

CONSIDERATIONS OF WAVE- TRANSMISSION FROM SOIL INTO STRUCTURE BASED ON NUMERICAL CALCULATIONS

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À minha família

...é meu ofício e exercício andar pelo mundo endireitando os tortos e desfazendo os agravos.

D.Quixote, XIX

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The Department for Environmental and Emission Protection, *iC-Consulenten* has important tasks as consulting problems related with issues of noise and ground vibration generated by train traffic. When is intended to construct new buildings near a rail line the issues inherent to the train operation may need to be carefully assessed. The ground-borne vibration, due to the train traffic, may cause problems in occupants' health and comfort in cases where those limits are exceeded, requiring measures to control it. One current countermeasure applied in the receiver (foundation slab) is the polymer material, the resilient bedding for buildings. However, this material presents an important shortcoming: the high costs of the material and its application. In order to find an effective and cheaper solution to solve this problem, or at least in cases where the ground vibration felt doesn't justify the application of such expensive countermeasures, Michael Österreicher proposed for my final thesis the study of the effect of the foundation geometry slab on the reduction of ground vibration, in order to find potential advantages with the slab foundation geometry.

Finished this report I would like to express my deep and sincere thanks to all people that somehow contributed in the elaboration of this report. Individually, I want address my thanks:

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ABSTRACT

The implementation of new communication networks in sensitive areas (densely populated, environmental protected areas, etc.), or the construction of new buildings near transportation routes reveals to be often a complex problem. Despite the potential benefits that new communication networks can bring for a given region, it is frequent the opposition from the population to new routes crossing their region. Such feeling is due to the deterioration of life quality that the operation will bring to those residents. Pollution, visual impact, noise, vibration are some aspects that contribute for the discomfort of the population.

Concerning to the train traffic, the noise, the air-borne noise (in the case of tunnels), the noise generated due to the vibration transmitted to the structure and the vibration are the main reasons of discomfort for the population which can be a cause of diseases.

The noise is generated from the contact of the train components and rail reaching the buildings by air. The vibration due to the train traffic results from the transmission of the wheel loads to the ground under and beneath the track, which propagates through the soil till reach the building foundations. In certain situations, the induced vibration by train traffic doesn't exceed the established limits, however the population claims against the vibration generated by the train traffic. This situation is explained through the phenomena of resonance, where the frequency vibration transmitted to the building can reach the natural frequency of some building components. The effect of "loudspeakers" of walls, with rattling of shelters, shows how this problem is amplified. This phenomenon is called the structural noise or re-radiated noise.

In order to solve this problem there are several solutions which can be applied in the source, in the propagation path and/or in the receiver. The mitigation measures applied in the source are the most common once present the better relation costs/effectiveness. However, when it is intended to construct new buildings near railway lines, in most of the cases is not possible to apply the countermeasures in the source, so the mitigation measures in the receiver should be taken.

A mitigation measure applied in the receiver is the rubber resilient bedding for buildings. This material is applied in the foundation slab and in the sustaining walls. This solution can in some cases reduce the vibration transmitted to the remaining building around 50% of the original vibration. However, the material itself and its application presents a high costs and in certain cases, even requires measures to reduce the vibration measured; so this doesn't justify the application of such expensive countermeasure.

In order to find a solution that can be economically more interesting, it was made a parametric study of the foundation slab thickness, using the finite differences software FLAC. This study was performed in models 2D and had as principal purpose to find a law that could be used to avoid excess of vibration.

This study concludes that by increasing linearly the foundation slab is achieved a reduction of the ground vibration absorbed in the order of 20~25%. Thus, for situations where the ground vibration slightly exceeds the limits the solution can pass by increasing the slab. It is a practical and economical option, since the cost of the material resilient bedding and its application present costs around 150€/m² and the concrete is 100€/m³. For buildings with a high implantation area the costs difference will be important.

KEYWORDS: ground-borne vibration, train traffic, FLAC, foundation slab thickness, reduction of ground vibration.

RESUMO

A implantação de novas vias de comunicação em zonas sensíveis (focos populacionais, zonas de protecção ambiental, etc.), ou a construção de novos edifícios junto a vias de comunicação revela ser muitas vezes um problema complexo. Apesar dos potenciais proveitos que novas redes de transporte podem trazer para determinada região, é frequente assitir-se à oposição da população à passagem de novos traçados nas suas localidades. Tal sentimento de oposição deve-se essencialmente ao deteriorar da qualidade de vida resultante da operação nessas vias de comunicação, tais como a poluição, impacto visual, ruído, vibração, etc...

No que diz respeito ao transporte ferroviário, o ruído transmitido “via aérea”, gerado pelas vibrações da estrutura, e as vibrações são o principal motivo de desconforto para as populações e que podem ser a causa de doenças.

O ruído transmitido “via aérea” é gerado pelo contacto dos material, das peças do comboio, propagando-se através do ar. As vibrações geradas pelo tráfego ferroviário resultam da carga transmitida pelo comboio ao solo envolvente propagando-se por este até alcançarem as fundações dos edifícios. Em determinadas situações as vibrações induzidas nos edifícios devido ao tráfego ferroviário não ultrapassam os limites estabelecidos, no entanto o tráfego ferroviário é objecto de grande contestação por parte da população. Esta situação resulta do fenómeno de ressonância que as vibrações possam produzir em certas partes do edifício. Quando as frequências das vibrações igualam a frequência natural de paredes, estas amplificarão o ruído, funcionando como “altifalantes”. Este ruído é conhecido como ruído estrutural ou “re-radiated noise”.

Com vista a resolver o problema das vibrações geradas pelo tráfego ferroviário existem diversas soluções a aplicar na fonte de vibração, no meio de propagação e/ou no receptor. As medidas aplicadas na fonte de vibração apresentam melhor relação custo/eficácia e como tal são as mais usadas no controlo da vibração. No entanto, em situações onde se pretende construir novos edificios perto de linhas ferroviárias já existentes ter-se-á de adoptar medidas no receptor, isto é, nas fundações do edifício.

Uma das medidas aplicadas nas fundações são as “camas elásticas” que consistem num polímero aplicado na laje de fundação e paredes de contenção. Esta medida revela-se de certo modo eficaz na medida em que reduz até 50% as vibrações transmitidas ao edifício. No entanto, esta medida apresenta custos elevados. Em certas situações as vibrações produzidas, apesar de necessitar de medidas de mitigação, não justificam a aplicação deste material.

Com vista a encontrar uma solução interessante em termos económicos recorreu-se ao software de diferenças finitas (FLAC), para se realizar um estudo paramétrico, em 2D, acerca da espessura da laje tendo em vista as alterações produzidas nas vibração recebidas pela laje.

Conclui-se com elaboração desta tese que incrementando linearmente a espessura da laje poder-se-á obter um redução efectiva das vibrações na ordem dos 20~25%. Assim para casos de menor gravidade, o incremento da espessura da laje da fundação usando betão corrente poderá constituir uma opção viável e económica. Tendo em conta que o custo e aplicação das “camas elásticas” terá um custo a rondar os 150€/m² e o preço do betão é de 100€/m³. Para edifícios com grande área de implantação o custo total será muito significativo.

PALAVRAS-CHAVE: vibração induzida, tráfego ferroviário, FLAC, espessura da laje da fundação, redução da vibração.

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NOTATION

$A(8)$	– Daily vibration exposure
$A_1(8)$	– Daily vibration exposure in direction 1
$a_w(t)$	– Weighted acceleration
c_c	– Critical damping
c_v	– Viscous damping ratio
C_p	– P-waves velocity
C_R	– Rayleigh waves velocity
C_S	– Shear waves velocity
D	– Dynamic magnification factor
$f(t)$	– Excitation load
$f_D(t)$	– Damping force
$f_i(t)$	– Inertial force
$f_s(t)$	– Spring force
G	– Shear modulus
$H(p)$	– Transfer function
$H_h(p)$	– High-pass filter
$H_l(p)$	– Low-pass filter
$H_t(t)$	– Pure weighting function
k	– Elastic constant
K	– Bulk modulus
k_1	– Coefficient dependent of the direction
m	– Lumped mass
$p(t)$	– Excitation load function
p_0	– Amplitude load
r	– Static defection
T	– Vibration period
T_i	– Exposure duration
$\ddot{u}_g(t)$	– Ground acceleration
$x(t)$	– Displacement
$\dot{x}(t)$	– Velocity
$\ddot{x}(t)$	– Acceleration

W_m - Band-limited frequency weighting

θ - Phase angle

ν - Poisson ratio

ξ - Critical coefficient ratio

ρ - Density

ω - Angular frequency

ϖ - Excitation frequency

ω_d - Damped frequency

ω_n - Natural frequency

ANSI – American National Standards Institute

DFT – Discrete Fourier Transform

FDM – Finite Difference Method

FEM – Finite Element Method

FFT – Fast Fourier Transform

FLAC – Fast Lagrangian Analysis of the Continua

HST – High Speed Train

ISO – International Standard Organization

MDOF – Multi-degree-of-freedom

r.m.s – Root-mean-square

SDOF – Single-degree-of-freedom

1

INTRODUCTION

1.1. BACKGROUND

In a society increasingly global the evolution of transportation networks are of critical importance. The numerous air links offered to european customers is a reflex of the politicians took that aim a bigger mobility within Europe and all over the world. With respect to the train transportation, European Union has being promoting different programs to encourage the member-states to build an efficient european high-speed network with interoperability between the different countries to suppress the actual weaknesses of the transportation system of persons and goods. The most recent and serious failure of the actual system happened with the eruption of the icelander volcano *Eyjafjallajökull* on the year 2010 stopping the air traffic in almost all Europe, causing large amounts of losses.

