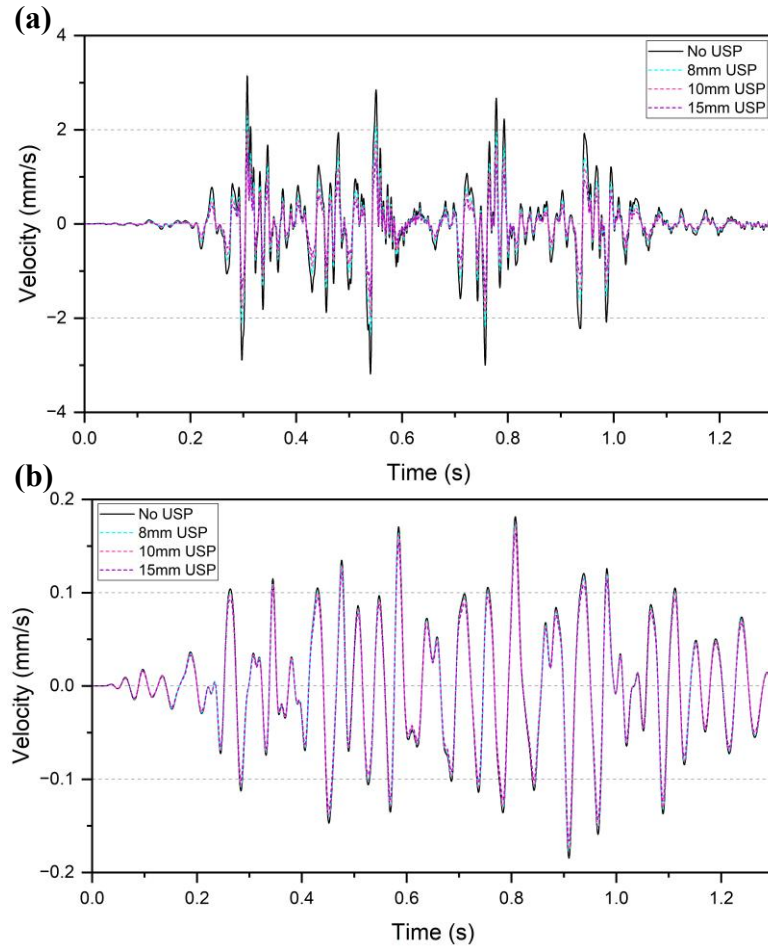


Figure 5.6: Velocity time histories of varying USP thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver

Figure 5.7 illustrates the vertical velocity time histories at near-field and far-field locations with varying USP stiffness. From the observation of Figure 5.7 (a), the addition of USPs has managed to reduce the peak amplitudes of velocities, but the differences between soft, medium, and stiff USPs are subtle. However, the decreased

stiffness achieved better attenuation with a consistent order (stiff USP < medium USP < soft USP). All time history traces maintain the fundamental pattern set by train passage, with peak times and oscillation frequencies being virtually identical across cases. Based on the far-field behaviour shown in Figure 5.7 (b), amplitude differences among all configurations become almost negligible. This demonstrates that the primary USP-induced vibration mitigation occurs within the near-field. However, the amplification caused by local resonance observed when using soft USPs in soft soil conditions is not seen in stiff soil conditions.

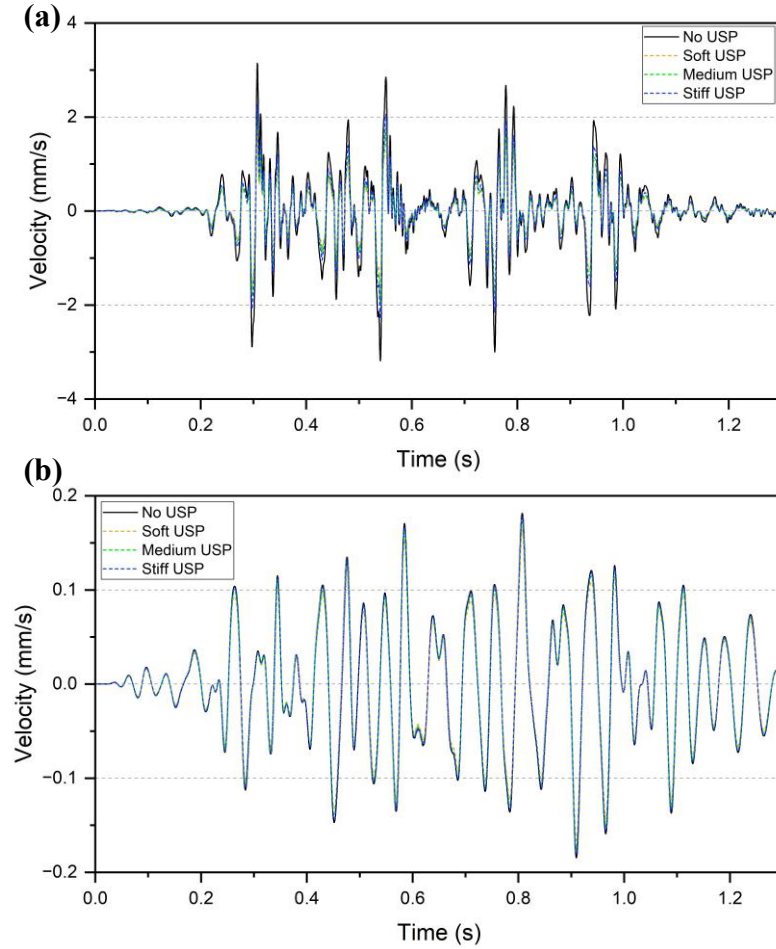


Figure 5.7: Velocity time histories of varying USP stiffness on stiff soil at (a) 4 m receiver and (b) 16 m receiver

5.3.2. Peak Decay and Quantification of Loss Along the Free Field

Figure 5.8 illustrates the PPV attenuation with varying USP thickness (Figure 5.8(a)) and stiffness (Figure 5.8(b)) as a function of distance from the track centre. As shown in Figure 5.8 (a), the presence of USPs leads to a significant reduction in PPV near the receiver, where the 15 mm USP decreased it from 3.2 mm/s to 1.5 mm/s, achieving more than 50% reduction. As the thickness decreases, the vibration attenuation effect also diminishes across all distances. Beyond 8 m, the differences among the three thicknesses rapidly converge, with all configurations resulting in similarly low PPV by 16 m and beyond. This emphasizes the dominance of geometric spreading and

inherent soil damping at larger distances. According to Figure 5.8 (b), all stiffness configurations demonstrate substantial PPV reduction at 4 – 8 m, with soft USPs achieving the greatest reduction, followed by medium and stiff USPs. Up to 12 m, the reduced PPV values for soft and medium USPs are approximately equal. Similarly, for the thickness case, there is a rapid convergence of all curves after 8 m. One important observation here is, the PPV recorded at 19 m receiver is slightly higher than the 16 m receiver. Although in general, PPV continuously decreases with the distance from the track, this increase can happen due to the wave reflection at boundary surfaces. The dashpots in the viscous boundary work best for normally incident waves and their absorption efficiency drops for oblique incident waves. Therefore, when waves hit the boundary at a grazing angle, part of the motion is not aligned with the dashpot directions, and a portion is reflected back into the domain. This reflected waves can artificially increase response at receivers close to the boundary surface.

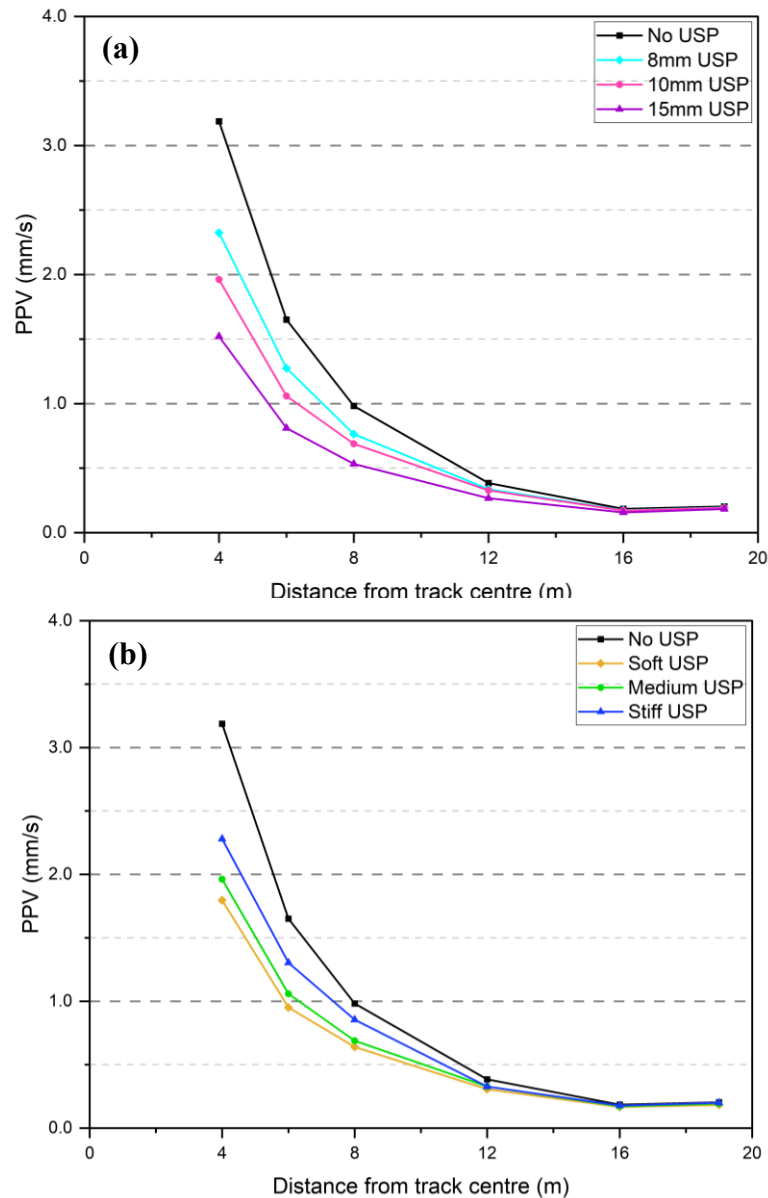


Figure 5.8: PPV attenuation along the free field of stiff soil with varying (a) USP thickness and (b) USP stiffness

To provide an in-depth understanding of this attenuation behaviour, IL achieved at each receiver for different USP configurations can be used. As shown in Table 5.3, at 4 m, the thickest mat (15 mm) reaches a peak IL of 6.43 dB compared to the no-USP case, followed by 10 mm (4.22 dB) and 8 mm (2.75 dB). Within the 4–8 m range, all thickness variations managed to maintain an IL above 2 dB. Beyond 16 m, the negligible effect of USP observed in previous plots is confirmed by the IL values in the table, where it drops below 0.63 dB for 8 mm and 10 mm USPs, while the 15mm USP stays closer to 1 dB (10% reduction). This clear ranking shows that increasing USP thickness significantly enhances performance in high-stiffness soils, especially in the near-field.

Table 5.3: Quantified vibration reduction of varying USP thickness in stiff soil

Receiver location (m)	8mm		10mm		15mm	
	IL (dB)	% reduction	IL (dB)	% reduction	IL (dB)	% reduction
4	2.75	27%	4.22	38%	6.43	52%
6	2.25	23%	3.86	36%	6.18	51%
8	2.19	22%	3.09	30%	5.32	46%
12	1.16	13%	1.41	15%	3.16	31%
16	0.53	6%	0.63	7%	1.41	15%
19	0.38	4%	0.45	5%	0.92	10%

According to the results in Table 5.4, the soft USP produces the highest IL at every receiver up to 8 m (4.97 - 3.73 dB between 4 m and 8 m). Medium stiffness comes close but is slightly lower in each case (4.22 dB at 4 m and 3.09 dB at 8 m). The stiffest USP consistently shows the lowest IL and percent reduction (2.91 dB at 4 m and 1.21 dB at 8 m). At receivers from 4 to 8 m, when the stiffness changes from stiff to medium, IL increases by about 1.8 dB, while this increase drops to about 0.75 dB when changing from medium to soft. This suggests that USP attenuation is less affected by stiffness than by thickness. Beyond 16 m, the IL drops below 1 dB across all stiffness levels, highlighting the minimal effect of USP in the far field of stiff soils.

