

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**TRAIN INDUCED VIBRATION AND ITS EFFECTS ON
BUILDING STRUCTURES**

EZGİ TEKERGÜL
**A THESIS SUBMITTED FOR THE DEGREE OF
MASTER OF SCIENCE**
DEPARTMENT OF CIVIL ENGINEERING
EARTHQUAKE AND STRUCTURAL ENGINEERING PROGRAMME

GEBZE
2020

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**TRAIN INDUCED VIBRATION AND ITS
EFFECTS ON BUILDING STRUCTURES**

EZGİ TEKERGÜL
A THESIS SUBMITTED FOR THE DEGREE OF
MASTER OF SCIENCE
DEPARTMENT OF CIVIL ENGINEERING
EARTHQUAKE AND STRUCTURAL ENGINEERING PROGRAMME

THESIS SUPERVISOR
ASSIST. PROF. DR. ABDULLAH CAN ZÜLFİKAR

GEBZE
2020

T.C.
GEBZE TEKNİK ÜNİVERSİTESİ
FEN BİLİMLERİ ENSTİTÜSÜ

TREN GEÇİŞİ KAYNAKLI TİTREŞİM VE
YAPILAR ÜZERİNDEKİ ETKİSİ

EZGİ TEKERGÜL
YÜKSEK LİSANS TEZİ
İNŞAAT MÜHENDİSLİĞİ ANABİLİM DALI
DEPREM VE YAPI MÜHENDİSLİĞİ PROGRAMI

DANIŞMANI
DR. ÖĞR. ÜYESİ ABDULLAH CAN ZÜLFİKAR

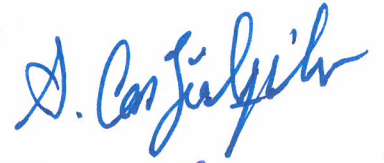
GEBZE
2020

GTÜ Fen Bilimleri Enstitüsü Yönetim Kurulu'nun 29/01/2020 tarih ve 2020/07 sayılı kararıyla oluşturulan jüri tarafından 14/02/2020 tarihinde tez savunma sınavı yapılan Ezgi TEKERGÜL' ün tez çalışması İnşaat Mühendisliği Anabilim Dalı/ Deprem ve Yapı Mühendisliği Programında YÜKSEK LİSANS tezi olarak kabul edilmiştir.

JÜRİ

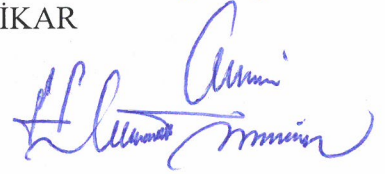
ÜYE

(TEZ DANIŞMANI) : Dr. Öğr. Üyesi Abdullah Can ZÜLFİKAR



ÜYE

: Prof. Dr. Erkan ÇELEBİ



ÜYE

: Dr. Öğr. Üyesi Saed MOGHIMI



ONAY

Gebze Teknik Üniversitesi Fen Bilimleri Enstitüsü Yönetim Kurulu'nun

...../...../..... tarih ve/..... sayılı kararı.

SUMMARY

With the advancement of technology, sources and job opportunities diversified in urban centers recently, and both inter-city transfers and urban transportation are expanding in direct proportion with population growth. Therefore, transportation and public transportation in a safe and fast manner become highly important day by day. Railway transportation is the most preferred and reliable system amongst transportation types. Although it has many advantages such as energy-saving, cost-effectiveness, reliability, and independence of weather conditions, it causes discomforting vibrations, especially in the urban city center. In my thesis, the effect of vibrations caused by train passes on the track line was investigated. Several field studies carried out, and an excavated trench barrier has used to reduce the impact of vibrations. Vibration levels due to freight and passenger trains with high-speed trains measured for three-components of the vibration motion, up to 40 meters from the railway line. The average V_{s30} value in the selected test site is 200m/s. The ground vibrations examined both in the time and frequency domain. Also, the results calculated separately for the free-field and barrier-field, accordingly compared with each other. To investigate the effect of vibrations on the structures, three different 2D frame system buildings modeled and linear time history analysis performed with the train records recorded from the field. Finally, the peak acceleration, velocity, and displacement values of each story of models examined and compared with the international standards.

Key Words: Train Vibration, Traffic-Induced Vibrations, Vibration Measurements, Wave Propagation

ÖZET

Teknolojinin gelişimiyle birlikte son yıllarda kent merkezlerinde artan olanaklar, iş imkanları, geçmiş yıllara göre ivmelenerek artış göstermektedir. Gerek şehirler arası transferler gerekse kent içi ulaşım ihtiyacı nüfus artışı ile doğru orantıda artmaktadır. Bu nedenle ulaşım ve yolcu taşımacılığı gün geçtikçe daha büyük önem arz etmektedir. Tren yolu ile taşımacılık ise ulaşım türleri arasında en tercih edilen ve güvenilir olan sistemdir. Enerji tasarrufu sağlaması, fiyat kalite performansı, güvenilirliği ve hava koşullarından bağımsız oluşu gibi birçok avantajı bulunuyor olmasına rağmen, özellikle şehir merkezi içinde kalan alanlarda yakın çevresinde konfor azaltıcı titreşime sebebiyet vermektedir. Bu çalışmada tren hattı üzerindeki tren geçişlerinden kaynaklı titreşimlerin zemine olan etkisi araştırılmıştır. Bir takım saha çalışmaları yürütülmüş, titreşimlerin etkisini azaltmak için kazılmış bir hendek bariyer kullanılmıştır. Yük, yolcu treni ve yüksek hızlı trenden kaynaklanan titreşim düzeyleri demiryolu hattından 40 m uzaklığa kadar, titreşim hareketinin üç farklı bileşeni için ölçülmüştür. Seçilen çalışma sahasındaki ortalama V_{s30} değeri 200 m/s olarak ölçülmüştür. Elde edilen kayıtlar hem zaman tanım alanında hem de frekans tanım alanında incelenmiştir. Sonuçlar serbest zemin ve hendekli zemin için ayrı ayrı hesaplanarak birbirleri ile karşılaştırılmıştır. Titreşimlerin yapılar üzerindeki etkisini incelemek için 3 farklı bina modellemesi yapılarak sahadan alınan tren kayıtları ile bu yapılar üzerinde lineer time history analizi yapılmıştır. Yapının kat bazında ivme, hız ve deplasman değerlerine bakılmış ve uluslararası alman standardıyla karşılaştırılmıştır.

Anahtar Kelimeler: Tren titreşimleri, Trafik kaynaklı titreşimler, Titreşim ölçümleri, Dalga yayılımı

ACKNOWLEDGEMENTS

I would like to thank to my supervisor, Asst. Prof. Dr. Abdullah Can Zülfikar for his guidance in this process.

I am also grateful to my friends Burcu Ayar and Ali Yeşilyurt, who supported me in overcoming obstacles I have been facing through my research and helped me achieve better results.

None the less, this thesis prepared under the project no:217M427 titled “Environmental Vibrations Induced by High-speed Trains and Mitigation of Structural Vibrations” funded by TUBITAK (The Scientific and Technological Research Council of Turkey). The support of TUBITAK is greatly appreciated.

And I would like to thank my family, my parents, and my sister, for supporting me throughout writing this thesis and in my life general.

TABLE of CONTENTS

	<u>Page</u>
SUMMARY	v
ÖZET	vi
ACKNOWLEDGEMENTS	vii
TABLE of CONTENTS	viii
LIST of ABBREVIATIONS and ACRONYMS	x
LIST of FIGURES	xi
LIST of TABLES	xiv
1. INTRODUCTION	1
1.1. Scope	2
1.2. Recent Studies	2
2. METHODOLOGY	5
2.1. Examination of Train Vibrations	6
2.2. Fieldworks and Evaluation	7
2.2.1. Testing Instrumentation Plans of Test Sites	7
2.2.2. Repeated Train Passes	10
2.3. Analysis Parameters of Train Records	11
2.4. Analysis of the Free-Field Motion	11
2.4.1. First Fieldwork Evaluation	11
2.4.2. Second Fieldwork Evaluation	15
2.4.3. Third Fieldwork Evaluation	20
2.5. Barrier Field Motion	25
2.5.1. First Fieldwork Evaluation	25
2.5.2. Second Fieldwork Evaluation	29
2.5.3. Third Fieldwork Evaluation	34
3. STRUCTURAL ANALYSIS ON BUILDING MODELS	39
3.1. Time History Analysis and Evaluation	41
3.1.1. Peak Acceleration Distributions of Building Models	44
3.1.2. Peak Velocity Distribution of Building Models	46
3.1.3. Peak Displacement Distribution of Building Models	48

3.2. Structural Response and Human Perception in The International Regulations	50
4. RESULTS AND DISCUSSION	55
4.1. PGA Graphs of Train Vibrations	55
4.2. Frame Model Structural Analysis Results	59
5. CONCLUSION	64
REFERENCES	66
BIOGRAPHY	68

LIST of ABBREVIATIONS and ACRONYMS

<u>Abbreviations</u>	<u>Explanations</u>
<u>and Acronyms</u>	
BFM	: Barrier field motion
FFM	: Free field motion
FT	: Freight train
F	: Frequency
Hz	: Hertz
HST	: High – speed train
m	: Meter
s	: Second
PT	: Passenger train
PGA	: Peak Ground Acceleration
PGV	: Peak Ground Velocity
PGD	: Peak Ground Displacement
SZ	: Free field sensor data
SZH	: Barrier field sensor data
(e-w)	: Parallel component of train vibration
(n-s)	: Perpendicular component of train vibration
(u-d)	: Vertical component of train vibration

LIST of FIGURES

<u>Figure No:</u>	<u>Page</u>
2.1: Train induced vibrations evaluation steps.	5
2.2: a) Location of railway line, field view (right).	6
2.3: a) Test site view, b) Accelerometer plantation.	7
2.4: Testing Instrumentation Plan of First Fieldwork.	8
2.5: Testing Instrumentation Plan of Second Fieldwork.	8
2.6: Testing Instrumentation Plan of Third Fieldwork.	9
2.7: Testing Instrumentation Plan of Third Fieldwork.	9
2.8: Acceleration-time graph of free – field parallel (e-w) direction.	12
2.9: Acceleration-time graph of free – field perpendicular (n-s) direction.	12
2.10: Velocity-Time graph of free – field parallel (e-w) direction.	13
2.11: Velocity-Time graph of free – field perpendicular (n-s) direction.	13
2.12: Fourier amplitude graph of free – field parallel (e-w) direction.	14
2.13: Fourier amplitude graph of free – field perpendicular (n-s) direction.	14
2.14: Acceleration-Time graph of free – field parallel (e-w) direction.	15
2.15: Acceleration-Time graph of free – field perpendicular (n-s) direction.	16
2.16: Acceleration-Time graph of free – field vertical (u-d) direction.	16
2.17: Velocity-Time graph of free – field parallel (e-w) direction.	17
2.18: Velocity-Time graph of free – field perpendicular (n-s) direction.	17
2.19: Velocity-Time graph of free – field vertical (u-d) direction.	18
2.20: Fourier amplitude graphs of free – field parallel (e-w) direction.	18
2.21: Fourier amplitude graph of free – field perpendicular (n-s) direction.	19
2.22: Fourier amplitude graphs of free – field vertical (u-d) direction.	19
2.23: Acceleration-Time graph of free – field parallel (e-w) direction.	20
2.24: Acceleration-Time graph of free – field perpendicular (n-s) direction.	21
2.25: Acceleration-Time graph of free – field vertical (u-d) direction.	21
2.26: Velocity-Time graph of free – field parallel (e-w) direction.	22
2.27: Velocity-Time graph of free – field perpendicular (n-s) direction.	22
2.28: Velocity-Time graph of free – field vertical (u-d) direction.	23
2.29: Fourier amplitude graph of free – field parallel (e-w) direction.	23
2.30: Fourier amplitude graph of free – field perpendicular (n-s) direction.	24

2.31:	Fourier amplitude graph of free – field vertical (u-d) direction.	24
2.32:	Acceleration-Time graph of barrier – field parallel (e-w) direction.	26
2.33:	Acceleration-Time graph of barrier – field perpendicular (n-s) direction.	26
2.34:	Velocity-Time graphs of barrier – field parallel (e-w) direction.	27
2.35:	Velocity-Time graph of barrier – field perpendicular (n-s) direction.	27
2.36:	Fourier amplitude graph of barrier – field parallel (e-w) direction.	28
2.37:	Fourier amplitude graph of barrier – field perpendicular (n-s) direction.	28
2.38:	Acceleration-Time graph of barrier – field parallel (e-w) direction.	29
2.39:	Acceleration-Time graph of barrier – field perpendicular (n-s) direction.	30
2.40:	Acceleration-Time graph of barrier – field vertical (u-d) direction.	30
2.41:	Velocity-Time graph of barrier – field parallel (e-w) direction.	31
2.42:	Velocity-Time graph of barrier – field perpendicular (n-s) direction.	31
2.43:	Velocity-Time graph of barrier – field vertical (u-d) direction.	32
2.44:	Fourier amplitude graph of barrier – field parallel (e-w) direction.	32
2.45:	Fourier amplitude graphs of barrier – field perpendicular (n-s) direction.	33
2.46:	Fourier amplitude graphs of barrier – field vertical (u-d) direction.	33
2.47:	Acceleration-Time graph of barrier – field parallel (e-w) direction.	34
2.48:	Acceleration-Time graph of barrier – field perpendicular (n-s) direction.	35
2.49:	Acceleration-Time graph of barrier – field vertical (u-d) direction.	35
2.50:	Velocity-Time Graph of barrier field motion parallel (e-w) direction.	36
2.51:	Velocity-Time graph of barrier – field perpendicular (n-s) direction.	36
2.52:	Velocity-Time graph of barrier – field vertical (u-d) direction.	37
2.53:	Fourier amplitude graph of barrier – field parallel (e-w) direction.	37
2.54:	Fourier amplitude graph of barrier – field perpendicular (n-s) direction.	38
2.55:	Fourier amplitude graph of barrier – field vertical (u-d) direction.	38
3.1:	Structural frame models.	40
3.2:	Freight Train acceleration records.	41
3.3:	Passenger Train acceleration records.	42
3.4:	High-Speed Train acceleration records of free field motion.	42
3.5:	High-Speed Train acceleration records of barrier field motion.	42
3.6:	Fourier amplitude spectrum of freight train.	43
3.7:	Fourier amplitude spectrum of passenger train.	43
3.8:	Fourier amplitude spectrum of high-speed train of free field motion.	43

3.9:	Fourier amplitude spectrum of high-speed train of barrier field motion.	43
3.10:	Acceleration distribution on storey levels of model 'A'.	44
3.11:	Acceleration distribution on storey levels of model 'B'.	45
3.12:	Acceleration distribution on storey levels of model 'C'.	45
3.13:	Velocity distribution on storey levels of model 'A'.	46
3.14:	Velocity distribution on storey levels of model 'B'.	47
3.15:	Velocity distribution on storey levels of model 'C'.	47
3.16:	Displacement distribution on storey levels of model 'A'.	48
3.17:	Displacement distribution on storey levels of Model 'B'.	49
3.18:	Displacement distribution on storey levels of Model 'C'.	49
3.19:	Curves for guideline values of short-term vibration at top floor.	51
3.20:	3 – storey frame model ‘A’ peak lateral top floor velocities.	51
3.21:	6 – storey frame model ‘B’ peak lateral top floor velocities.	52
3.22:	9 – storey frame model ‘C’ peak lateral top floor velocities.	53
4.1:	Comparison of the Pga values in parallel (e-w) direction 1st track line.	55
4.2:	Comparison of the Pga values in perpendicular (n-s) direction 1st track line.	56
4.3:	Comparison of the Pga values for the first fieldwork in parallel (e-w) direction 2nd track line.	57
4.4:	Comparison of the Pga values for the first fieldwork in perpendicular (n-s) direction 2nd track line.	57
4.5:	Comparison of free - field motion with pavement stone motion.	58
4.6:	Acceleration distribution of model ‘C’.	59
4.7:	Velocity distribution of model ‘C’.	60
4.8:	Displacement distribution of model ‘C’.	61
4.9:	Comparison of peak lateral velocities with guideline values.	62
4.10:	Comparison of peak velocities for human perception.	63

LIST of TABLES

<u>Table No:</u>	<u>Page</u>
2.1: Repeated train passes for the first fieldwork.	10
2.2: Repeated train passes for the second fieldwork.	10
2.3: Repeated train passes for the third fieldwork.	10
3.1: Material properties of the frame models.	39
3.2: Mode values of structural frame models.	40
3.3: Selected train records for the analytical approach.	41
3.4: Guideline values for vibration velocity.	50
3.5: Peak lateral velocity values of structural models.	53
3.6: Vibration and human perception of motion.	54

1. INTRODUCTION

With urbanization increasingly widespread, the need for public transport and transportation are becoming primary subjects. Along with the enhancement of technology, the importance of transfer by train augmented lately. In both urban and intercity transportation; railway transportation capacity which is high-speed trains, and subway systems, increasing day by day. The countries in Europe such as Italy, Spain, Germany, Portugal, etc. provide public transportation by high-speed trains [1]. It is also possible to travel between countries by high-speed train in Europe. Along with that high-speed railway transport in Turkey become active between Eskişehir and Ankara cities for the first time in 2009 [2]. Following that Ankara-Konya railway line became operational in 2011. Eventually, an almost 530 km long high-speed railway line between Istanbul and Eskişehir, has assumed a large share in transportation by connecting many city centers since 2014. [2]. Additionally, there are also ongoing high-speed railway projects which are Ankara- Sivas, and Ankara- İzmir railway line projects [2]. As an example of transportation by train in the urban center, the 76.3 km long “Marmaray” train line provides transportation between the far east and the far west end of Istanbul with an average of 100 trips per day [2]. With the increase of this transportation capacity, the railway line passes through the city centers, and even with the formation of new settlements in the areas reached by railway, there is a bilateral growth. This caused the operation of high-speed trains adjacent to the old settlements, after the railway renewal and maintenance of the lines where the old railroad is located.

Recently, high-speed trains were planned with a maximum of 250 km/h speed to be reached in Turkey [2]. Railway platform vibrates under high-frequency dynamic loads occurred by railway cars, and transfers these vibrations beneath the soil layers and accordingly affects surrounding structures indirectly. Along with this, ground motions not only can cause damage to buildings but also affect human comfort by causing discomfort vibrations. Hence, to investigate the damage level of train transitions through soil-structure interaction and wave propagation under a variety of soil types, have importance in terms of protection of nearby buildings and as a preliminary step to reduce vibration effects [3]. However, since high-speed trains are implemented in our country recently, researches are generally focused on reducing platform vibration and noise pollution [3]. The dynamic behavior of the ground, the

interaction of the structure-ground, and the variation of the wave propagations are not well studied. With this study, it is aimed to eliminate the deficiency in the researches and to be the first step for the next stages.

1.1. Scope

Railway car movement induces vibrations on a track and these vibrations convey through track structure and indirectly affect the soil beneath them dynamically. Most of the vibrations that occurred by train passes are transferred by Rayleigh Surface waves. Dynamic stresses and deflections will be amplified with Resonance when the critical velocity equal to the velocity of Rayleigh waves [4]. An engineering approach suggested solving this problem, an isolation structure perpendicular to the vibration waves such as trench barriers that have an optimum depth for specified to its soil conditions where located. Therefore, the impact of wave propagation of vibrations on structures could be reduced.

This study aims to evaluate the effect of various train vibrations from high-speed, freight and passenger trains on their surroundings and ways of vibration mitigations through the field tests and via analysis. Accordingly, some field tests conducted on a predetermined site by taking into consideration local soil conditions. Train vibrations recorded from a close distance to railway tracks. Several accelerometers used to examine different parameters, which are distance, soil types and barrier effect.

To evaluate the response of a building under dynamic vibrations, building types with different heights were modeled and performed numerical analysis on the vibration records taken from the field tests.

The environmental impacts derived from the periodic railway transportation were analyzed in terms of building safety and the limit values compared with the worldwide known standards.

1.2. Recent Studies

The effects of widespread urbanization, associated with the developing technology, triggered high standards in transportation. However, the augment in the traffic and the capacity of railway transportation in the urban center brought along

some problems such as unwanted vibration. M. Bata mentioned about the traffic vibrations as

“These vibrations generated by passing vehicles cause, through the movement of foundations, forced vibrations of the building.”

in their study [5]. In the case of unwanted situations, traffic vibrations can cause damage to sensitive buildings which are monumental, etc. Since ground vibrations and their propagation from their sources are related to complex parameters, the subject was rarely taken into account in the planning process.

However, since it is generally more expensive and difficult to come up with a solution after such vibrations occur, vibration-reducing operations should be included in the planning process. K.R. Massarsch studied on reducing vibrations which are originated from railway traffic [6].

In Italy, having numerous amounts of historical buildings caused demand for protection of cultural heritage. M. Crispino and M. Dapuzzo examined the traffic induced-vibrations in a heritage building in Naples using vehicle type and peak particle velocity as measurement parameters. And compared with values obtained by a modified prediction model [7].

Another research about the protection of a historical building in Rome studied by P. Clemente and D. Rinaldis carried out an experimental method about wave propagation in the soil and presented a vibration reduction method [8].

Within all the developments in transportation, these ground vibrations on structural damage in buildings can no longer be neglected [9]. The impact of vibrations depends on the type of source, the soil configuration, the propagation and the distance from the source. G. Kouroussis et al. pointed out that two problems commonly examined, which are the human perception and the damages on buildings. Moreover, they focused on human exposure inside buildings [9]. Furthermore, ground motions caused by traffic analyzed on typical reinforced concrete structure models analytically and the results compared with the known standards vibration levels in some researches [10], [11].

S. U. Dikmen carried out a study for near-surface attenuation and used both traffic vibrations records and earthquake records at near-surface and engineering bedrock then compared the results with each other [12]. In 2013, E. Çelebi and O.

Kırtel studied on the non-linear 2-D finite element model, taking into account of the effect of local plastic deformation of the soil behavior on the dynamic response of vibrating coupled soil-structure system for mitigation of train induced vibration [13].

Also, to evaluate the ground vibrations, several countries have adopted vibration standards [14]. These standards focus on two main titles which are the evaluation of human comfort and evaluation of potential structural damage. Standards such as ISO 2631-2, AS2670.2, mainly have human comfort limits while German Standard DIN4150-3, Swiss Standard SN640 312-a and British Standard BS 7385-2 have specified on structural damage limit values [15]-[19].

2. METHODOLOGY

The followed method in this study is shown below with the flow chart. Determining the importance of examining train vibrations, planning fieldwork, creating site plans, analysis of vibrations; analysis of time and frequency domain, peak acceleration and velocity values and comparative analysis of ground behaviors were carried out. Furthermore, a 2D frame system modeled and linear time history analyses carried out by injecting the vibrations obtained from the field study on these models. Finally, the results were evaluated by calculating the acceleration, velocity, and displacement response values on a floor basis and plotting the graphics (Figure 2.1).

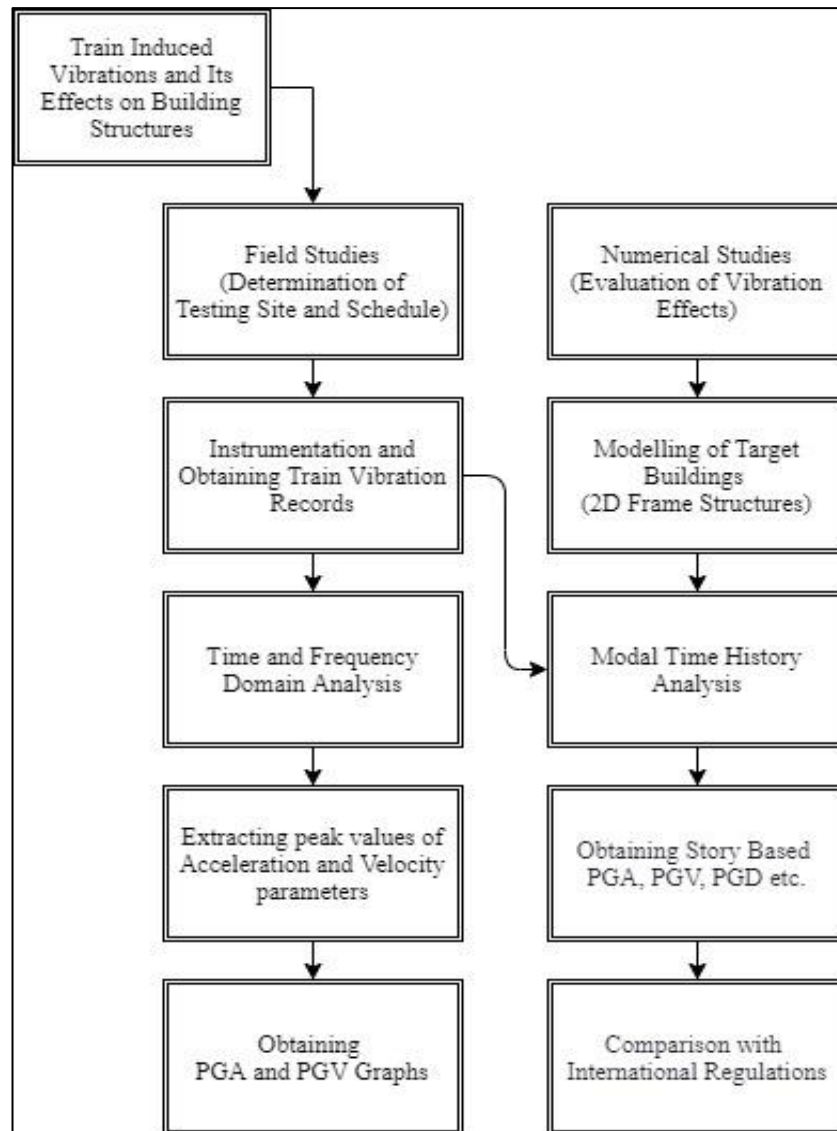


Figure 2.1: Train induced vibrations evaluation steps.

2.1. Examination of Train Vibrations

First of all, a field study was needed to examine and evaluate the wave propagation of vibrations on the ground generated from train transitions. A university hotel building was chosen as the test site to examine the impact of the railway traffic on its surroundings and the nearby structures in the Kırkpınar district in Sakarya Province. This campus selected because the hotel is located very close to the railway and also because of its weak soil condition. By considering the local soil conditions, vibrations measured of the bilateral train passes at the speed of 200-240 km/h in the area where the Istanbul-Ankara high-speed train line is located. The soil condition of the area determined for the measurement was weak and exposed to speeds exceeding 200 km/h made it suitable for the study. During the field-study period, high-speed trains, passenger trains, and freight trains pass observed. The vs30 ground parameter of the region is approximately under 300 m/sec.

The selected high-speed train line part on the area shown on the map marked with yellow (Figure 2.2).

There are four different train lines in the area. Two of them are bilateral high-speed trains tracks, which are the two closest lines to the building. The third track line is for the freight train passes, the passenger (suburban) train passes through the fourth line. Three different field studies conducted at different times to compare the obtained data from the studies and measure its accuracy. The accelerometers seated around one of the guest houses, located 8-10 m from the train line for testing.



Figure 2.2: a) Location of railway line, b) Field view.

2.2. Fieldworks and Evaluation

Three different field studies conducted on the same site at different times. The general view and the location of the test site relative to the railway shown in the figure below (Figure 2.3).

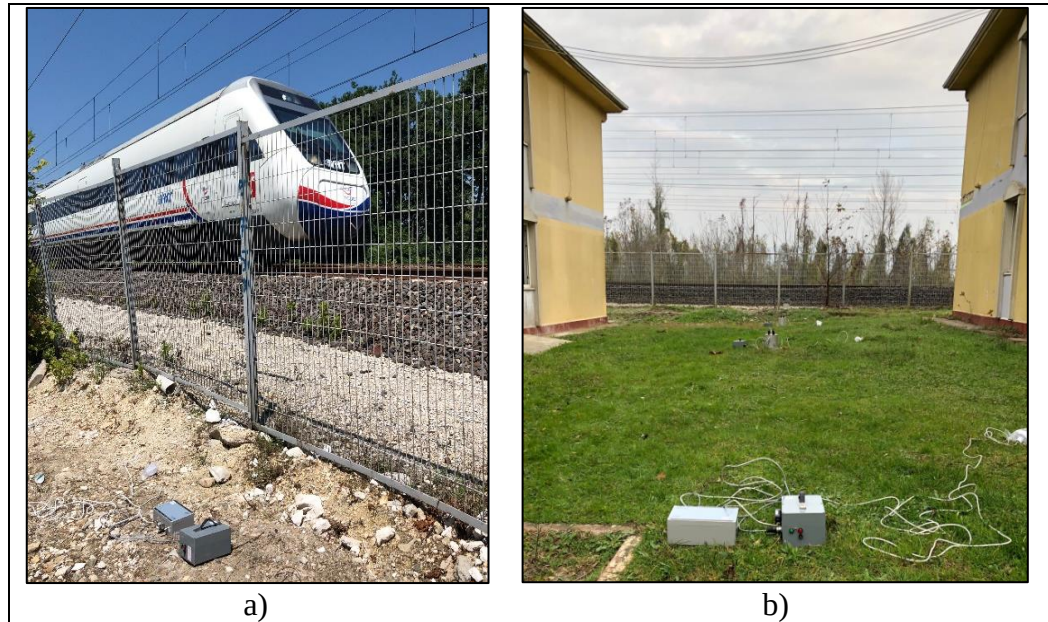


Figure 2.3: a) Test site view, b) Accelerometer plantation.

2.2.1. Testing Instrumentation Plans of Test Sites

The accelerometers arranged at a constant distance of seven meters from the train line. Four of the sensors are located on the left side of the building to observe the free-field motion; another four of them placed behind an excavated barrier in the area, to examine the impact of the trench on vibrations, parallel to the accelerometers taking the free-field motion vibration. The last four accelerometers are located inside the building to evaluate the change depending on the distance and story levels in the building.

In this thesis, the results of eight accelerometers outside the building were evaluated. The testing instrumentation plan for the first and second fieldworks can be seen below (Figure 2.4), (Figure 2.5).