The extensive investments made by the governments to improve the actual railway system, in new infrastructures to enable the people and goods to move in a modern, sustainable and efficient with reduced costs, incremented the necessity to create more effective railway network and new machines, more powerful and faster. However, the train transportation brings some problems for the railway line neighborhood related with its operation, as the ground-borne vibration, ground-borne noise (in the case of tunnels) and the air-borne noise. These problems become more serious when the railway line crosses sensitive areas as inhabited or environmental protected areas.

In 1847 with the first railways in England between the cities Manchester and London, Sir James South was charged to evaluate the consequences of the passage of trains near the *Royal Observatory of Greenwich*. The method to evaluate was based on studying the effects of ground-borne vibration caused by the passage of the train at different velocities and to find velocity limits in order to minimize the vibration effects.

Particularly since the 1960s, environmental noise has become an increasingly important issue. Noise is often identified as a source of dissatisfaction with the living environment by residents. With the increasing of noise levels the population became aware of the potential noise impact that new railway lines could bring. Opposition to new projects that can cross their “neighborhood” is the evidence that the populations are afraid to loose life quality.

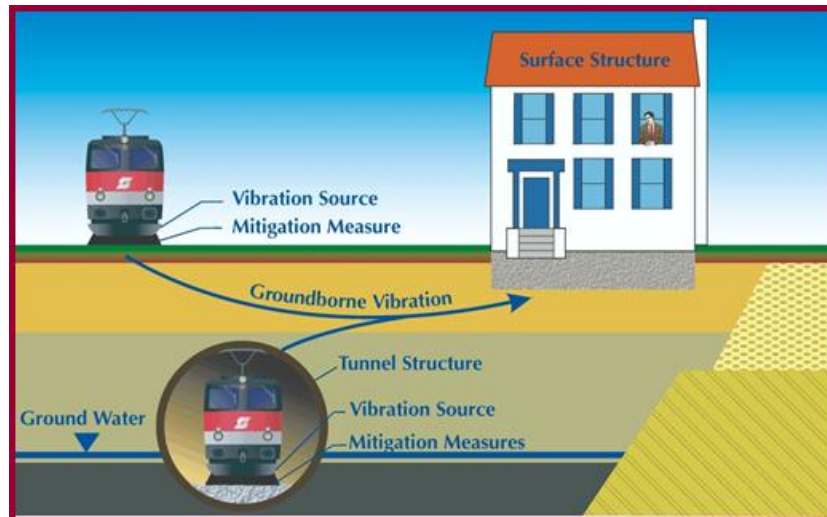


Fig. 1– Ground-borne vibration (Unterberger, 2004)

Nowadays, new railway projects take seriously the effects caused by the train traffic on the life quality of the populations around the railways, being very important to project the new railway in order to serve the interest of the owner and respect the life standards concerned with air-borne noise and ground-borne vibration. To solve these problems it shouldn't be studied separately from each other the different aspects of train traffic, once many times they have a common source and often the countermeasures taken can solve part of the two problems.

As the theme of the thesis indicates, this study will be focused on the problem related with ground-borne vibration. This problem can be subdivided into three categories according the different types of railway situation (Fig. 1): the heavy axle-load freight traffic, travelling at relatively low speeds, causing high amplitude vibration at the track; the high speed passenger trains that travel at speeds higher than the wave speed of vibration in the ground and in the embankment; and the trains running in tunnels which transmit vibration to buildings above and around them. To do the study of ground-borne vibration, the problem should be interpreted in three distinct parts: the generation of waves as result of the interaction between the railway track with the soil beneath while the train passes; the propagation path traversed by the seismic waves; and the interaction between the soil and the foundations of the building.

To perform this study it will be used the FLAC – Fast Lagrangian Analysis of the Continua, developed by Peter Cundall, Itasca Software, in 1986. The code is an explicit two-dimensional finite difference program that performs a Lagrangian analysis.

1.2. OBJECTIVES AND ORGANIZATION OF THE THESIS

The study of the present topic, ground-borne vibration due to the train traffic as mentioned, can be subdivided into three categories. This thesis will be focused in the interaction between soil and the foundation of the building. Using a software developed for geotechnical analysis to perform static and dynamic analysis, FLAC – Fast Lagrangian Analysis of Continua, the influence of the foundation slab characteristics in wave reception will be studied, specially the thickness. The study will be done through the calculation of several models 2D with the slab located along the propagation path. The slab modeled should present a constant thickness along its length. The slab will support a constant

tension that represents the weight of the building. The building itself won't be modeling because the FLAC software proved that is not reliable, Travanca [10].

With this report it is expected that in some cases where the ground vibration, due to the train traffic or another source, exceeds the legal limits, a solution can pass by increasing the foundation slab, avoiding this way expensive mitigation measures against the ground vibration, especially the mitigation measures that are applied in the receiver.

For example, the material Cibatur is placed between layers of concrete, significantly reducing the ground vibration. The material plus its application in the foundation slab costs around $150\text{€}/\text{m}^2$. In cases where isn't necessary such reduction, because in the source there are already countermeasures in the source, however insufficient, or the building construction site is located far away that doesn't justify an expensive countermeasure. So, with an increase of the foundation slab it is possible to reduce costs of installation and conservation, construction times, etc. So the use of a thick slab to reduce the ground vibration can be an attractive solution, since the cost of material (a m^3 of concrete, in Austria, costs around 90€) and its application would be cheaper and faster. This report expects to answer to this answer.

The organization of the thesis is as follows:

The chapter 1 gives a background of problems caused by train traffic and describes the objectives of the report.

In chapter 2, the aspects related with the ground-borne vibration will be discussed. The three components of the generation, propagation and reception will be analyzed and its influence in all process. The effects of vibration on human response and impacts on buildings are described and the limits imposed by the austrian standards. In the cases which the legal limits are not respected are presented some countermeasures and its application framework.

The chapter 3 will present the fundamental concepts that are important to understand the dynamic analysis. The dynamic material model that was assumed is the elastic, isotropic scheme, since the strain levels produced by the ground vibration due to train traffic are very lower. It is developed the expressions for dynamic analysis and for signal analysis. It is also presented the seismic waves that carry the energy produced by the train traffic.

In the chapter 4 it is presented the study of the ground-borne vibration. Firstly the FLAC software, terms and concepts, as its calculation algorithm is presented. This software presents a finite difference method (FDM) which presents advantages and disadvantages when compared with finite element method (FEM) that it will be explained forward. It is also presented the FLAC characteristics that make it a powerful tool to evaluate the vibration propagation through the half-space as the silent boundaries, Rayleigh damping, etc. Also the models characteristics which interest calculate will be introduced, as the element size, its dimensions and the dynamic load applied.

Chapter 5 presents the material used in *iC-Consulenten* to do the assessment of ground vibration. Additionally, is presented some resumed data, obtained from measurements done. These results are of extreme importance, since it is based on that it possible to have substantiated conclusion of the model results.

Chapter 6 presents the results obtained. As it was stated before, the aim of this report is to obtain the variation law of ground vibration reception with the variation of the slab thickness, for this reason the results presented are not absolute values but relative values. I.e., the decreasing of vibration reception

when increasing the slab thickness is made in function of a thickness slab of 0,5m, since corresponds to the high value for the 4 thicknesses admitted. The variation of the frequency received for different thicknesses will be also studied in this report.

Finally in the chapter 7, a global appreciation of the work is done.

2

STATE OF THE ART

2.1. INTRODUCTION

It is consensual that studies done about the ground-borne vibration due to the train traffic and the prediction of their influence on buildings, sensible areas or humans within a building can be subdivided in three main “propagation components”, with different complexity according the location, type of train or project phase. These three parts consist of the source, the propagation path through the soil and the receiver and its wave propagation within the building. The generation of vibrations due to the passage of trains on the track line or vehicles on the road is the source; the propagation path corresponds to the path traveled by the waves, from the source to the receiver; the receivers are residential areas, a single building, etc. This chapter consists of a brief review concerning the subject of ground-borne vibration due to train traffic and the importance of each component dealing with this topic.

2.2. GROUND-BORNE VIBRATION DUE TO THE TRAIN TRAFFIC

The wave-transmission from soil into structure is a complex process which involves several factors, many of them are unknown or approaches based on similar cases. The study of the transmission of train-induced ground vibrations can be separated into several distinctive stages; each of them represents its own characteristics to provide an input to the next stage in the process. In the Fig. 2, one can see the different parts concerning this subject. The stages of this process are divided in the source, the propagation path and the receiver. At this step, it is assumed that the distance between the railway track and the building is large enough to consider that the presence of the building doesn't affect the vibration generation.

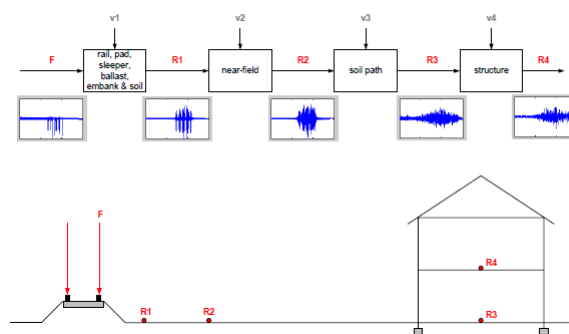


Fig. 2 – Distinctive stages of the study of train-induced ground vibration [1]

Due to the fact that there are a lot of different train construction types, with various technologies in different states of operation, the relative importance of the components responsible for the generation of waves isn't unanimous, and varies according the model and stage of the project.

Also the propagation path, which corresponds to distance between the source, the axis of the railway line, and the receiver, the building foundations, is very difficult to determine exactly in its properties. In finite elements models (FEM), it is usually assumed that the soil is constituted by one or more homogeneous layers. This assumption doesn't correspond to the reality. In spite of the use of representative values, there are small volumes of soil with completely different characteristics inducing changes on the comportment of the waves. Due to these uncertainties described above, measurements should be applied to determine the important soil parameters. Usually this is done by seismic investigations (for example refraction seismic).