Table 5.4: Quantified vibration reduction of varying USP stiffness in stiff soil

Receiver location (m)	Soft		Medium		Stiff	
	IL (dB)	% reduction	IL (dB)	% reduction	IL (dB)	% reduction
4	4.97	44%	4.22	38%	2.91	28%
6	4.79	42%	3.86	36%	2.05	21%
8	3.73	35%	3.09	30%	1.21	13%
12	1.96	20%	1.41	15%	1.41	15%

16	0.92	10%	0.63	7%	0.35	4%
19	0.95	10%	0.45	5%	0.18	2%

5.3.3. Frequency-Specific Analysis of the Loss

Figure 5.9 illustrates the one-third octave band spectra of insertion loss (IL) for vertical velocity as a function of USP thickness in stiff soil. According to Figure 5.9 (a), all curves remain slightly below zero, indicating minimal or negligible amplification at very low frequencies. There is no pronounced negative IL (amplification risk) as observed in soft soil, reaffirming that stiff soils shift the dynamic system resonance beyond the primary excitation band of train loads. From 45 Hz upward, a sharp positive transition appears, reaching peak ILs of 5.45, 8.85, and 2.8 dB at frequency bands of 45–56 Hz, 56–71 Hz, and 71–88 Hz, respectively, recorded for 15 mm USP, while the peaks reduce with decreasing thickness. According to the far-field response shown in Figure 5.9 (b), the overall IL values are consistently lower due to propagation losses and soil damping, but the order (15 mm > 10 mm > 8 mm) is preserved. The 15 mm USP still provides a peak IL close to 6 dB at 56–71 Hz.

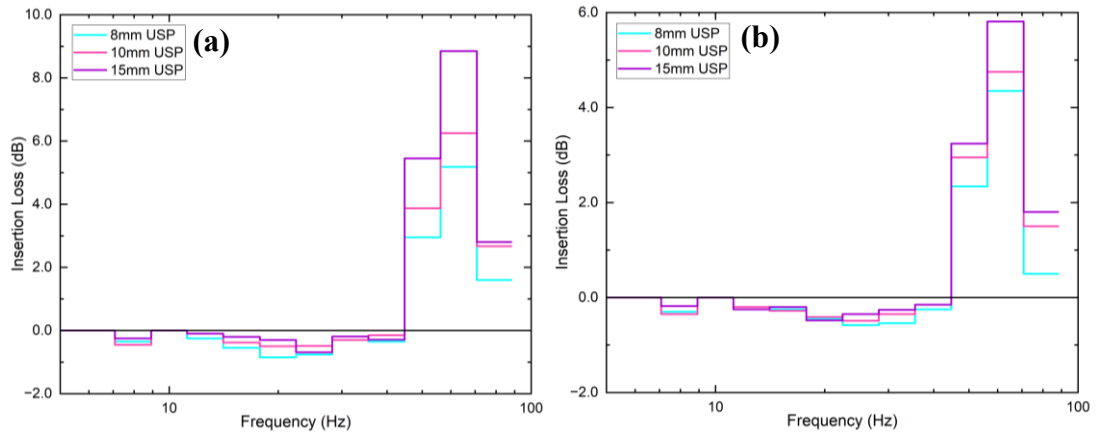


Figure 5.9: One-third octave spectrum of the insertion loss of vertical velocity across USP thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver

The one-third spectrum of the IL for different USP stiffnesses is shown in Figure 5.10. In both near-field and far-field measurements, slightly negative ILs are observed for frequencies up to 45 Hz. The transition of IL from negative to positive occurs at 45 Hz across all stiffness cases, while the soft USP reaches this transition at 36 Hz. According to Figure 5.10 (a), the soft USP produces the highest mid-band IL, peaking near 7.5 dB between 56 and 71 Hz. The medium and stiff configurations follow, with maximum values around 6.25 dB and 3.8 dB, respectively. In the 71–88 Hz band, IL drops to a range between 1.56 and 2.67 dB, with the medium USP achieving the highest. As shown in the far-field response in Figure 5.10 (b), negative ILs from 0 to -0.5 dB are observed for frequencies up to 45 Hz. Beyond that, positive ILs are maintained, with maximum values recorded as 5.2 dB for the soft USP, 4.75 dB for the medium USP, and 3.46 dB for the stiff USP in the 56–71 Hz frequency band.

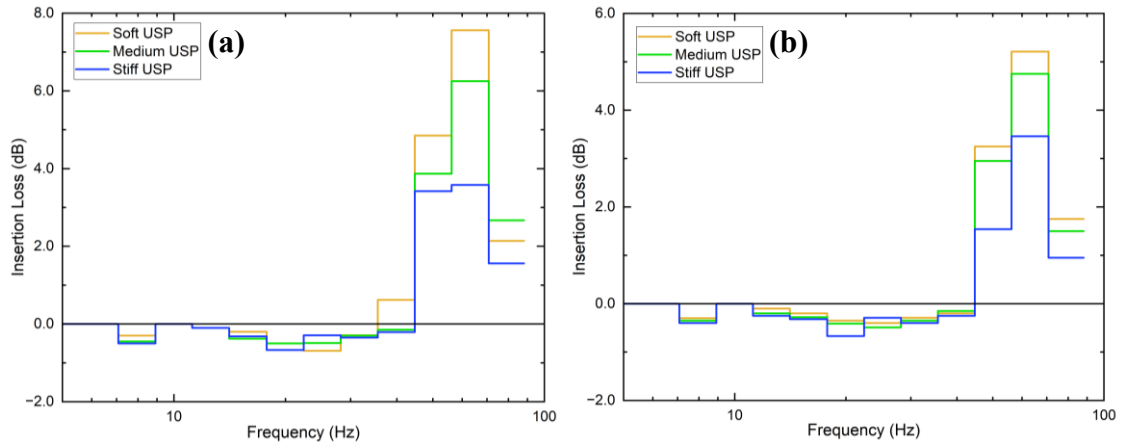


Figure 5.10: One-third octave spectrum of the insertion loss of vertical velocity across USP stiffness on stiff soil at (a) 4m receiver and (b) 16m receiver

5.4. Chapter Summary

This chapter evaluates how USPs change ground vibration for soft and stiff soils. Results show a clear thickness effect and a secondary stiffness effect: thicker pads give the larger reductions, while the best stiffness depends on soil, medium pads suit soft soil and soft pads work best on stiff soil. Benefits are strongest in the near field (within about 10 m) and fade with distance; far-field gains are small. Frequency-band results indicate that USPs mainly reduce vibration in the mid-frequency range important for perception. Overall, USPs provide useful mitigation but their performance is sensitive to soil condition and pad tuning; careful selection (thickness first, then stiffness) is needed, and very soft pads on soft ground should be avoided.

CHAPTER 6

BALLAST MATS: PARAMETRIC STUDY AND PERFORMANCE EVALUATION

6.1. Introduction

This chapter presents parametric results for Ballast Mats (BM) under two ground conditions: soft and stiff. For each soil type, the response with BMs is analysed by varying their thickness and stiffness. The analysis includes (i) changes in the pass-by waveform near and away from the track, and (ii) the decay of peak and energy levels with distance, with IL used to measure performance at each receiver. The one-third-octave IL spectrum is examined to identify the frequency bands where reductions occur. The chapter concludes with a comparison between soft and stiff soils, highlighting where BMs are most effective.

6.2. Effect of Ballast Mat on Soft Soil

6.2.1. Effect on the Waveform

Figure 6.1 illustrates the changes in the time history of the vertical velocity as the BM thickness increases at near-field and far-field receivers. At both receivers, the waveform with BM follows the same response as the baseline condition, with the amplitude decreasing as the thickness increases. Compared to USP, a noticeable reduction in amplitudes is observed at both receivers, indicating that the effectiveness of BM extends through the far-field as well. A phase shift of approximately one-hundredth of a second was recorded in the results; however, this is negligible in the overall time history.

Similarly, Figure 6.2 shows the variation of vertical velocity time histories across the considered stiffness conditions. As in the previous plot, the response follows the same pattern as the baseline condition with no mitigation measures. A noticeable reduction in velocity amplitudes is observed for soft and medium stiffnesses, while the stiff BM follows the baseline response with minimal amplitude reduction throughout.

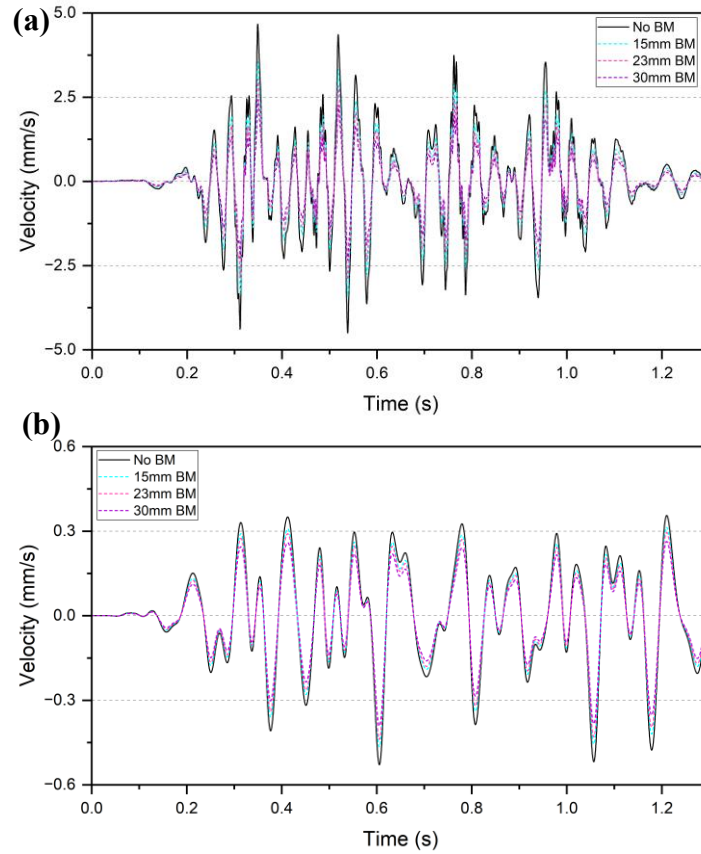


Figure 6.1: Velocity time histories of varying BM thickness on soft soil at (a) 4 m receiver and (b) 16 m receiver (right)

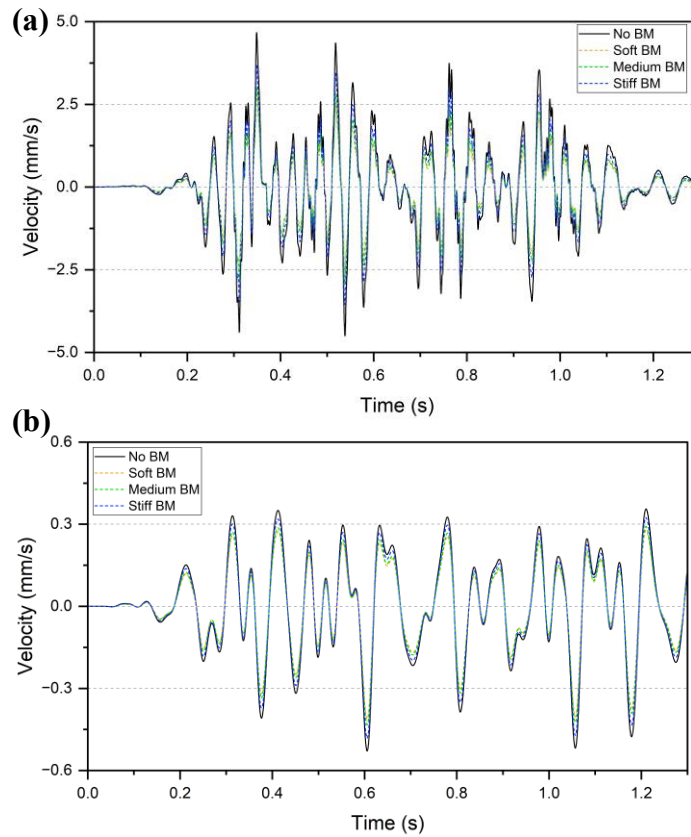


Figure 6.2: Velocity time histories of varying BM stiffness on soft soil at (a) 4 m receiver and (b) 16 m receiver

6.2.2. Peak Decay and Quantification of Loss Along the Free Field

Figure 6.3 (a) shows the variation of PPV across the free field with changing BM thickness, while Figure 6.3 (b) illustrates the PPV variation for different BM stiffnesses. The overall behaviour of these figures suggests that inserting BM significantly reduces PPV, with greater attenuation in the near-field, diminishing as it moves to the far-field. However, compared to USP, BMs managed to demonstrate a noticeable reduction in the far-field. When stiffness is changed from stiff to medium, a considerable decrease in PPV is observed compared to the change from medium to soft. For the case of thickness, when increasing it from 23 mm to 30 mm, a higher reduction of PPV is gained compared to the thickness increase from 15mm to 23mm.