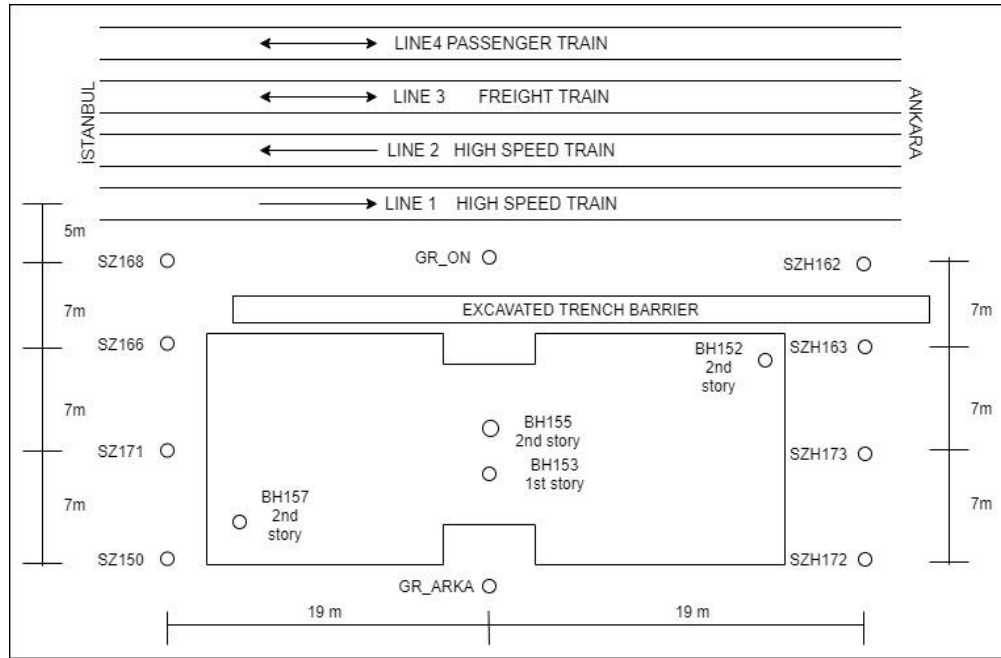


Figure 2.4: Testing instrumentation plan of first fieldwork.

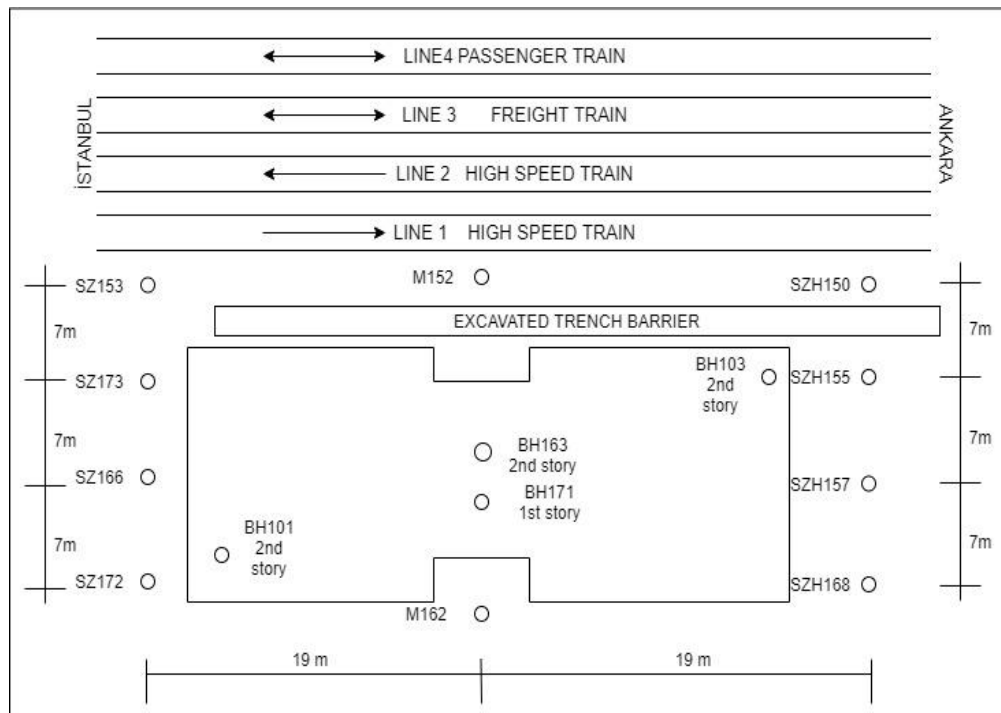


Figure 2.5: Testing instrumentation plan of second fieldwork.

During the third field study, since the barrier on the site was filled and turned into the pavement with cobblestone, the instrumentation plan was reorganized to be separate from the first two tests. Hence there are two types of positioning. The first one is to observe the vibrations change with the distance from the line (Figure 2.6).

Lastly, the second type is the adjacent sequence, parallel to the railway line to observe the horizontal changes (Figure 2.7).

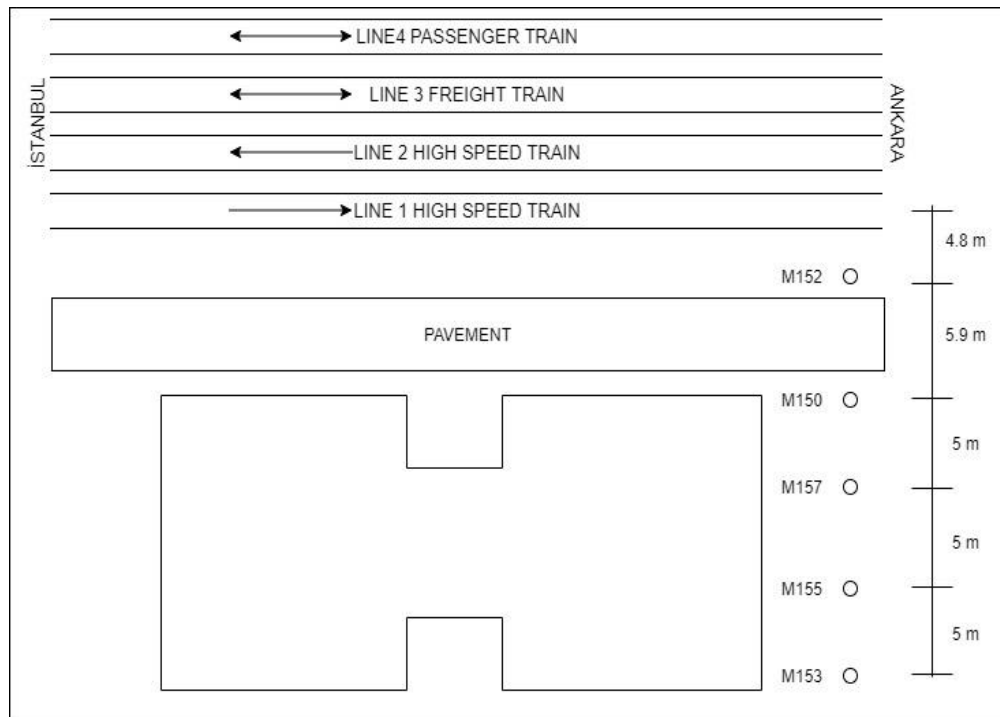


Figure 2.6: Testing Instrumentation Plan of Third Fieldwork.

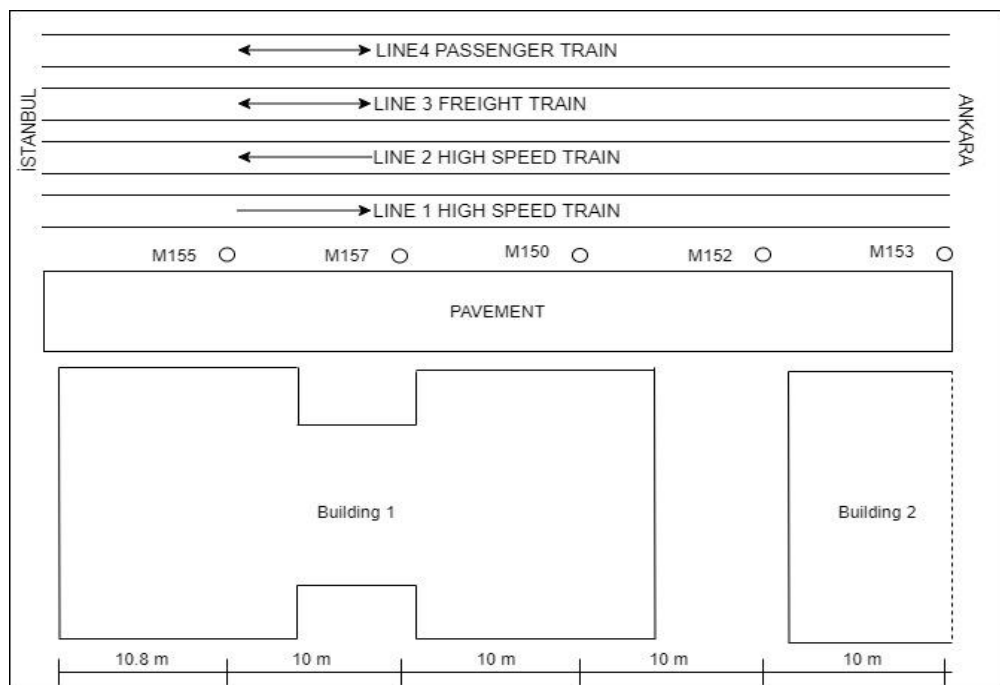


Figure 2.7: Testing Instrumentation Plan of Third Fieldwork.

2.2.2. Repeated Train Passes

During the first fieldwork, eight different high – speed trains, four different freight trains, and three passenger trains, fifteen trains vibrations in total, have recorded throughout the recording time (Table 2.1).

Table 2.1: Repeated train passes for the first fieldwork.

Direction	High-Speed Train	Freight	Passenger	Total
İstanbul	4	3	1	8
Ankara	4	1	2	7
				15

In the second fieldwork, eight different high – speed train records, three different freight trains, and three different passenger trains, along with a total of fourteen train passes, were recorded during the recording period (Table 2.2).

Table 2.2: Repeated train passes for the second fieldwork.

Direction	High-Speed Train	Freight	Passenger	Total
İstanbul	4	3	1	8
Ankara	4	-	2	6
				14

In the third study, two different instrumentation plans created, and at least two train transitions recorded for each recording plan. However, since the sequences planning adapted with high-speed train passages, the accelerometers relocated during the recording time. Therefore, the recording of freight/passenger train was not possible (Table 2.3).

Table 2.3: Repeated train passes for the third fieldwork.

Direction	High-Speed Train	Freight	Passenger	Total
İstanbul	3	-	-	3
Ankara	6	-	-	6
				9

2.3. Analysis Parameters of Train Records

Considering the passing times of the trains are different from each other, the vibration times they create will also vary. The vibrations examined one by one and record times specified for each train type individually for high-speed trains, freight trains, and passenger trains. The time interval set to record 200 data per second so that the frequency content of the vibrations can be examined more precisely ($\Delta t=0.005$). Velocity- time data obtained by integrating from the acceleration-time data recorded from the stations. Meanwhile, the Fourier amplitude spectrum of the records were examined with Baseline Correction, and fourth order Butterworth bandpass filtering in the frequency range between 10 ~ 100 Hz. To examine the different interactions on the ground, the sensors coded as the free-field motion (SZ), barrier-field motion (SZH) according to the determined measurement locations.

2.4. Analysis of the Free-Field Motion

The main component examined in free ground motion (FFM), is to observe the changes in acceleration, velocity and Fourier contents received from each recorder depending on the distance. Based on the first accelerometer set adjacent to the train line, the other three sensors placed in the same direction with an interval of 7 meters. Analyzes performed for parallel direction of train passes (e-w), perpendicular direction of train passes (n-s), and in the vertical direction of train passes (u-d). As the graphs are similar for all transitions, the acceleration and velocity-time graphs with Fourier graphs are shown below only for the first track line, which is closest to the test site and where the peak values are obtained.

2.4.1. First Fieldwork Evaluation

The records were analyzed in the time domain, and a decrease in the acceleration and velocity values noticed from the sensors as they move away from the train line.

The acceleration values for the parallel direction of the track line; while 23.37 gal obtained from the nearest sensor, this value decreased as it moved away from the line and obtained as 2.35 gal (Figure 2.8).

In the perpendicular direction of the track line, the acceleration values obtained from all four sensors were higher. The peak value is 23 gal at the nearest point and 4.43 gal on the last accelerometer (Figure 2.9). There is a distinctive decrease between the first alignment and the other alignments. Since the first sensor located next to the railway lines, the values are therefore greater.

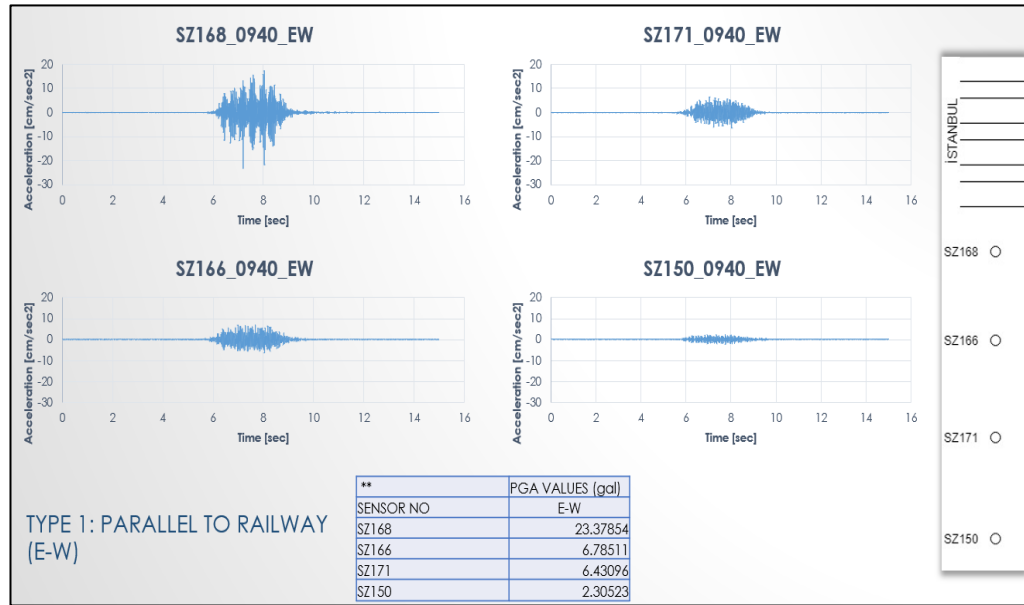


Figure 2.8: Acceleration-time graph of free – field parallel (e-w) direction.

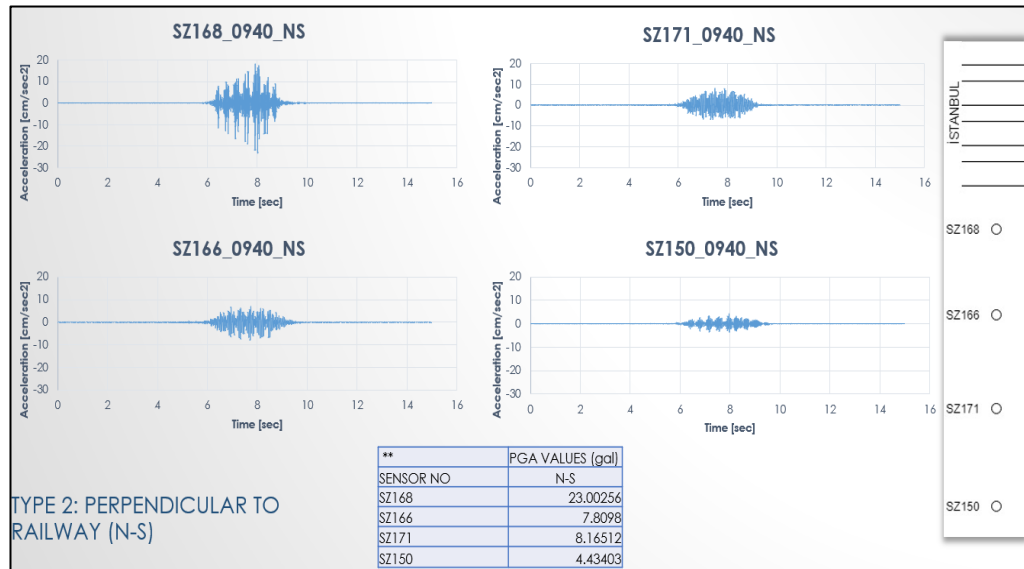


Figure 2.9: Acceleration-time graph of free – field perpendicular (n-s) direction.

There is a decrease in velocity values in the parallel direction of the train line. The velocity value obtained from the first accelerometer is 0.044 cm/s and the velocity

value obtained from the fourth accelerometer is 0.013 cm/s. In the perpendicular direction of the line, reached higher values similar to the acceleration values. While the velocity at the nearest sensor was 0.049 cm/s, the last sensor was 0.017 cm/s (Figure 2.10), (Figure 2.11).

As a result, the perpendicular component of the train vibrations is more effective in the time domain than the parallel component of the train vibrations is observed.

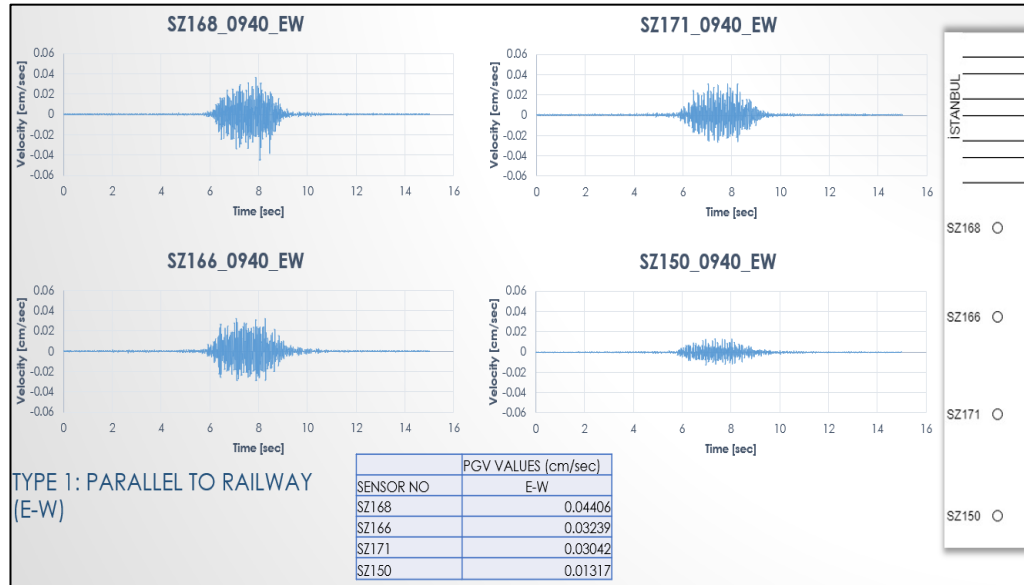


Figure 2.10: Velocity-Time graph of free – field parallel (e-w) direction.

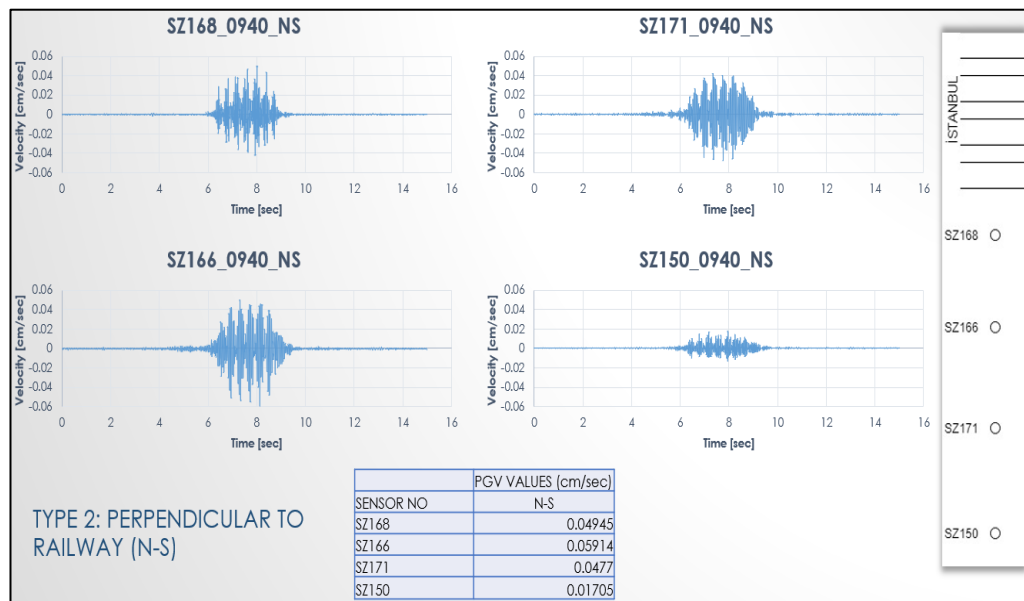


Figure 2.11: Velocity-Time graph of free – field perpendicular (n-s) direction.

For the frequency domain analysis, the dominant Fourier amplitude values in the parallel direction for free ground vibrations found around 80 Hz for the adjacent station, and decreases to the range of 20 ~ 30 Hz along with the distance. There was not any regular decrease with distance for the perpendicular direction. While the dominant amplitude was 60 Hz at the nearest point, it was 20 ~ 30 Hz according to the results obtained from the second and third accelerometers, and 40~50 Hz at the farthest point. Accordingly, the distance cannot be said to have a direct effect on the Fourier amplitude (Figure 2.12), (Figure 2.13).

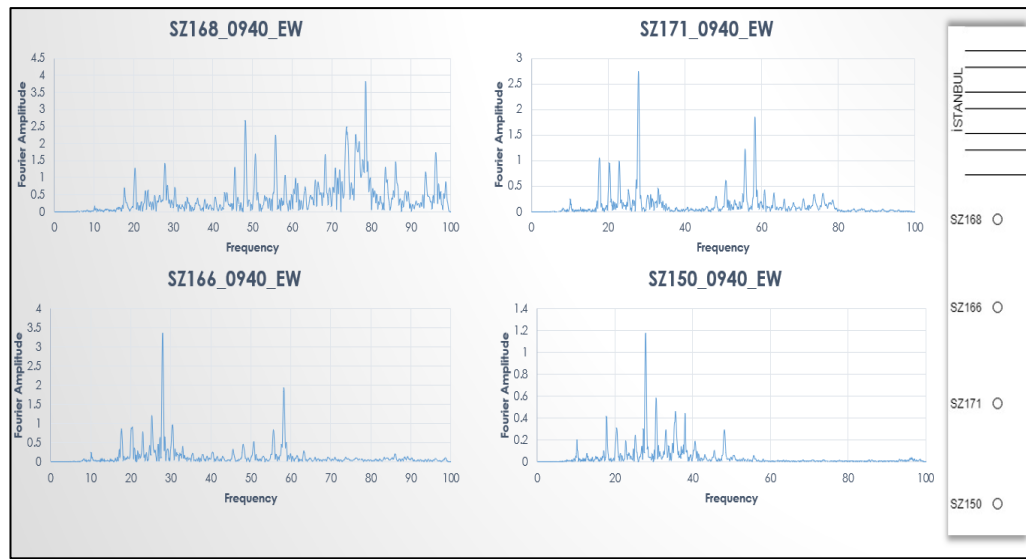


Figure 2.12: Fourier amplitude graph of free – field parallel (e-w) direction.

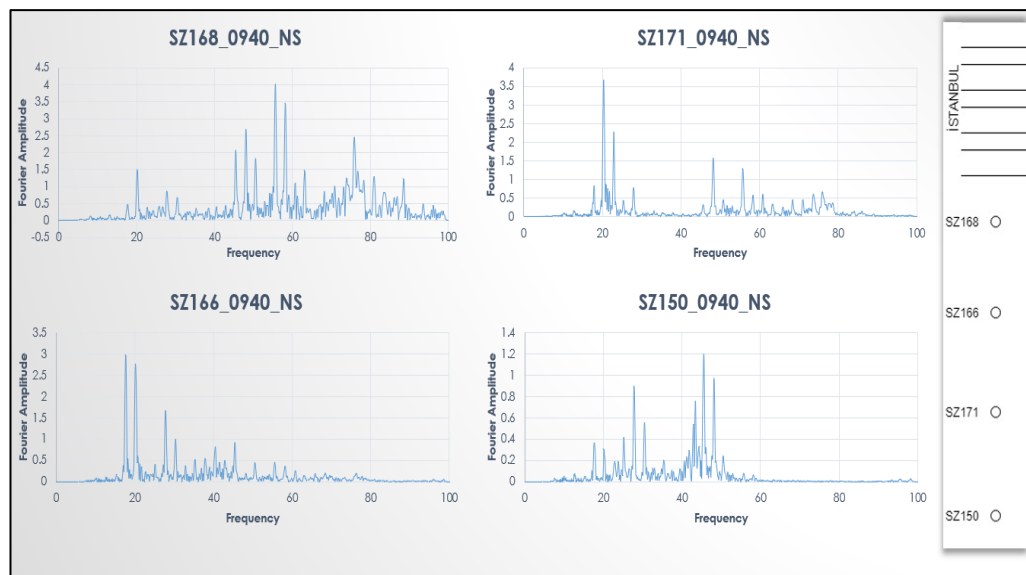


Figure 2.13: Fourier amplitude graph of free – field perpendicular (n-s) direction.

2.4.2. Second Fieldwork Evaluation

The second fieldwork conducted to check the obtained data and also for the completion of the missing parts from the first fieldwork. If the vibrations examined in the time domain, a decrease in the acceleration and velocity values received from the sensors with the distance from the train line.

The accelerations in the parallel direction obtained as 13.45 gal from the nearest sensor and decreasing with the distance, the second is 8.75 gal, the third is 2.84 gal and the last acceleration obtained as 2.35 gal (Figure 2.14).

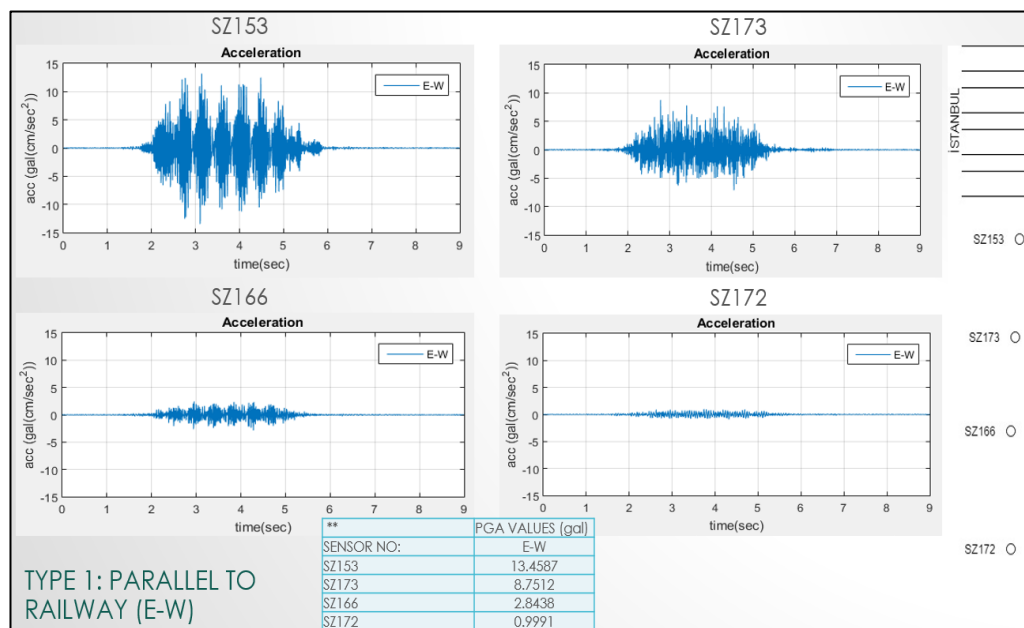


Figure 2.14: Acceleration-Time graph of free – field parallel (e-w) direction.

And also, the nearest point is 9.03 gal, the second value is 9.11 gal, the third acceleration value is 4.79 gal, the last measurement is 1.76 gal for the perpendicular direction (Figure 2.15).

Lastly, peak acceleration is 11.75 gal, the second value is 11.98 gal, the third acceleration value is 3.64 gal, and the minimum value is 2.08 gal for the vertical direction (Figure 2.16).

As a result, there is a noticeable decrease for acceleration values in three directions, but also a small increase occurred in the second alignment for perpendicular and vertical direction. This change may be caused due to soil conditions or due to the sensor were not fixed properly on the ground.

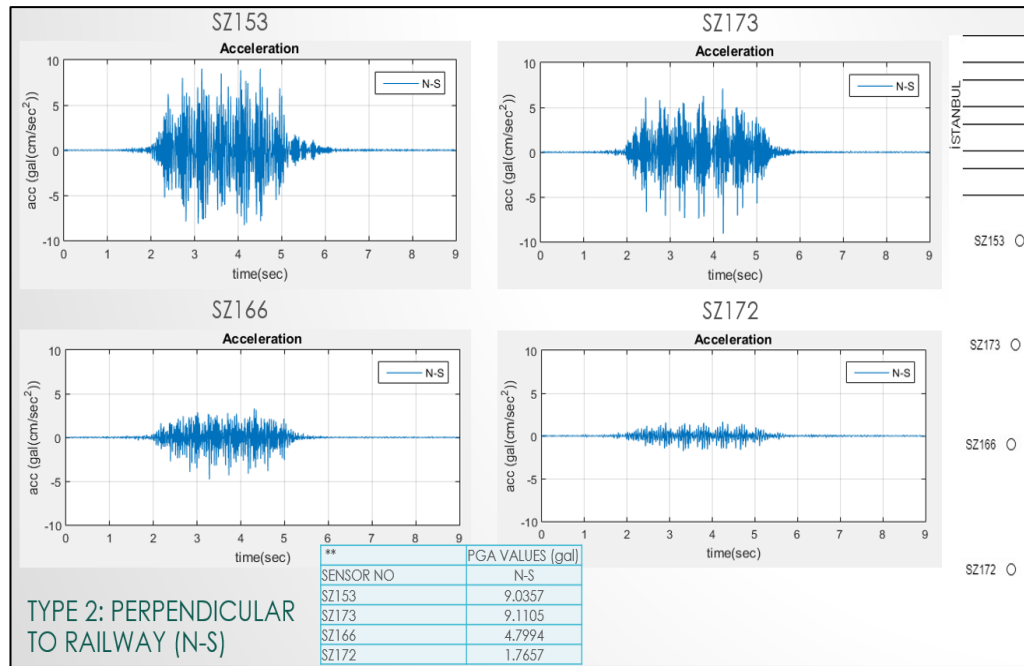


Figure 2.15: Acceleration-Time graph of free – field perpendicular (n-s) direction.