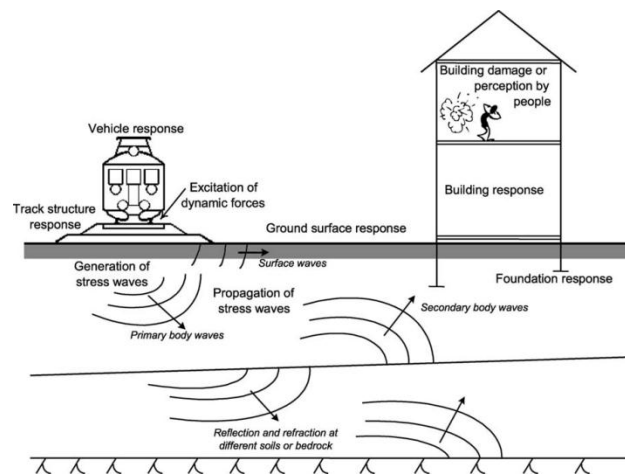


Fig. 3 - Wave-transmission of ground-born vibration [2]

The receiver represents the inhabited area or special buildings, like hospitals or schools. However, the inhabitants of buildings don't experience the ground-borne vibrations with the same severity and sensibility. Although the ground floor (foundation) is the receiver of ground vibration is not necessary the most affected by vibration, since the different natural frequencies of the different floors can cause higher vibrations due to the phenomenon of resonance.

2.2.1. VIBRATION SOURCE

The vibration source mainly consists of the train properties (for example the suspension) and the type and the constitution of the track way itself. On this topic there is a respectable research about the importance of train suspension system, the railway type of construction, the importance of the embankment, the importance of the maintenance, etc. In this section the authors Bahrekazemi [1] and Hall [2] which can provide general notion about the vibration source will be presented.

According to Bahrekazemi [1], the vibration source of the ground-borne vibration due to train traffic is approved as a result of the interaction between the moving train itself and the rail way tracks. To find out how the vibration are generated it is essential to understand how each component of the vibration source works: the interaction between the moving train, the track and the soil mounted on, the load of

the moving train is transmitted through the train suspension to the track and finally to the ground under and around the track.

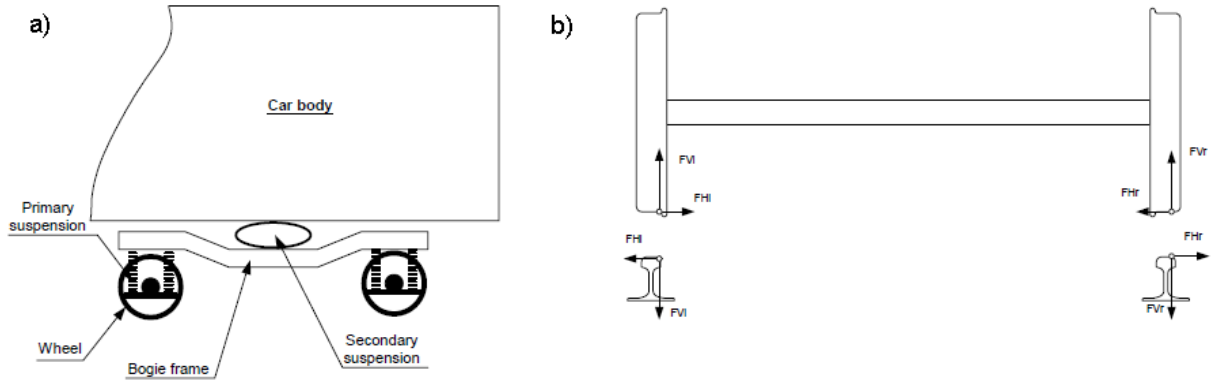


Fig. 4 – a) Train transmission system [1]; b) Wheel forces [1]

Although there are many types of trains with a different state of maintenance, different technology standards, maximum speed level or the type of traffic, the basic transmission of loads to the track is similar in all of them. The main parts of the train considering the excitation of vibrations are schematically shown in Fig. 4 a). The load of the moving train is transmitted to the bogie through secondary suspension which in the new trains consists of an air bag. The load is then transmitted to the wheels via primary suspension and the wheels in turn transfer the load to the track lines as shown in Fig. 4 b).

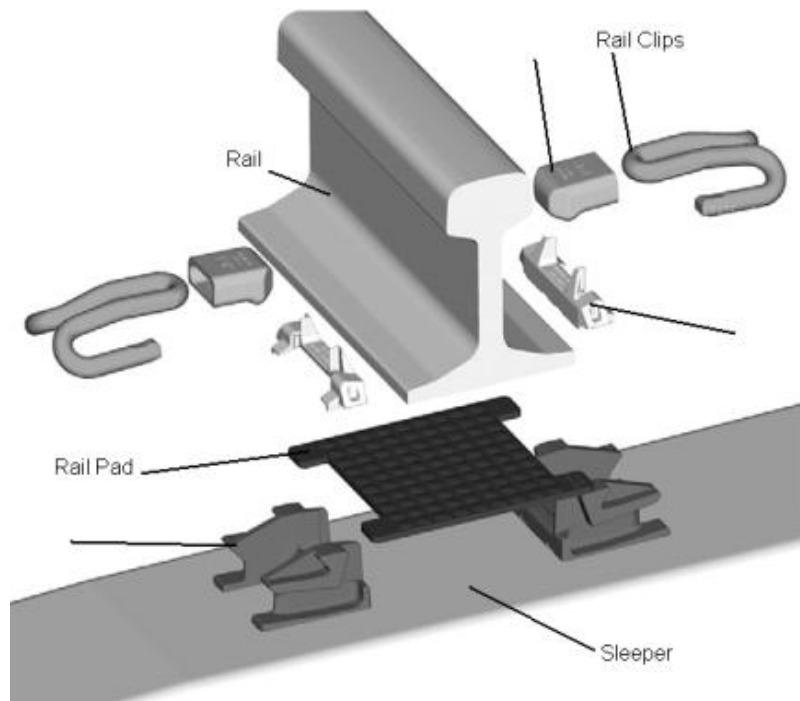


Fig. 5 – Rail fastening system

The transmission of the load from the track to the ground beneath and around the track line depends of the type on track way ballast. In order to control the ground-borne vibration and the ground-borne noise, different methods have been developed for the construction of the tracks. Its importance has grown following the emergence of trains more powerful and faster reflecting it in new materials, as the case of rubbers, and the installation of sleepers welded.

The different trackway types can be subdivided in two main groups: the ballasted tracks, shown in Fig. 7 a), and the non-ballasted tracks, Fig. 7 b) and c). The ballast track is the most common type of railway technology applied in the railroad networks. In the Fig. 7 a) the rails are held to sleepers which in turn are supported in a layer of stones (ballast). The Fig. 5 shows detail drawing of a typical rail fastening system. The ballast thickness is typically 30 cm to 40cm under the sleepers overlying in subballast which usually is 40 cm. An important component of the track, which has a considerable influence on the rolling noise behavior, is the rail pad. It is made of rubber and is located between the rail foot and the sleepers. These pads have the primary function of protecting the sleepers, particularly the concrete ones, from high impact loads which may cause cracking, and at same time have an important role on vibration and noise reduction of resulted by interaction between the rail and the sleepers, due to their soft stiffness (Fig. 6).

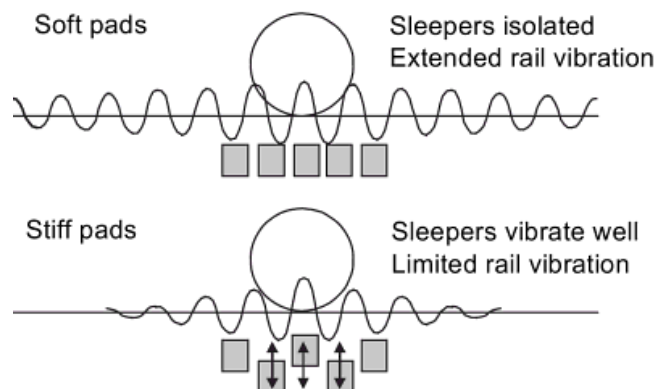


Fig. 6 – Effect of ballast pads

The non-ballast, slab and floating slab track, have been used increasingly in recent years especially in rail way tunnels. Its use reveals to be more efficient in reducing the ground-borne vibration and the noise vibration than in ballasted tracks, due to construction of the track by different layers with different materials which can filter the different frequencies excited due to the passage of the train. The effective reduction of the vibration from the passage of trains and the low maintenance required provided by non-ballasted tracks, as for example the floating slab makes it interesting to install in tunnels and in structures as bridges. However, due to the costs of installation, its implementation should be made only in areas where it is important to have a significant reduction of the problems already enounced.

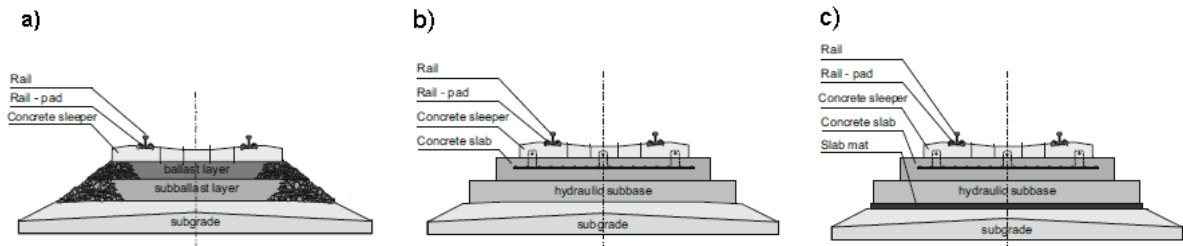


Fig. 7 – a) Ballasted track; b) Slab track; c) Floating slab track [3]

It is important due to the fact that there are many types of tracks with different types of technology, with increasing costs according to their complexity and efficiency, to make an accurate evaluation of the vibrations generated by the passage of trains and the occupation of the area, presently and hereafter, around the line. In the case of track line with no measures against the ground-borne vibration and noise vibration, the construction that will arise around the line will need countermeasures in the receiver (building) that will be more expensive and less effective.