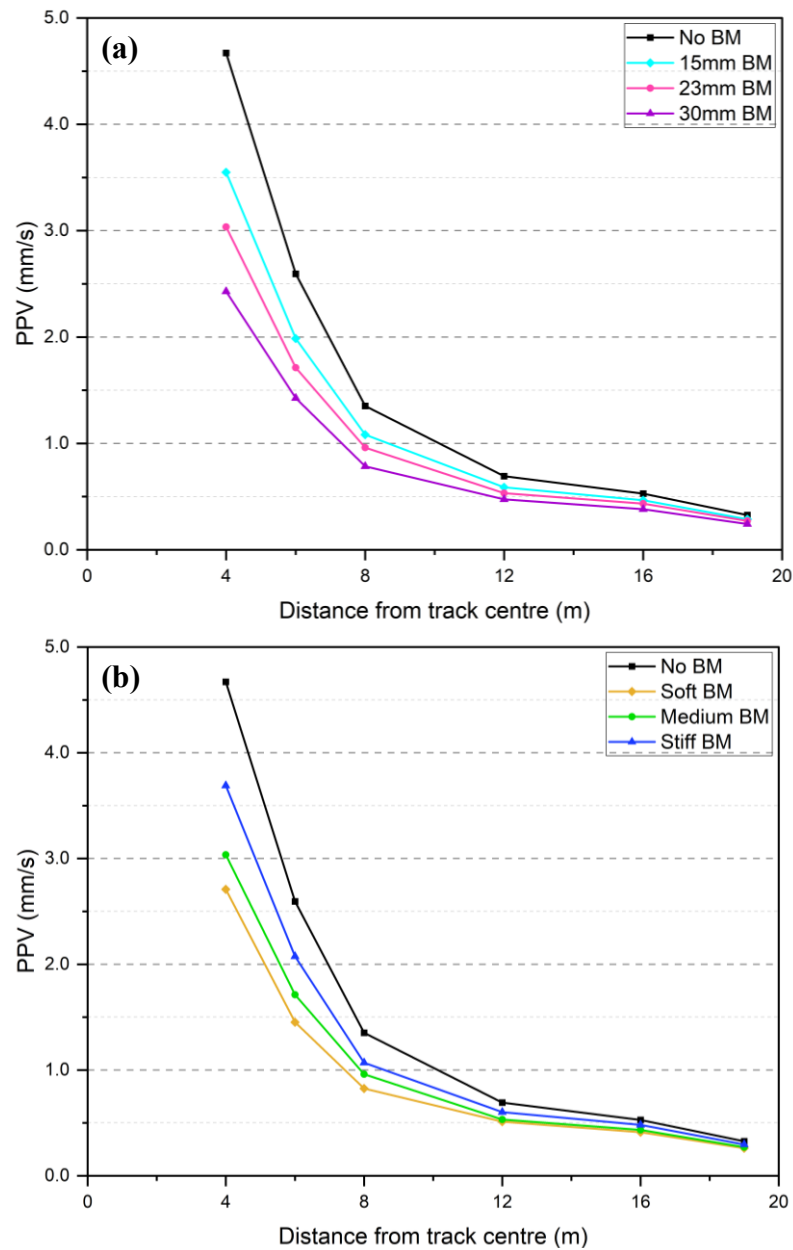


Figure 6.3: PPV attenuation along the free field of soft soil with varying (a) BM thickness and (b) BM stiffness

For a detailed discussion on attenuation performance, they are quantified using IL and percentage reduction, as shown in Table 6.1 for different thicknesses and Table 6.2 for varying stiffness levels. With the installation of BM, an IL above 2 dB is achieved in the near-field across all cases. Increasing the thickness to 30 mm results in an IL of up to 5.68 dB at a 4 m receiver, and with a soft stiffness for the BM, an IL of 4.73 dB can be obtained. In the far-field (beyond 16 m), most cases produce an IL above 1 dB, corresponding to about a 10% reduction. However, the IL from stiff BM drops below 1 dB beyond 16 m. Increasing the thickness of the BM yields higher vibration reduction (5.68 dB for 30 mm thickness) compared to enhancing its stiffness (4.73 dB for stiff BM). These results suggest that combining thickness and stiffness adjustments can help achieve a desired level of vibration reduction.

Table 6.1: Quantified vibration reduction of varying BM thickness in soft soil

Receiver location (m)	15mm		23mm		30mm	
	IL (dB)	% reduction	IL (dB)	% reduction	IL (dB)	% reduction
4	2.38	24%	3.74	35%	5.68	48%
6	2.32	23%	3.61	34%	5.19	45%
8	1.94	20%	2.97	29%	4.73	42%
12	1.41	15%	2.27	23%	3.27	31%
16	1.11	12%	1.72	18%	2.84	28%
19	1.11	12%	1.62	17%	2.57	26%

Table 6.2: Quantified vibration reduction of varying BM stiffness in soft soil

Receiver location (m)	Soft		Medium		Stiff	
	IL (dB)	% reduction	IL (dB)	% reduction	IL (dB)	% reduction
4	4.73	42%	3.74	35%	2.05	21%
6	5.04	44%	3.61	34%	1.94	20%
8	4.29	39%	2.97	29%	2.05	21%
12	2.62	26%	2.27	23%	1.21	13%
16	2.16	22%	1.72	18%	0.82	9%
19	1.94	20%	1.62	17%	0.86	9%

6.2.3. Frequency-Specific Analysis of the Loss

Figure 6.4 shows the variation of IL across frequency bands for different thicknesses at near-field and far-field receivers. The BM effect is observed at frequencies above 7 Hz at both receivers. From 7 to 28 Hz, the IL turns negative in the near field and the far field, and it changes from negative to positive at 35 Hz.

As shown in Figure 6.4 (a), negative IL has steadily increased from 7 Hz to 28 Hz, where the negative value decreased as the BM thickness increased. This indicates that, within this frequency range, excitation increases with the installation of BMs and that the magnitude of excitation is inversely proportional to the BM thickness. Once the IL turns positive, it gradually increases at each frequency band, reaching its maximum value in the 56–71 Hz range, with an 11.34 dB loss for a 30 mm thickness. However, a comparatively low IL is recorded for the 15 mm thickness, with a maximum value around 4 dB. For the 71–88 Hz band, IL decreases to a range of 2.7–4 dB across different thicknesses.

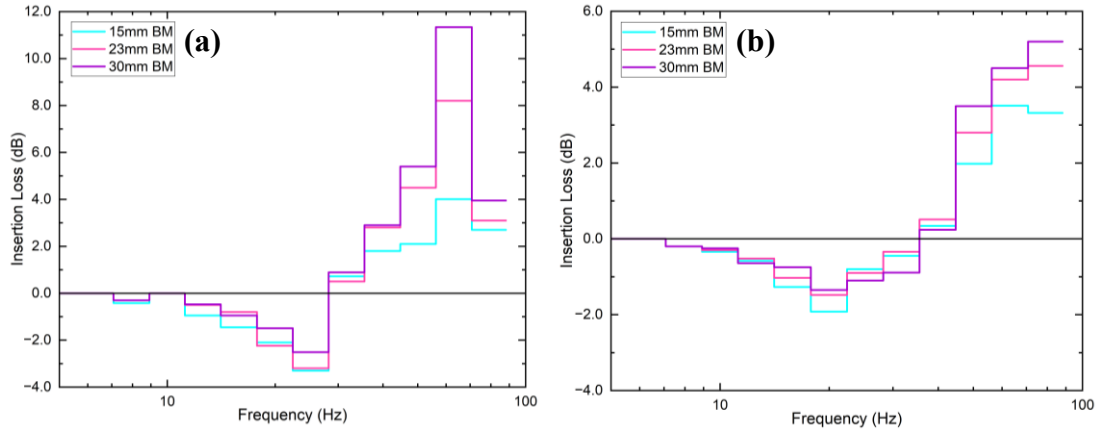


Figure 6.4: One-third octave spectrum of the insertion loss of vertical velocity across BM thickness on soft soil at (a) 4m receiver and (b) 16m receiver

Based on Figure 6.4 (b), the negative ILs are observed from 7 to 35 Hz. However, their maximum negative IL occurs within the 18 to 22.5 Hz range, with values between -1.35 dB and -1.92 dB. The negative IL decreases from 22.5 Hz to 35 Hz before turning positive. At 35 to 45 Hz, all thicknesses show positive ILs less than 1 dB before experiencing a significant increase in the next frequency band, reaching up to 4 dB for 30 mm thick BM. Beyond this, IL continues to rise, although for a 15 mm thickness, it slightly decreases in the highest frequency band compared to the previous one.

Figure 6.5 illustrates the variation of IL across frequency bands for different stiffness levels at near-field and far-field receivers. Both plots follow the IL variation trend

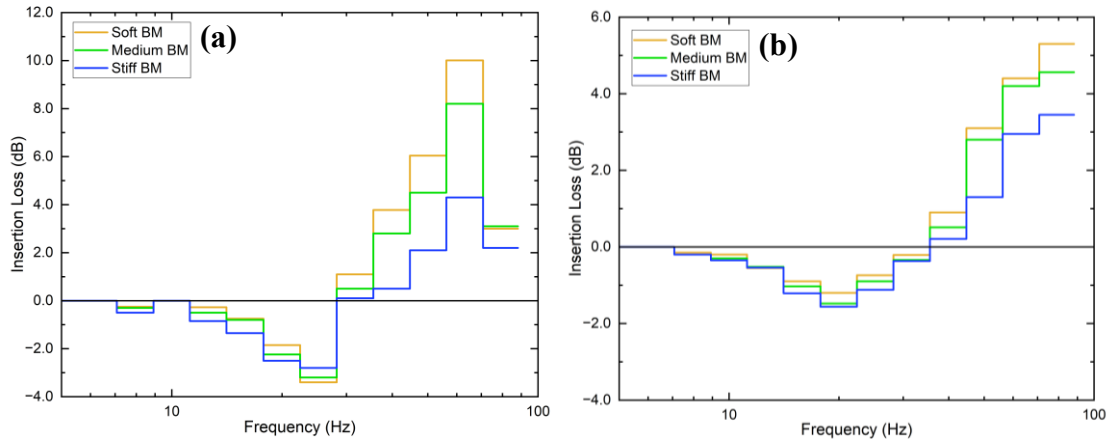


Figure 6.5: One-third octave spectrum of the insertion loss of vertical velocity across BM thickness on soft soil at (a) 4m receiver and (b) 16m receiver

discussed in Figure 6.4. Under varied stiffness levels, the positive ILs increased as the BM stiffness changed from stiff to soft. In the near-field (Figure 6.5(a)), the variation of ILs at different stiffnesses looks more consistent compared to the far-field (Figure 6.5(b)). The IL for medium and soft BM has taken close values across frequency bands, while comparably low values are observed for the stiff BM beyond 45 Hz.