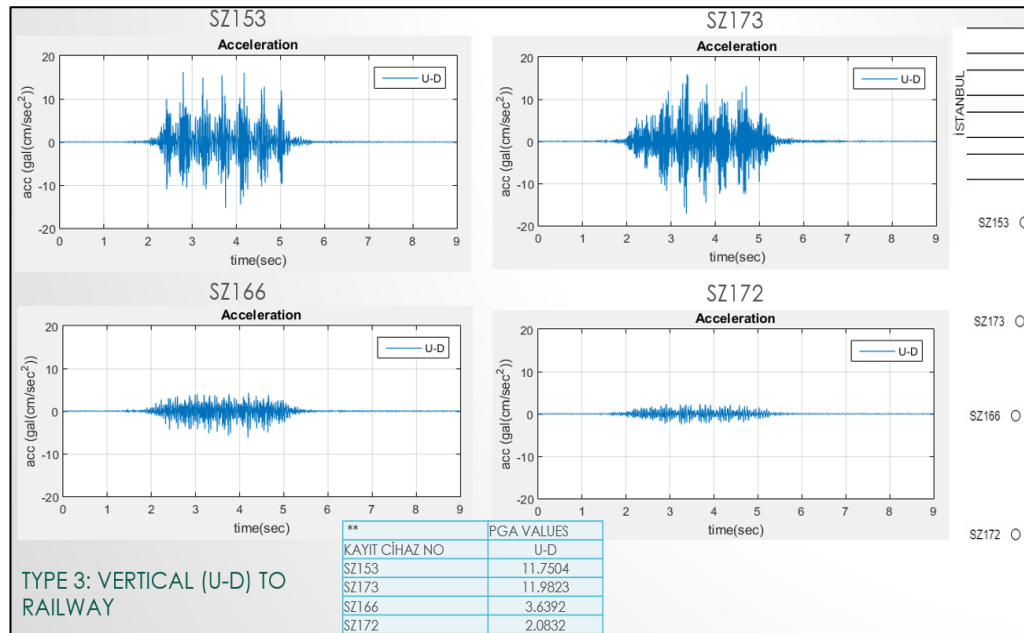


Figure 2.16: Acceleration-Time graph of free – field vertical (u-d) direction.

Another parameter is the velocity changes between the stations. There is a decrease in velocity values for all three directions. The obtained parallel direction values from the first sensor are 0.065 cm/s, 0.0334 cm/s from the second sensor, 0.0115 cm/s from the third sensor and 0.007 cm/s in the fourth sensor (Figure 2.17).

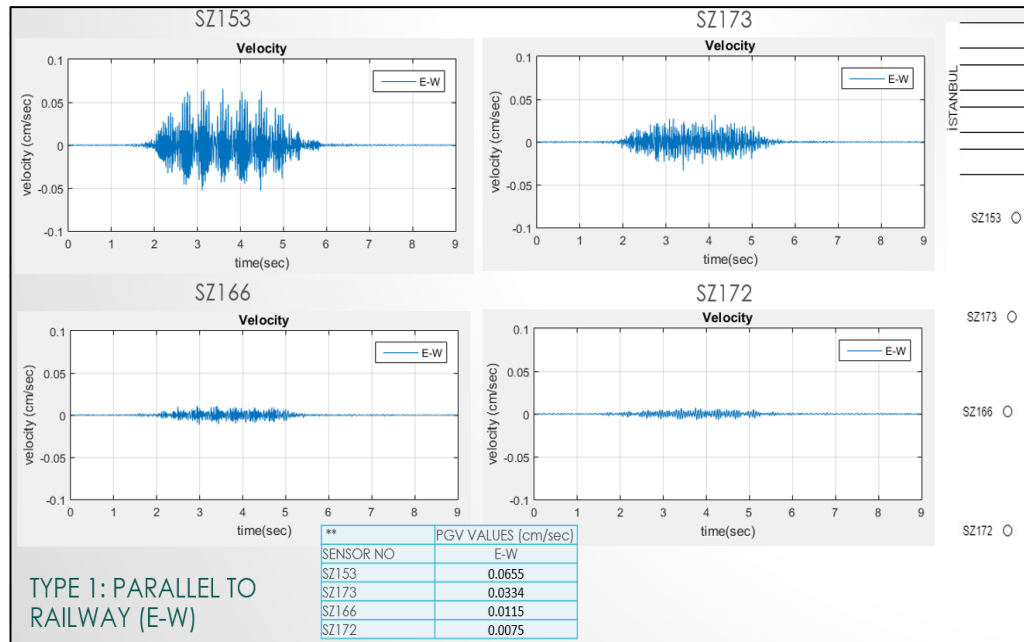


Figure 2.17: Velocity-Time graph of free – field parallel (e-w) direction.

Furthermore, the velocities in the perpendicular direction 0.051 cm/s for the first sensor, 0.037 cm/s from the second sensor, 0.021 cm/s from the third and, 0.012 cm/s for the last sensor (figure 2.18). Also, for the vertical direction, the first accelerometer resulted in 0.075 cm/s, similarly decreasing to 0.06 cm/s for second alignment, to 0.04 for third alignment, and finally to 0.017 cm/s for fourth alignment (Figure 2.19).

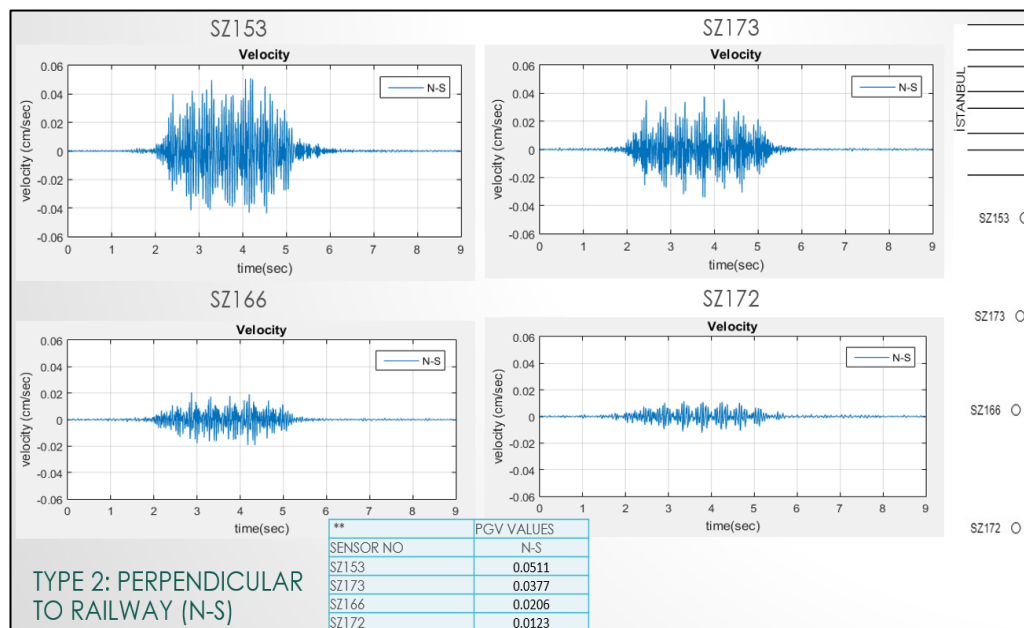


Figure 2.18: Velocity-Time graph of free – field perpendicular (n-s) direction.

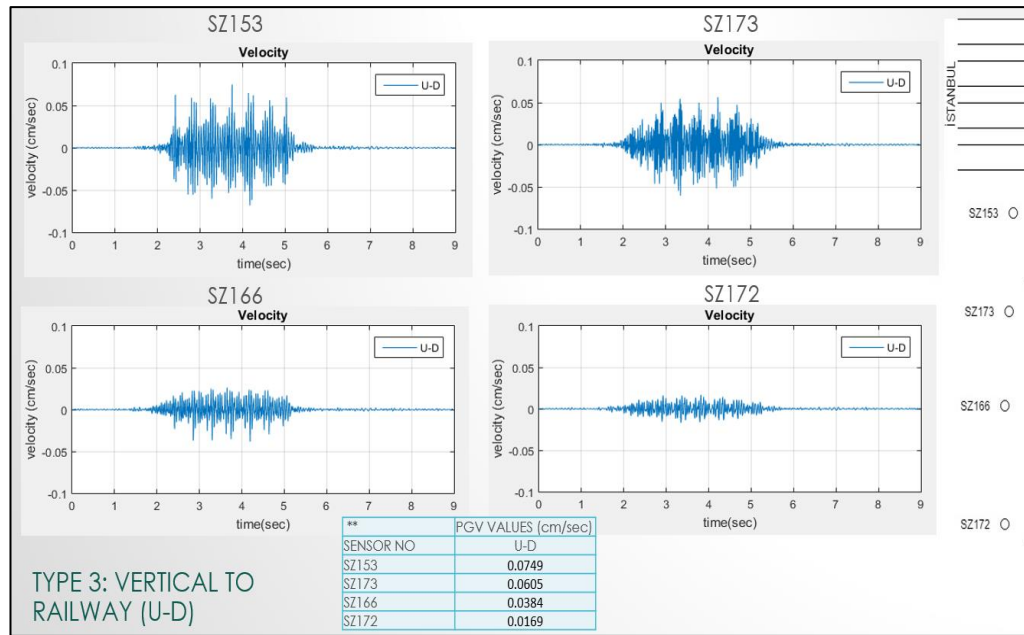


Figure 2.19: Velocity-Time graph of free – field vertical (u-d) direction.

The vertical component has the highest value of all the others from the station adjacent to the railway track line for the velocity parameters. In general, the values show similarity by distance from the railroad for all directions.

The evaluations under the frequency domain are found as the dominant Fourier amplitude values of parallel direction is around 40 ~ 50 Hz in the first three accelerometers, and 20 Hz in the fourth accelerometer (Figure 2.20).

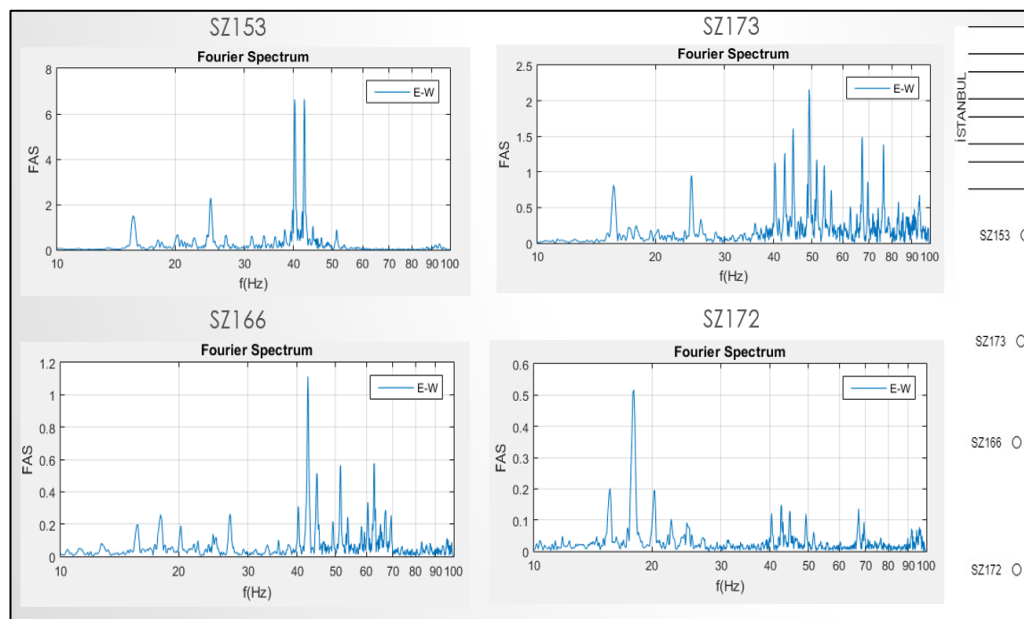


Figure 2.20: Fourier amplitude graphs of free – field parallel (e-w) direction.

Perpendicular direction results are related to parallel ones, found while the dominant amplitude is 40 ~ 50 Hz in the first three stations, it is around 20 ~ 30 Hz at the farthest point (Figure 2.21).

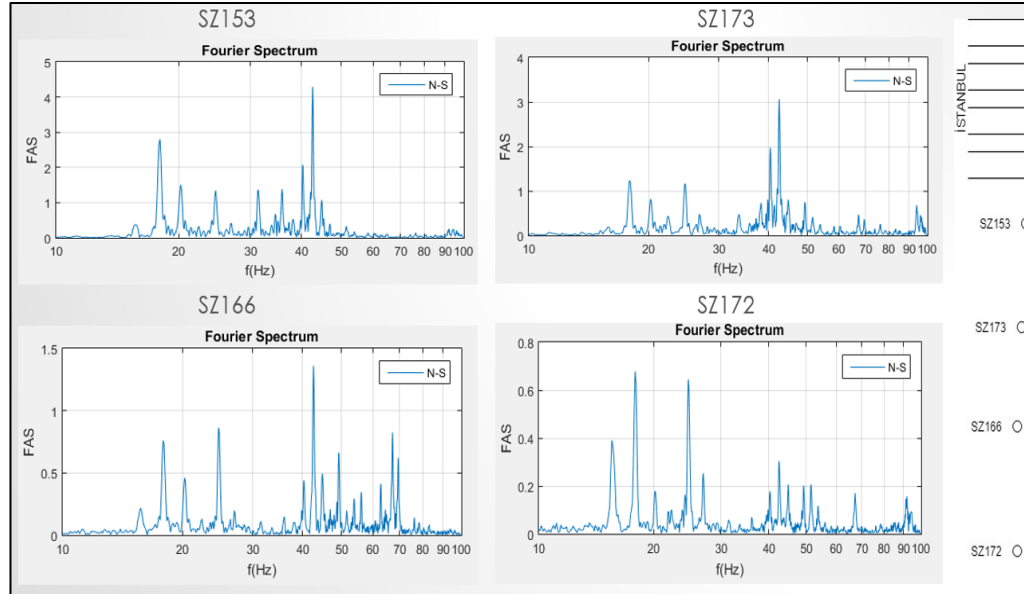


Figure 2.21: Fourier amplitude graph of free – field perpendicular (n-s) direction.

Lastly, in the vertical direction, dominant amplitudes are between 20 ~ 30 Hz or 40 ~ 50 Hz for all accelerometers. Therefore, the distance having a direct effect on the Fourier amplitude cannot be said (Figure 2.22).

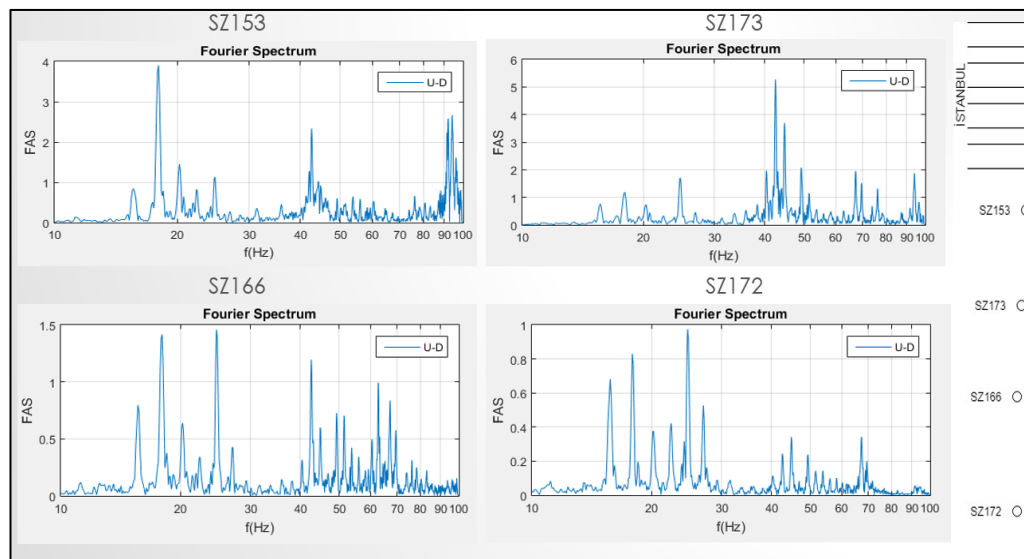


Figure 2.22: Fourier amplitude graphs of free – field vertical (u-d) direction.

2.4.3. Third Fieldwork Evaluation

This study has a different layout than the first two studies due to the changes in the field conditions. Free field behavior examined with five accelerometers placed parallel to the track line. Since the distances to the track line are the same, approximate values expected to come out.

In the time domain analysis, similar results obtained in four accelerometers for the parallel direction to railway as expected. Peak acceleration found as 7.80gal, all four sensors resulted similarly and minimum value found as 5.18 gal. This difference may be due to the interaction of adjacent sensors with each other (Figure 2.23).

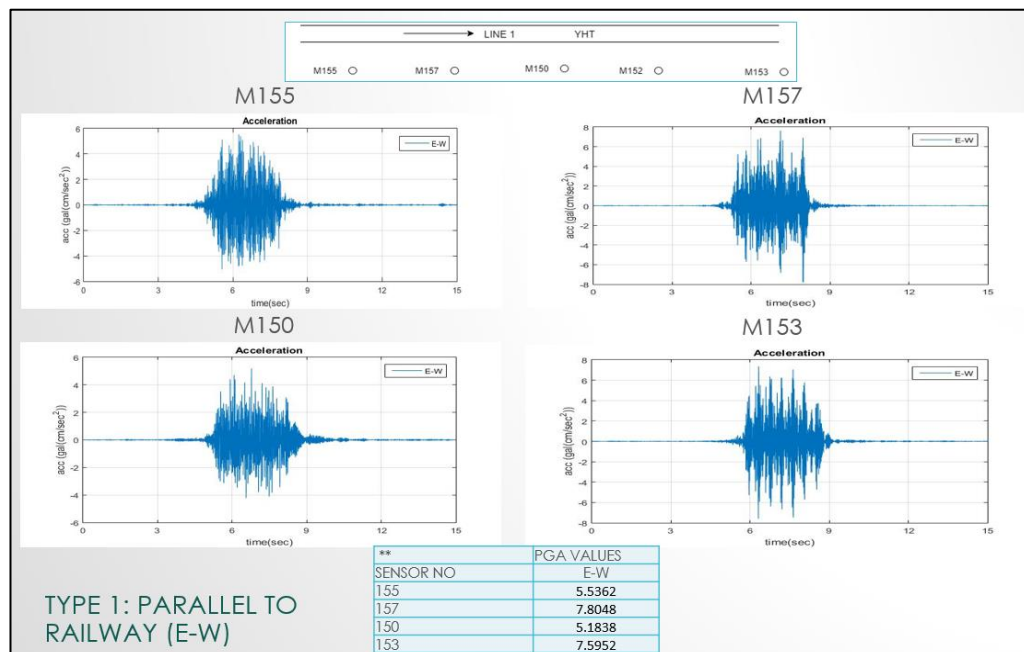


Figure 2.23: Acceleration-Time graph of free – field parallel (e-w) direction.

Moreover, perpendicular component accelerations are higher than the other directions. The peak value is 15.18 gal, two of the devices show similarity, while one sensor had min acceleration value as 6.95 gal (Figure 2.24).

For the vertical component, the results are also close to other horizontal directions. Min acceleration value is 4.49 gal and peak acceleration value is 7.33 gal (Figure 2.25).

Furthermore, the obtained results from the M150 station are the minimum values for all three directions. This can be explained by the interaction of the ground and the accelerometer.

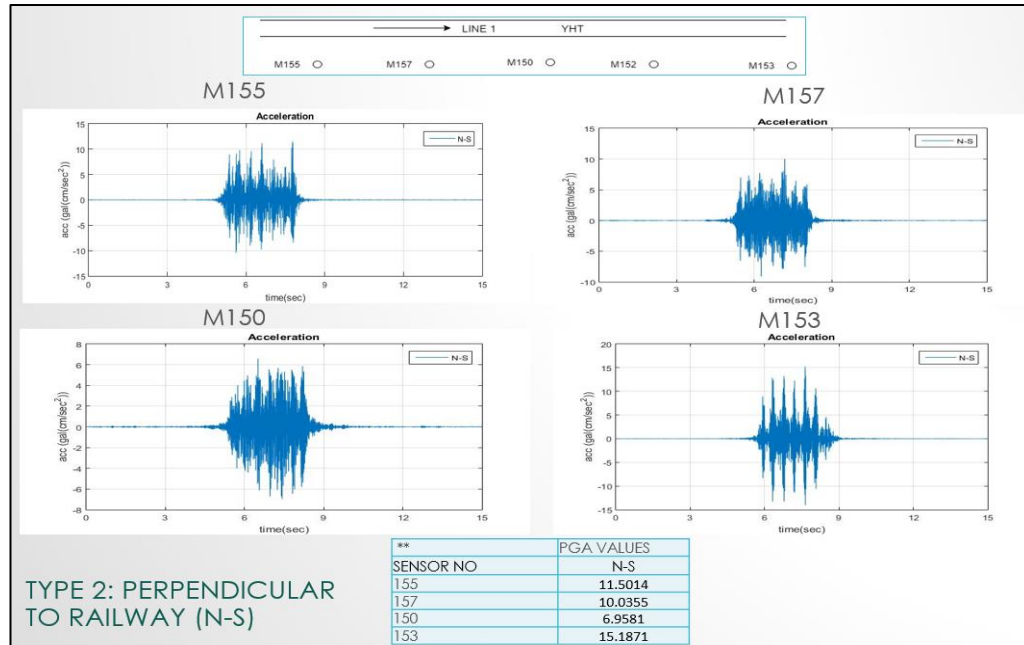


Figure 2.24: Acceleration-Time graph of free – field perpendicular (n-s) direction.

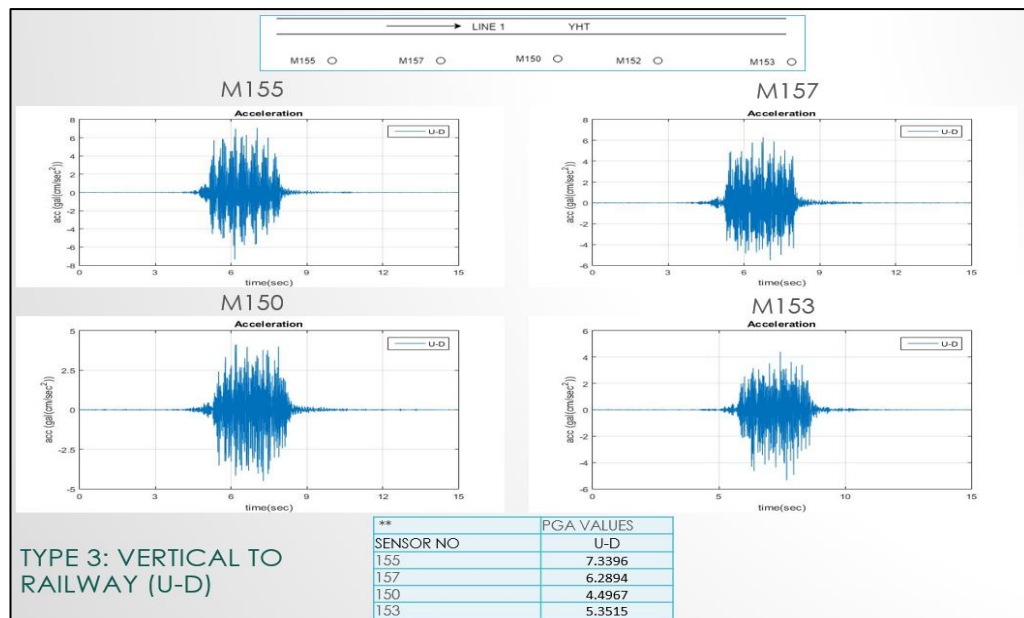


Figure 2.25: Acceleration-Time graph of free – field vertical (u-d) direction.

Considering the instrumentation was parallel with the train route, approximate results expected from all the recorded stations for the velocities. And as expected, all

values were approximate to each other. Peak velocity for direction is 0.037cm/s (Figure 2.26), for perpendicular direction 0.052 cm/s (Figure 2.27), and for vertical direction is 0.049 cm/s (Figure 2.28). For the velocities, perpendicular component received the highest value, while the results were generally close to each other.

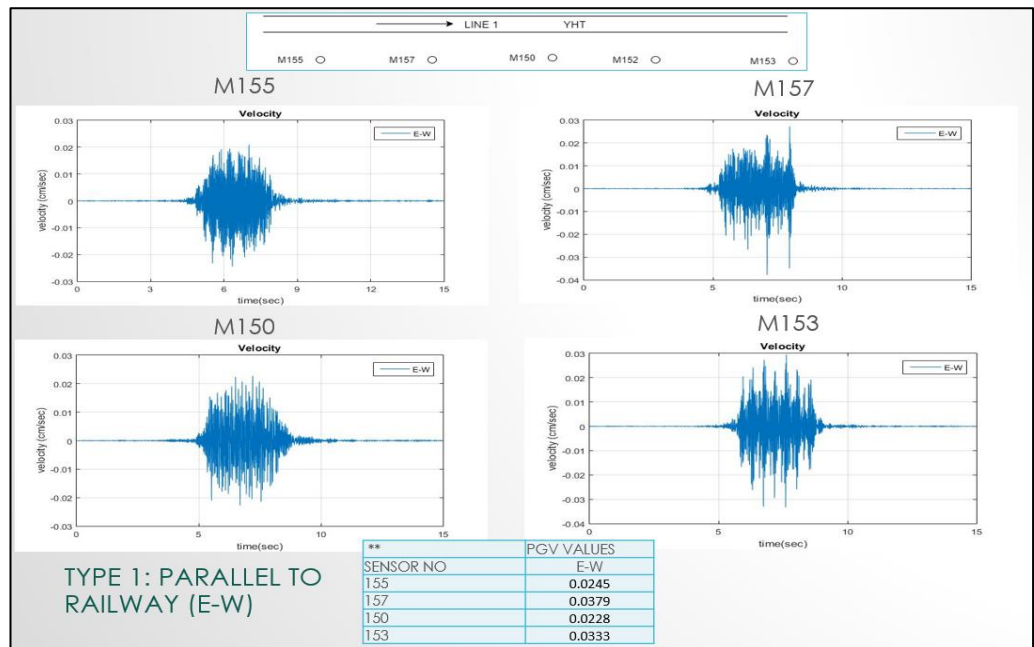


Figure 2.26: Velocity-Time graph of free – field parallel (e-w) direction.

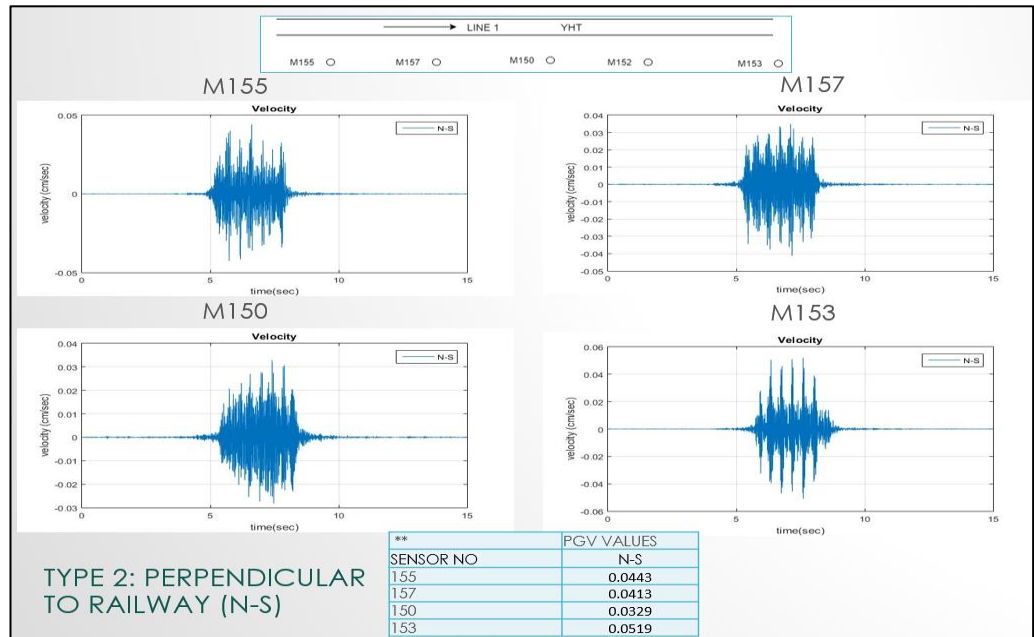


Figure 2.27: Velocity-Time graph of free – field perpendicular (n-s) direction.