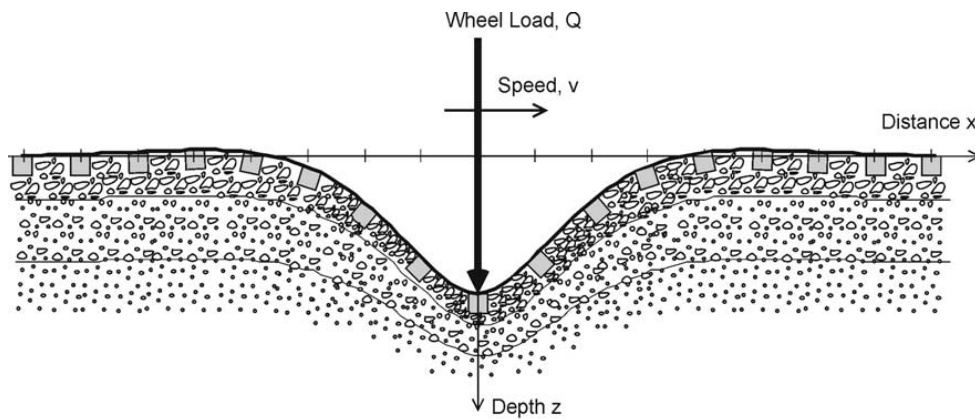


Fig. 8 – Variation of stress patterns (Hall)

The vibration excitation, according to Hall (2000) has as a principal factor: the variations of stress patterns in the ground beneath and around the track caused by the load of the train movement, showed in Fig. 8. A variation of the ground reaction (this mechanism is also called track structure response), caused by the moving stress pattern will induce stress waves, even in the absence of imperfections or periodic irregularities in the vehicle or track (Dawn and Stanwaorth, 1979) jointly with many other sources supplementing vibrations caused by moving stress patterns.

The Table 1 presents different factors which have influence on train-induced ground vibration. This is a collection from several authors brought together by Hall. Some of these features will only produce local effects while others will create a regular stress pattern moving with the train. The extent of these features will depend on the train velocity and its weight.

Table 1– Different factors influencing the level and characteristics of train-induced ground vibrations [2]

Stress waves induced by the track structure response
Axle weight
Spacing of wheel axles
Speed of train
Vibration source at wheel-rail interface
Unsteady riding of the vehicle (bouncing, rolling, pitching)
Dynamic properties of the vehicle bogie
Wheels defects (eccentricity, imbalance, flats)
Misalignment of motors
Acceleration and deceleration of train
Discontinuity on the track
Rail defects (unevenness, waviness)
Spacing and interval of rail joints
Switches
Curves and tilting track (centrifugal forces)
Variable support
Geometry, stiffness and spacing of sleepers
Geometry, stiffness and heterogeneity of the ballast
Stiffness and geometry of the ground

2.2.2. PROPAGATION PATH

The vibration generated in and around the railway track due to train traffic will propagate to the surrounding soil. The geometrical damping, the stiffness and the material damping of the surrounding soil and rocks will determinate the degree and extent spread of the ground vibrations. A variety of waves are induced in the media, but for the purpose of the subject ground-borne vibration the focus is based on the study of compression waves, shear waves and especially the Rayleigh waves. Each of these wave-types has different propagation velocities as well as the particle motion and will diminish differently in intensity with the distance increasing from the source. The propagation of different types of waves is further influenced by reflection and refraction as the propagating waves pass through different materials, as it's represented in the Fig. 3.

The distribution of ground-borne energy depends strongly on the magnitude excitation and will neither be equal through the space or the types of waves. If the excitation is localized in a small area, compared with shear wave wavelength, the R-waves will carry most of the energy. Wood (1968) proved that 67% of the energy in small areas is carried by surface waves. If, however, the excitation

area is large, body waves will be dominant (Heckle, Hauck & Wettchureck, 1996). The Fig. 9 shows the distribution of the different types of waves [1].

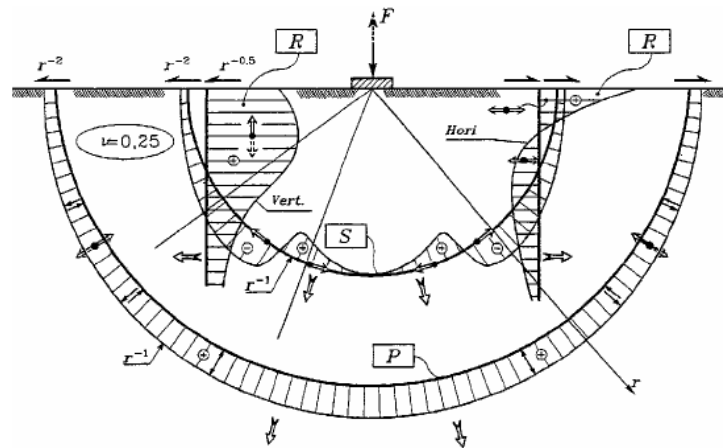


Fig. 9– Propagation path (Woods, 1968)

In the study of railway traffic induced ground-borne vibration most of the energy will be carried by the lower frequency waves: the R-waves. This happens because the dimensions of the embankment are generally small compared with the shear wave wavelength, so the embankment will work as a filter of high frequency waves (Bodare, 1993).

2.2.3. RECEIVER

The foundations of a certain structure works as the receiver of the ground-borne vibrations. The importance of the vibration absorbed by the foundations, as was expect, depends on several aspects, as the type of construction, type of foundations and the type of soil where the foundation is located, respectively. In some soft soils, like soft clays, the vibrations due to the train traffic can be felt at 200 m from the source.

According with Barehkazemi [1], only the low vibration frequencies can be absorbed by the building foundations. From the range of vibration frequencies produced by train traffic, the frequencies between 1-60Hz are potentially absorbed by foundations and transmitted to the remaining structure.

The vibration generated due to the train traffic propagates until the building foundations. From the building foundations, the vibration propagates to the remaining structure. The principal objective of this thesis will be focused in the foundation transmission characteristics. It will be discussed the influence of the length and thickness of the foundation slab, with the finite differences software (FLAC).

2.3. EFFECTS OF VIBRATION

Noise and vibration assessments are very important for the environmental impact assessment process for mass transit transports. The ground waves travel through the various soil and rock layers to the foundations of nearby buildings. After being received, the vibrations propagate through the structure. Generally the ground vibrations itself are not the principal problem for inhabited areas. The real

problem occurs when the ground vibrations contents of frequencies which are similar or equal to the natural frequency of the floors and/or walls in buildings. This can on the one hand cause the effect of loudspeaker, the so-called re-radiated noise or structure-born noise, schematized in Fig. 10. On the other hand the vibrations itself can be perceptible. Because of that, the effects of ground-borne vibration due to the train or road traffic make the analysis of the real effect on the occupants inside the building very difficult.

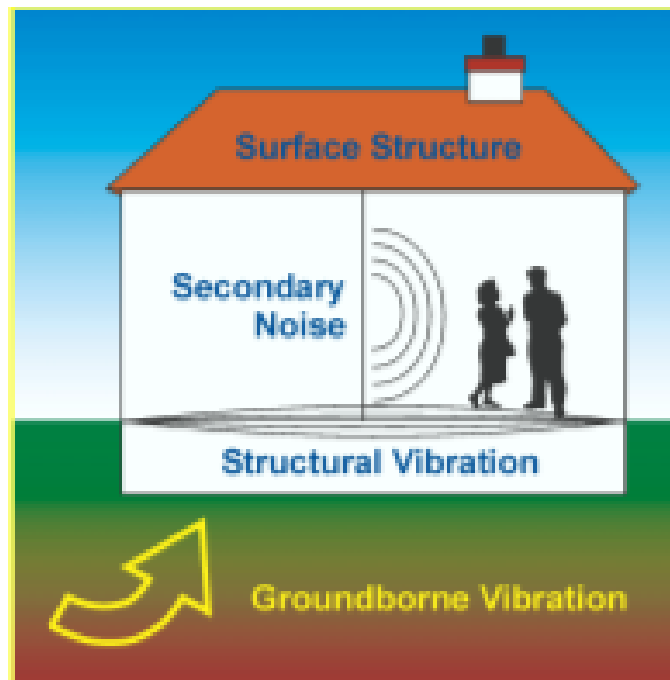


Fig. 10 – Effects of vibration (Unterberger, 2004)

2.3.1. HUMAN RESPONSE

One of the major problems to develop a suitable criteria for assessment of ground-borne vibration is that there has been relatively little research into human response to vibration, in particular human annoyance due to building vibrations. In 1983, the American National Standards Institute (ANSI) developed a criteria which describes a connection between the human response and a measured vibration value. The results basically show that it was not possible to measure the human response based only on the magnitude of the measured vibration. In some cases the occupants of a building complain it, even when the vibration values recorded were lower than a given threshold. The criterion developed by ANSI, make an evaluation of the human response combining the physical aspects of the ground vibration such as amplitude, frequency and duration with psychological aspects related with the population such as gender, age, occupation, etc.

The data represented in Fig. 11, present in DOT-293630-1, illustrates the relationship between the vibration velocity measured in 22 objects and the subjective response of the occupants to the vibration. This data was assembled from vibration measurements connected with subjective ratings by researchers and occupants. The data shown in Fig. 11 indicates that residential vibration exceeding approximately 0.143 mm/s (75dB) is unacceptable for a repetitive vibration source, such as a train that passes every 5 to 15 minutes. Additionally, it shows a curve indicating the percent of people

annoyed by vibration from HST in Japan. The conclusion of this study indicates that a particle velocity over than 0.143 mm/s will find the vibration annoying.