6.3. Effect of Ballast Mat on Stiff Soil

6.3.1. Effect on the Waveform

Similar to previous cases, inclusion of BMs has not affected the overall waveform pattern in the stiff soil with thickness and stiffness being varied. However, compared to the response in the soft soil, a higher reduction of velocity amplitudes is observed across thickness, as illustrated in Figure 6.6, and across stiffness, as illustrated in Figure 6.7. A very small phase shift was noted in the results of higher BM thicknesses, which, however, cannot be distinguished in the overall waveform. Overall, increased thickness and lower stiffness have facilitated the reduction of velocity amplitudes transferred to the ground.

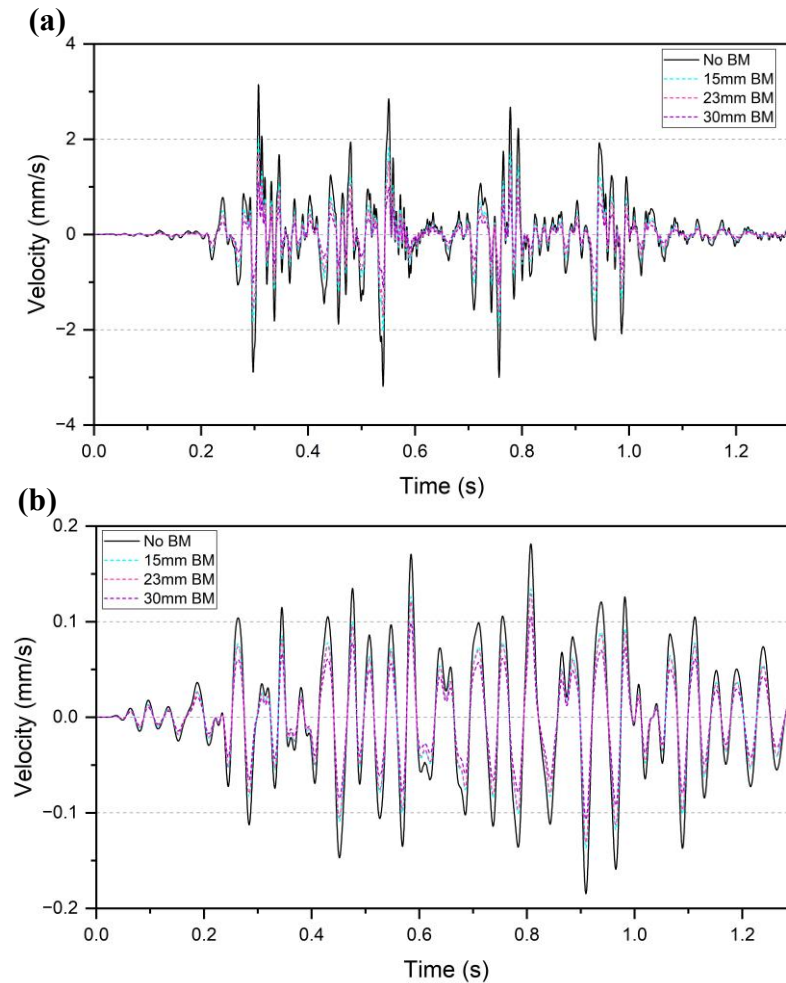


Figure 6.6: Velocity time histories of varying BM thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver

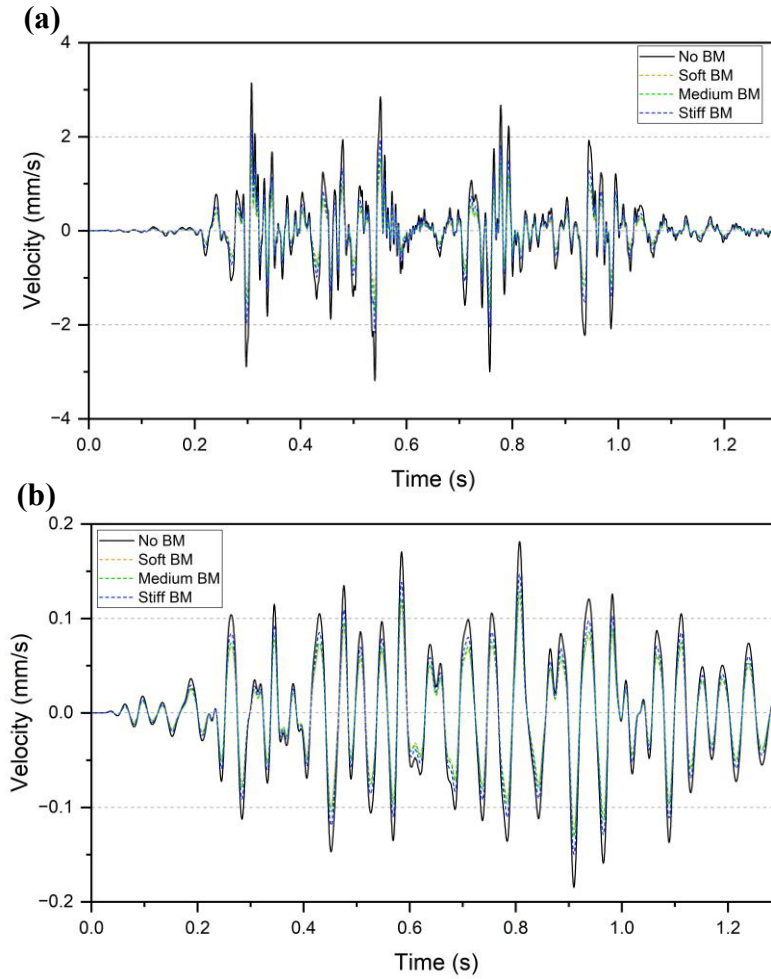


Figure 6.7: Velocity time histories of varying BM stiffness on stiff soil at (a) 4 m receiver and (b) 16 m receiver

6.3.2. Peak Decay and Quantification of Loss Along the Free Field

Compared to soft soil, reduced PPV is observed in stiff soil at each receiver. However, BMs further decrease PPV as their thickness increases, shown in Figure 6.8 (a), and their stiffness becomes softer, as shown in Figure 6.8 (b). Overall, attenuation is higher in the near-field and decreases with distance in the far-field. Still, noticeable attenuation is observed in the furthest receiver of the soil domain (19 m). Installing a 15 mm BM reduces PPV at the 4 m receiver from 3.2 mm/s to 2 mm/s, while a stiff BM lowers it to 2.2 mm/s, with these values being quite close. When stiffness is reduced, PPV drops by 0.7 mm/s, which is less than the nearly 1 mm/s reduction achieved by increasing the thickness to 30 mm. This indicates that thickness has a greater influence on attenuation than stiffness. Additionally, similar to the USP on stiff soil condition, artificially increased response is observed at furthest receiver (19 m). This is probably caused by waves striking at oblique incidence.

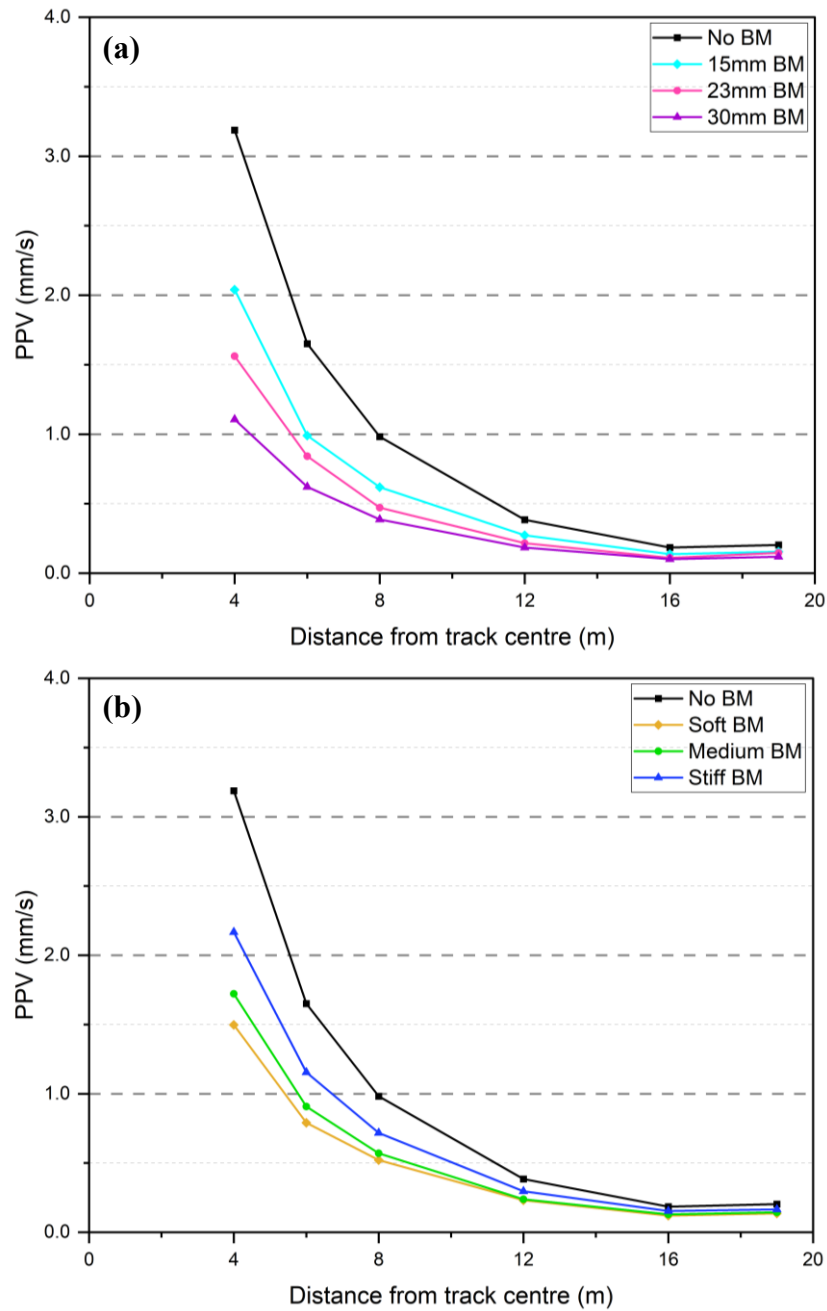


Figure 6.8: PPV attenuation along the free field of stiff soil with varying (a) BM thickness and (b) BM stiffness

Above vibration attenuation can be quantified using IL as in Table 6.3. The 15 mm BM achieved an IL ranging from 2.4 dB to 4.44 dB, 23 mm BM achieved IL of 2.97 to 5.35 dB, and 30 mm BM had an IL between 4.74 dB and 9.19 dB. The 30 mm BM significantly outperformed other thicknesses, with the 4 m receiver reaching an IL of 9.19 dB and the 19 m receiver showing a loss of 4.73 dB. This IL at the 19 m receiver is higher than the maximum IL achieved by the 15 mm BM. This confirms the behaviour observed in Figure 6.8 (a). The IL at the far-field exceeding 2 dB indicates that, in stiff soil conditions, the effect of BM extends over a considerable distance from the track.