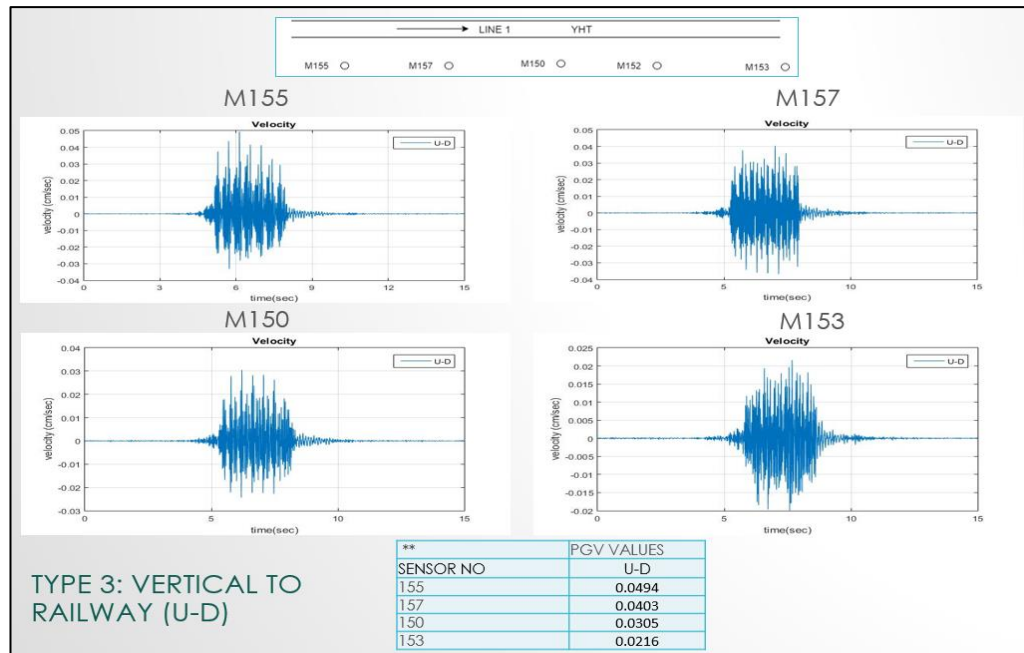


Figure 2.28: Velocity-Time graph of free – field vertical (u-d) direction.

In the frequency domain analysis, the dominant amplitude values for the parallel direction are between 50~60 Hz for three stations. For M150 however, it found between 20~30 Hz and 40~50 Hz (Figure 2.29). The dominant amplitudes are between 60~70 Hz in the same three stations for perpendicular direction M150 also resulted differently as in the range of 40~50 Hz (Figure 2.30).

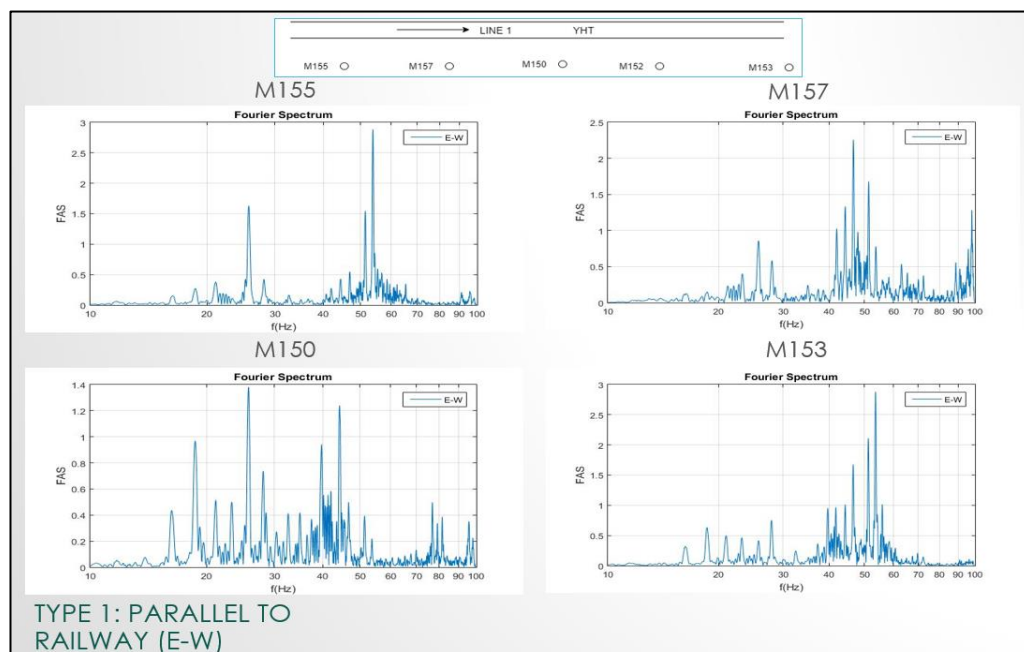


Figure 2.29: Fourier amplitude graph of free – field parallel (e-w) direction.

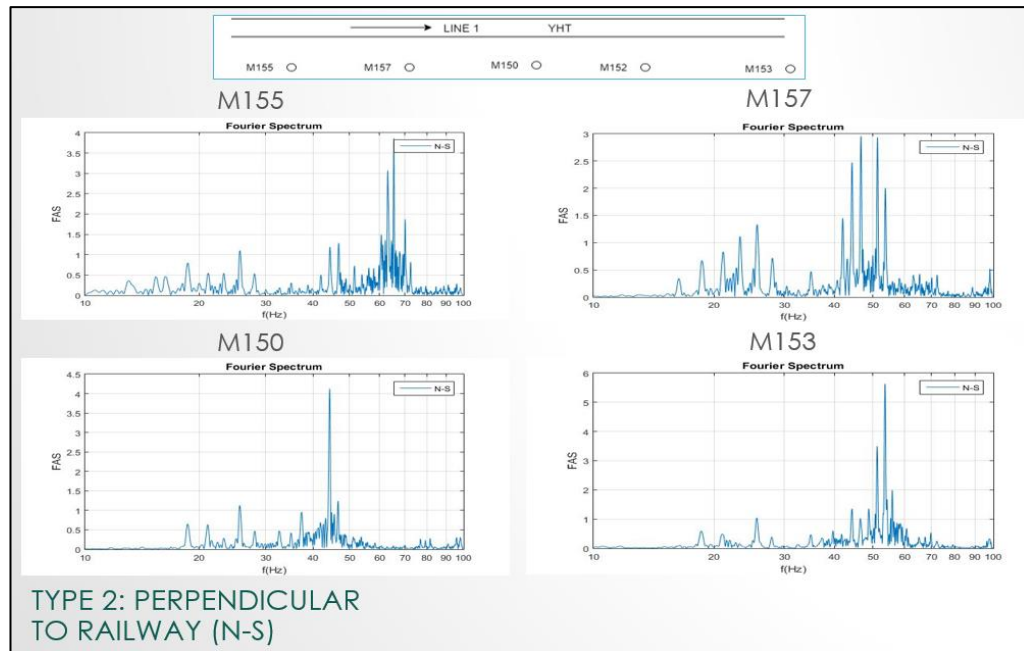


Figure 2.30: Fourier amplitude graph of free – field perpendicular (n-s) direction.

If it is evaluated that the peak acceleration and peak velocity values are different from the other stations in M150 it can be concluded that there is a difference due to the soil interaction since the measurements are consistent in itself. In the vertical direction, the dominant amplitudes are in the range of 20~30 Hz for all stations (Figure 2.31).

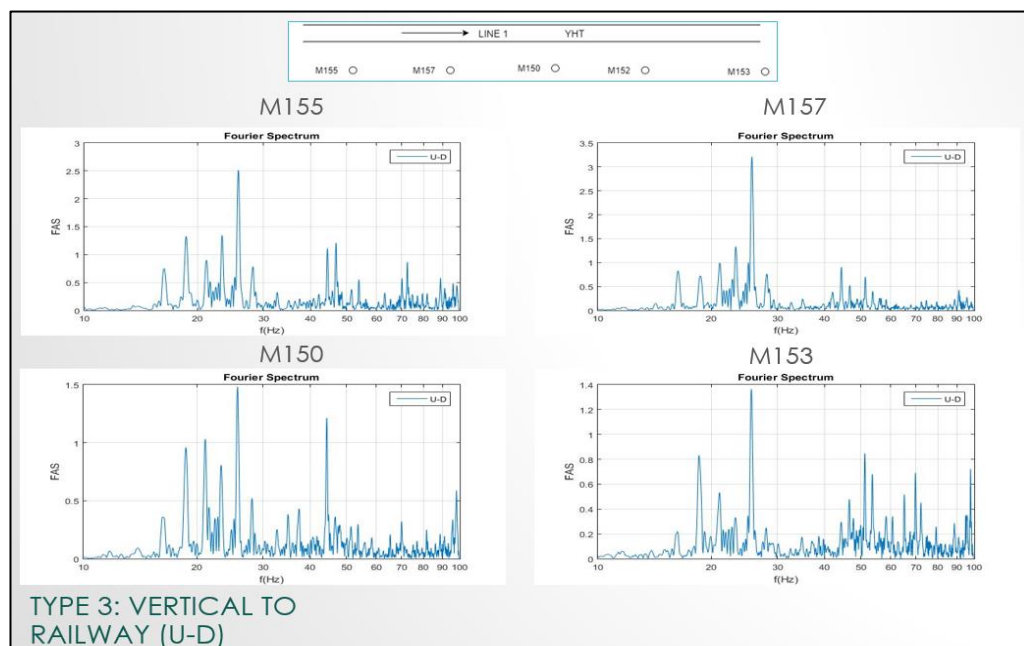


Figure 2.31: Fourier amplitude graph of free – field vertical (u-d) direction.

2.5. Barrier Field Motion

The main component examined in the barrier field free ground motion (BFM), is to observe the changes in acceleration, velocity and Fourier contents received from each recorder depending on the distance. Based on the first accelerometer set adjacent to the train line, the other three sensors placed in the same direction with an interval of 7 meters. The first accelerometer placed in front of the barrier independently from the barrier effect, the others are placed behind the barrier to observe the reduction by the barrier.

Analyzes performed for parallel direction to the train line, perpendicular direction to the train line, and in the vertical direction to the train line. As the graphs are similar for all transitions, the acceleration and velocity-time graphs with Fourier graphs are shown below only for the single-track line.

2.5.1. First Fieldwork Evaluation

Due to the malfunction of the accelerometer positioned nearest to the railroad during the study, it was only able to record from the first train pass to the second railway line. The acceleration and velocity-time charts and Fourier charts are shown below for the Istanbul (2nd railway line) direction.

There is a general decrease in acceleration and velocity values from the sensors with the distance from the railway, except for the amplification in the third sensor. For the parallel direction, while the 4.38 gal obtained from the nearest sensor, this value decreased with distance to 1.148gal. The acceleration value in the third alignment, which is expected to decrease, however, found as 3.20 gal (Figure 2.32).

For the perpendicular component, the values are higher than the parallel horizontal component. Unlike the parallel direction, there was not any amplification in this direction (Figure 2.33). The nearest sensor was found 11.0 gal, the second sensor was found 3.86 gal, and the acceleration values decreased to 2.96 gal at the third sensor, and the last alignment was found 1.56 gal.

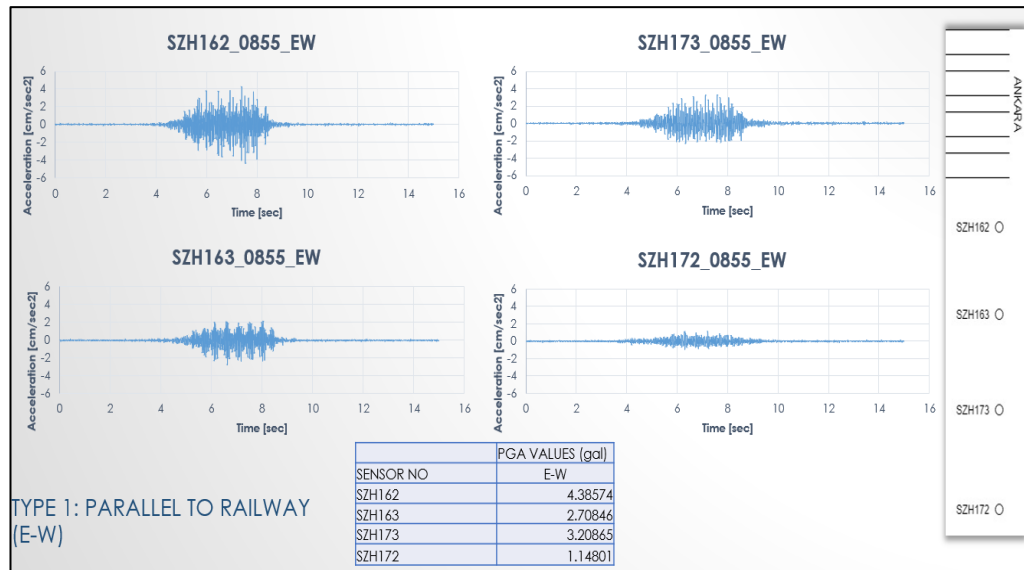


Figure 2.32: Acceleration-Time graph of barrier – field parallel (e-w) direction.

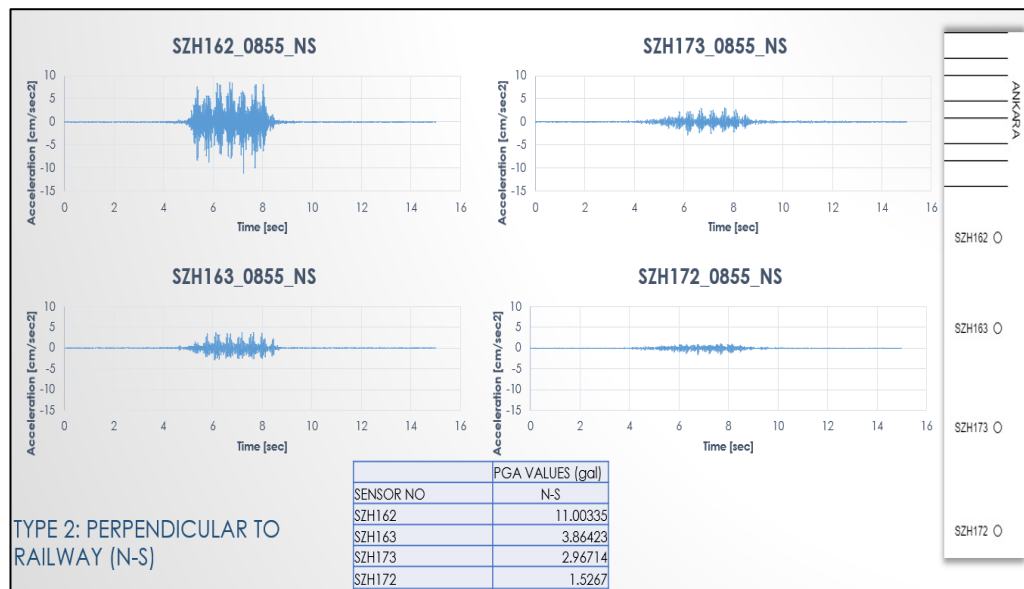


Figure 2.33: Acceleration-Time graph of barrier – field perpendicular (n-s) direction.

There was a decrease in velocity values in all directions. The velocity of the first accelerometer was 0.020 cm/s and of the fourth accelerometer was found 0.008 cm/s. A similar amplification observed in the third alignment for velocity value (Figure 2.34).

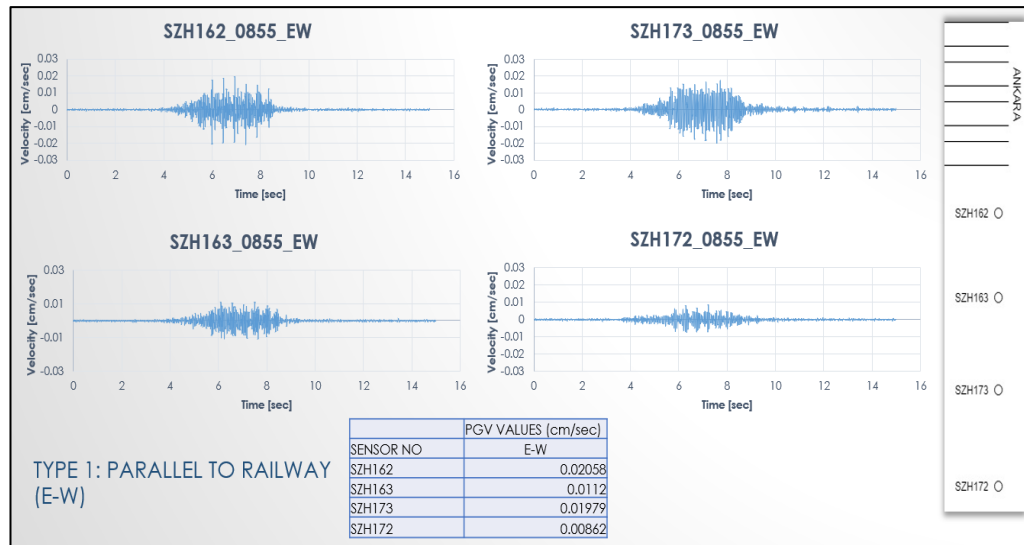


Figure 2.34: Velocity-Time graph of barrier – field parallel (e-w) direction.

For perpendicular direction, another amplification occurred in the second alignment. The velocity values are 0.017 cm/s at the nearest point, and 0.008 cm/s at the end (Figure 2.35).

As a result, the trench affected both acceleration and velocity values, and both parameters were found smaller.

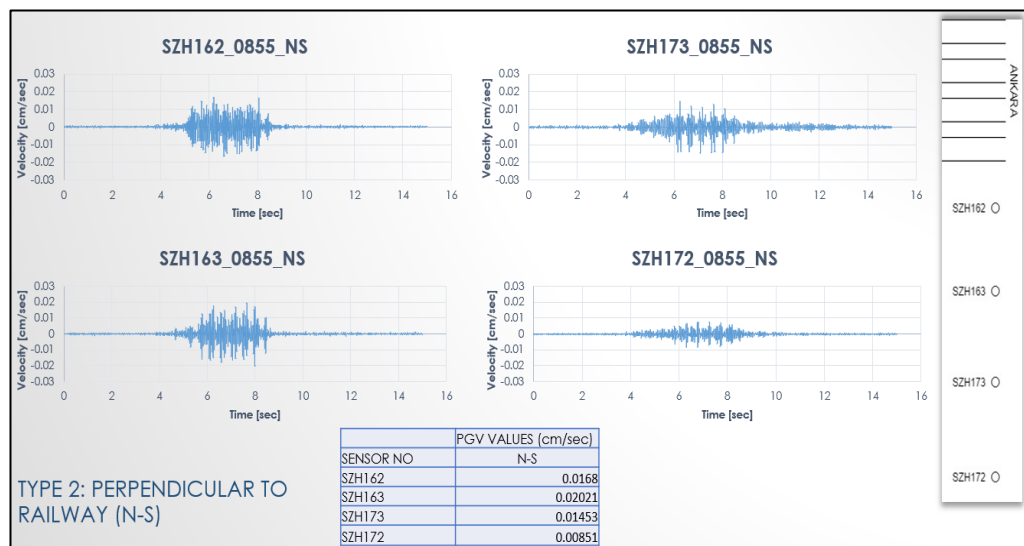


Figure 2.35 Velocity-Time graph of barrier – field perpendicular (n-s) direction.

In frequency domain analysis, Fourier amplitude values in the parallel direction found. The dominant amplitude was around 40 Hz at the nearest point, and it decreased to 20 Hz with the distance from the track line. The values, including the nearest sensor,

were lower because the Fourier amplitudes examined were the records passing through the second railway line (Figure 2.36).

However, this situation changes in the perpendicular direction. The dominant amplitude found as 90 Hz at the nearest point, in the second accelerometer, the dominant amplitude found as 40 Hz, and in the third and fourth accelerometers, it varied between 20~40 Hz. Accordingly, it can be said that the type of vibration that is perpendicular to the frequency content is more effective. However, the direct relation of distance with frequency content could not be observed (Figure 2.37).

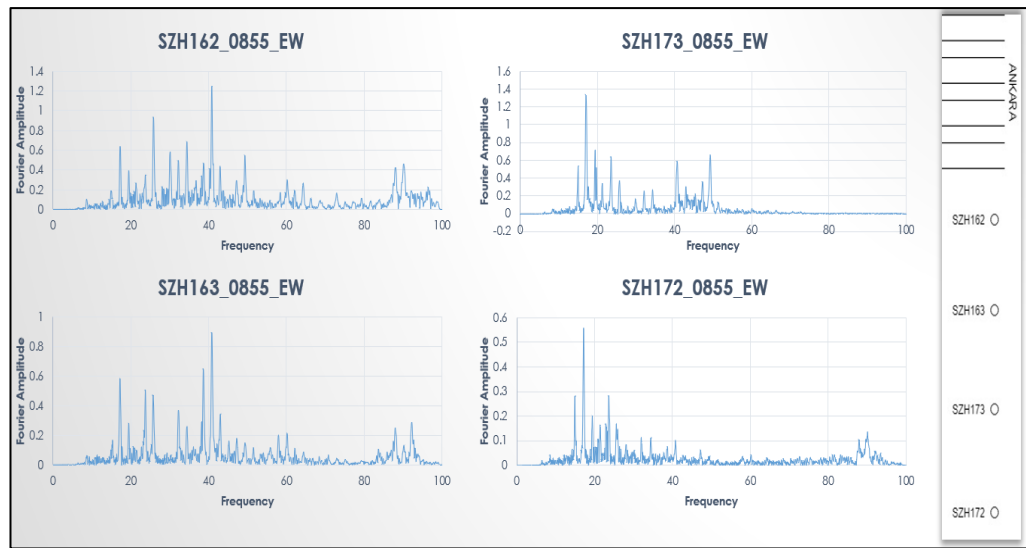


Figure 2.36: Fourier amplitude graph of barrier – field parallel (e-w) direction.

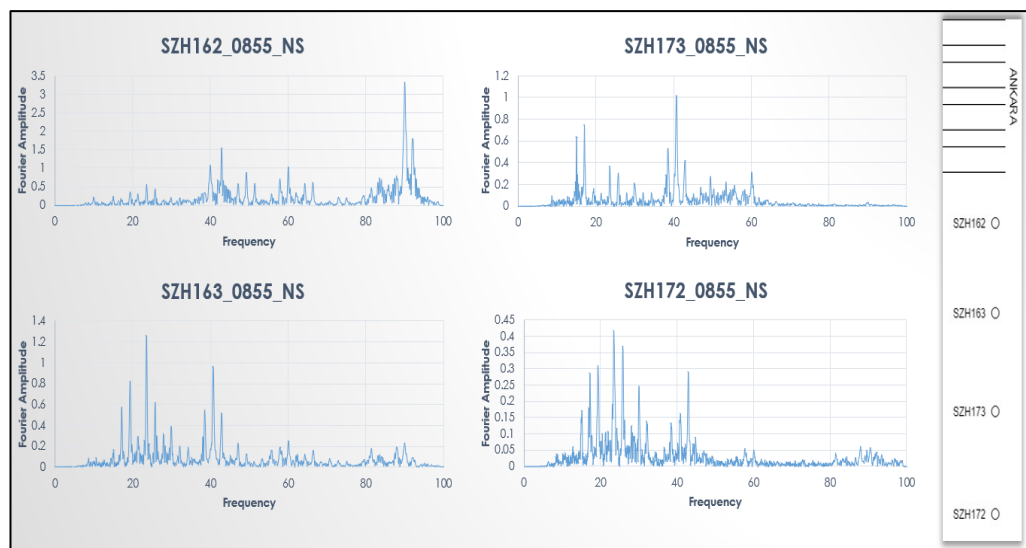


Figure 2.37: Fourier amplitude graph of barrier – field perpendicular (n-s) direction.

2.5.2. Second Fieldwork Evaluation

In this evaluation, it is aimed to compare the data of the third alignment for both two fieldworks to see if there is any amplification occurred on the ground. However, the instrument could not record any data due to the power supply failure.

In the time domain analysis, a 10.25gal acceleration value obtained from the nearest sensor in the parallel direction. This value decreased to 1.59 gal at the fourth alignment. As the first accelerometer is located in front of the barrier, there is a significant difference between the first two accelerometers (Figure 2.38).

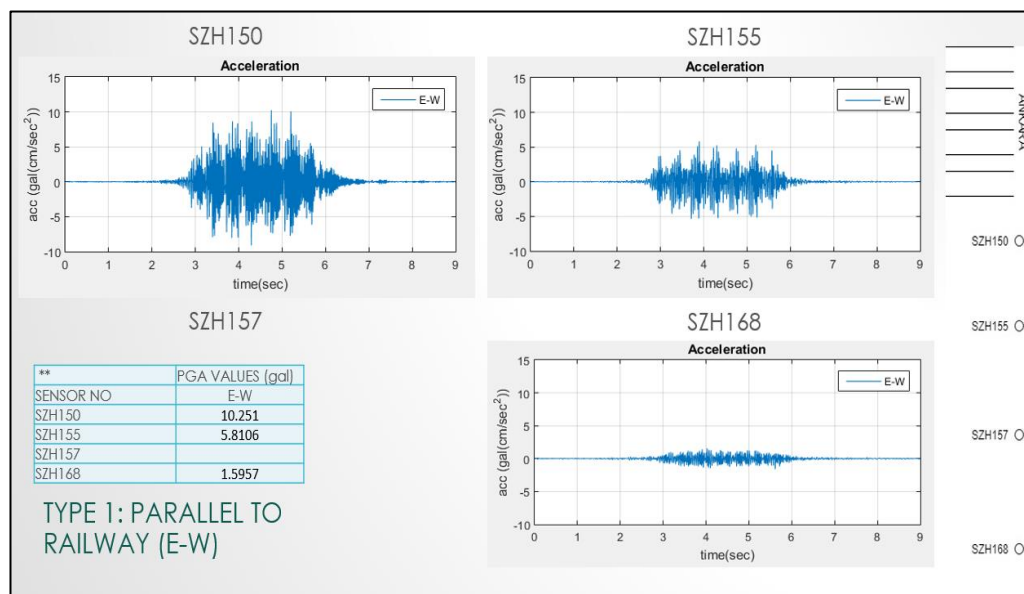


Figure 2.38: Acceleration-Time graph of barrier – field parallel (e-w) direction.

The peak acceleration obtained as 10.73 gal from the first station at perpendicular direction. With the distance from the railway, it decreased and found 2.13 gal in the last alignment (Figure 2.39). In the vertical direction, the peak acceleration value found 8.37 gal, similarly decreasing and calculated 3.79gal in the last alignment (Figure 2.40). The most effective vibration component obtained as perpendicular direction to the train line in this study.

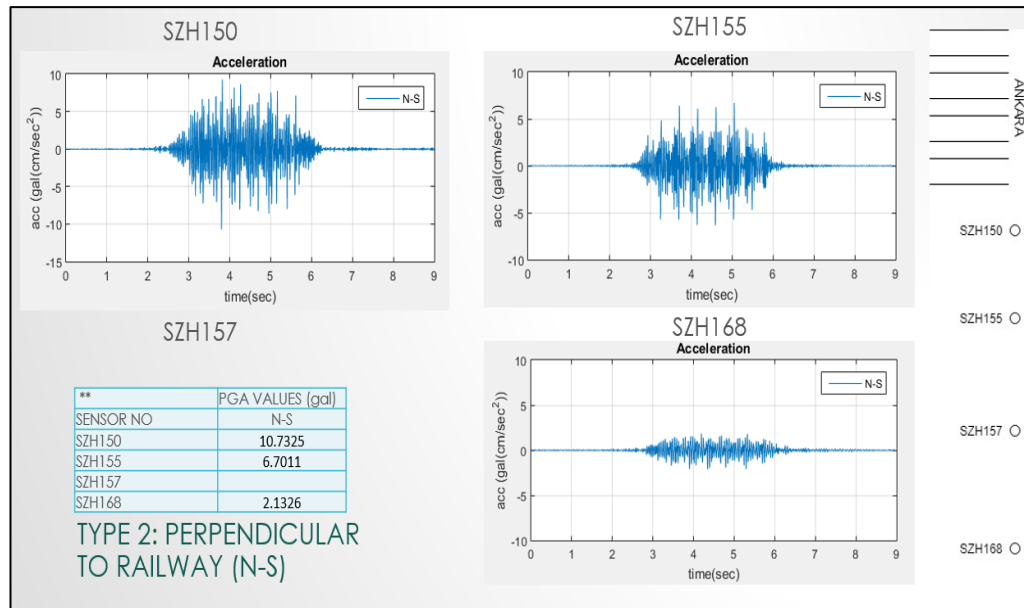


Figure 2.39: Acceleration-Time graph of barrier – field perpendicular (n-s) direction.

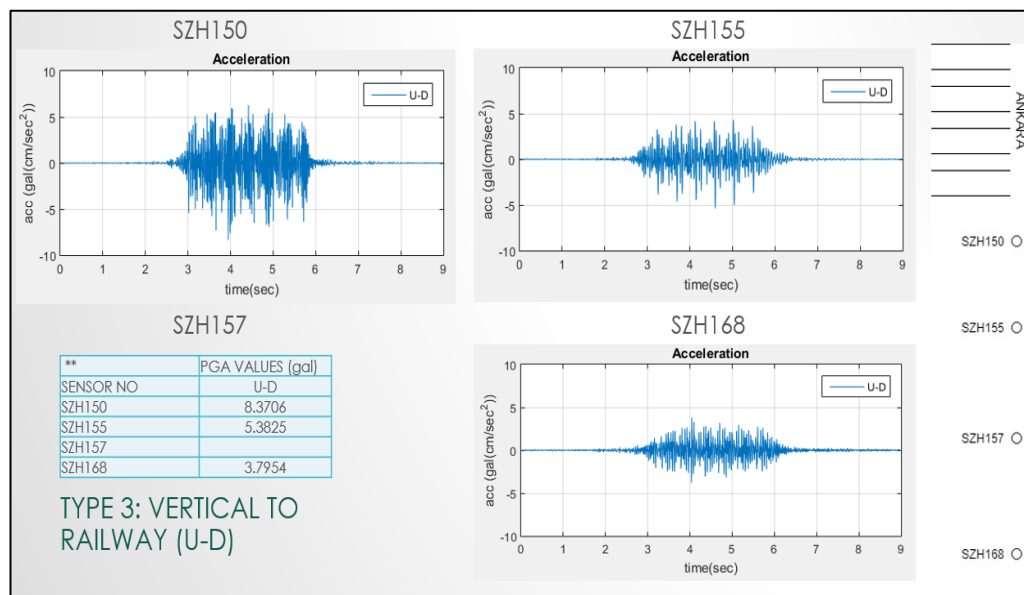


Figure 2.40: Acceleration-Time graph of barrier – field vertical (u-d) direction.