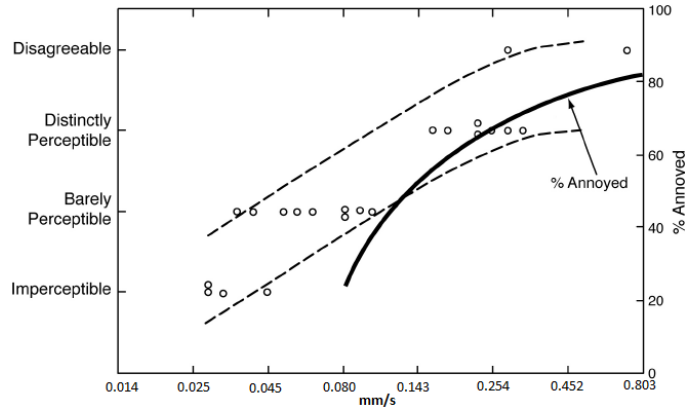


Fig. 11 – Response to residential building vibration 4 to 15 rapid transit trains per hour [4]

The International Standardization Organization (ISO), in 1979, adopted similar criteria to the ANSI, and in 2003 was revised resulting in ISO 2631-1: 2003. This document assesses the human response due to the ground-borne vibration according several factors which can be divided into two categories. One concerns the physical aspects of the ground-borne vibration such as amplitude, duration and frequency content; another category contain the factors related with the population, such as age, gender, population type, etc.

In order to measure the human response, ISO 2631-1: 2003 suggests calculating the *r.m.s.* (root-mean-square) values from the measured vibration. This value of *r.m.s.* can be obtained through the following equation

$$r.m.s = \sqrt{\frac{1}{t_2 - t_1} \int_{t_1}^{t_2} f(t)^2 dt} \quad (1)$$

where $f(t)$ represents a function within a time window t_1 and t_2 . The basic value for the evaluation of the human response according to this standard is the magnitude of the measured acceleration. Vibration measurement assessment is usually made by placing accelerometers on the vibrating surface (e.g. foundation, first floor or in another point that should be evaluated). So the weighted *r.m.s.* acceleration shall be calculated in accordance to the following equation.

$$a_w = \sqrt{\frac{1}{T} \int_0^T a_w(t)^2 dt} \quad (2)$$

where T represent the duration of the measurements, in seconds, and $a_w(t)$ is the weighted acceleration (translational or rotational) as a function of time, in m/s^2 and rad/s^2 , respectively.

Frequency Weighting

The manner in which vibration affects health, comfort, perception and motion sickness is mainly dependent on the vibration frequency content of the vibration. Different frequency weightings are required for the different axes of vibration. A special frequency weighting is included for evaluation of low-frequency vibration affecting motion sickness, for example ships [4]. The frequency weightings can be calculated by analogue or numerical methods, through transfer functions.

Therefore measured data should be “frequency-weighted” to give greater preeminence to frequencies where the human body is more sensible. The frequency weighting relates with the effects of periodic, random or transient vibration on the health of a person in normal health exposed to whole-body vibration during travel, at work, sleeping and leisure activities.

The standard ISO 2631-1: 2003, presents a couple transfer functions that permit to filter the signal measured in the site or when it is not possible to measure the signal, for example, before the construction of the railway line or the building is finished, the acceleration can be obtained from transfer functions from similar cases.

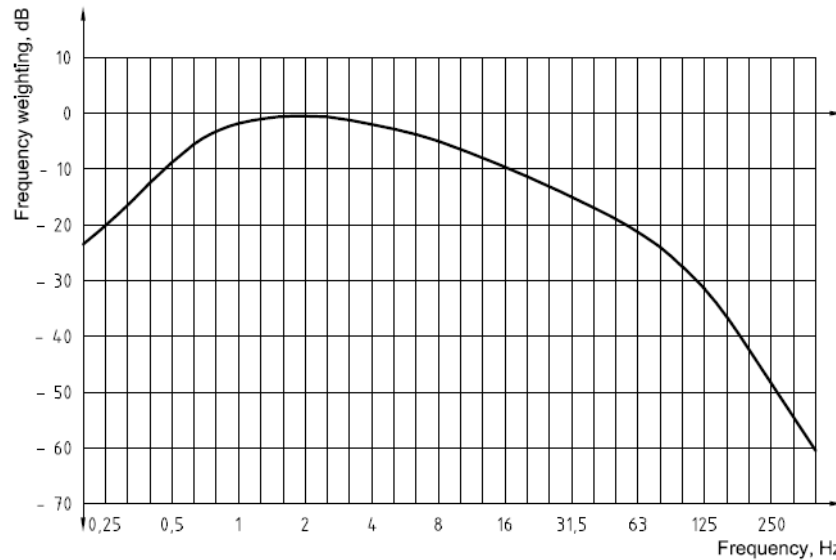


Fig. 12 – Frequency weighting [5]

The filters presented in this standard are the band-limiting (two-pole filter with Butterworth characteristic) and the pure frequency-weighting. The band-limiting is performed using filters type ‘high-pass’ and ‘low-pass’ their expressions are shown in Eq.(3) and Eq.(5), respectively.

$$H_h(p) = \frac{1}{1 + \sqrt{2} \cdot \omega_1/p + (\omega_1/p)^2} \quad (3)$$

$$|H_h(p)| = \sqrt{\frac{f^4}{f^4 + f_1^4}} \quad (4)$$

where $f_1 = 10^{-0,1} \approx 0,7943Hz$

$$H_l(p) = \frac{1}{1 + \sqrt{2} \cdot p/\omega_2 + (p/\omega_2)^2} \quad (5)$$

$$|H_l(p)| = \sqrt{\frac{f_2^4}{f^4 + f_2^4}} \quad (6)$$

where $f_2 = 100Hz$. For acceleration as the input quantity, the pure frequency weighting filter,

$$H_t(p) = \frac{1}{1 + p/\omega_3} \quad (7)$$

$$|H_t(p)| = \sqrt{\frac{f_3^2}{f^2 + f_3^2}} \quad (8)$$

where $f_3 = \frac{1}{0,028 \times 2\pi} \approx 5,684Hz$

The transfer function, $H(p)$ of the band-limited frequency weighting W_m is given by product of the high-pass filter $H_h(p)$, the low-pass filter $H_l(p)$ and the pure weighting function $H_t(p)$.

$$H(p) = H_h(t) \cdot H_l(t) \cdot H_t(t) \quad (9)$$

The modulus (magnitude) $|H(p)|$ is shown schematically in Fig. 12.

So basically the calculation of the a_w value which represents and value for the human reaction to vibrations is calculated as follows:

In the first step a frequency weighting is applied to the time history. Practically this is done by applying a band-pass filter with the corner frequencies of the high and the low-pass filter and afterwards a high-pass filter with the parameters of the pure weighting filter is applied. Finally it is necessary to apply a moving average filter to the frequency filtered time history. For this moving average filter it is necessary to use an exponential window instead of the rectangle window. The window size of this window should be, depending on the type of signal (trains, cars, etc), 1s.

According with [4], there are not sufficient data to present a reliable relationship between the vibration exposure and the health risk. So, ISO 2631-1:1997 recommend an evaluation of the exposure to vibration based on the calculation of daily vibration exposure $A(8)$ expressed on equivalent continuous acceleration over an 8h-period. The daily vibration exposure $A_l(8)$, in m/s^2 , for each direction l is defined by

$$A_l(8) = \sqrt{k_l \frac{1}{T_0} \sum_i a_{wli}^2 T_i} \quad (10)$$

where k_l represents a coefficient depending of the direction, T_0 is the reference duration of 8h (28800s) and l represents the direction in study. Assuming the vibration exposures are related with energy, it is possible compare two different daily exposures according to exposure time and the weighted *r. m. s.* acceleration respectively

$$a_{w1} \cdot T_1^{1/2} = a_{w2} \cdot T_2^{1/2} \quad (11)$$

More conservative studies indicate a time dependence according the following relationship

$$a_{w1} \cdot T_1^{1/4} = a_{w2} \cdot T_2^{1/4} \quad (12)$$

where a_{w1} and a_{w2} are the weighted *r. m. s.* acceleration values for two different exposures with T_1 and T_2 durations respectively.

The data shown in the Fig. 13 represents a health guidance caution zone limited by the dashed and solid lines correspondent to the Eqs. (11) and (12). It should be noted that the health guidance caution zones for both equations corresponds to time expositions from 4h to 8h are the same, wherein exist most occupational observations.

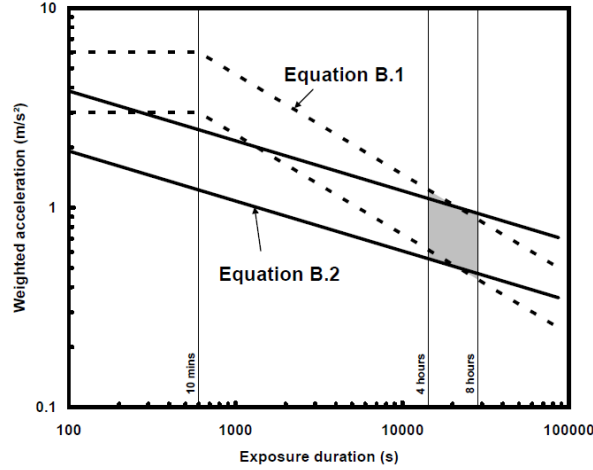


Fig. 13 – Health guidance caution zone [4]

When the vibration exposure consists of two or more periods of exposure to different magnitudes and durations, the energy-equivalent vibration magnitude can be evaluated according to the following Eqs. (13) and (14). The Eqs.(13) and (14) should be used when the analysis response adopt the relations showed by Eqs (11) and (12), respectively.

$$a_{w,e} = \left[\frac{\sum a_{wi}^2 \cdot T_i}{\sum T_i} \right]^{\frac{1}{2}} \quad (13)$$

$$a_{w,e} = \left[\frac{\sum a_{wi}^4 \cdot T_i}{\sum T_i} \right]^{\frac{1}{4}} \quad (14)$$

where $a_{w,e}$ is the equivalent vibration magnitude (*r.m.s.* acceleration in m/s^2) and $a_{w,i}$ is the vibration magnitude (*r.m.s.* acceleration in m/s^2) for exposure duration T_i .