Table 6.3: Quantified vibration reduction of varying BM thickness in stiff soil

Receiver location (m)	15mm		23mm		30mm	
	IL (dB)	% reduction	IL (dB)	% reduction	IL (dB)	% reduction
4	3.88	36%	5.35	46%	9.19	65%
6	4.44	40%	5.19	45%	8.50	62%
8	4.01	37%	4.73	42%	8.11	61%
12	2.97	29%	4.15	38%	6.38	52%
16	2.62	26%	3.10	30%	5.19	45%
19	2.38	24%	2.97	29%	4.73	42%

The quantified IL in Table 6.4 indicates that soft and medium stiffnesses achieved IL values of approximately over 3 dB at all receivers, with the highest IL recorded at 6.56 dB for soft BM and 5.35 dB for medium BM. The IL recorded for stiff BMs is comparatively lower than other stiffness levels, with a maximum of 3.35 dB at the receiver nearest to the track, while it drops below 2 dB at the 16 m receiver. However, the stiff BM might be safer for track elements, including rail, sleepers, and ballast, as it transmits considerable energy toward the soil without trapping it within the track elements, thus reducing the potential for degradation.

Table 6.4: Quantified vibration reduction of varying BM stiffness in stiff soil

Receiver location (m)	Soft		Medium		Stiff	
	IL (dB)	% reduction	IL (dB)	% reduction	IL (dB)	% reduction
4	6.56	53%	5.35	46%	3.35	32%
6	6.39	52%	5.19	45%	3.10	30%
8	5.51	47%	4.73	42%	2.73	27%
12	4.44	40%	4.15	38%	2.27	23%
16	3.74	35%	3.10	30%	1.62	17%
19	3.48	33%	2.97	29%	1.83	19%

6.3.3. Frequency-Specific Analysis of the Loss

Figure 6.9 illustrates how IL varies across the considered frequency bands at near-field (Figure 6.9(a)) and far-field (Figure 6.9(b)). The trend of the one-third spectrum of IL in stiff soil aligns with that in soft soil conditions. All BM thickness configurations show negative loss up to 28 Hz in the near field, with higher negative values for 30 mm thickness, and up to 35 Hz in the far field, with higher negative values for 15 mm thickness. This suggests minor amplification of ground vibration at low frequencies. IL becomes positive for frequencies above 28 Hz in the near field and above 35 Hz in the far field, with higher IL values observed in thicker BMs. The IL continues to rise across the frequency bands, reaching maximum IL at 56–71 Hz in the near field and 71–88 Hz in the far field. However, in the near field, the IL decreases to a range of 3.6–5.2 dB at the highest frequency band.

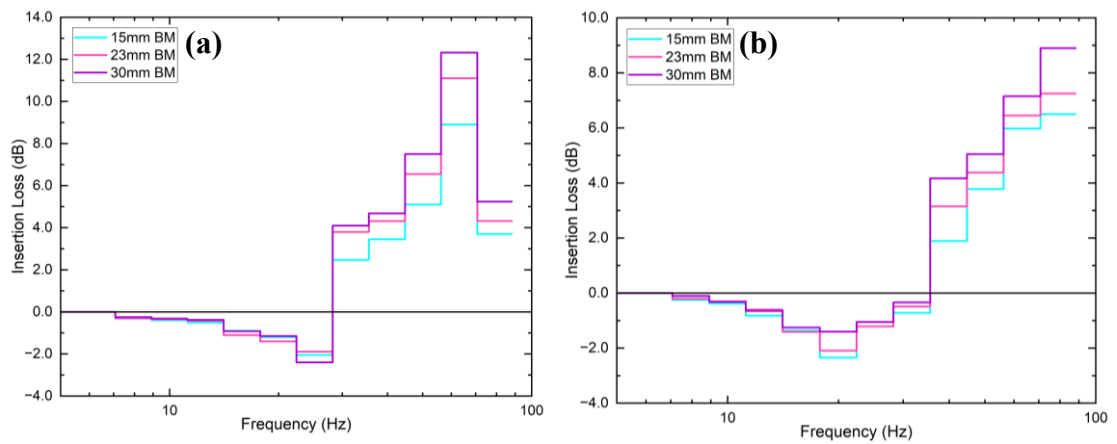


Figure 6.9: One-third octave spectrum of the insertion loss of vertical velocity across BM thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver

Figure 6.10 shows the one-third octave spectrum of the IL across BM stiffness in both near-field and far-field conditions. The trend is similar to what is seen in soft soil scenarios. Negative IL appears in the lower frequency bands, where increasing BM stiffness also enhances the resonance effect. A shift from negative to positive IL occurs at the previously mentioned frequency levels, with a gradual increase across the

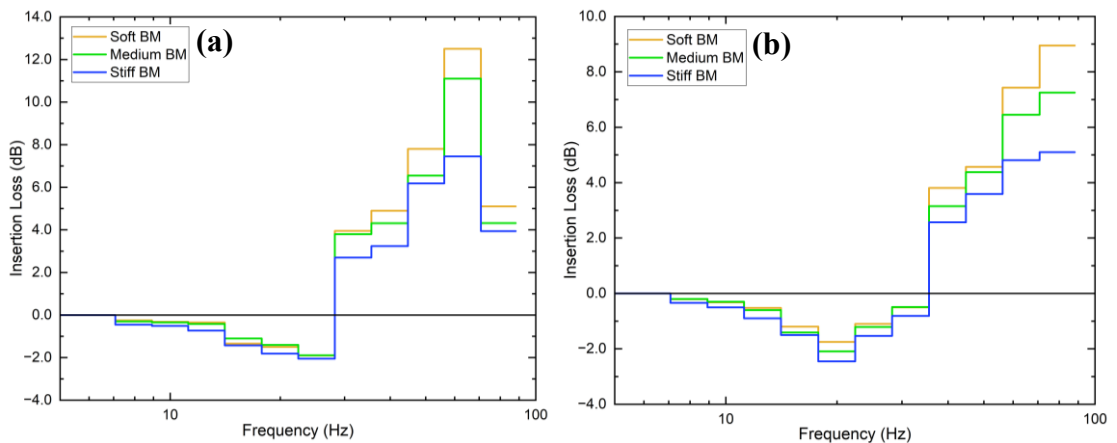


Figure 6.10: One-third octave spectrum of the insertion loss of vertical velocity across BM thickness on stiff soil at (a) 4 m receiver and (b) 16 m receiver

spectrum. Soft BMs reach the highest IL of about 12.5 dB within 56–71 Hz in the near field and 9 dB within 71–88 Hz in the far field. As BM stiffness increases, negative IL becomes more prominent while positive IL declines. The performance order matches expectations that greater compliance improves vibration isolation at higher frequencies.

6.4. Chapter Summary

This chapter assesses BMs on soft and stiff soils. Results show a strong thickness effect (30 mm > 23 mm > 15 mm) and a clear stiffness effect (soft > medium > stiff), with the best performance on stiff soil. Unlike USPs, BMs give robust reductions across distance, largest in the near field but still measurable in the far field, so they are the more effective and consistent option overall. Frequency-band plots indicate the main benefits in the mid-frequency range relevant to human perception. Practical notes are included: thicker/softer mats improve isolation but may increase track compliance, so selection should balance mitigation with serviceability.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1. Conclusion

This research developed and validated a comprehensive three-dimensional (3D) finite element (FE) model for the simulation and mitigation of train-induced ground vibration, with a focus on the effects of Under Sleeper Pads (USP) and Ballast Mats (BM) under contrasting soil conditions. The model demonstrated robust alignment with published field data across multiple validation metrics, including waveform correspondence, spectral content, running root mean square (RMS), and attenuation profiles, confirming its reliability for advanced parametric studies. The parametric investigations yielded several key conclusions for the considered mitigation measures on soft and stiff soil conditions.

In relation to the USP performance on soft soil, near-field vibration reduction was thickness-dependent, with 15 mm pads providing the highest insertion loss: 3.89 dB at 4 m, declining to 3.76 dB at 8 m. The 8 mm and 10 mm variants achieved 1.94–2.62 dB in the same range. Beyond 12 m, insertion loss dropped below 1–2 dB for all thicknesses, with far-field performance (16–19 m) dominated by soil damping and geometric spreading. Frequency specific study revealed that use of soft USP in soft soil condition can yield a coupled system whose natural frequency coincides with dominant excitation frequencies (typically 25–40 Hz), causing local resonance that hinder the damping effect of USP to provide a negligible reduction in vibration excitation. This behaviour is observed by Thadsanamoorthy (2023) also, where reducing the stiffness of the subsoil has shown some resonance effect in ground vibrations. Overall, Medium stiffness USPs outperformed soft and stiff alternatives in the near field, achieving 2.5–2.6 dB at 4–6 m. Frequency-band analysis confirmed that all USP configurations delivered positive insertion loss primarily above 45 Hz, with peak effectiveness in the 56–71 Hz range (up to 9 dB for 15 mm pads).

Considering the USP performance in stiff soil, thickness was the dominant parameter where near-field insertion losses at 4 m were 2.75 dB (8 mm), 4.22 dB (10 mm), and 6.43 dB (15 mm). At 6 m, values were 2.25, 3.86, and 6.18 dB, maintaining the same hierarchy. Stiffness variation showed soft pads delivering the highest insertion loss (4.97 dB at 4 m, 4.79 dB at 6 m), followed by medium (4.22, 3.86 dB) and stiff (2.91, 2.05 dB). Beyond 16 m, all configurations converged below 1 dB. low-frequency amplification was observed in stiff soil, with positive insertion loss beginning around 45 Hz (36 Hz for soft pads). Peak benefits occurred at 56–71 Hz, reaching 7–9 dB in the near field and 5–6 dB at 16 m.

Again, thickness effects were pronounced in the BM performance in soft soil. 15 mm mats provided approximately 2 dB insertion loss at 4–8 m; 23 mm yielded 3–4 dB; and 30 mm achieved 4.7–5.7 dB. Far-field benefits (12–19 m) remained measurable at 1–3 dB for 23–30 mm variants. Stiffness classification showed soft mats delivering 4–

5 dB in the near field, medium 3–4 dB, and stiff about 2 dB. At 16–19 m, soft and medium configurations maintained 1–2 dB, while stiff mats dropped below 1 dB. Frequency analysis revealed small negative insertion loss in the 7–35 Hz band, transitioning to positive above 35 Hz and peaking at 56–71 Hz, where thicker and softer mats provided the greatest gains. BMs substantially outperformed USPs on soft soil, particularly in the far field, demonstrating more robust attenuation characteristics across distance.

BM performance in stiff soil is also thickness dominated, where 15 mm mats yielded 2.4–4.44 dB; 23 mm provided 3.0–5.35 dB; and 30 mm achieved 4.74–9.2 dB across all receivers. At 4 m, the 30 mm mat delivered 9.19 dB insertion loss, maintaining 4.73 dB even at 19 m. Soft and medium stiffness mats exceeded 3 dB at all receivers (peaks: 6.56 dB soft, 5.35 dB medium), while stiff variants reached only 3.35 dB at 4 m and dropped below 2 dB by 16 m. Frequency-band analysis showed minimal negative insertion loss below 28–35 Hz, with strong positive insertion loss above these thresholds, peaking at 56–71 Hz (in near field reaching 12.5 dB for soft mats and in far field reaching 9 dB). Stiff soil consistently enabled superior mitigation effectiveness, with BM insertion losses 3–8 dB higher than equivalent USP configurations and benefits sustained to distances exceeding 19 m.

Based on the above information, following generalized conclusions can be derived.