For velocity values caused by train vibrations, the peak value for the parallel direction obtained from the first device 0.05 cm/s, second value is 0.03 cm/s, and the values from the last station decreased similarly to the first study and found 0.011 cm/s (Figure 2.41).

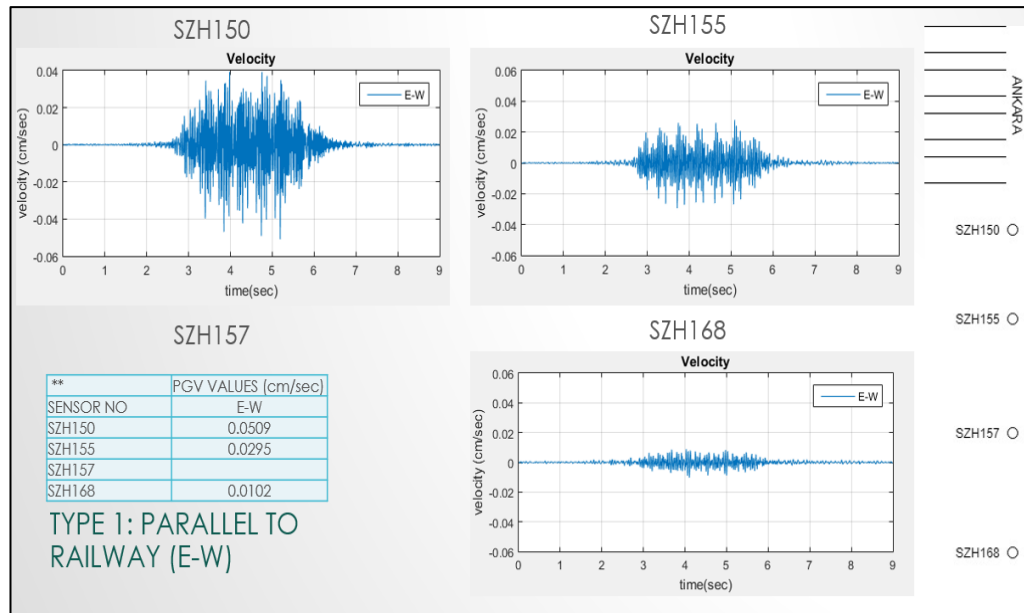


Figure 2.41: Velocity-Time graph of barrier – field parallel (e-w) direction.

Also, for perpendicular values in order, the first sensor found as 0.054 cm/s, the second sensor is 0.043 cm/s and the last sensor 0.014 cm/s (Figure 2.42). Even though “SZH157” could not record any vibrations, the other sensors show a nominal decrease with the distance from the railway line.

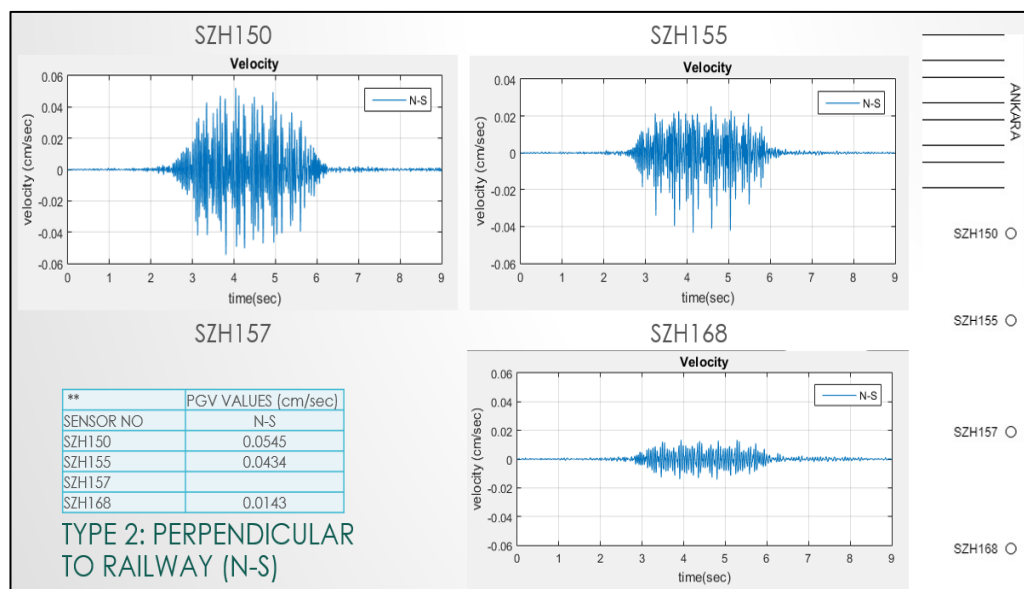


Figure 2.42: Velocity-Time graph of barrier – field perpendicular (n-s) direction.

For the velocity values in the vertical direction, the first value is 0.043 cm/s, the second value is 0.036 cm/s, while the value at the last sensor is 0.023 cm/s (Figure 2.43).

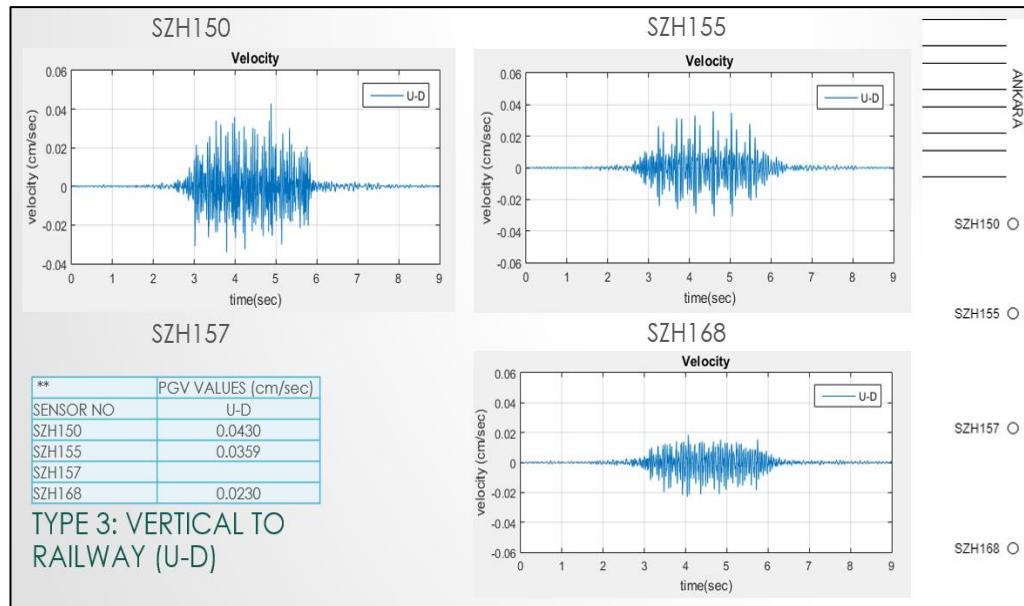


Figure 2.43: Velocity-Time graph of barrier – field vertical (u-d) direction.

In the frequency domain analysis, the Fourier amplitude values in the parallel direction calculated. The dominant amplitude at the nearest point is around 40 Hz, and when the distance increase from the line, two separate dominant amplitudes found between 20~30 Hz and 50 Hz (Figure 2.44).

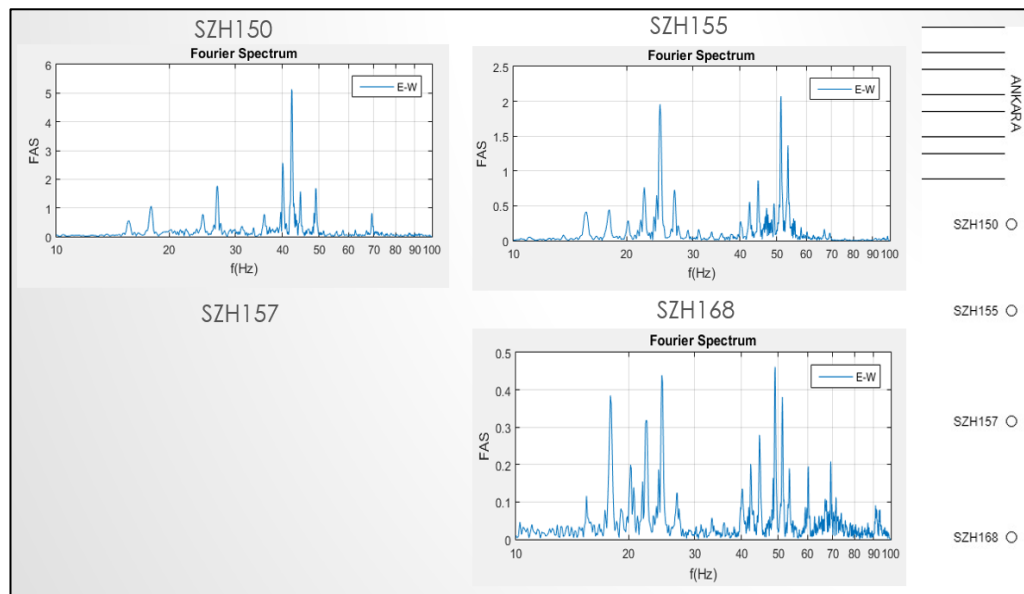


Figure 2.44: Fourier amplitude graph of barrier – field parallel (e-w) direction.

The dominant amplitude is 20~30 Hz in the nearest point in the perpendicular direction, it found between 20~30 Hz and 40~50 Hz in the second accelerometer (Figure 2.45).

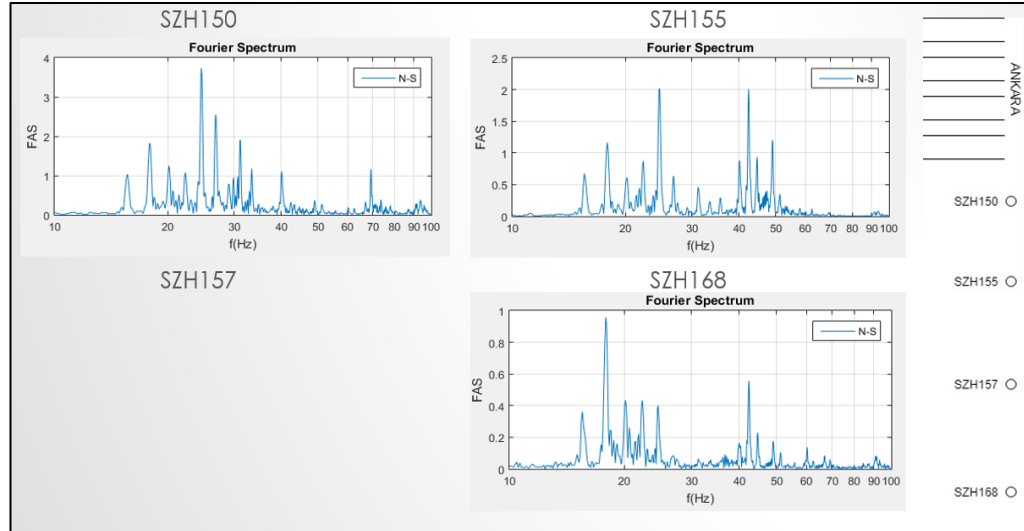


Figure 2.45: Fourier amplitude graphs of barrier – field perpendicular (n-s) direction.

Multiple dominant amplitudes occurred in the vertical direction. This might be caused from the accelerometers that are not fixed properly to the ground. However, the direct relation of distance with frequency content could not be observed (Figure 2.46).

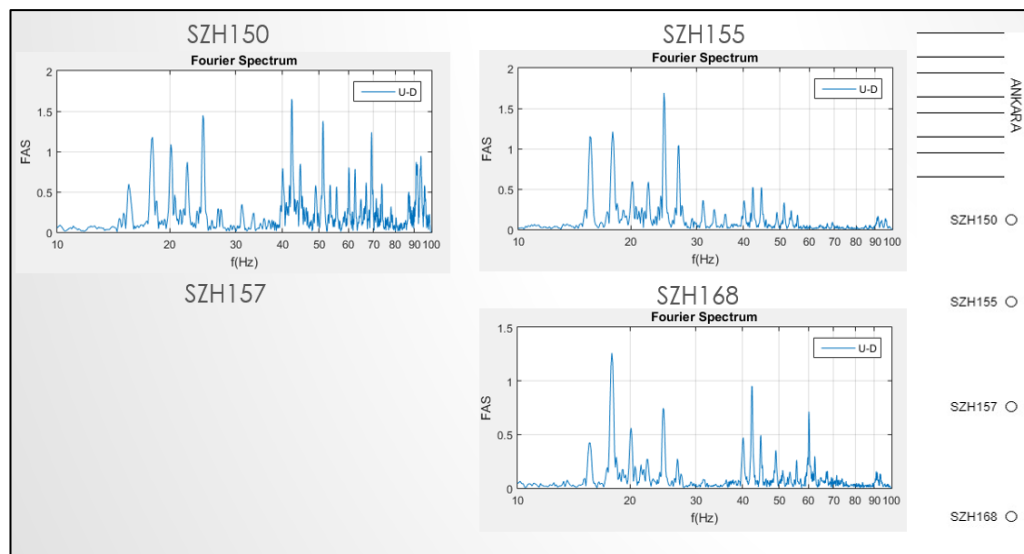


Figure 2.46: Fourier amplitude graphs of barrier – field vertical (u-d) direction.

2.5.3. Third Fieldwork Evaluation

Considering that the pavement will have a barrier effect, it is evaluated in the barrier motion section. One of the two train passages' record could not be obtained due to a malfunction in the sensor numbered "152" located adjacent to the railway. Graphics shown in this chapter are for the remaining four sensors.

When the graphs analyzed in the time domain, a decrease in the acceleration values received from the sensors is with the distance from the train line.

For the acceleration values in the parallel direction, 3.13 gal obtained in M150. As we moved away from the line, this value decreased and found 2.48 and 2.49 gal in the middle devices, and 1.89gal in the last sensor (Figure 2.47).

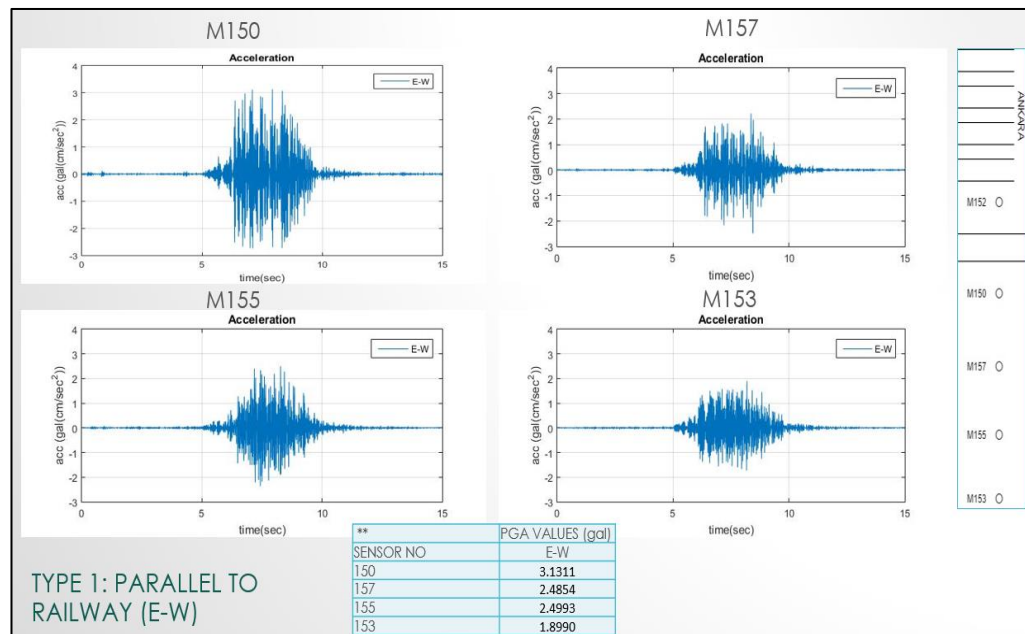


Figure 2.47: Acceleration-Time graph of barrier – field parallel (e-w) direction.

The peak acceleration value was found 4.34 gal in the perpendicular direction. The value decreased to 2.47 gal in the third alignment. The fourth sensor decreased to 2.38 gal and in the last accelerometer was found as 1.53 gal (Figure 2.48).

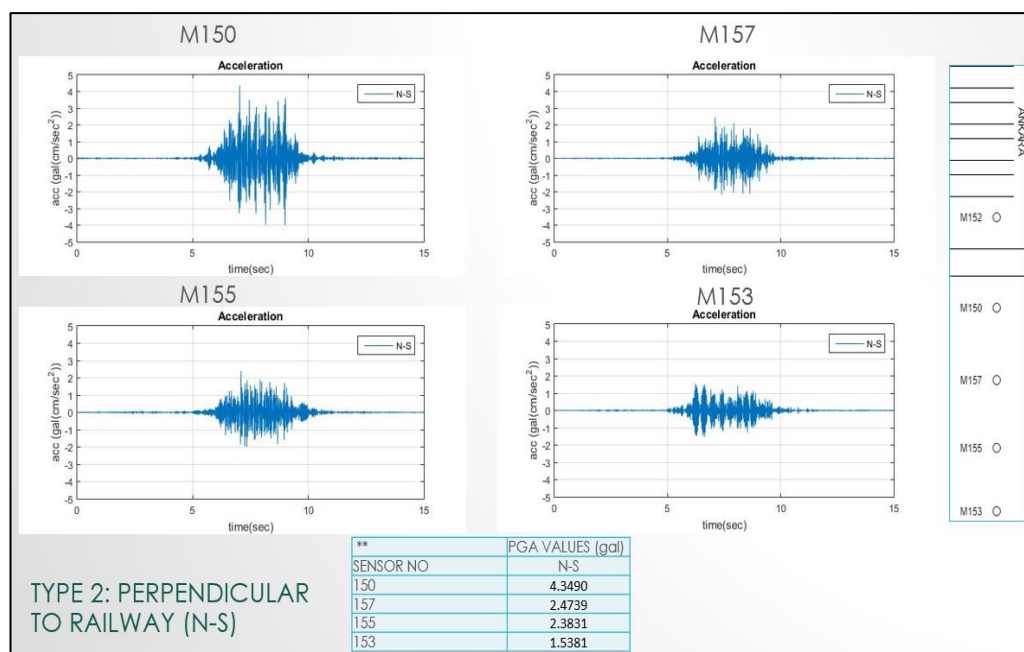


Figure 2.48 Acceleration-Time graph of barrier – field perpendicular (n-s) direction.

In the vertical direction, values found on the second sensor as 5.47 gal, 3.70 gal found on the third sensor, 3.27 gal on the fourth sensor and 1.55 gal found on the farthest sensor (Figure 2.49). Thus, the pavement had an effect on acceleration values similar to barrier effect.

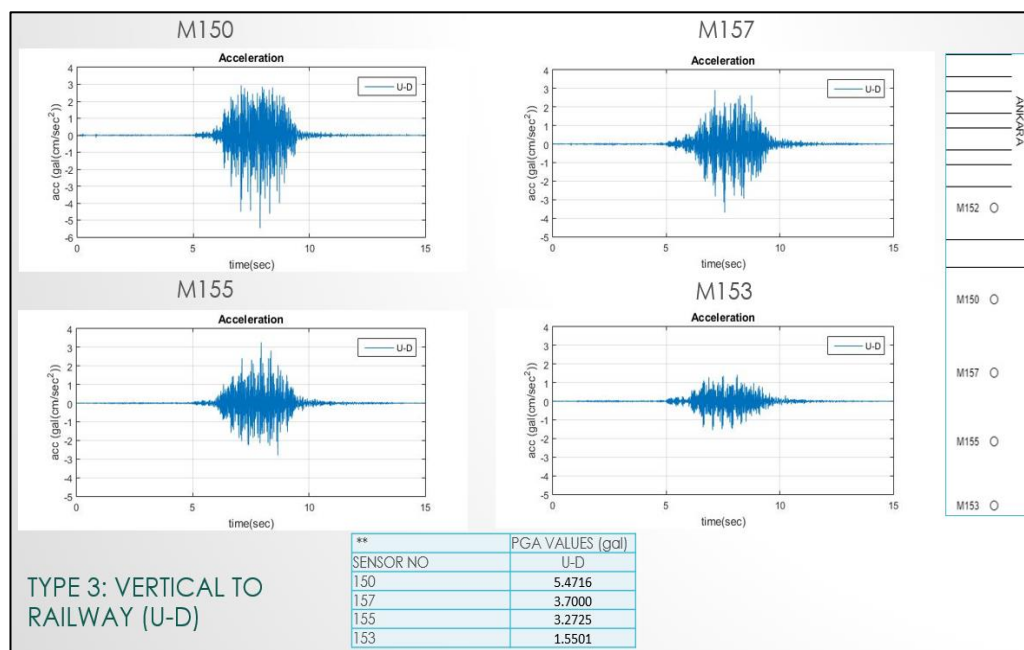


Figure 2.49: Acceleration-Time graph of barrier – field vertical (u-d) direction.

The velocity values caused by train vibrations are 0.019 cm/s the first station and 0.011 cm/s the last station for parallel direction (Figure 2.50). Moreover, the perpendicular values are 0.029 cm/s in the first sensor and 0.009 cm/s in the last sensor (Figure 2.51).

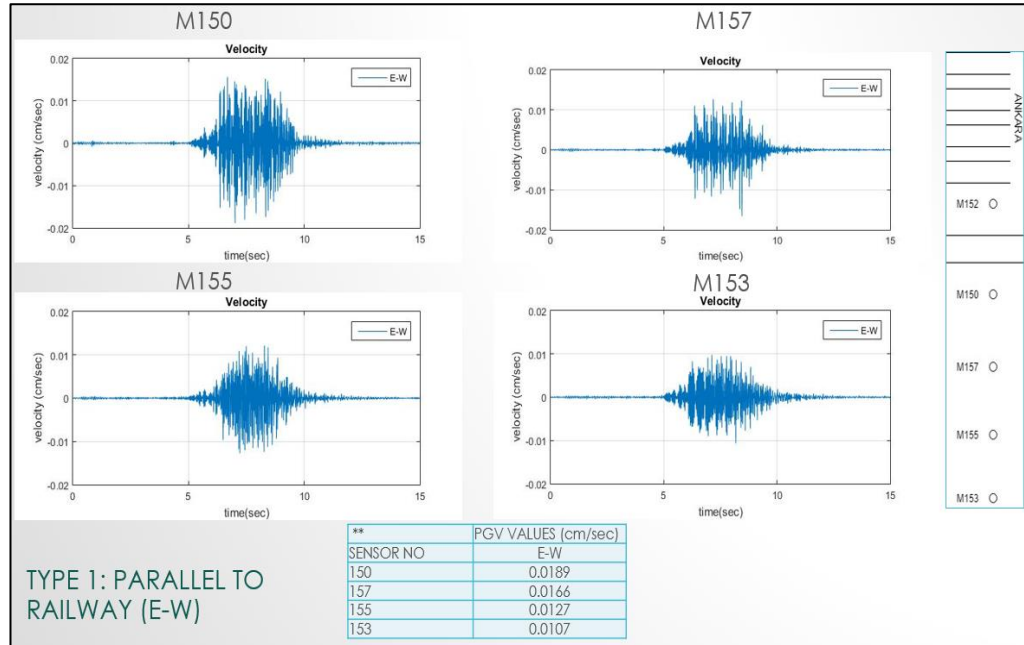


Figure 2.50: Velocity-Time Graph of barrier field motion parallel (e-w) direction.

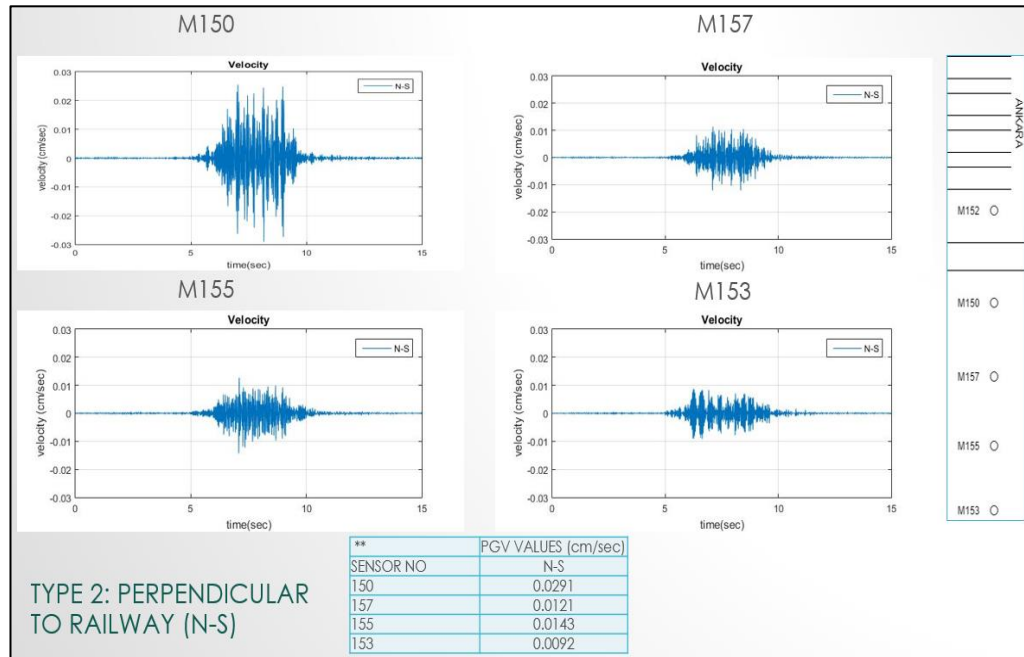


Figure 2.51: Velocity-Time graph of barrier – field perpendicular (n-s) direction.

For the vertical direction, the value adjacent to the line is 0.024 cm/s, and the last sensor is 0.010 cm/s (Figure 2.52). The decrease with the distance from railway tracks can be seen for all directions.

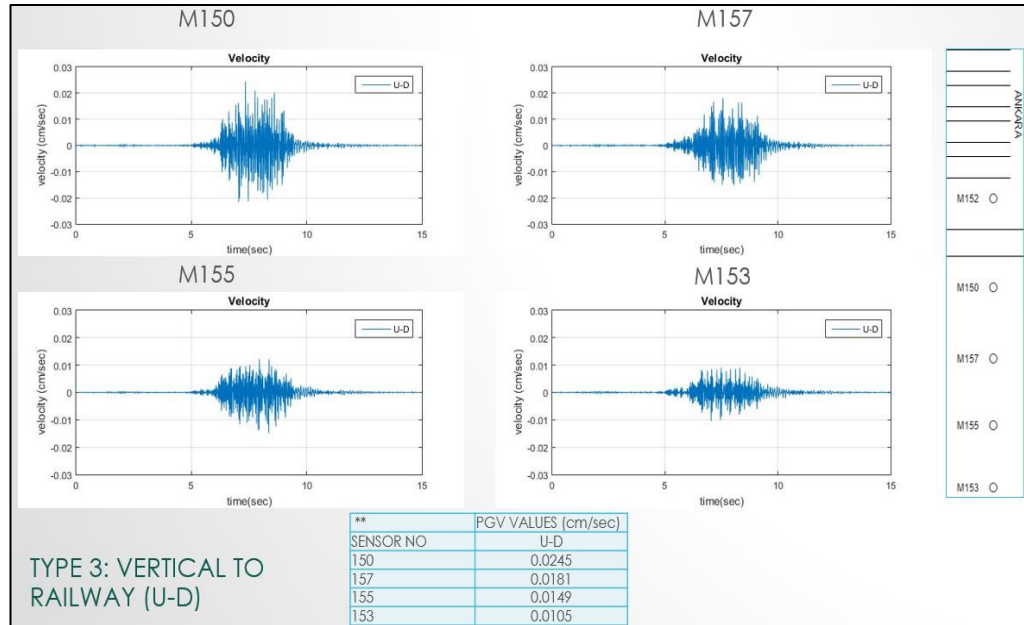


Figure 2.52: Velocity-Time graph of barrier – field vertical (u-d) direction.

In the frequency domain analysis, the Fourier amplitude values in the parallel direction, the dominant amplitude is 30 Hz in the nearest station and it varies between 20 to 40 Hz in the second and third stations (Figure 2.53).

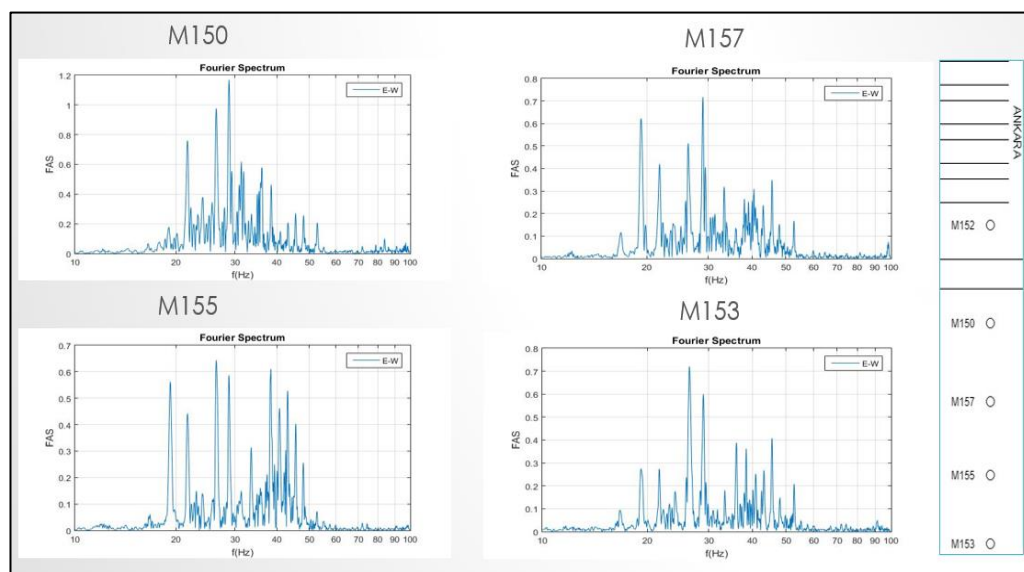


Figure 2.53: Fourier amplitude graph of barrier – field parallel (e-w) direction.