2.3.2. VIBRATION IMPACTS ON BUILDINGS

Only extremely high levels of ground-borne vibration or a large number of cycles can in certain circumstances (for example, blasting) give rise to risk of damage to building structure. Building damage is not a factor for normal transportation project, with the exception of blasting and pile-driving during construction. The vibration levels required are of the order of 10 to 100 times larger than those associated with human perception, thus the levels of vibration necessary to damage a building, even cosmetically, would be intolerable to occupants [1]. Annoyance from vibrations often occurs when the vibration exceeds the threshold of perception by only a small margin. A vibration level that causes annoyance will be well below the damage threshold for normal buildings.

Barehkazemi [1] shows that only the buildings subjected to a particle velocity lower than 50 mm/s , the probability to damage structurally the building due to ground-borne vibrations is 5%. In velocities lower than 15 mm/s the risk of damage is null.

2.4. MITIGATION METHODS

The ground-borne vibration can be controlled at different levels along the transmission path between the source and the receiver. For this problem there are several countermeasure solutions that can solve the problem totally or in part. In spite of this thesis be exclusively about the ground-borne vibration, in an evaluation of the alterations caused due to the traffic train, it should be taken into account that the noise due to the train traffic and the noise provoked by the ground vibration, the re-radiated noise. As it was analyzed in the preceding sub-chapter, the ground vibration is not the main problem for the inhabitants, but the effects that provoke in the buildings when the frequencies content of the transmitted vibrations are equal or approximately to the natural frequencies of the building components.

There are no generally applicable mitigations measures for problems of ground-borne vibration due to the train traffic. One method to reduce ground motion would be to impose a speed reduction and/or limit the maximum allowed weight per train wagon. However, this is not the most efficient solution to this problem. Reducing the speed and/or weight of the train is most suitable if the critical wave velocity in the soil is being approached and/or if it is the near-field that is of most concern. The treatment depends on the dominant mechanism of vibration excitation and on the interaction of the ground and track structure. Given the long wavelengths of vibration in both the track and ground, large-scale civil engineering measures must be adopted, as for example wave barriers. The analysis of mechanisms presented in this sub-chapter gives clues to measures that could be effective but these must be appropriately designed for particular locations.

The measures taken in the source of the generation of vibration, in the propagation path and in the receiver represent the three groups of possibilities to reduce the annoying levels of ground-borne vibration to the population. The first group includes those that reduce the vibration in the source; the second and third group represents the possibilities to reduce the vibration focused on the propagation path and receiver, respectively. In the next sections it will be presented some possibilities to reduce the ground-borne vibration, in the case of possibilities taken into the source and countermeasures taken in the propagation path and in the receiver.

2.4.1. MITIGATION MEASURES IN THE SOURCE

There is a wide variety of methods that can be applied to the source, on the interaction train-track-ground, in order to reduce the ground motion. There are simple methods or more complex and cost intensive methods, although they are more effective. The application of each of these methods depends on the severity of the problem, the type of railway track, technology employed, etc.

An important measure taken in order to reduce the ground vibration and noise is the continuous maintenance of the track and rail components.

Usually several methods are applied, in the source, and/or along the system of generation and reception of ground vibration, being more effective and cheaper. The design of the railway tracks should include rail grinding, wheel truing, continuous welded rail in addition with some of the below methods presented (J.T. Nelson, 1995).

- Resilient direct fixation fasteners

This type of fasteners (Fig. 14) has long been used by United States transit systems for new construction. It can provide a moderate degree of vibration isolation, low maintenance and better rail head positioning control than ballast track. There are fasteners with static stiffness of direct

fixation which can vary between 10MN/m and 50MN/m. The utilization of soft fasteners permits the reduction of ground-borne vibration above 30Hz and the ground-borne noise.



Fig. 14 – Resilient direct fixation fasteners

- Ballast mats

Ballast mats (Fig. 15) have been recommended for surface ballast-and-tie track located adjacent to sensitive residential structures. Conventional installations of ballast mat are in subway with concrete base, for which vibration insertion losses are predicted to be high [6].



Fig. 15 – Ballast mat application

- Floating slab track

Floating slab systems consist of two basic types: continuous cast-in-place floating slabs and discontinuous double tie pre-cast floating slabs (Fig. 16). Such as the ballast mats, this process is used in sensitive inhabited areas or in structures, as subway crossovers near residential structure. The continuous slab is constructed by placing a sheet metal form on elastomer isolators, and filling the sheet metal form with concrete. This process may involve two concrete pours, one by the structural contractor and the other by the track-work installer [7].

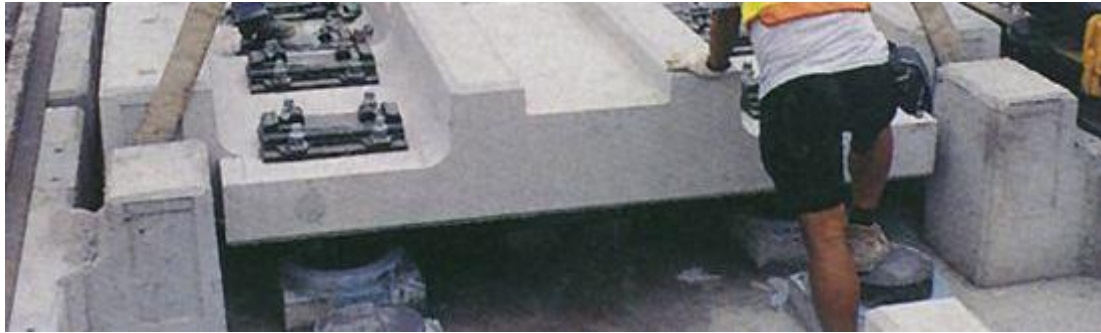


Fig. 16 – Floating slab track

- Rail Straightness

Rail straightness is of fundamental importance to control the ground motion in critical areas. Roller straightened rails have been observed to produce ground vibration frequency components coincident with the roller straightener pitch diameter. So, it's highly recommended a “super-straight” rail, in sensitive areas, where the floating slabs are not practical for economical or construction reasons [2].

- Modification of car design especially the primary suspension system

Primary suspension is a very important topic related with the excitation of ground-borne vibration. Generally, vehicles with vertically soft suspension produce a lower level of vibration than vehicles with stiff suspension. Nowadays, the technological research of new trains, specially the HST, has developed train transmissions which possibilities a lower level of ground-borne vibrations with no sacrifice of the security and passenger comfort [1].

- Stabilization of soil under the track embankment

This solution was studied by Robert Hildebrand, 2003, and is a good methodology to solve ground vibration in soft soils. It is successful for frequencies lower than 20Hz, however, it has that the problem that it can lead to amplifications in some audible bands [7]. Lime-cement columns are a particular form of soil stabilization in which dry lime and cement are pneumatically injected and blended into natural clay.

2.4.2. MITIGATION MEASURES IN THE PATH

The methods taken to reduce the ground-borne vibration due to the train traffic in the path are applied in some particular situations. This type of methods is, comparably to the methods applied at the source and receiver more expensive and less effective. Its application is restricted to situations where the track line already exists and it is intended to build new buildings or situations where the ground vibrations exceeds the limits due to alterations of train traffic, and so defaulted the ground vibration levels of health and comfort standards. Usually, the interruption of the train traffic to install mitigation measures in the source is not an option, because the cost of that interruption, which could take months, would cost more than the installation of possible countermeasures at the propagate path or in the receiver.

These countermeasures taken in the path consists of wave barriers in the ground, open or filled. The main objective of these barriers is reducing the intensity of the vibrations and the size of the affected area around the railway line. This purpose is reached through the reflection, damping and scattering of the incoming waves. Its degree of reduction is dependent from the relation between the wave barrier and the wavelengths of the vibration [9]. So, for clay soil type where is generated a huge wavelength, the depth of the wave barrier need to be considerable, which makes impracticable the construction of a wave barrier.

Generally open wave barriers (for example slots) are more effective for stiff soils near the railway line. The distance, geometry, as the filling material, should consider the type of soil and the embankment geometry.

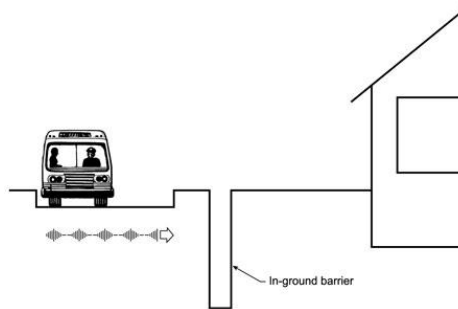


Fig. 17 - Wave barriers

2.4.3. MITIGATION MEASURES IN THE RECEIVER

If there are only a few buildings affected by the excessive ground vibration, possibly the most economical possible countermeasure should be taken in the receiver, i.e. in new building foundations. There are several possible methods available to be applied in the receiver; however the principle is basically the same. The foundations of the building are isolated using an elastic support system (Fig. 14), where the building is basically considered as a rigid body supported by a large number of springs and dampers.



Fig. 18 – Building elastically supported [10]

The profiled Cibatur mat, shown in Fig. 19, consists of a fabric reinforced elastomeric plate (sandwich method) that has underneath truncated cone shape spring elements. The surface of the top layer is not only insensitive to weather but also resistant to abrasion, oil and ozone. Natural rubber of high quality is used for the spring elements which have excellent dynamic properties. As presented in Fig. 22, this mat is placed between concrete lawyers. For the project showed in Fig. 21 and Fig. 22, the concrete layers are 1.0m thick.