- Both USPs and BMs are significantly more effective in stiff soil than soft soil, achieving larger insertion loss and vibration reduction in the near-field and extending attenuation benefits further into the far field. In soft soil, near-field amplification (negative insertion loss) can occur, especially with soft or thick USPs, due to coupled system resonances, whereas stiff soil consistently enables strong vibration mitigation without amplification risk.
- Thicker and softer BMs provide the greatest reductions in vibration levels, with 30 mm thick mats and soft stiffness classes achieving up to 9–12 dB insertion loss near the track in stiff soil. For USPs, medium stiffness and the thickest pads provided the optimal balance in soft soil, maximizing reduction while minimizing resonance amplification; in stiff soil, the softest USPs performed best.
- All mitigation effects are strongest in the near-field (within 8–10 m of the track) and diminish with distance due to geometric spreading and inherent soil damping. However, BMs consistently outperformed USPs in the far field, especially on stiff soil, providing robust performance up to the 19 m mark.
- For both USP and BM, the most substantial vibration reductions were observed in the mid-frequency bands (typically 30–80 Hz), which are critical for human perception and structural impact. Frequency-selective insertion loss analysis confirmed that thicker and softer mitigation elements shift attenuation benefits toward these crucial bands.

7.2. Recommendations

Based on the findings of this study, following recommendations can be made.

For soft soil conditions ($E = 140$ MPa),

- Select medium stiffness USPs with 10–15 mm thickness to maximize near-field reduction (2.5–3.9 dB) while avoiding low-frequency amplification. Avoid soft pads within 8 m of the track where resonance risks are highest. Expect limited benefits beyond 16 m (< 1 dB insertion loss).
- For BMs, prioritize 30 mm thickness with soft or medium stiffness for optimal performance (4.7–5.7 dB near field; 1–3 dB far field). If construction constraints limit thickness, 23 mm medium mats provide reasonable compromise at 3–4 dB near field and 1–2 dB far field. Avoid stiff mats, which deliver < 1 dB in the far field.
- Design for improvements primarily in the 45–80 Hz range, where positive insertion loss is strongest and most consistent.

For stiff soil conditions ($E = 720$ MPa),

- For USPs, Specify 15 mm thickness with soft or medium stiffness for maximum near-field reduction (6.43 dB at 4 m). For balanced performance with cost constraints, 10 mm medium stiffness delivers 4.22 dB (38% reduction). Stiff pads offer minimal benefit (2.91 dB) and should be avoided where vibration control is prioritized.
- When vibration reduction is the primary objective, deploy 30 mm soft or medium BMs to achieve peak insertion losses (9.2 dB near field; 4.7 dB at 19 m). For projects with build-height or settlement limitations, 23 mm medium mats provide 3.0–5.35 dB across the vibration field. Stiff mats, while offering lower vibration benefit (3.35 dB maximum), may be considered where track component protection dominates design criteria.
- Stiff soil enables aggressive mitigation strategies without amplification risk, supporting use of softest available materials.

However, the above recommendations are applicable to performance of USPs and BMs under high-speed train passage on ballasted embankments within the tested soil profile.

Although validation and the main parametric study used high-speed data from abroad, the model is directly adaptable to Sri Lankan conventional rail, which mostly runs at sub-critical speeds (less than the Rayleigh wave speed of the soil).

- Since most Sri Lankan lines are at-grade ballasted embankments, the FE model can be applied with minimal changes by updating the layer thicknesses to match local track build-ups.
- Adjust the material properties (rail, sleepers, ballast, sub-ballast, subgrade) to reflect the target site. Where layered soils exist, model the subsoil as layered

and assign appropriate stiffness, density, damping (per layer) rather than a single homogeneous stratum.

- Replace the train inputs in the VDLOAD with the dynamic properties of the relevant Sri Lankan rolling stock (axle loads, bogie spacing, car masses, speed).
- The model is validated for sub-critical conditions; Sri Lankan operating speeds are lower and within this regime. Therefore, it can be used to predict train-induced ground vibration in Sri Lanka with good accuracy, subject to the above localisations.

7.3. Future Work

To build upon this study, the following future directions are proposed:

- Extend the validation for lower speeds that are available in Sri Lanka and check for the performance of USPs and BMs under those speeds.
- Test stratified soil profiles to see how layers change vibration paths, create waveguide amplification, and shift resonant bands. USP/BM performance should be re-checked in these conditions, ideally with field data from sites that have good geophysical profiles and sensors at different depths.
- Trains running near the soil Rayleigh speed can cause strong amplification. The model should be extended to near-critical and super-critical speeds to track how mechanisms change and to see if USPs/BMs still help, or if alternatives (soil stiffening, track stiffness tuning, barriers) are needed. A small parametric study varying speed, axle spacing, and soil wave velocities would give practical guidance for high-speed lines on weak ground.
- Study applicability of machine learning models using a wide range of data set generated from FE simulations in providing near-instant estimates for different soils, trains, and mitigation settings.

REFERENCES

- Abiram, P. (2021). *Characteristic Analysis of Embedded Wave Barrier Material Beside the Railway Track* [BSc Thesis]. University of Moratuwa.
- Adam, D., Vogel, A., & Zimmermann, A. (2007). Ground improvement techniques beneath existing rail tracks. *Proceedings of the Institution of Civil Engineers - Ground Improvement*, 11(4), 229–235.
<https://doi.org/10.1680/grim.2007.11.4.229>
- Alves Costa, P., Calçada, R., & Silva Cardoso, A. (2012a). Ballast mats for the reduction of railway traffic vibrations. Numerical study. *Soil Dynamics and Earthquake Engineering*, 42, 137–150.
<https://doi.org/10.1016/j.soildyn.2012.06.014>
- Alves Costa, P., Calçada, R., & Silva Cardoso, A. (2012b). Ballast mats for the reduction of railway traffic vibrations. Numerical study. *Soil Dynamics and Earthquake Engineering*, 42, 137–150.
<https://doi.org/10.1016/j.soildyn.2012.06.014>
- Andersen, L., & Jones, C. J. C. (2006). Coupled boundary and finite element analysis of vibration from railway tunnels—A comparison of two- and three-dimensional models. *Journal of Sound and Vibration*, 293(3–5), 611–625.
<https://doi.org/10.1016/j.jsv.2005.08.044>
- Arachchi, S.S.L., (2023). *Investigate the Methods of Controlling the Train Induced Ground Vibrations* [BSc Thesis]. University of Moratuwa.
- Auersch, L. (2005). The excitation of ground vibration by rail traffic: Theory of vehicle–track–soil interaction and measurements on high-speed lines. *Journal of Sound and Vibration*, 284(1–2), 103–132.
<https://doi.org/10.1016/j.jsv.2004.06.017>
- Auersch, L. (2006). Two- and Three-dimensional Methods for the Assessment of Ballast Mats, Ballast Plates and Other Isolators of Railway Vibration. *The International Journal of Acoustics and Vibration*, 11(4).
<https://doi.org/10.20855/ijav.2006.11.4200>
- Auersch, L. (2021). Train-induced ground vibration due to the irregularities of the soil. *Soil Dynamics and Earthquake Engineering*, 140, 106438.
<https://doi.org/10.1016/j.soildyn.2020.106438>
- Avillez, J., Frost, M., Cawser, S., Skinner, C., El-Hamalawi, A., & Shields, P. (2012). Procedures for Estimating Environmental Impact From Railway Induced

Vibration: A Review. *ASME 2012 Noise Control and Acoustics Division Conference*, 381–392. <https://doi.org/10.1115/NCAD2012-1083>

Bahrekazemi, M. (2004). *Train-induced ground vibration and its prediction* [Doctoral disssertation]. Royal Institute of Technology.

Barbosa, J. M. D. O., Park, J., & Kausel, E. (2012). Perfectly matched layers in the thin layer method. *Computer Methods in Applied Mechanics and Engineering*, 217–220, 262–274. <https://doi.org/10.1016/j.cma.2011.12.006>

Berenger, J.-P. (1994). A perfectly matched layer for the absorption of electromagnetic waves. *Journal of Computational Physics*, 114(2), 185–200. <https://doi.org/10.1006/jcph.1994.1159>

Bettess, P., & Zienkiewicz, O. C. (1977). Diffraction and refraction of surface waves using finite and infinite elements. *International Journal for Numerical Methods in Engineering*, 11(8), 1271–1290. <https://doi.org/10.1002/nme.1620110808>

Bewes, O. G., Jakielaszek, L. J., & Richardson, M. L. (2015). An Assessment of the Effectiveness of Replacing Slab Track to Control Groundborne Noise and Vibration in Buildings above an Existing Railway Tunnel. In J. C. O. Nielsen, D. Anderson, P.-E. Gautier, M. Iida, J. T. Nelson, D. Thompson, T. Tielkes, D. A. Towers, & P. De Vos (Eds.), *Noise and Vibration Mitigation for Rail Transportation Systems* (Vol. 126, pp. 393–400). Springer Berlin Heidelberg. https://doi.org/10.1007/978-3-662-44832-8_47

Bongini, E., Muller, R., Garburg, R., & Pieringer, A. (2012). *Design guide and technology assessment of the track mitigation measures*. (Nos. SCP0-GA-2010–265754). RIVAS Project.

Cao, Z., Cai, Y., & Han, J. (2014). Mitigation of Ground Vibration Generated by High-Speed Trains on Saturated Poroelastic Ground with Under-Sleeper Pads. *Journal of Transportation Engineering*, 140(1), 12–22. [https://doi.org/10.1061/\(ASCE\)TE.1943-5436.0000610](https://doi.org/10.1061/(ASCE)TE.1943-5436.0000610)

Çati, Y., Gökçeli, S., Anil, Ö., & Korkmaz, C. S. (2020). Experimental and numerical investigation of usp for optimization of transition zone of railway. *Engineering Structures*, 209, 109971. <https://doi.org/10.1016/j.engstruct.2019.109971>

Çelebi, E., Fırat, S., & Çankaya, İ. (2006). The effectiveness of wave barriers on the dynamic stiffness coefficients of foundations using boundary element method. *Applied Mathematics and Computation*, 180(2), 683–699. <https://doi.org/10.1016/j.amc.2006.01.008>

Central Environmental Authority: Pollution Control Division (2008), Interim ABOP and Vibration Standard

Chai, S., Tian, W., Yu, L., & Wang, H. (2020). Numerical Study of Ground Vibrations Caused by Cylindrical Wave Propagation in a Rock Mass with a Structural Plane. *Shock and Vibration*, 2020, 1–9. <https://doi.org/10.1155/2020/4681932>

Chen, A., Cheng, F., Wu, D., & Tang, X. (2019). Ground vibration propagation and attenuation of vibrating compaction. *Journal of Vibroengineering*, 21(5), 1342–1352. <https://doi.org/10.21595/jve.2019.20388>

Connolly, D., Giannopoulos, A., Fan, W., Woodward, P. K., & Forde, M. C. (2013). Optimising low acoustic impedance back-fill material wave barrier dimensions to shield structures from ground borne high speed rail vibrations. *Construction and Building Materials*, 44, 557–564. <https://doi.org/10.1016/j.conbuildmat.2013.03.034>

Connolly, D., Giannopoulos, A., & Forde, M. C. (2013). Numerical modelling of ground borne vibrations from high speed rail lines on embankments. *Soil Dynamics and Earthquake Engineering*, 46, 13–19. <https://doi.org/10.1016/j.soildyn.2012.12.003>

Connolly, D. P., & Costa, P. A. (2020). Geodynamics of very high speed transport systems. *Soil Dynamics and Earthquake Engineering*, 130, 105982. <https://doi.org/10.1016/j.soildyn.2019.105982>

Connolly, D. P., Kouroussis, G., Giannopoulos, A., Verlinden, O., Woodward, P. K., & Forde, M. C. (2014). Assessment of railway vibrations using an efficient scoping model. *Soil Dynamics and Earthquake Engineering*, 58, 37–47. <https://doi.org/10.1016/j.soildyn.2013.12.003>

Connolly, D. P., Kouroussis, G., Laghrouche, O., Ho, C. L., & Forde, M. C. (2015a). Benchmarking railway vibrations – Track, vehicle, ground and building effects. *Construction and Building Materials*, 92, 64–81. <https://doi.org/10.1016/j.conbuildmat.2014.07.042>

Connolly, D. P., Kouroussis, G., Laghrouche, O., Ho, C. L., & Forde, M. C. (2015b). Benchmarking railway vibrations – Track, vehicle, ground and building effects. *Construction and Building Materials*, 92, 64–81. <https://doi.org/10.1016/j.conbuildmat.2014.07.042>