The dominant amplitude obtained approximately 20~30 Hz at the nearest point for perpendicular direction, and it varies in the range of 30~40 Hz in other alignments (Figure 2.54).

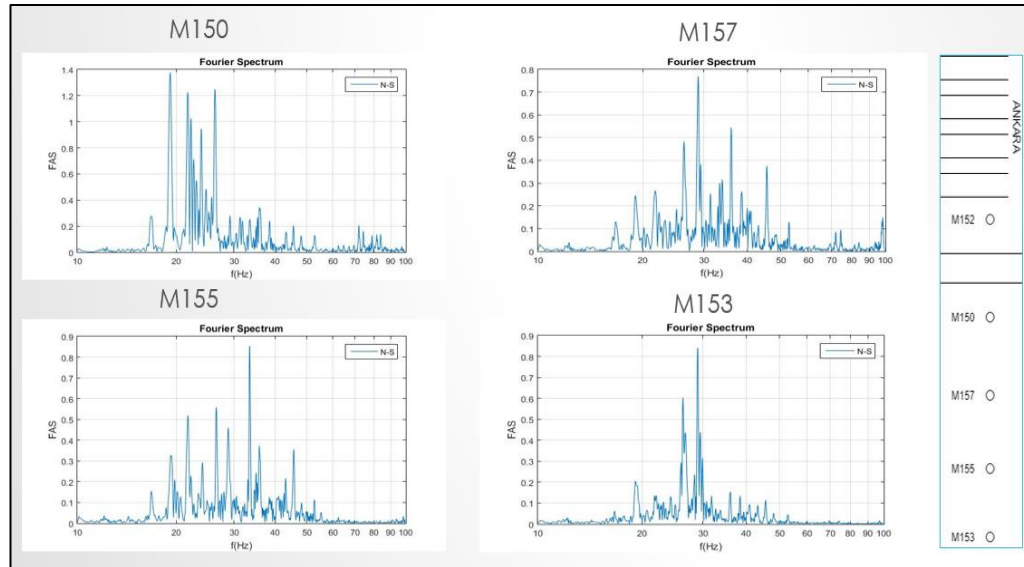


Figure 2.54: Fourier amplitude graph of barrier – field perpendicular (n-s) direction.

As in all other fieldworks, multiple dominant amplitudes occurred in the vertical direction. An increase in the dominant amplitude value also noticed in the third alignment regardless of the line distance (Figure 2.55).

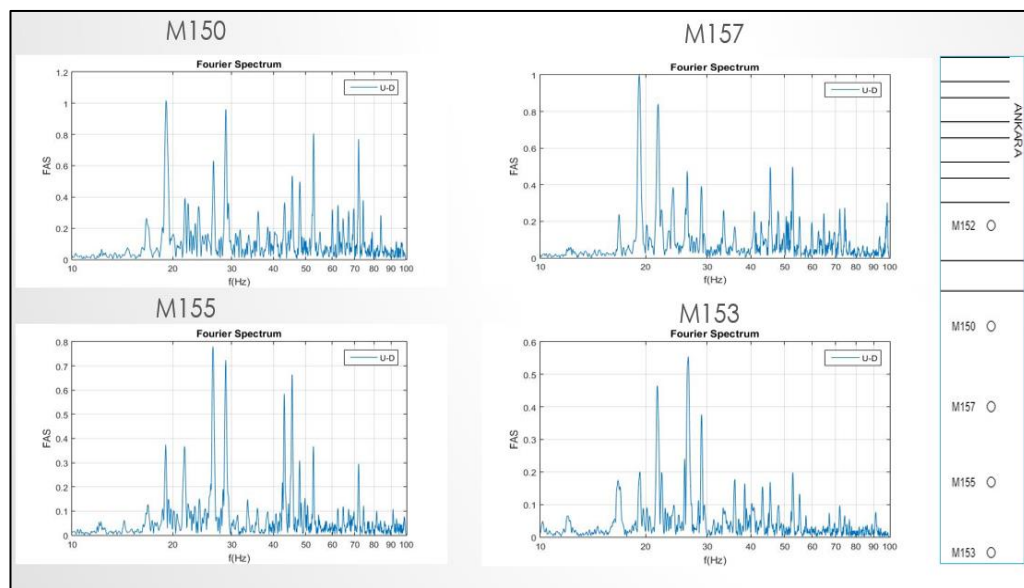


Figure 2.55: Fourier amplitude graph of barrier – field vertical (u-d) direction.

3. STRUCTURAL ANALYSIS ON BUILDING MODELS

Recently, developments in technology made easier the construction of high-rise buildings and led the construction industry began to progress in this direction as well. Therefore, different types of building stocks in urban centers have appeared. The vibration frequencies of reinforced concrete buildings vary depending on their height. Considering that the urban transportation capacity is increasing and due to the resonance effect, the responses of the structures to the vibrations are vital. Hence environmental vibration effects on structures studied in this thesis.

In this study, to analyze the effect of the vibrations obtained from the field studies, three different reinforced concrete building frame models designed, which are, 3-storey, 6-storey and 9 storey frames. For all three structures, frame system type, dead and live loads, structure geometry, material properties assigned the same and only building heights differed. The plans of the A, B, and C models are shown below (Figure 3.1). The width of spans was taken as 5 m, first-floor height was taken 3.5 m, and other floors had fixed 3 m. long. Material properties and load types are given in the table below (Table 3.1).

Table 3.1: Material properties of the frame models.

Properties:		
Concrete Type	:	C25/30 MPa
Rebar	:	S420
Columns	:	30x30 cm
Beams	:	30x30 cm
Dead Load	:	3.5 kN/m
Live Load	:	1.5 kN/m

For modeling of the frame systems with three, six and nine stories, a finite element software analysis program has been utilized for the analysis. Considering the response of the structures would be relatively small, the linear elastic response has taken into consideration. Modal linear time history analysis carried out by affecting the vibrations of the high-speed train, freight and passenger train transitions to all

building models. The analysis performed regardless of kinematic interaction between soil and the structure.

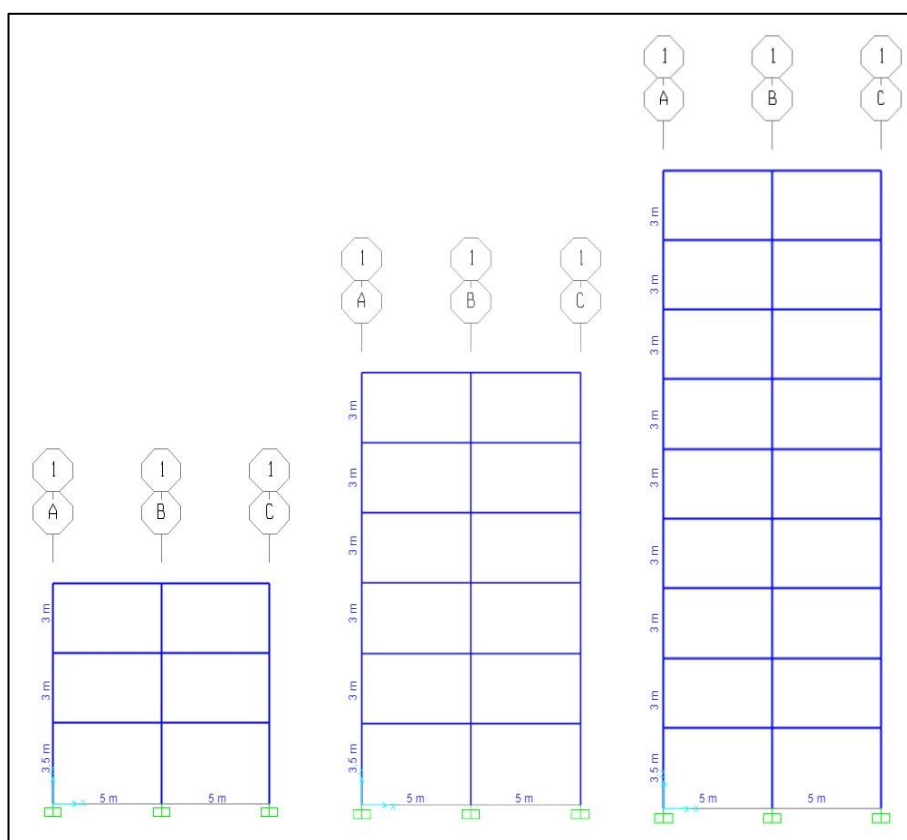


Figure 3.1: Structural frame models.

Subsequently, the distribution of the peak acceleration, peak velocity, and peak displacement parameters of these models extracted and plotted. Periods and frequencies of the buildings according to their modes are shown in the table below (Table 3.2).

Table 3.2: Mode values of structural frame models.

Model No:	Story	Mode 1		Mode 2		Mode3	
		T (sec)	f (Hz)	T (sec)	f (Hz)	T (sec)	f (Hz)
Model A	3 Story	0.424	2.355	0.129	7.694	0.071	14.081
Model B	6 Story	0.834	1.198	0.266	3.746	0.149	6.693
Model C	9 Story	1.251	0.799	0.407	2.455	0.232	4.291

3.1. Time History Analysis and Evaluation

To see the effects of different train types on the structures together, a linear time history analysis performed using four different train recordings. For the evaluations, the station that recorded the highest horizontal accelerations in all three fieldworks chosen to see the maximum effect. Actually, the measured vertical component values of train recordings had the peak results but, because of the buildings' weight, vertical vibrations can be neglected. The peak values of the train crossings are given in the table below (Table 3.3). Station "A" coded recordings are from first fieldwork and Station "B" coded Barrier Field Motion recording is taken from second fieldwork.

Table 3.3: Selected train records for the analytical approach.

Train Types	Station codes	Track Line Location	Peak Acceleration (gal)
High-Speed Train	A-FFM-HST	1 st track line	27.44923
High-Speed Train	B-BFM-HST	1 st track line	12.9219
Freight Train	A-FFM-FT	3 rd track line	8.4094
Passenger Train	A-FFM-PT	4 th track line	3.91653

Selected acceleration- time records of trains are shown in figures (Figure 3.2), (Figure 3.3), (Figure 3.4), (Figure 3.5). As can be seen from acceleration – time histories, train vibrations have different characteristics. Freight trains and passenger trains periods are longer than high-speed trains.

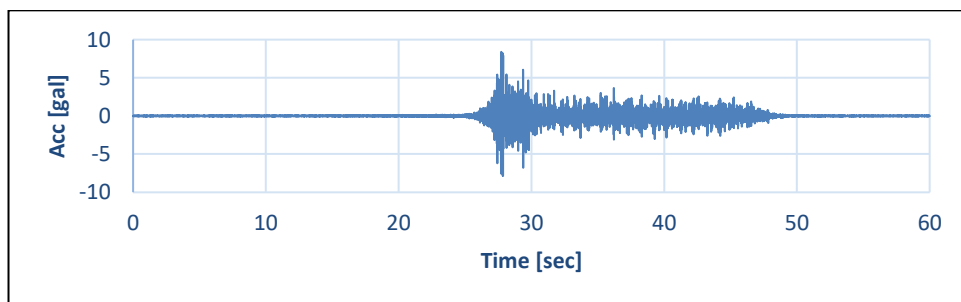


Figure 3.2: Freight Train acceleration records.

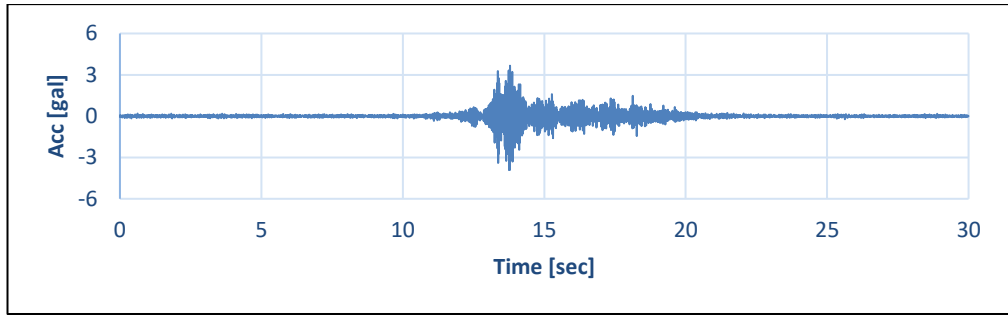


Figure 3.3: Passenger Train acceleration records.

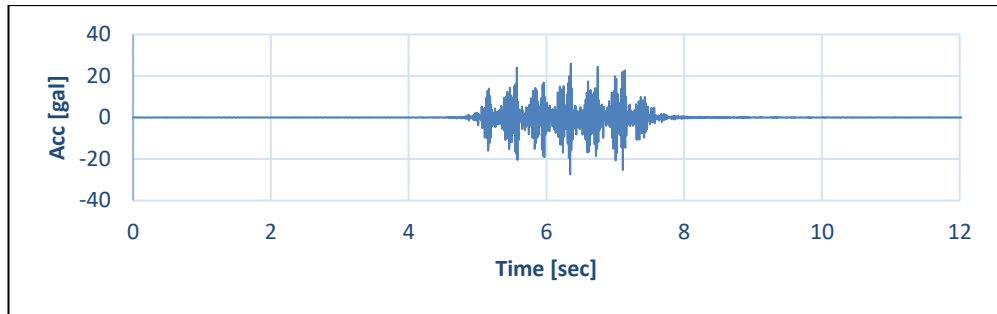


Figure 3.4: High-Speed Train acceleration records of free field motion.

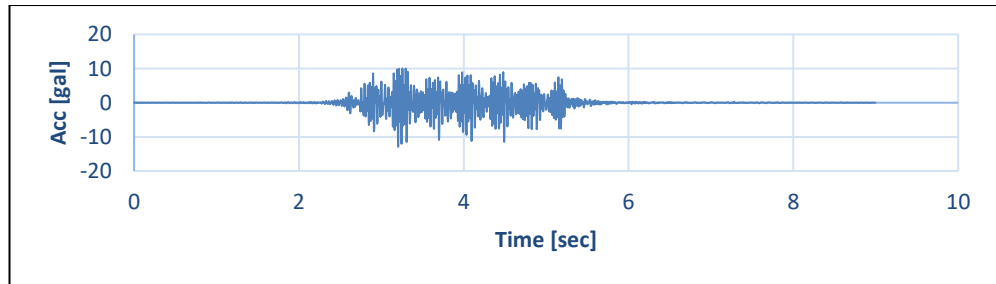


Figure 3.5: High-Speed Train acceleration records of barrier field motion.

Fourier Amplitude Spectrums of input motions can be seen below (Figure 3.6), (Figure 3.7), (Figure 3.8), (Figure 3.9). High-speed trains have greater amplitudes comparing to freight and passenger trains.

The dominant frequencies for high-speed trains in free field recording resulted as 60 Hz, while barrier field recording resulted as 50 Hz. Moreover, the freight train's dominant frequency is at 60 Hz and for the passenger train, it has found at 40 Hz.

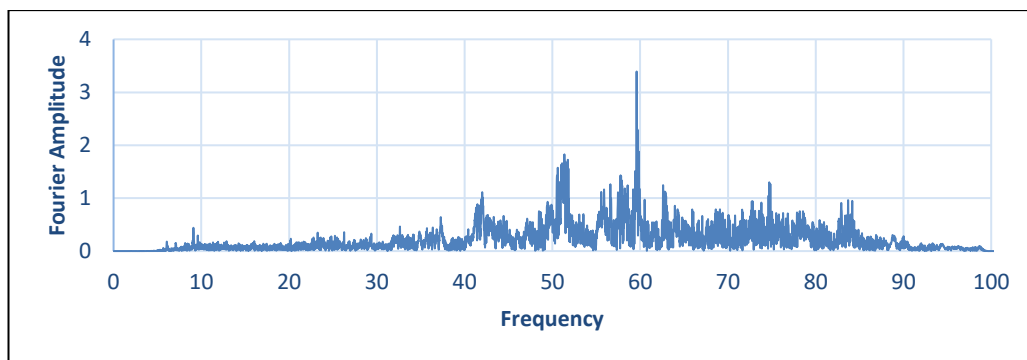


Figure 3.6: Fourier amplitude spectrum of freight train.

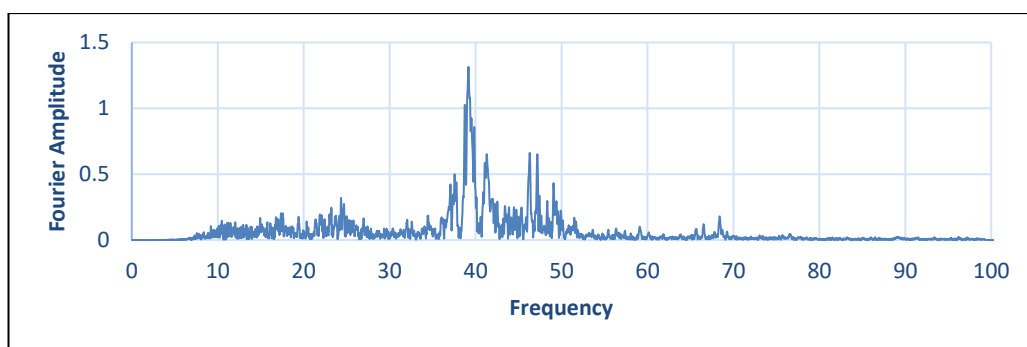


Figure 3.7: Fourier amplitude spectrum of passenger train.

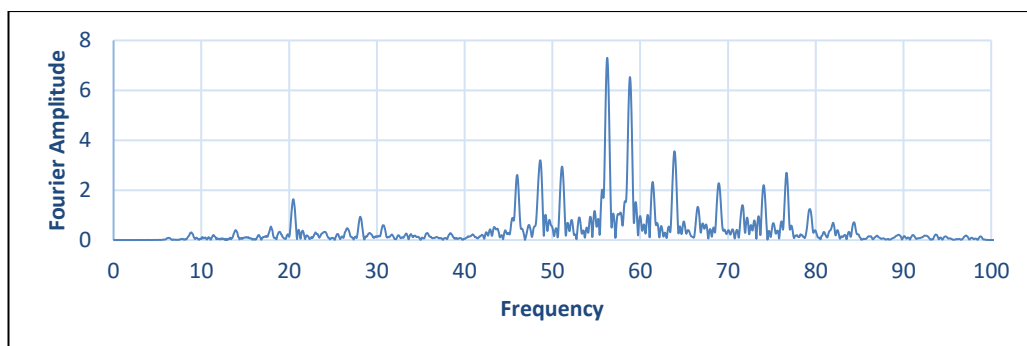


Figure 3.8: Fourier amplitude spectrum of high-speed train of free field motion.

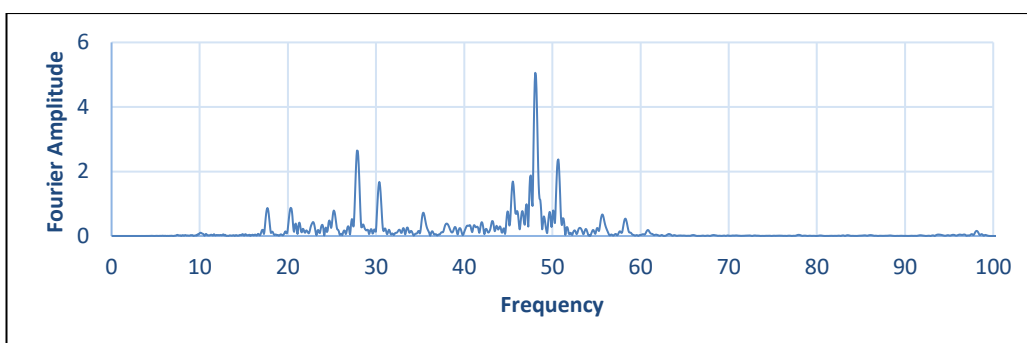


Figure 3.9: Fourier amplitude spectrum of high-speed train of barrier field motion.

Finally, all input motions injected to the models as time history functions. Five percent damping ratio assumed for the analysis. The analysis performed regardless of the kinematic interaction of soil and structures. In addition, only the lateral responses taken into consideration in the analysis.

The outputs extracted from the analysis program and envelope curves drawn along the story levels of the building by extracting the maximum acceleration, velocity and displacement values. In this way, the floors on which the peak values occurred were determined. Velocity response spectrums obtained and maximum absolute velocities calculated. Then the results compared with the international regulations.

3.1.1. Peak Acceleration Distributions of Building Models

As a result of the analysis of the building models, the following graphs obtained by drawing the maximum and minimum values along the story levels for three different train types (Figure 3.10), (Figure 3.11), (Figure 3.12). As can be seen in the graphics, the peak acceleration value for all three building models occurred on the first or second floors, and all the values are almost approximate to one another across the entire building.

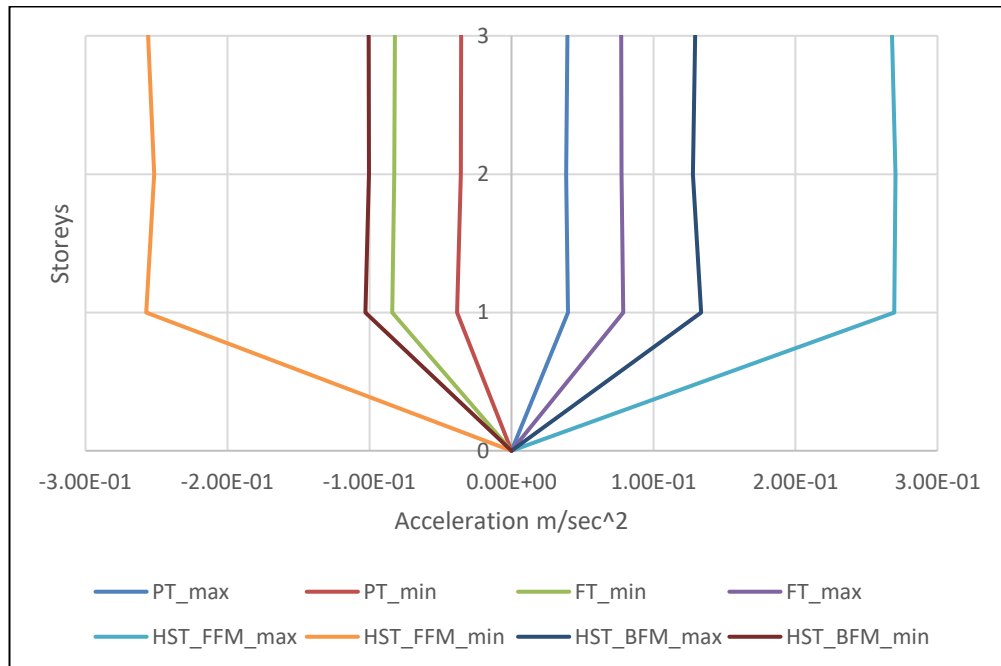


Figure 3.10: Acceleration distribution on storey levels of model 'A'.

According to these results, the peak accelerations were obtained from the high-speed train for three models. Followed by the freight train and lastly suburban train. The results of the analysis were directly proportional to their distance from the recording stations.

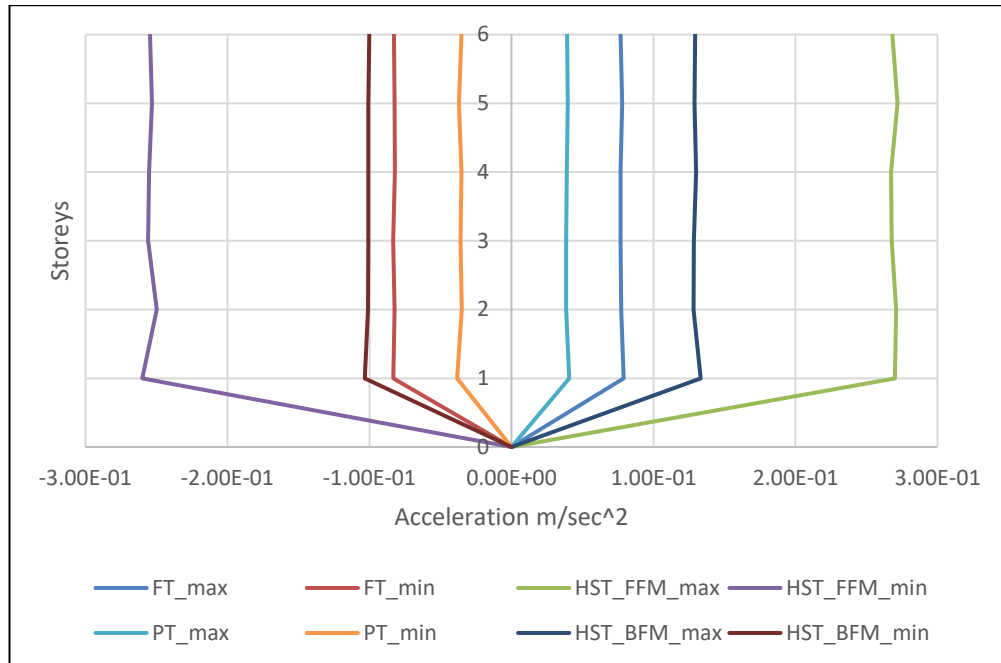


Figure 3.11: Acceleration distribution on storey levels of model 'B'.

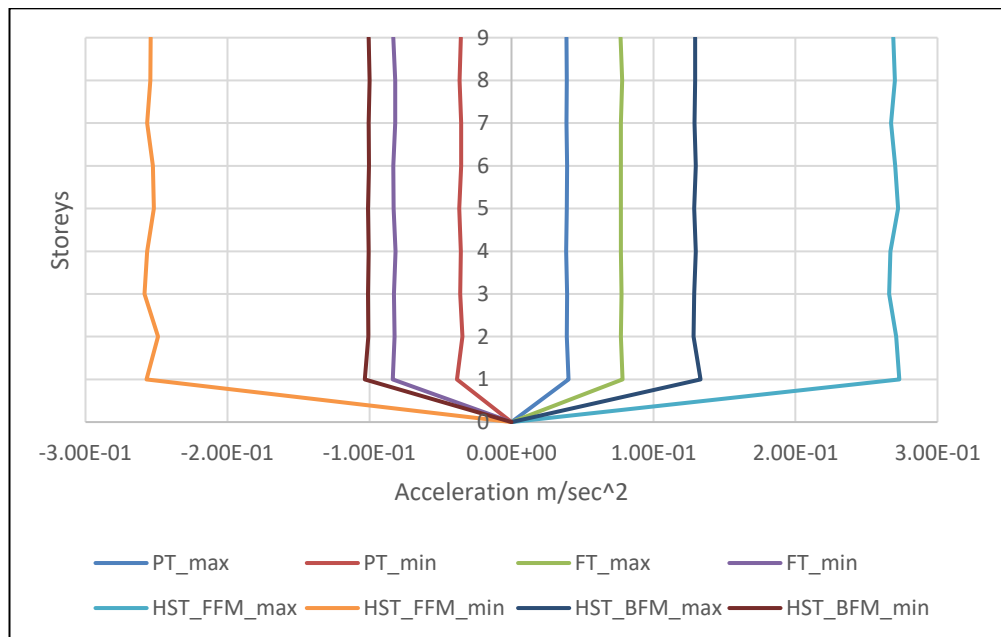


Figure 3.12: Acceleration distribution on storey levels of model 'C'.

The reason why all the accelerations are so close to each other can be explained by the fact that buildings are normally deformed at a lower frequency content. The vibration levels obtained from the train records were higher than the vibrations of the buildings.

3.1.2. Peak Velocity Distribution of Building Models

When the records of trains applied on the sample structures with three different frequencies, the maximum and minimum velocities on the storey levels found below. The three – storey frame model 'A' is seen in (Figure 3.13), the six – storey frame model 'B' is seen in (Figure 3.14), the nine – story frame model 'C' is seen in (Figure 3.15).

According to the results, the highest velocities reached in the high-speed train effect for all building models as comparison of the train types. This was followed by the freight and passenger train. When we evaluate the freight and passenger train among themselves, the highest velocity values in the model 'A' formed on the roof floor were obtained.

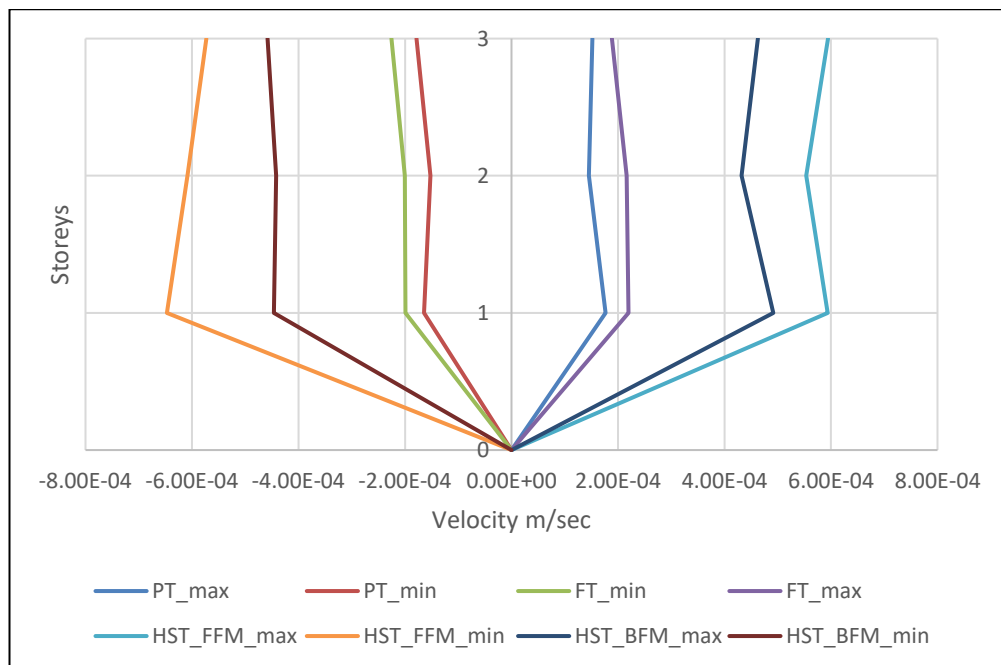


Figure 3.13: Velocity distribution on storey levels of model 'A'.