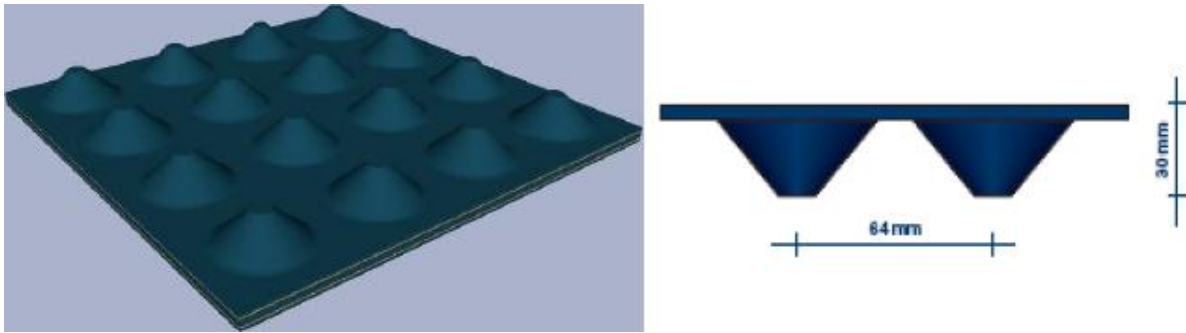


Fig. 19 – Countermeasure (Cibatur) applied in the foundations

To model this countermeasure should be taken into account the building weight, the natural frequency of building and the excitation frequency. In Fig. 20 is represented the insulation effect produced by this product. It is possible to conclude that for high excitation frequencies Cibatur reveals to be very effective. However for lower vibration frequencies this countermeasure has negative effects amplifying the vibration on the structure. This aspect isn't presented in the following figure because of marketing.

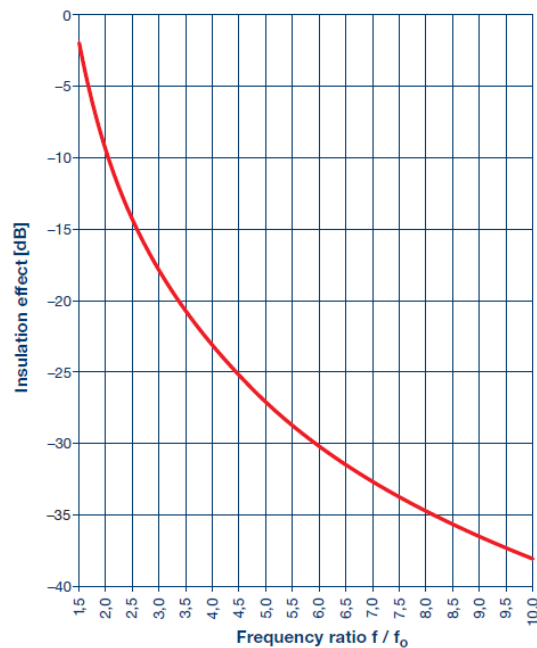


Fig. 20 – Vibration reduction of Cibatur (www.calenberg-ingenieure.de)



Fig. 21 – Application of bedding elastic for buildings (*iC – Consulanten*)



Fig. 22 – Lateral view of bedding elastic for buildings (*iC – Consulanten*)

3

DYNAMIC ANALYSIS

3.1. INTRODUCTION

The analysis of ground-borne vibrations due to train traffic can be studied idealizing the train as a moving load or a set of moving loads on the surface as well as in the tunnel situation. Its amplitude corresponds to the weight transmitted by the wheels (wheels force) of the train to the track and its frequency corresponds to the time between loads (wheels). Additionally there are a lot of other loads caused by wheel-rail contact and discontinuities along the trackway.

Another aspect that should be taken into account about this issue has to do with the dynamic and *quasi-static* components of the loads. For large distances the dynamic component of the loads grows importance, the *quasi-static* component is important for analysis at small distances.

This chapter deals with some important basics on “Dynamic Analysis” concerning some relevant expressions developed for a single-degree-of-freedom (SDOF), and fundamentals of dynamics which are important for ground-borne vibration due to train traffic. However, the complete development of these expressions can be found in books as in “Dynamics of Structures” from Anil K. Chopra or “Vibration, Fundamentals and Practice” from de Silva.

3.2. DYNAMIC MATERIAL MODEL

The dynamic material models should represent the behavior of the soil under dynamic excitation. For this thesis a linear elastic behavior of the soil and its material is considered. The linear elastic model represents the simplest type of stress-strain-time relationship applied, where the stress-strain relation is considered to be approximately linear proportional.

It is the shear strain experimented for the soil that will determine the mathematical models developed to describe the dynamic response of the soil. Usually, the shear strain exhibit in the propagation path due to the vibrations caused by train traffic is below 10^{-4} . At this level of shear strain, the behavior of the soil can be characterized through a linear elastic model to describe the dynamic response.

If larger deformations between 10^{-4} and 10^{-2} as it happens to soil under seismic activity, the soil will show an elasto-plastic behavior, i.e. irrecoverable deformations. To describe this soil response the equivalent material model is more appropriate [2]. However, its application in this issue is not appropriated since the vibration due to train traffic that does not cause significant changes in the shear modulus of the soil. Its application can be found in the study of rubber-modified asphalt concrete as a ballast material [3]; or in the study of the ground vibration under the railway, where the soil can

experiment high deformations. Wherein, the linear hysteretic damping (damping type independent of the frequency) is used to capture the dissipating mechanism of each material.

3.3. EQUATION OF MOTION

The equation of motion aims to represent the coordinates of the mass m , at any instant t , for each degree-of-freedom of the system. The minimum number of independent coordinates required to determine completely the positions of all parts of a system at any instant of time defines the degree of freedom of the system. The single degree-of-freedom (SDOF) system represents the simplest model which can be idealized to study the motion of an inertia body. The forces acting in the direction of the degree-of-freedom are the applied load $f(t)$, and three resisting forces resulting from the motion, i.e, the inertial force $f_I(t)$, the damping force $f_D(t)$, and the spring force $f_S(t)$.

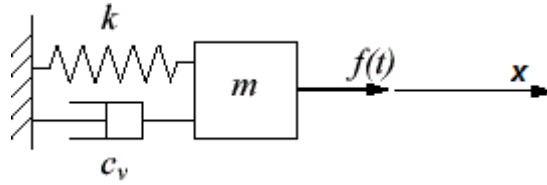


Fig. 23 – Single-degree-of-freedom system

In Fig. 23 a schematically body is represented with lumped mass, m , excited by the applied load $f(t)$, with the resisting forces represented by a spring with elastic constant k and a viscous damper with a viscous damping ratio c_v .

The equation of motion will represent the dynamic equilibrium between these forces,

$$f_I(t) + f_D(t) + f_S(t) = f(t) \quad (13)$$

where, $f_I(t)$, $f_D(t)$ and $f_k(t)$ represent inertial, damping and elastic force respectively; and $f(t)$ represents the excitation force.

Each of the forces represented in the left hand side of this equation are function of the displacement $x(t)$ or one of its derivatives (velocity and acceleration). In accordance with D'Alembert principle, the inertial force, as it is shown in Fig. 23, is the product of the mass and acceleration,

$$f_I(t) = m \cdot \ddot{x}(t) \quad (14)$$

$$\ddot{x}(t) = \frac{d^2 x(t)}{dt^2} \quad (15)$$

$x(t)$ and $\ddot{x}(t)$ represent the mass displacement in function of time and their corresponding second derivative, the acceleration, respectively.

Assuming viscous damping mechanism, the damping force is the product of viscous damping coefficient c_v and the velocity $\dot{x}(t)$,

$$f_D(t) = c_v \cdot \dot{x}(t) \quad (16)$$

$$\dot{x}(t) = \frac{dx(t)}{dt} \quad (17)$$

The viscous damping, considered in this sub-chapter, will be object of analysis in this thesis.

Finally, the elastic force is the product of the spring stiffness k , and the displacement $x(t)$,

$$f_S(t) = k \cdot x(t) \quad (18)$$

If the Eqs. (14), (16) and (18) substitute into Eq. (13), then the equation of motion for a SDOF system viscously damped, with mass m , excited by a dynamic force, $f(t)$, is

$$m \cdot \ddot{x}(t) + c_v \cdot \dot{x}(t) + k \cdot x(t) = f(t) \quad (19)$$

Damping ratio

In order to study the damping in certain structure, generally a system under free vibration with viscous damping should be considered. As it was stated before the viscous damping force Eq. (16) is proportional to the velocity $\dot{x}(t)$. To solve the differential Eq. (13), through the exponential form, we have

$$x(t) = C \cdot e^{s \cdot t} \quad (20)$$

where C and s are undetermined constants. Insert this equation into Eq. (19) and assuming that the external force is null $f(t) = 0$, leads to the characteristic equation

$$m \cdot s^2 + c_v \cdot s + k = 0 \quad (21)$$

The roots of the Eq. (21) are

$$s_{1,2} = \frac{-c \pm \sqrt{c^2 - 4 \cdot m \cdot k}}{2 \cdot m} \quad (22)$$

The critical damping corresponds to

$$c_c^2 - 4 \cdot m \cdot k = 0 \quad (23)$$

The natural frequency of the system can be expressed in function of the stiffness k , and the mass m , of the system

$$\omega_n = \sqrt{k/m} \quad (24)$$

Substituting the stiffness and the mass for ω_n in Eq. (23) and arranging in order to the critical damping coefficient c_c , we obtain

$$c_c = 2 \cdot m \cdot \omega_n \quad (25)$$

The quotient between the damping coefficient and the critical coefficient damping corresponds to damping ratio as its show in the following Eq. (26)

$$\xi \equiv \frac{c_v}{c_c} = \frac{c_v}{2 \cdot m \cdot \omega_n} \quad (26)$$

3.4. RESPONSE OF A SYSTEM UNDER VIBRATION

Whenever external energy is applied to an oscillating system, the system is said to undergo forced vibration. These applied forces or displacements may have several natures: harmonic, periodic, non-periodic, random, etc. This chapter introduces the formulation of the general equation of SDOF under harmonic load, to understand the phenomena of resonance, and the formulation for general conditions of excitation. The response of a system under vibration can be subdivided into two parts: the transient response and the steady response. The first component refers to vibration due to the initial conditions when the analysis of vibration starts, i.e., the initial displacement and initial velocity. The second component refers to the response of the structure due to the force or excitation applied to the system during or after its application.