Costa, P. A., Calçada, R., & Cardoso, A. S. (2012). Influence of train dynamic modelling strategy on the prediction of track–ground vibrations induced by railway traffic. *Proceedings of the Institution of Mechanical Engineers, Part*

F: Journal of Rail and Rapid Transit, 226(4), 434–450.
<https://doi.org/10.1177/0954409711433686>

- Coulier, P., Dijckmans, A., Jiang, J., Thompson, D. J., Degrande, G., & Lombaert, G. (2015). Stiff Wave Barriers for the Mitigation of Railway Induced Vibrations. In J. C. O. Nielsen, D. Anderson, P.-E. Gautier, M. Iida, J. T. Nelson, D. Thompson, T. Tielkes, D. A. Towers, & P. De Vos (Eds.), *Noise and Vibration Mitigation for Rail Transportation Systems* (Vol. 126, pp. 539–546). Springer Berlin Heidelberg. https://doi.org/10.1007/978-3-662-44832-8_63
- Croy, I., Smith, M. G., & Waye, K. P. (2013a). Effects of train noise and vibration on human heart rate during sleep: An experimental study. *BMJ Open*, 3(5), e002655. <https://doi.org/10.1136/bmjopen-2013-002655>
- Croy, I., Smith, M. G., & Waye, K. P. (2013b). Effects of train noise and vibration on human heart rate during sleep: An experimental study. *BMJ Open*, 3(5), e002655. <https://doi.org/10.1136/bmjopen-2013-002655>
- Degrande, G., & Schillemans, L. (2001). Free Field Vibrations During the Passage of a Thalys High-Speed Train at Variable Speed. *Journal of Sound and Vibration*, 247(1), 131–144. <https://doi.org/10.1006/jsvi.2001.3718>
- Dieterman, H. A., & Metrikine, A. (1997). Critical Velocities of a Harmonic Load Moving Uniformly Along an Elastic Layer. *Journal of Applied Mechanics*, 64(3), 596–600. <https://doi.org/10.1115/1.2788934>
- Dijckmans, A., Coulier, P., Jiang, J., Toward, M. G. R., Thompson, D. J., Degrande, G., & Lombaert, G. (2015). Mitigation of railway induced ground vibration by heavy masses next to the track. *Soil Dynamics and Earthquake Engineering*, 75, 158–170. <https://doi.org/10.1016/j.soildyn.2015.04.003>
- Dominguez, J. (1993). *Boundary Elements in Dynamics*. Computational Mechanics Publications and Elsevier Applied Science.
- Dong, K., Connolly, D. P., Laghrouche, O., Woodward, P. K., & Alves Costa, P. (2019). Non-linear soil behaviour on high speed rail lines. *Computers and Geotechnics*, 112, 302–318. <https://doi.org/10.1016/j.compgeo.2019.03.028>
- Eitzenberger, A. (2008). *Train-induced Vibrations in Tunnels – A Review*. Luleå University of Technology.
- Ekevid, T., & Wiberg, N.-E. (2002). Wave propagation related to high-speed train. *Computer Methods in Applied Mechanics and Engineering*, 191(36), 3947–3964. [https://doi.org/10.1016/S0045-7825\(02\)00341-9](https://doi.org/10.1016/S0045-7825(02)00341-9)

- Erlingsson, S. (1993). *Dynamic soil analysis with an application to rock induced vibrations in Ullevi Stadium* [Ph.D. Dissertation]. Royal Institute of Technology.
- Ferro, E., Harkness, J., & Le Pen, L. (2020). The influence of sleeper material characteristics on railway track behaviour: Concrete vs composite sleeper. *Transportation Geotechnics*, 23, 100348. <https://doi.org/10.1016/j.trgeo.2020.100348>
- François, S., Schevenels, M., Galvín, P., Lombaert, G., & Degrande, G. (2010). A 2.5D coupled FE–BE methodology for the dynamic interaction between longitudinally invariant structures and a layered halfspace. *Computer Methods in Applied Mechanics and Engineering*, 199(23–24), 1536–1548. <https://doi.org/10.1016/j.cma.2010.01.001>
- François, S., Schevenels, M., Thyssen, B., Borgions, J., & Degrande, G. (2012). Design and efficiency of a composite vibration isolating screen in soil. *Soil Dynamics and Earthquake Engineering*, 39, 113–127. <https://doi.org/10.1016/j.soildyn.2012.03.007>
- Galvín, P., & Romero, A. (2014). A 3D time domain numerical model based on half-space Green’s function for soil–structure interaction analysis. *Computational Mechanics*, 53(5), 1073–1085. <https://doi.org/10.1007/s00466-013-0949-1>
- Galvín, P., Romero, A., & Domínguez, J. (2010a). Fully three-dimensional analysis of high-speed train–track–soil–structure dynamic interaction. *Journal of Sound and Vibration*, 329(24), 5147–5163. <https://doi.org/10.1016/j.jsv.2010.06.016>
- Galvín, P., Romero, A., & Domínguez, J. (2010b). Fully three-dimensional analysis of high-speed train–track–soil–structure dynamic interaction. *Journal of Sound and Vibration*, 329(24), 5147–5163. <https://doi.org/10.1016/j.jsv.2010.06.016>
- García-Bennett, A., Jones, C. J. C., & Thompson, D. J. (2012). A Numerical Investigation of Railway Ground Vibration Mitigation Using a Trench in a Layered Soil. In T. Maeda, P.-E. Gautier, C. E. Hanson, B. Hemsworth, J. T. Nelson, B. Schulte-Werning, D. Thompson, & P. Vos (Eds.), *Noise and Vibration Mitigation for Rail Transportation Systems* (Vol. 118, pp. 315–322). Springer Japan. https://doi.org/10.1007/978-4-431-53927-8_37
- Giannopoulos, A., Melling, S., & Connolly, D. (2012, September 3). *A Second Order PML Implementation for FDTD Seismic Modelling*. Near Surface Geoscience 2012 – 18th European Meeting of Environmental and Engineering Geophysics, Paris, France. <https://doi.org/10.3997/2214-4609.20143325>

- Grassie, S. L., Gregory, R. W., Harrison, D., & Johnson, K. L. (1982). The Dynamic Response of Railway Track to High Frequency Vertical Excitation. *Journal of Mechanical Engineering Science*, 24(2), 77–90. https://doi.org/10.1243/JMES_JOUR_1982_024_016_02
- Graves, R. W. (1996). Simulating seismic wave propagation in 3D elastic media using staggered-grid finite differences. *Bulletin of the Seismological Society of America*, 86(4), 1091–1106. <https://doi.org/10.1785/BSSA0860041091>
- Grootenhuis, P. (1977). *Floating track slab isolation for railways*. 51(3), 443–448.
- Hall, L. (2000). *Simulations and Analyses of Traininduced Ground Vibrations* [Doctoral disssertation]. Royal Institute of Technology.
- Hanson, C., Towers, D., & Meister, L. (2006). *Transit Noise and Vibration Impact Assessment* (No. FTA-VA-90-1003-06). Federal Transit Administration.
- Hassoun, M., Ulgen, D., Toygar, O., & Durmaz, S. (2019). *Numerical study on reducing the ground borne vibrations using open trench and rubber-chips filled trench wave barriers*. 1st Mediterranean young geotechnical engineers conference, Mulga, Turkey.
- Hemsworth, B. (2000). Reducing Ground borne Vibrations: State-Of-The-Art Study. *Journal of Sound and Vibration*, 231(3), 703–709. <https://doi.org/10.1006/jsvi.1999.2642>
- Hildebrand, R. (2001). *Countermeasures Against Railway Ground and Track Vibrations* [Doctoral thesis]. KTH Royal Institute of Technology.
- Holm, G., Andreasson, B., Bengtsson, P., Bodare, A., & Eriksson, H. (2002). *Mitigation of track and ground vibrations by high speed trains at Ledsgård, Sweden*. Swedish National Road and Transport Research Institute (VTI).
- Hunt, H. E. M., Hussein, M. F. M., & Talbot, J. P. (2003). *Insertion Loss Models for Evaluating The Performance of Vibration Counter-Measures for Underground Railways*. 10th International Congress On Sound And Vibration, ICSV10.
- Insa, R., Salvador, P., Inarejos, J., & Roda, A. (2012). Analysis of the influence of under sleeper pads on the railway vehicle/track dynamic interaction in transition zones. *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit*, 226(4), 409–420. <https://doi.org/10.1177/0954409711430174>
- International Organization for Standardization. (1990). *Mechanical vibration and shock—Vibration of buildings—Guidelines for the measurement of vibrations*

and evaluation of their effects on structures (ISO 4866:1990). Geneva, Switzerland: ISO.

International Organization for Standardization. (2010). *Mechanical vibration and shock—Vibration of fixed structures—Guidelines for the measurement of vibrations and evaluation of their effects on structures* (ISO 4866:2010). Geneva, Switzerland: ISO.

International Organization for Standardization. (1997). *Mechanical vibration and shock—Evaluation of human exposure to whole-body vibration—Part 1: General requirements* (ISO 2631-1:1997). Geneva, Switzerland: ISO.

International Organization for Standardization. (2003). *Mechanical vibration and shock—Evaluation of human exposure to whole-body vibration—Part 1: General requirements* (ISO 2631-1:1997/Amd 1:2010). Geneva, Switzerland: ISO.

International Organization for Standardization. (2000). *Mechanical vibration and shock—Vibration and shock in buildings with sensitive equipment—Part 1: Measurement and evaluation* (ISO/TS 10811-1:2000). Geneva, Switzerland: ISO.

International Organization for Standardization. (2000). *Mechanical vibration and shock—Vibration and shock in buildings with sensitive equipment—Part 2: Classification* (ISO/TS 10811-2:2000). Geneva, Switzerland: ISO.

International Union of Railways. (2008). *Earthworks and track-bed layers for railway lines* (UIC Code 719 R). Paris, France: UIC.

International Union of Railways. (2022). *Track and structure recommendations for the use of ballasted track* (UIC IRS 70719-1). Paris, France: UIC.

International Union of Railways. (2017). *Railway induced vibration—State of the art report*. Paris, France: UIC.

Jean, P., Guigou, C., & Villot, M. (2004). A 2.5D BEM Model for Ground-Structure Interaction. *Building Acoustics*, 11(3), 157–173. <https://doi.org/10.1260/1351010042250376>

Johnson, K. (1985). *Contact mechanics*. Cambridge University Press.