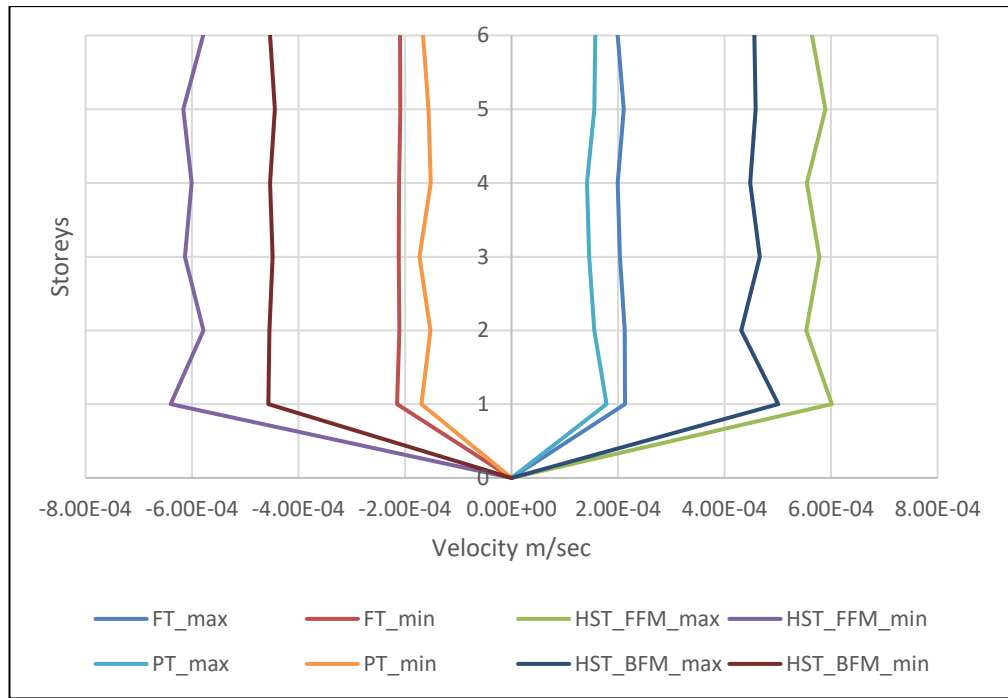


Figure 3.14: Velocity distribution on storey levels of model 'B'.

However, in high-speed train results, this situation occurred on the first floor, similarly the acceleration values. In the 'B' model, the absolute peak velocity for all three trains achieved on the first floor. And in the 'C' model, the highest velocity value found on the first floor.

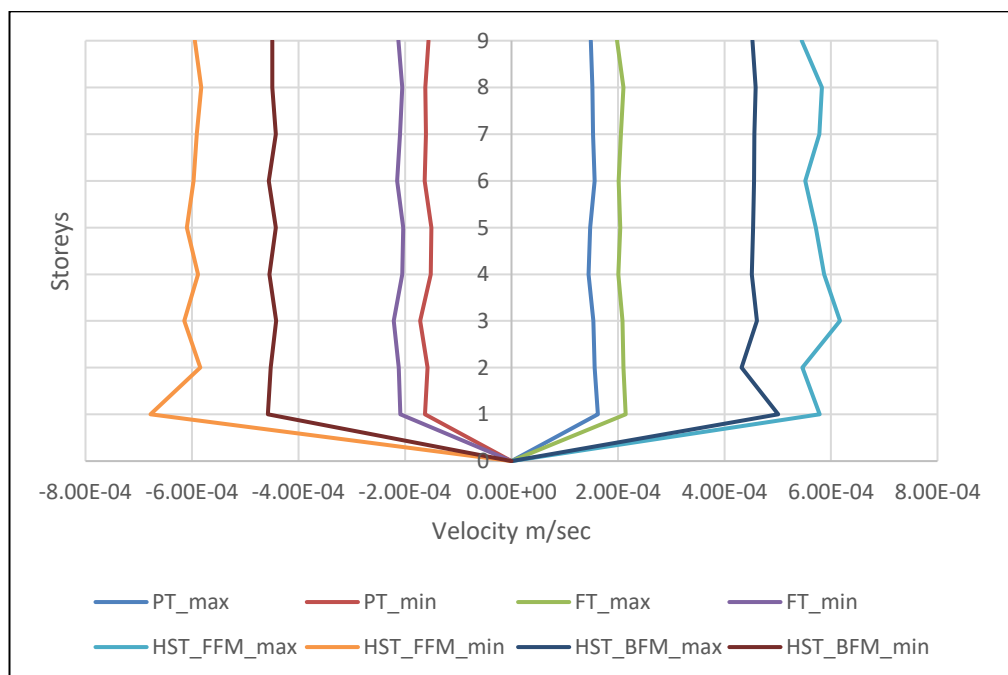


Figure 3.15: Velocity distribution on storey levels of model 'C'.

3.1.3. Peak Displacement Distribution of Building Models

Finally, displacement values examined for all story levels. Accordingly, envelope curves drew by finding displacement values on each floor horizontally. The peak displacements for all models obtained from the records of high-speed trains.

For the three-story frame model 'A', the maximum displacement in the passenger train and high-speed train was on the first floor. And on the freight train, the maximum displacement was on the second floor. Displacement values from freight and passenger trains resulted very similar to each other (Figure 3.16).

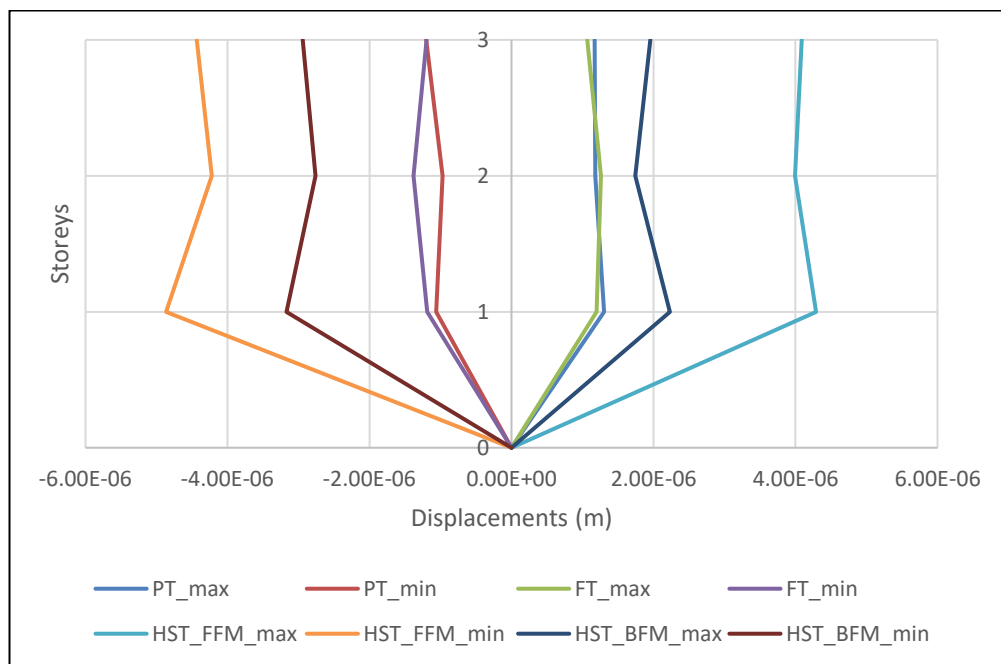


Figure 3.16: Displacement distribution on storey levels of model 'A'.

For the model 'B', the maximum and minimum displacement values resulting from freight and passenger trains were found very close to each other. The displacement resulting from the high-speed train reached the peak value on the third floor. Barrier motion recording of high – speed train reached to peak value at first floor (Figure 3.17).

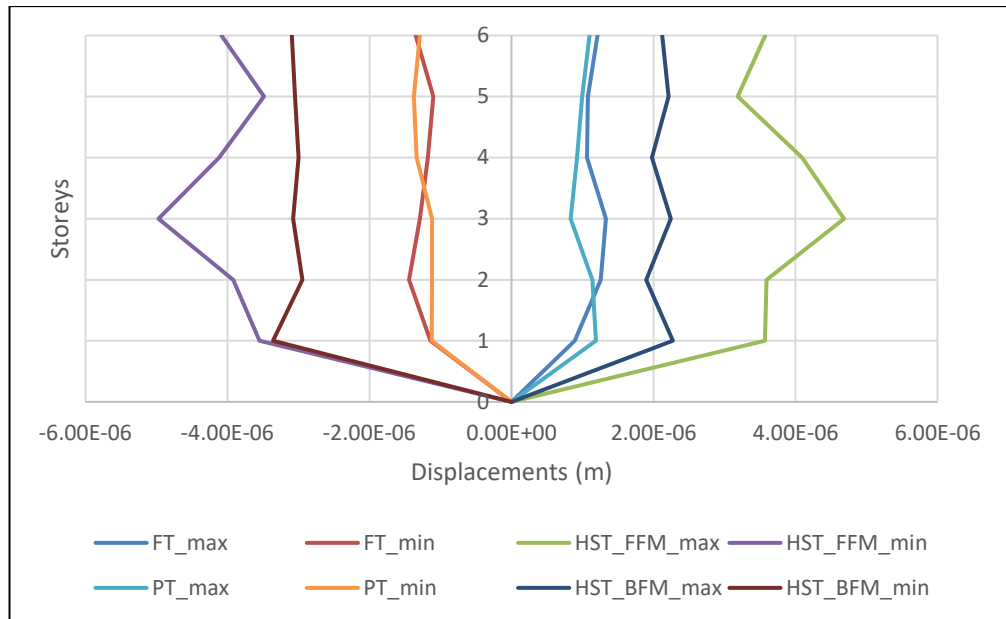


Figure 3.17: Displacement distribution on storey levels of Model 'B'.

Lastly, the model 'C' was examined. The displacement values obtained from the freight and passenger trains reached peak values at the rooftop floor as expected. However, in the high-speed train, peak displacement occurred on the third floor as in the other two models (Figure 3.18).

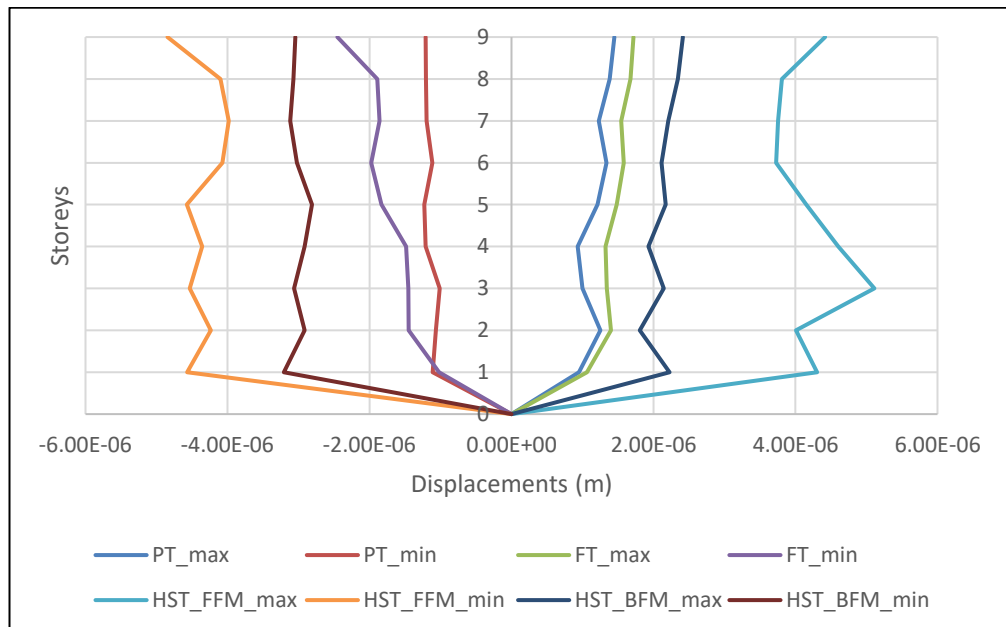


Figure 3.18: Displacement distribution on storey levels of Model 'C'.

These displacements may be caused by the content of recordings. The freight and passenger trains have longer periods than a high-speed train. Therefore, unlike freight trains and passenger trains, the period of the high-speed train vibration is rather too short, that recording ends without the opportunity for the building to oscillate.

3.2. Structural Response and Human Perception in The International Regulations

According to DIN4150-3 standard, evaluations based on the maximum absolute value of the velocity signals for the three of the components (x, y, z) of the unweighted velocity signals. Three types of lines determined for different types of structures. First line indicates industrial buildings and commercial purpose buildings, second line indicates dwellings and similar type of buildings and third line indicates sensitive buildings, such as historical buildings [16]. These guideline values are shown in the table below (Table 3.4). Specified curves for vibration shown below (Figure 3.19).

Table 3.4: Guideline Values for Vibration Velocity.

Line	Type of Structure	Velocity Values, v_i (mm/s)			
		Vibration at the foundation			Vibration at highest floor for all frequencies
		1 Hz to 10 Hz	10 Hz to 50 Hz	50 Hz to 100 Hz	
1	Buildings used for commercial purposes, industrial buildings, and buildings of similar design	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design	5	5 to 15	15 to 20	15
3	Structures that because of their sensitivity to vibration (e.g. listed buildings under preservation order)	3	3 to 8	8 to 10	8

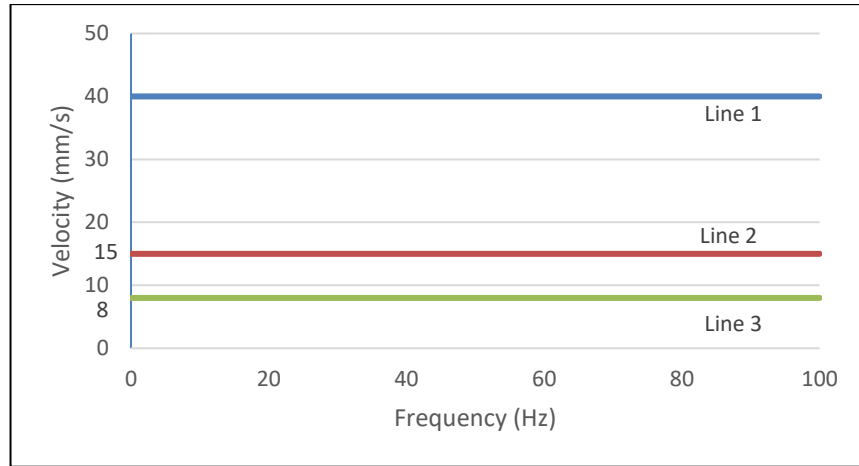


Figure 3.19: Curves for guideline values of short-term vibration at top floor.

Accordingly, for the evaluation of the structural response, the outputs of velocity response values of frame models on the top floor subjected to each input motions are extracted. Peak lateral velocity values for 3 storey frame model subjected to the recorded input motions results are shown below (Figure 3.20).

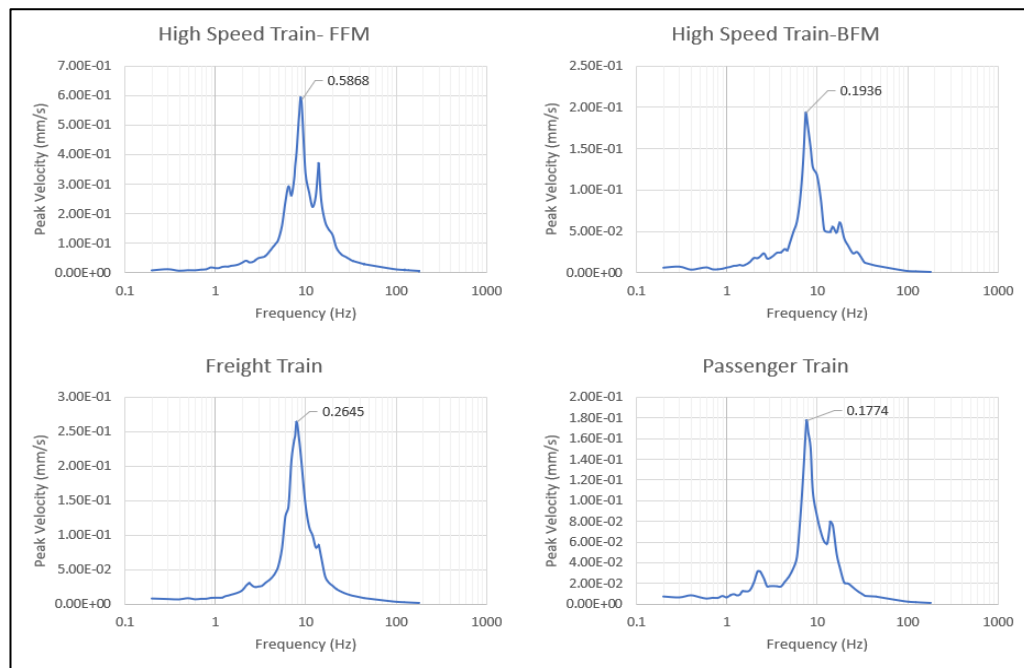


Figure 3.20: 3 – storey frame model ‘A’ peak lateral top floor velocities.

As can be seen in the figures, high-speed train result has the larger response to vibration motion. Followed by the freight and barrier motion and lastly passenger train

motion. The results show that the barrier decreased the response values for 3 story buildings.

Peak lateral velocity values for 6 – storey frame model subjected to the recorded input motions results can be seen in this section (Figure 3.21). The results are similar to frame model A. Free field recording of the high-speed train has the larger response among other recordings and also barrier motion response is rather small. Therefore, barrier reduces the vibration effect.

Peak lateral response velocity of 9 – story frame models can be seen in (Figure 3.22). The results show similarity with the other frame models. H. Hao et al [10] indicated in their study as

“An earthquake motion generally occurs at structural fundamental frequencies, while a blast-induced motion occurs at higher frequencies”.

The dominant frequency of the train induced ground motions subjected to the models is between 40-60 Hz, and peak velocities occurred close to 8-10 Hz. Therefore, all three frame models affected as such larger than seismic effect and lower than blast-induced vibration effect. The largest velocities occurred at 3-storey building. Since the heights of the buildings increase, the dominant frequency of the buildings gets minor values, which are too low than the dominant frequency of the input ground motions.

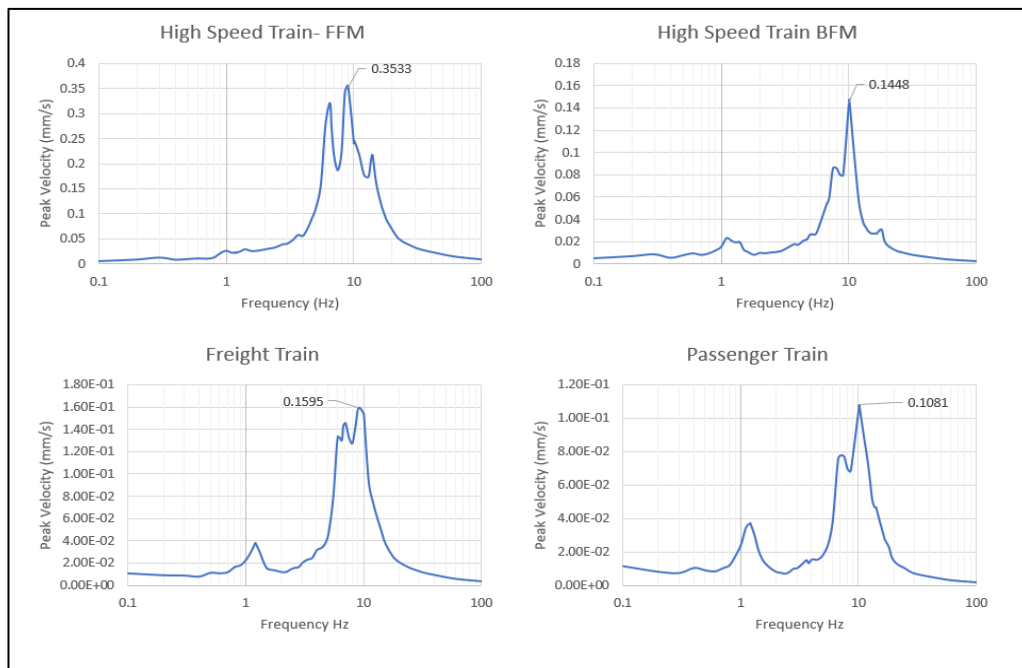


Figure 3.21: 6 – storey frame model ‘B’ peak lateral top floor velocities.

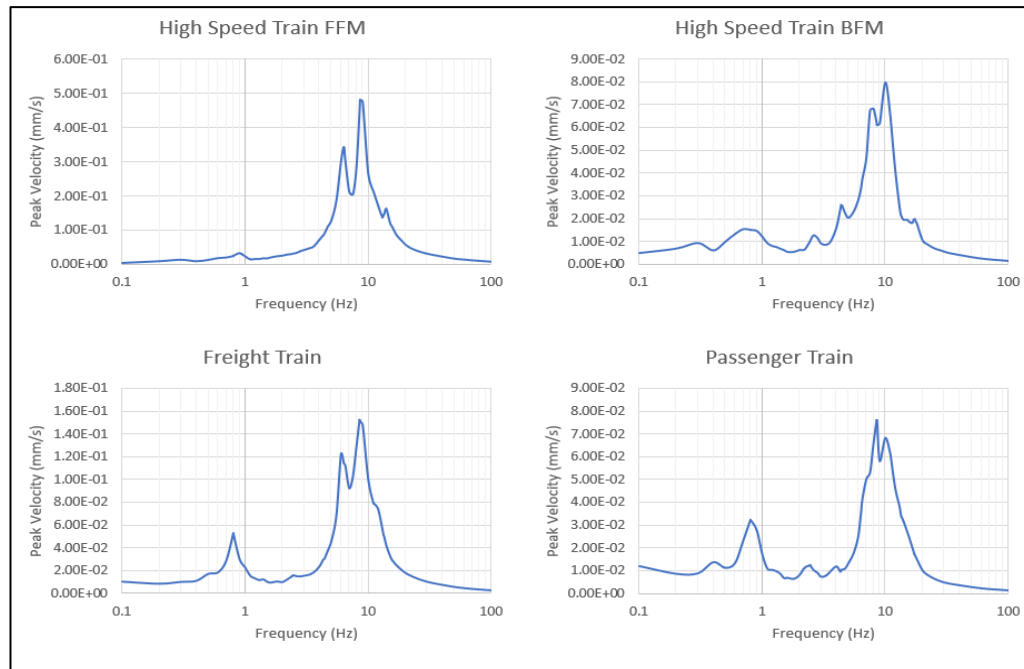


Figure 3.22: 9 – storey frame model ‘C’ peak lateral top floor velocities.

The peak response velocities put together in a tabular form to compare (Table 3.5). As can be seen from the table, the peak velocity resulted in 3 storey models subjected to high speed train free field records. Even the largest response value is under the guideline values of German vibration regulation (DIN4150-3) [16].

Table 3.5: Peak lateral velocity values of structural models.

Train Type	3 – Storey	6 – Storey	9 – Storey
	Velocity (mm/s)	Velocity (mm/s)	Velocity (mm/s)
High-speed Train (Free Field)	0.5868	0.3533	0.4820
High-speed Train (Barrier Field)	0.1936	0.1474	0.0797
Freight Train	0.2645	0.1595	0.1524
Passenger Train	0.1774	0.1081	0.0758

Thus, the vibration motions did not cause any structural response for this study. On the other hand, considering the human perception, values are adequate to be noticed by humans according to Australian vibration regulation (AS2670.2) [19] (Table 3.6).

Table 3.6: Vibration and human perception of motion.

Approximate Vibration Level (mm/s)	Degree of Perception
0.10	Not felt
0.15	Threshold of perception
0.35	Barely noticeable
1.0	Noticeable
2.2	Easily noticeable
6.0	Strongly noticeable

4. RESULTS AND DISCUSSION

In this section, peak acceleration graphs plotted from all train crossings and evaluated. Also, comparative graphics obtained from building analysis evaluated. And lastly, the results of analysis compared with the International Standards.

4.1. PGA Graphs of Train Vibrations

Peak acceleration values of all train crossings obtained and displayed on the chart depending on the distance. The graphs plotted for each railway line. Below, the values of the free ground and barrier ground for Ankara direction (first track line) are given together. Free ground transitions are indicated by a circular symbol on the graph and trenched ground transitions by a triangular symbol. The peak values in the first accelerometer seem obvious for all train crossings in parallel direction. It is observed that there is an amplification in the third alignment. The difference between the free field and barrier field is obvious with distance away from the train line. The recordings of first fieldwork from all stations were higher on the graph (Figure 4.1).

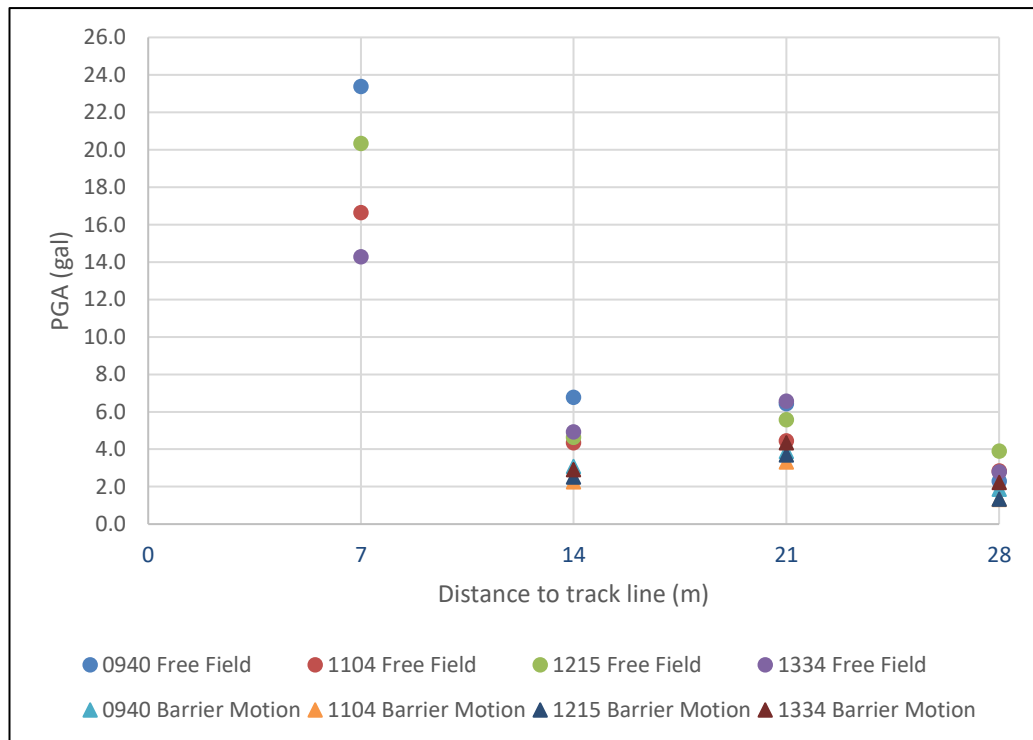


Figure 4.1: Comparison of the Pga values in parallel (e-w) direction 1st track line.

In the perpendicular direction, there is a regular decrease in values as it moves away from the rails (Figure 4.2). Train passes obtained different acceleration values for each direction in the closing areas, however, these values become similar to each other with the distance away from the line. Considering that the passing trains are at different velocities, it can be said that this situation changes with the effect of the velocity whereas nearest the line.

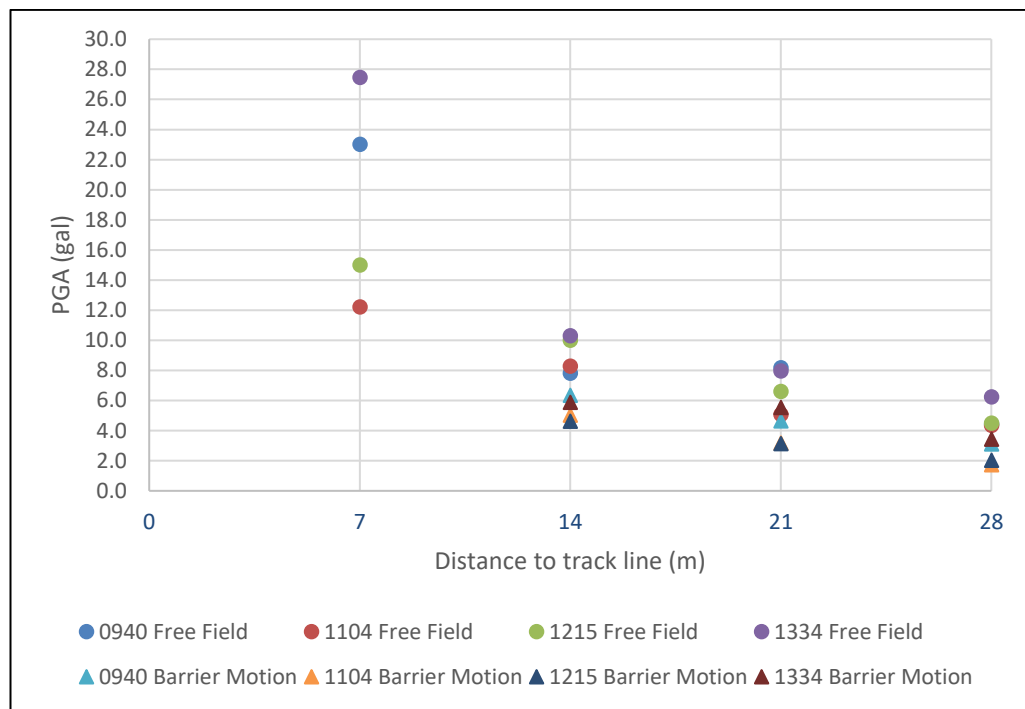


Figure 4.2: Comparison of the Pga values in perpendicular (n-s) direction 1st track – line.

Below, the values of the free field and barrier field are given together for the direction of Istanbul (second track line). Free ground transitions are indicated by a circular symbol on the graph and trenched ground transitions by a triangular symbol. The second line is ten meters away from the first alignment. Free ground values are higher than trenched ground values. Recordings from all stations are higher on the graph below. This difference gradually closes as it moves away from the line. In the third alignment, the PGA values amplified except for one recording (Figure 4.3).