Jones, C. J. C., Thompson, D. J., & Petyt, M. (2002). A model for ground vibration from railway tunnels. *Proceedings of the Institution of Civil Engineers - Transport*, 153(2), 121–129. <https://doi.org/10.1680/tran.2002.153.2.121>

Kaewunruen, S., Aikawa, A., & Remennikov, A. M. (2017). Vibration Attenuation at Rail Joints through under Sleeper Pads. *Procedia Engineering*, 189, 193–198. <https://doi.org/10.1016/j.proeng.2017.05.031>

- Katou, M., Matsuoka, T., Yoshioka, O., Sanada, Y., & Miyoshi, T. (2008). Numerical simulation study of ground vibrations using forces from wheels of a running high-speed train. *Journal of Sound and Vibration*, 318(4–5), 830–849. <https://doi.org/10.1016/j.jsv.2008.04.053>
- Kattis, S. E., Polyzos, D., & Beskos, D. E. (1999). Vibration isolation by a row of piles using a 3-D frequency domain BEM. *International Journal for Numerical Methods in Engineering*, 46(5), 713–728. [https://doi.org/10.1002/\(SICI\)1097-0207\(19991020\)46:5%253C713::AID-NME693%253E3.0.CO;2-U](https://doi.org/10.1002/(SICI)1097-0207(19991020)46:5%253C713::AID-NME693%253E3.0.CO;2-U)
- Kouroussis, G., Connolly, D. P., & Verlinden, O. (2014). Railway-induced ground vibrations – a review of vehicle effects. *International Journal of Rail Transportation*, 2(2), 69–110. <https://doi.org/10.1080/23248378.2014.897791>
- Kouroussis, G., Van Parys, L., Conti, C., & Verlinden, O. (2013). *Prediction of ground vibrations induced by urban railway traffic: An analysis of the coupling assumptions between vehicle, track, soil, and buildings*. 18(4), 163–172.
- Kouroussis, G., Verlinden, O., & Conti, C. (2011). Free field vibrations caused by high-speed lines: Measurement and time domain simulation. *Soil Dynamics and Earthquake Engineering*, 31(4), 692–707. <https://doi.org/10.1016/j.soildyn.2010.11.012>
- Kouroussis, G., Verlinden, O., & Conti, C. (2012). Efficiency of resilient wheels on the alleviation of railway ground vibrations. *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit*, 226(4), 381–396. <https://doi.org/10.1177/0954409711429210>
- Krylov, V., & Ferguson, C. (1994). Calculation of low-frequency ground vibrations from railway trains. *Applied Acoustics*, 42(3), 199–213. [https://doi.org/10.1016/0003-682X\(94\)90109-0](https://doi.org/10.1016/0003-682X(94)90109-0)
- Lakus'ic', S., & Ahac, M. (2012). *Rail traffic noise and vibration mitigation measures in urban areas* (Technical Gazette No. 19(2); pp. 427–435).
- Lakusic, S., Ahac, M., & Haladin, I. (2010). *Experimental investigation of railway track with under sleeper pad*. 10th Slovenian Road and Transportation Congress, Portorož, Slovenia.
- Lamb, H. (1904). On the propagation of tremors over the surface of an elastic solid. *Philosophical Transactions of the Royal Society of London, Series A*, 203(359–371), 1–42. <https://doi.org/10.1098/rsta.1904.0013>
- Li, L., Nimbalkar, S., & Zhong, R. (2018). Finite element model of ballasted railway with infinite boundaries considering effects of moving train loads and Rayleigh

- waves. *Soil Dynamics and Earthquake Engineering*, 114, 147–153. <https://doi.org/10.1016/j.soildyn.2018.06.033>
- Lombaert, G., & Degrande, G. (2009a). Ground-borne vibration due to static and dynamic axle loads of InterCity and high-speed trains. *Journal of Sound and Vibration*, 319(3–5), 1036–1066. <https://doi.org/10.1016/j.jsv.2008.07.003>
- Lombaert, G., & Degrande, G. (2009b). Ground-borne vibration due to static and dynamic axle loads of InterCity and high-speed trains. *Journal of Sound and Vibration*, 319(3–5), 1036–1066. <https://doi.org/10.1016/j.jsv.2008.07.003>
- Lu, F., Gao, Q., Lin, J. H., & Williams, F. W. (2006). Non-stationary random ground vibration due to loads moving along a railway track. *Journal of Sound and Vibration*, 298(1–2), 30–42. <https://doi.org/10.1016/j.jsv.2006.04.041>
- Lysmer, J. (1978). *Analytical procedures in soil dynamics*. 3, 1267–1317.
- Madshus, C., & Kaynia, A. M. (2000). High-Speed Railway Lines on Soft Ground: Dynamic Behaviour at Critical Train Speed. *Journal of Sound and Vibration*, 231(3), 689–701. <https://doi.org/10.1006/jsvi.1999.2647>
- Massarsch, K. R., & Fellenius, B. H. (2008). *Ground Vibrations Induced by Impact Pile Driving*. 3.
- Mateus, E. P., Gomes Da Silva, M. D. R., Ribeiro, A. B., & Marriott, P. J. (2008). Qualitative mass spectrometric analysis of the volatile fraction of creosote-treated railway wood sleepers by using comprehensive two-dimensional gas chromatography. *Journal of Chromatography A*, 1178(1–2), 215–222. <https://doi.org/10.1016/j.chroma.2007.11.069>
- Mirza, A. A., Frid, A., Nielsen, J. C. O., & Jones, C. J. C. (2012). Ground Vibration Induced by Railway Traffic – The Influence of Vehicle Parameters. In T. Maeda, P.-E. Gautier, C. E. Hanson, B. Hemsworth, J. T. Nelson, B. Schulte-Werning, D. Thompson, & P. Vos (Eds.), *Noise and Vibration Mitigation for Rail Transportation Systems* (Vol. 118, pp. 259–266). Springer Japan. https://doi.org/10.1007/978-4-431-53927-8_30
- Nelson, J. T., & Saurenman, H. J. (1983). State-of-the-art review: Prediction and control of groundborne noise and vibration from rail transit trains (Report No. DOT-TSC-UMTA-83-3). Cambridge, MA: U.S. Department of Transportation, Transportation Systems Center.
- Nielsen, J. C. O., & Johansson, A. (2000). Out-of-round railway wheels-a literature survey. *Proceedings of the Institution of Mechanical Engineers, Part F:*

Journal of Rail and Rapid Transit, 214(2), 79–91.
<https://doi.org/10.1243/0954409001531351>

Olivier, B., Connolly, D. P., Alves Costa, P., & Kouroussis, G. (2016). The effect of embankment on high speed rail ground vibrations. *International Journal of Rail Transportation*, 4(4), 229–246.
<https://doi.org/10.1080/23248378.2016.1220844>

Ouakka, S., Verlinden, O., & Kouroussis, G. (2022). Railway ground vibration and mitigation measures: Benchmarking of best practices. *Railway Engineering Science*, 30(1), 1–22. <https://doi.org/10.1007/s40534-021-00264-9>

Paul de Vos, S. (2017). *Railway induced vibration—State of the art report*. UIC-ETC (Railway Technical Publications).

Peterie, S. L., Miller, R. D., & Ivanov, J. (2014). Seismology and Its Applications in Kansas. *Public Information Circular*, 37.

Petyt, M. (2010). *Introduction to finite element vibration analysis*. Cambridge university press.

Pita, A. L., Teixeira, P. F., & Robuste, F. (2004). High speed and track deterioration: The role of vertical stiffness of the track. *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit*, 218(1), 31–40. <https://doi.org/10.1243/095440904322804411>

Romero, A., Galvin, P., & Dominguez, J. (2010). Fully 3D analysis of HST-track-soil-structure dynamic interaction. *Proceedings of Isma 2010*.

Sandqvist, N., & Milicevic, M. (2020). *Soil-Structure Interaction Analysis of Portal Frame Railway Bridges: Numerical Analysis of Two Case Study Bridges* [Master Thesis]. KTH Royal Institute of Technology.

Sankeeth, S. (2022). *Numerical Analysis of the Vibration Behavior of the Sri Lankan Railway Track System* [BSc Thesis]. University of Moratuwa.

Sehner, M., Valdés Nava, L. E., Seidl-Nigsch, M., & Loy, H. (2023). Vibration Mitigation: Under-Ballast Mats in Heavy-Haul Applications. *Practice Periodical on Structural Design and Construction*, 28(4), 05023004. <https://doi.org/10.1061/PPSCFX.SCENG-1258>

Sheng, X. (2001). *Ground vibrations generated from trains* [PhD thesis]. University of Southampton.

- Sheng, X., Jones, C. J. C., & Petyt, M. (1999a). Ground Vibration Generated by a Harmonic Load Acting on a Railway Track. *Journal of Sound and Vibration*, 225(1), 3–28. <https://doi.org/10.1006/jsvi.1999.2232>
- Sheng, X., Jones, C. J. C., & Petyt, M. (1999b). Ground Vibration Generated by a Load Moving Along a Railway Track. *Journal of Sound and Vibration*, 228(1), 129–156. <https://doi.org/10.1006/jsvi.1999.2406>
- Sheng, X., Jones, C. J. C., & Thompson, D. J. (2005). Modelling ground vibration from railways using wavenumber finite- and boundary-element methods. *Proceedings of the Royal Society A: Mathematical, Physical and Engineering Sciences*, 461(2059), 2043–2070. <https://doi.org/10.1098/rspa.2005.1450>
- Sheng, X., Jones, C. J. C., & Thompson, D. J. (2006). Prediction of ground vibration from trains using the wavenumber finite and boundary element methods. *Journal of Sound and Vibration*, 293(3–5), 575–586. <https://doi.org/10.1016/j.jsv.2005.08.040>
- Smith, M., Ögren, M., & Hammar, O. et al. (2015). *Physiological reaction thresholds to vibration during sleep*. Euronoise 2015, Maastricht.
- Sol-Sánchez, M., Moreno-Navarro, F., & Rubio-Gámez, M. C. (2015). The use of elastic elements in railway tracks: A state of the art review. *Construction and Building Materials*, 75, 293–305. <https://doi.org/10.1016/j.conbuildmat.2014.11.027>
- vinkin, M. R. (2008). *Soil and Structure Vibrations from Construction and Industrial Sources*. 8.
- Thadsanamoorthy, P. (2023). *Investigation of Ground Vibrations due to Moving Trains* [MSc Thesis]. University of Moratuwa.
- Thompson, D. (2009). *Railway noise and vibration. Mechanisms, modelling and means of control*. Elsevier Ltd.
- Thompson, D. J., Jiang, J., Toward, M. G. R., Hussein, M. F. M., Ntotsios, E., Dijckmans, A., Coulier, P., Lombaert, G., & Degrande, G. (2016). Reducing railway-induced ground-borne vibration by using open trenches and soft-filled barriers. *Soil Dynamics and Earthquake Engineering*, 88, 45–59. <https://doi.org/10.1016/j.soildyn.2016.05.009>
- Thompson, D. J., Kouroussis, G., & Ntotsios, E. (2019). Modelling, simulation and evaluation of ground vibration caused by rail vehicles. *Vehicle System Dynamics*, 57(7), 936–983. <https://doi.org/10.1080/00423114.2019.1602274>

- Thornely-Taylor, R. M. (2004). The prediction of vibration, groundborne noise and structure-radiated noise using finite difference methods. *Proc 8th International Workshop on Railway Noise*.
- UIC. (2007). *Railway induced vibration—State of the art report*. International Union of Railways.
- UIC. (2011). *RIVAS Railway Induced Vibration Abatement Solutions Collaborative project*. International Union of Railways.
- Waddington, D., Woodcock, J., Smith, M. G., Janssen, S., & Persson Waye, K. (2015). CargoVibes: Human response to vibration due to freight rail traffic. *International Journal of Rail Transportation*, 3(4), 233–248. <https://doi.org/10.1080/23248378.2015.1076623>
- Wheel/Rail Noise* (No. Report 23). (1997). Transportation research board.
- Wilson, G. P., Saurenman, H. J., & Nelson, J. T. (1983). Control of ground-borne noise and vibration. *Journal of Sound and Vibration*, 87(2), 339–350. [https://doi.org/10.1016/0022-460X\(83\)90573-4](https://doi.org/10.1016/0022-460X(83)90573-4)
- Wu, Y., Du, X., Zhang, H., Wen, Z., & Jin, X. (2017). Experimental analysis of the mechanism of high-order polygonal wear of wheels of a high-speed train. *Journal of Zhejiang University-SCIENCE A*, 18(8), 579–592. <https://doi.org/10.1631/jzus.A1600741>
- Yang, Y. B., Hung, H. H., & Chang, D. W. (2003). Train-induced wave propagation in layered soils using finite/infinite element simulation. *Soil Dynamics and Earthquake Engineering*, 23(4), 263–278. [https://doi.org/10.1016/S0267-7261\(03\)00003-4](https://doi.org/10.1016/S0267-7261(03)00003-4)
- Yuan, H., Zhu, R., Huang, J., Li, J., & Zou, C. (2021). Ground vibration mitigation by using ballast mat. *Vibroengineering Procedia*, 37, 19–23. <https://doi.org/10.21595/vp.2021.21902>