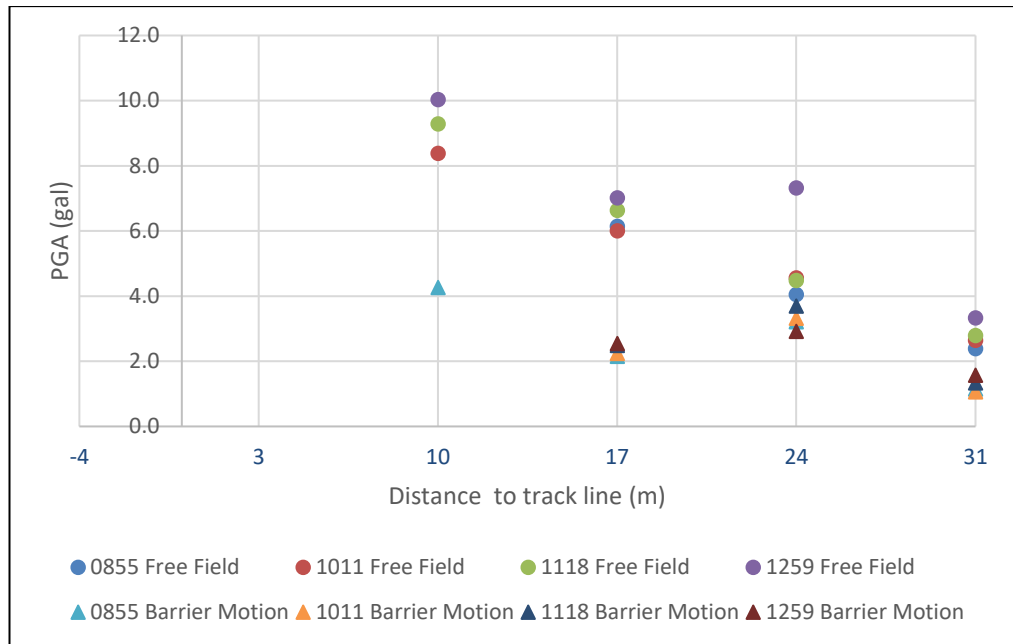


Figure 4.3: Comparison of the Pga values for the first fieldwork in parallel (e-w) direction 2nd track line.

In the free field results of perpendicular direction, an amplification occurred in the fourth alignment. In the barrier field motion, a similar situation can be seen in the records taken from the third alignment (Figure 4.4).

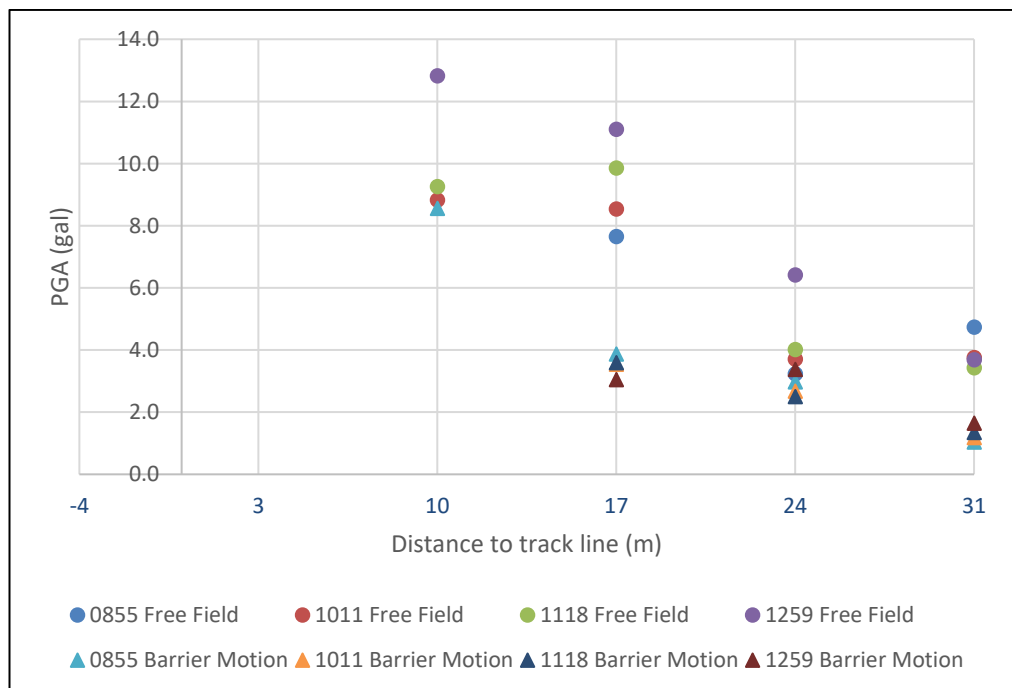


Figure 4.4: Comparison of the Pga values for the first fieldwork in perpendicular (n-s) direction 2nd track line.

Train passes obtained different acceleration values for each direction in the nearest stations, but it observed that these values approached each other as they moved away from the line. Considering that the passing trains are at different velocities, it can be said that this situation changes with the effect of the velocity near to the line.

In addition, in the third fieldwork, a different layout implemented due to the changes in the field. The barrier which used to observe the barrier effect was filled and turned into cobblestone pavement. Thus, with the third study, whether paving stones had an effect on vibrations just like a barrier has been observed. This section includes the comparison of free field motion from the first and second fieldworks and pavement field motion from the third fieldwork (Figure 4.5).

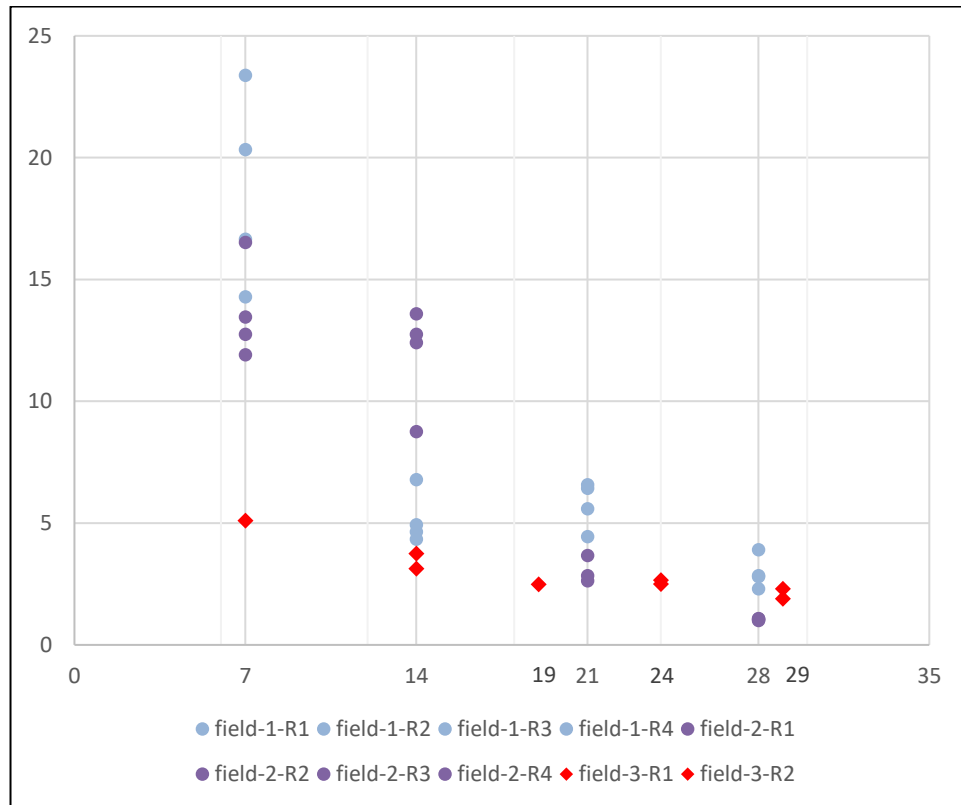


Figure 4.5 Comparison of free - field motion with pavement stone motion.

The blue dots indicate the first fieldwork and purple dots indicate the second fieldwork. The red diamonds belong to third fieldwork. Sensor alignments for first and second fieldwork start at seven meters away from the first railway line and finish at 28 meters with a bay 7 meters away from each other. For the third fieldwork, however, five sensors used for the alignment. Hence, distances showed a difference. The sequence follows with “7, 14, 19, 24, 29” meters away from the railway.

As can be seen in the Figure, red diamonds are below the blue and purple dots in the first fourteen meters of the alignment. A reduction can be mentioned for 14 meters. After that, however, the values seem rather close to each other and a few free field values measured lower than the pavement field. As a result, cobblestone pavement may affect the near site vibrations at a distance. After that, it shows no effect at all.

4.2. Frame Model Structural Analysis Results

As a result of the analysis of the building models, the following envelope graphs obtained for four different train types. Due to the similarity, only the results of model C added to this chapter. As mentioned earlier in the graphics, the peak acceleration value occurred on the first floor, and all the values are almost approximate to each other on all story levels (Figure 4.6).

According to these results, high-speed trains had the most effect on the building models. Followed by the freight train and lastly suburban train. The railway lines of recordings differed from each other. Therefore, the analysis resulted as expected, directly proportional to their distance from the recording stations.

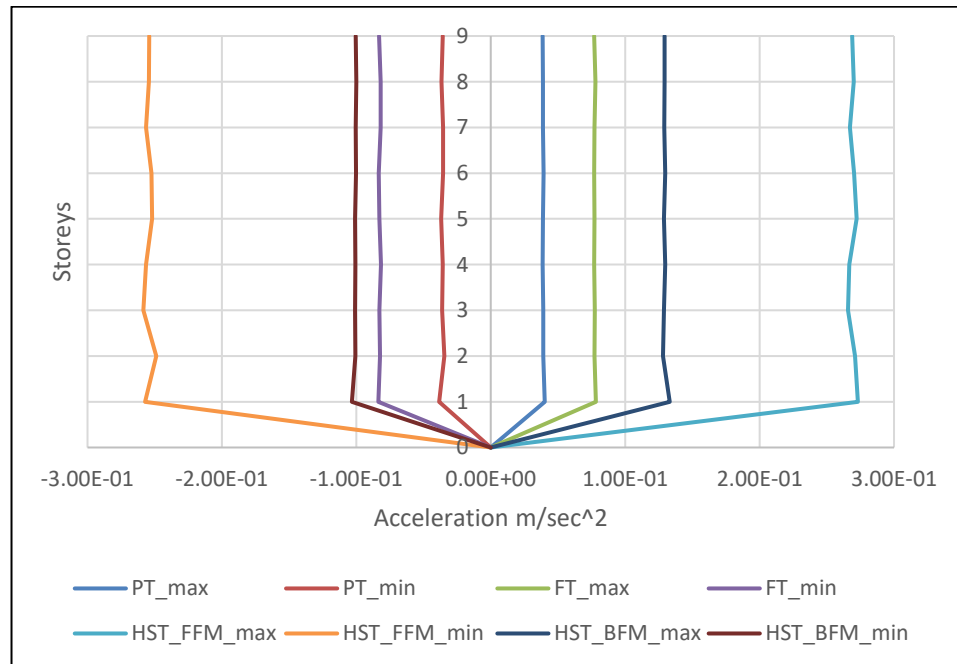


Figure 4.6: Acceleration distribution of model 'C'.

Normally, peak values are expected on the top floors but, in this model, peak acceleration and velocity occurred on the first floor (Figure 4.7). This can be explained that buildings are mostly deformed at a lower frequency content. Accordingly, the dominant frequency of train vibrations was between 40-60 Hz. Thus, vibrations do not have any significant effect on structures.

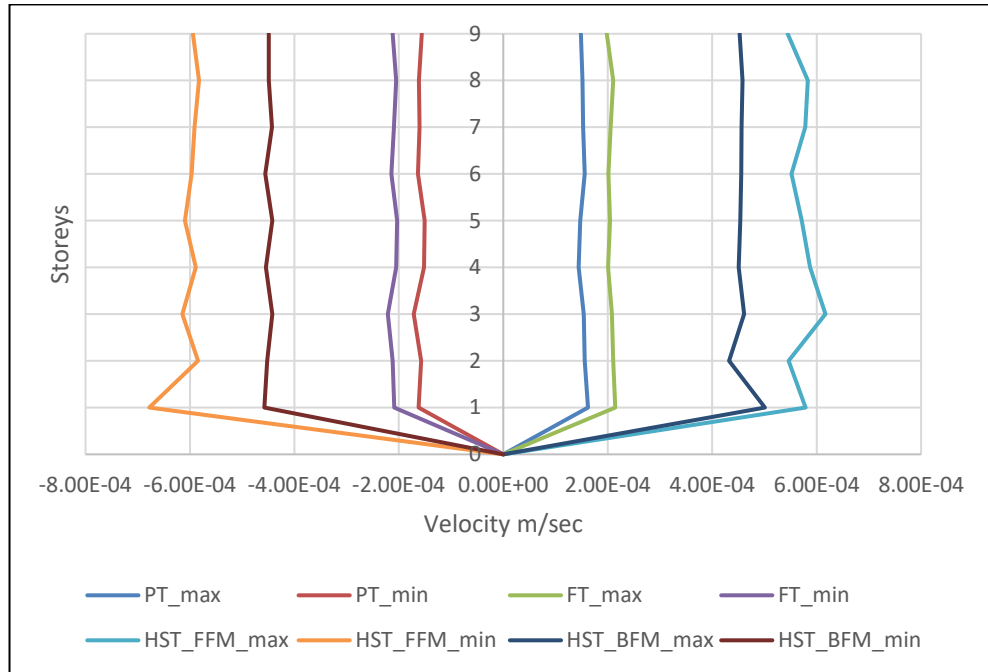


Figure 4.7: Velocity distribution of model 'C'.

Lastly, the displacement values evaluated for models. The results for model C obtained from the freight and passenger trains reached peak values at the rooftop floor as expected. However, models B and C resulted differently, reached peak displacements, like the high-speed trains, on the middle floors (Figure 4.8). This can only be explained by the content of recordings. First of all, freight and passenger trains passing periods are longer than a high-speed train. When a freight train or passenger train passes, they generate approximately ten seconds of vibration. On the other hand, a high-speed train generates only three seconds of vibration after a transition. The period of the vibration is rather too short, that recording ends without the opportunity for the building to oscillate. Thus, expected displacement could not be reached.

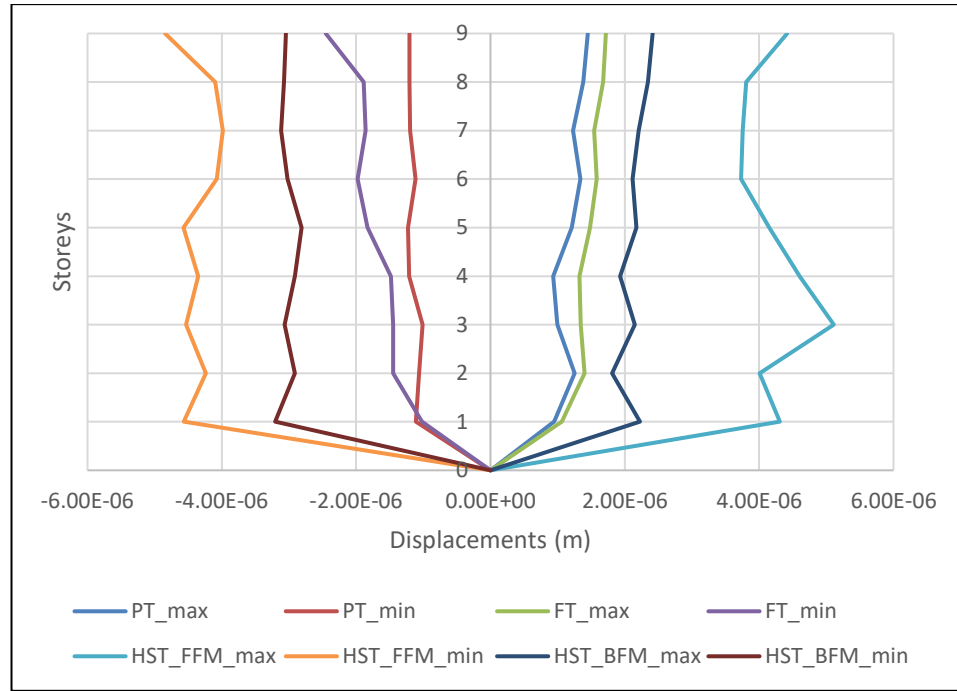


Figure 4.8: Displacement distribution of model 'C'.

4.2.1. Comparison of The Results with International Regulations

For the evaluation of the structural response, the outputs of velocity response values of frame models on the top floor subjected to each input motions are extracted. Most of the peak velocity values occurred in 8-10 Hz frequencies. The content of the recording may be answering these frequency levels. Since subjected recordings have a short period, they show similarity to blast-induced vibrations. As mentioned earlier, an earthquake motion generally occurs at structural fundamental frequencies, while a blast-induced motion occurs at higher frequencies [10]. The dominant frequency of the train induced ground motions subjected to the models is between 40-60 Hz, and peak velocities occurred close to 8-10 Hz. Therefore, all three frame models affected as such larger than seismic effect and lower than blast-induced vibration effect. The largest velocities occurred at the 3-story building. Since the heights of the buildings increase, the dominant frequency of the buildings gets minor values, which are too low than the dominant frequency of the input ground motions. Hence, velocities decrease with distance.

German Structural Vibration Standard namely, DIN4150-3, determined the guidance level of vibrations using peak velocity values in structures. In this study, vibrations at the floor for all frequencies evaluated in each frame model. The figure

below adapted from the tabular form of the guidance values in the standard (Figure 4.9). Line 1 is opposed to the sensitive buildings, line 2 is for dwellings and similar use of buildings and lastly, line 3 is for industrial buildings. The obtained response velocities can be seen in the figure. The values resulted under the guidelines of a structural damage. The largest velocity obtained is 0.5868 mm/s from high-speed train and that is under the line 3, even though all models were designed as RC type. As a result, train induced vibrations did not cause any structural response.

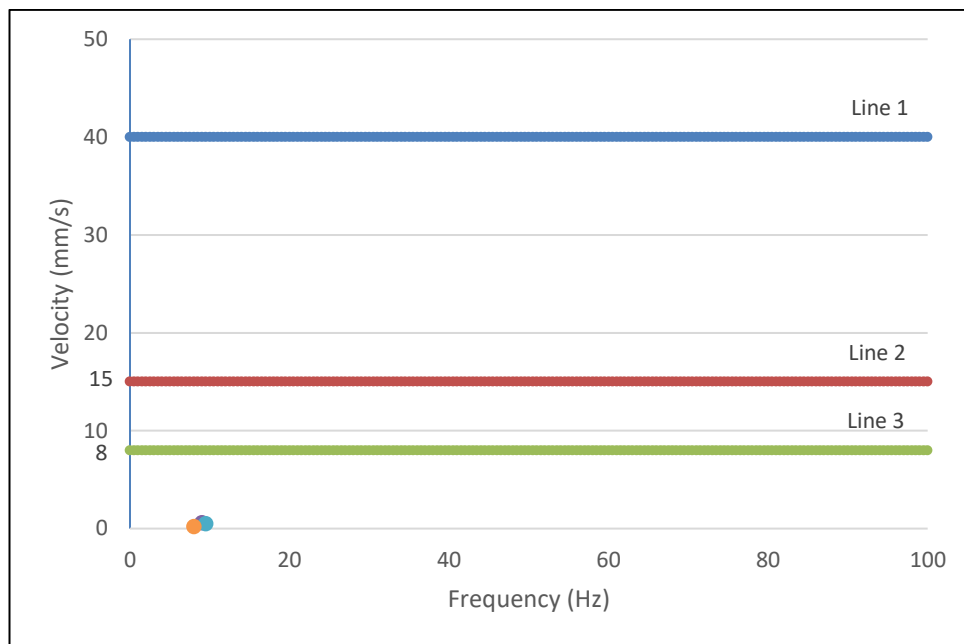


Figure 4.9: Comparison of peak lateral velocities with guideline values.

On the other hand, values are quite effective to be noticed by humans [19]. In this part, vibration levels compared with the Australian Vibration Standard, namely AS2670.2. The tabular form of vibration levels shown earlier in (Table 3.6) transformed into a graphic for comparisons.

Lastly, peak velocity values for a high-speed train, freight train, and passenger train plotted for comparison. According to the figure, nine out of twelve recordings are over “threshold of perception level” and three of the high-speed train records are also above “barely noticeable level”. This figure shows that a structure near a railway line is affected by train induced ground motion for human perception (Figure 4.10).

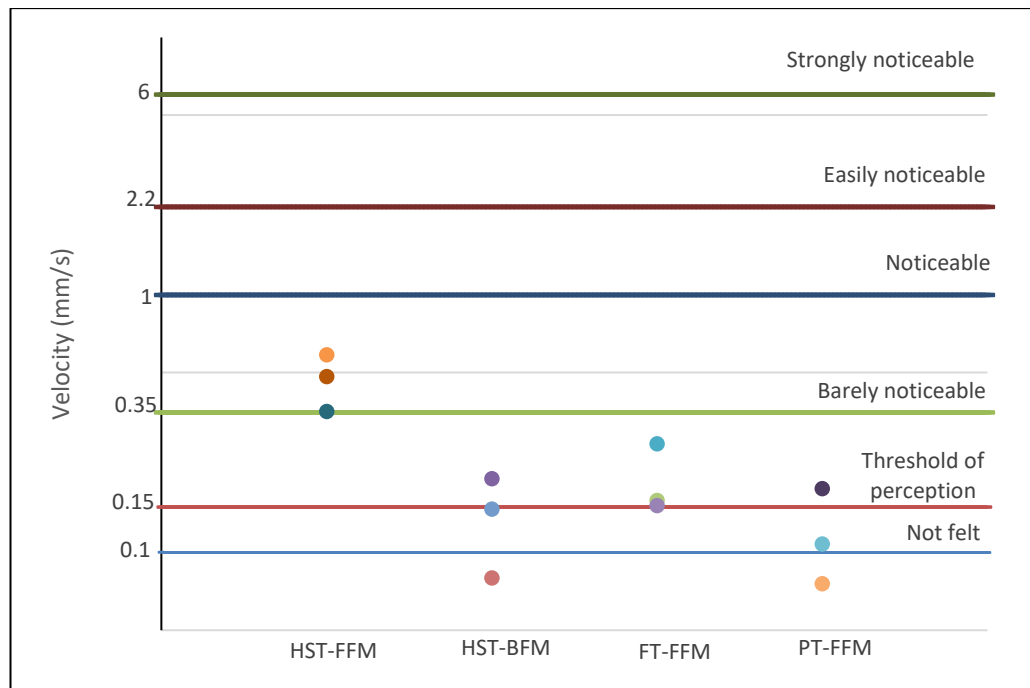


Figure 4.10: Comparison of peak velocities for human perception.

5. CONCLUSION

High-speed trains are recently being operated in Turkey, and more ongoing projects exist. The effects of train induced vibrations caused by high-speed trains taken into consideration gradually, especially with the increase of the railway lines passing through the urban center. In this thesis, the ground vibrations originating from the train and the effects of these vibrations on the structures examined. In the first phase, fieldwork carried out for the examination of ground vibrations. In these tests, the excavated trench barrier used to measure the vibration-reducing effect with the free field behavior. These recorded vibrations evaluated according to their acceleration, velocity, and frequency contents, as well as whether the train type was a high-speed, freight or passenger train. Three sample two-dimensional frame system structure is modeled for the interaction of the obtained results with the buildings. The distribution of the peak acceleration, velocity and displacement values obtained from the analysis results according to the story levels plotted and the results evaluated. Accordingly, the following results obtained:

- Ground vibrations effect on free field motion significantly reduced by the barrier.
- For the horizontal components, perpendicular component of the vibrations found more effective depending on the acceleration values, however, the vertical component achieved the higher values.
- The peak acceleration and peak velocity values decreased with the distance increased to the train line. The distance reduced the peak values as expected.
- The conducted field studies showed that the Pavement effect reduced the vibration level within the 14 meters from the track line. No clear pavement effect has been observed after 14 meters.
- Frequency contents are around 80 Hz from the stations adjacent to the train track line while in the stations aligned away from the railway line, it obtained around 20~40 Hz.
- The direct relationship of frequency content due to vibrations and distance has not been observed.
- The highest impact on building models caused by high-speed train passages comparing with the other trains.

- As the vibration duration of the high-speed train records subjected to models are too short as 3 seconds, the vibration ends before reaching to the top floor. Therefore, peak displacements occur on the intermediate floors and could not reach the expected displacements on the upper floors.
- Peak acceleration, peak velocity, and peak displacement values found very small in all three building models. The vibrations did not have a significant structural impact.
- The frequency contents of building models evaluated. Velocity response spectrum of models obtained and dominant frequencies occurred in 8-10 Hz for peak velocities which are in general between the typical frequency ranges of seismic motion and blast-induced motion. Since the duration of the records are short, the results show similarity to blast-type vibration motion.
- Models response velocities compared with DIN4150-3 for structural response and the obtained values were lower than the code assigned lowest limit values. Hence, the recorded train vibrations have not shown any significant structural effect on the well-designed RC type structures.
- Furthermore, response velocities compared with AS2670.2 for human perception, and approximately 75% of recordings exceeded the threshold perception limit. 25% of recordings exceeded also barely noticeable level. Thus, it can be said that, for buildings near the railway lines are affected in human perception level from train induced vibrations.
- Finally, barrier affected the vibration levels and reduced the velocity values significantly. Therefore, barriers can be suggested for buildings adjacent to the railway lines.

REFERENCES

- [1] Web 1, (2019), <https://www.railway-technology.com/features/feature-the-10-fastest-high-speed-trains-in-europe/> (Date: 31/12/2019).
- [2] Web 2, (2019), <http://www.tcdd.gov.tr/> (Date: 01/12/2019).
- [3] Celebi E., Zulfikar A.C., Kirtel O., Goktepe F., (2018), “In Situ Measurement of Environmental Vibrations Induced by High-speed Trains and Mitigation of Structural Vibrations”, Project No: 217M427, TÜBİTAK, Turkey.
- [4] Tekergul E., Zulfikar A. C., Celebi E., Kirtel O., Goktepe F., (2019), “Effect of Trench Barrier on Free Field Motion Due to The Train and High-Speed Train Passages”, 7th Eccomas Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, 862-870, Crete, Greece, 24-26 June.
- [5] Bata M., (1971), “Effects on buildings of vibrations caused by traffic”, Building Science Vol.6 Issue 4, 221-246.
- [6] Massarsch K. R., (2004), “Mitigation of Traffic Induced Ground Vibrations”, 11th International Conference on Soil Dynamics and Earthquake Engineering and 3rd on Earthquake Geotechnical Engineering, Keynote Lecture, Berkeley, 7-9 January.
- [7] Crispino M., D’apuzzo M., (2001), “Measurement and Prediction of Traffic-Induced Vibrations in a Heritage Building”, Journal of Sound and Vibration, 246 (2), 319-335.
- [8] Clemente P., Rinaldis D., (1998), “Protection of a Monumental Building Against Traffic-Induced Vibrations”, Soil Dynamics and Earthquake Engineering 17 (5), 289-296.
- [9] Kouroussis G., Florentin J., Conti C., Verlinden O., (2014), “Building Vibrations Induced by Railways: An Analysis of Commonly Used Evaluation Standards”, The 21st International Congress on Sound and Vibration ICSV21, Beijing, China, 13-17 July.
- [10] Hao H., Ang T. C., Shen J., (2000), “Building Vibration to Traffic-Induced Ground Motion”, Building and Environment 36, 321-336.
- [11] Yesilyurt A., Zulfikar A. C., Alcik H., Akram M. R., (2019), “Parametric Study for Assessment of Traffic-Induced Vibrations on Residential Buildings in Istanbul, Turkey”, (Submitted for Evaluation).
- [12] Dikmen S. U., Pinar A., Edinçliler A., (2015), “Near-surface Attenuation Using Traffic-Induced Seismic Noise at a Downhole Array”, Journal of Seismology 20 (1), 375-384.

- [13] Celebi E., Kirtel O., (2013), “Non-linear 2-D FE modeling for prediction of screening performance of thin-walled trench barriers in mitigation of train-induced ground vibrations”, *Construction and Building Materials* 42, 122-131.
- [14] Hunaidi O., (2000), “Traffic Vibrations in Buildings”, *Construction Technology Update* 39, Institute for Research in Construction, National Research Council of Canada.
- [15] Standard 1, ISO 2631-2, (1989), *Mechanical Vibration and Shock-Evaluation of Human Exposure to Whole Body Vibration Part-2: Continuous and Shock Induced Vibration in Buildings (1-80 Hz)*, Washington, USA.
- [16] Standard 2, German Standard DIN4150-3, (1999), *Structural Vibration-Part 3: Effects of Vibration on structures*.
- [17] Standard 3, Swiss Standard SN640 312, (1978), *Effects of Vibration on Construction*.
- [18] Standard 4, British Standard BS 7385-2 (1993), *Evaluation and Measurement for Vibration in Building Part 2: Guide to Damage Levels from Ground Borne Vibration*.
- [19] Standard 5, Australian Standard AS 2670.2 (1990), *Evaluation of Human Exposure to Whole-Body Vibration Part 2: Continuous and Shock Induced Vibration in Buildings (1-80 Hz)*, Australia.

BIOGRAPHY

Ezgi Tekergul, Master of Science student in Gebze Technical University. She was born in 1989 in Eskişehir. She graduated from Kılıçoğlu Anatolian High School. She graduated from the Civil Engineering department at Osmangazi University. She majored on the Earthquake and Structural Engineering. She took a part in a TUBİTAK research project titled “Environmental Vibrations Induced by High-speed Trains and Mitigation of Structural Vibrations” recently. She worked as a technical office engineer for Erdoğan Construction Ltd. Şti. in province of Eskişehir, between 2011-2012. She is been working for AFAD, Disaster and Emergency Management Presidency, as a civil engineer and also as structural engineer for AFAD INSARAG certified urban search and rescue team since 2013.