Response under harmonic vibration

In the following Eq. (27) the general response of a SDOF under harmonic excitation, in time domain, with viscous damping and nonzero initial conditions is presented. The development of the equation in

any fundamental dynamics book can be shown easily. The importance to treat this expression concerns about the resonance and its importance for systems under dynamic loads.

$$x(t) = e^{-\xi\omega_n t}(A \cos(\omega_a t) + B \sin(\omega_a t)) + \frac{p_0}{k} \frac{1}{(1-r^2)^2 + (2\xi r)^2} [(1-r^2) \sin(\omega t) - 2\xi r \cos(\omega t)] \quad (27)$$

where ω_a and ω represents the damped frequency of the system and the excitation frequency, respectively; A and B correspond to initial parameters; p_0 corresponds to the amplitude load; and r is the static deflection. The first two terms of the Eq. (27) refers to the transient response and its importance is relatively small, since it damps out quickly, so it is usually of little interest and won't be more referred. The second term refers to the steady response due to the excitation load. To obtain the damped frequency of any structure we have

$$\omega_a = \omega_n \sqrt{1 - \xi^2} \quad (28)$$

In the case of soils, the damping ratio has as range between $\xi = 2\sim 3\%$, so we can consider that $\omega_a \cong \omega_n$.

The static deflection corresponds to the quotient between the excitation frequency and the natural/damped frequency of the system,

$$r = \frac{\omega}{\omega_n} \quad (29)$$

The steady-state displacement behavior can be interpreted easily by plotting two corresponding rotating vectors in the complex plane as shown in the Fig. 24, where their components along the real axis are identical to the two terms corresponding to the steady-state in the Eq. (27).

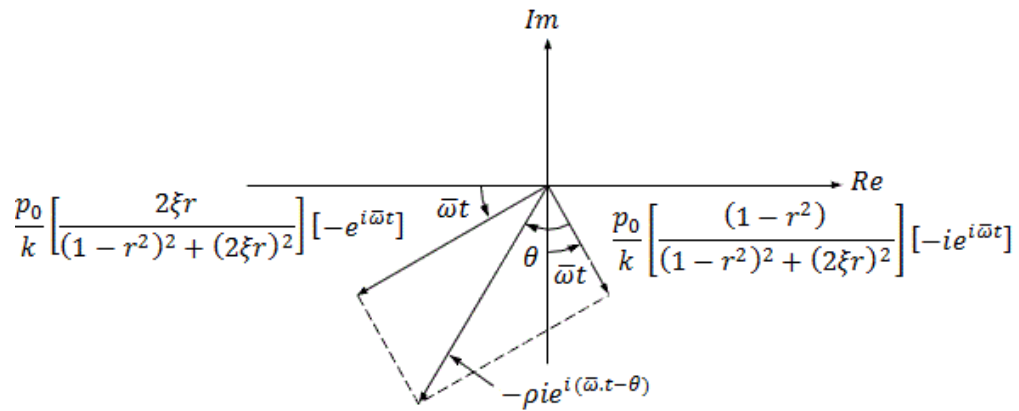


Fig. 24 – Steady state displacement response

The real component to the resultant vector, $-\rho \cdot i \cdot e^{i(\omega t - \theta)}$ gives the real steady response in the form

$$x_p(t) = \rho \cdot \sin(\omega t - \theta) \quad (30)$$

having an amplitude

$$\rho = \frac{p_0}{k} [(1 - r^2)^2 + (2\xi r)^2]^{-1/2} \quad (31)$$

and a phase angle θ , which the response lags behind the applied loading

$$\theta = \tan^{-1} \left[\frac{2\xi r}{1 - r^2} \right] \quad (32)$$

It should be understood that this phase angle is limited to the range $0 \leq \theta \leq 180^\circ$.

The ratio of the resultant harmonic response amplitude to the static displacement which would be produced by the force p_0 will be called by dynamic magnification factor D ; thus:

$$D \equiv \frac{\rho}{p_0/k} = [(1 - r^2)^2 + (2\xi r)^2]^{-1/2} \quad (33)$$

It is seen that both the dynamic magnification factor D and the phase angle θ vary with the frequency ratio r and the damping ratio ξ . Plots of D vs. r and θ vs. r are shown in Fig. 25, for discrete values of damping ξ .

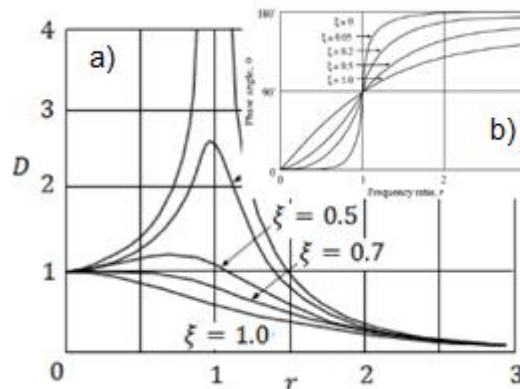


Fig. 25 - a) variation of dynamic magnification factor; and b) variation of phase angle; with damping and frequency

Depending on the damping ratio, a system can be classified in three different categories. With damping ratio $\xi = 1$, the system returns to its equilibrium position without oscillating. This case is merely theoretical, once there are no systems with critical damping, and it's called critically-damped system.

The overcritically-damped system has a damping coefficient bigger than the critical coefficient. The system will not oscillate, and in comparison with critically-damped system will return to the equilibrium position lower.

In undercritically-damped systems, $\xi < 1$, the system will have a oscillatory motion. This type of systems is the majority of study cases in Civil Engineering, because the structures – buildings, bridges, dams, etc.- all fall into this category, once their damping ratio is less than 0.10. The soil, as the structures mentioned before, also has a reduced damping ratio. Depending on the type of soil the damping ratio presents its higher value for soft clays, 0.05 and presents its lowest value for rocks, around 0.01.

Response to general dynamic loading

When the applied force $p(t)$ or ground excitation $\ddot{u}_g(t)$ varies arbitrarily with the time or if the system is non-linear, it's not possible to develop an analytical expression to describe its motion, as it happens with soils. To evaluate such motions, there are several methods that can be used. According with Clough, the methods available to analyze the response of SDOF under general dynamic loading can be subdivided into two main groups, the numerical time-stepping methods for integration of differential equations and the superposition methods.

The superposition method represents the response through a superposition of functions. This type of analysis is done in the time-domain or in the frequency-domain. The frequency-domain is applicable even when the signals are not periodic. Using the Fourier transform it's possible to transform a time signal into a frequency *spectrum*. The time-stepping methods are usually used in numerical evaluation because it permits a good accuracy without a large computational effort.

Fourier series

When there is a periodic but non-harmonic function, it's possible to represent this function by Fourier series as an infinite sum of sine and cosine terms. If $p(t)$ is a periodic function with period T , its Fourier series representation is given in trigonometric form by

$$p(t) = \frac{a_0}{2} + \sum_{m=1}^{\infty} a_m \cos(m \cdot \omega \cdot t) + b_m \sin(m \cdot \omega \cdot t) \quad (34)$$

where $\omega = 2\pi/T$ is the angular frequency. The constant coefficients a_0 are the average value of $p(t)$, and a_m , b_m are the amplitudes of m th of frequency $m\omega$. The constant coefficients can be obtained through the following equations

$$a_0 = \frac{2}{T} \int_0^T p(t) dt \quad (35)$$

$$a_m = \frac{2}{T} \int_0^T p(t) \cos(m \cdot \omega \cdot t) dt \quad (36)$$

$$b_m = \frac{2}{T} \int_0^T p(t) \sin(m \cdot \omega \cdot t) dt \quad (37)$$

The steady response of SDOF due to harmonic loading is

$$x(t) = \frac{1}{k} \left[a_0 + \sum_{n=1}^{\infty} \frac{1}{1 - r_n^2} (a_n \cos(n\bar{\omega} t + b_n \sin(n\bar{\omega} t)) \right] \quad (38)$$

The Fourier series can also be represented in exponential form. Substituting the Euler relations into Eq. (34) and rearranging, we obtain the excitation function

$$p(t) = \sum_{n=1}^{\infty} \overline{P}_n \cdot e^{in\bar{\omega}t} \quad (39)$$

where

$$\overline{P}_n = \overline{P}_n(\bar{\omega}) = \frac{1}{T} \int_0^T p(t) \cdot e^{-in\bar{\omega}t} dt = \overline{A}(n\bar{\omega}) - i\overline{B}(n\bar{\omega}) = \overline{A} - i\overline{B} \quad (40)$$

with

$$\overline{A}_n = \overline{A}_n(\bar{\omega}) = \frac{1}{T} \left[\int_0^T p(t) \cos(n\bar{\omega}t) dt \right] \quad (41)$$

and

$$\overline{B}_n = \overline{B}_n(\bar{\omega}) = \frac{1}{T} \left[\int_0^T p(t) \sin(n\bar{\omega}t) dt \right] \quad (42)$$

The imaginary terms will cancel mutually when the sum varies from $-\infty$ to $+\infty$.

Using the steady state response to an excitation response $\bar{\omega}_n = n\bar{\omega}$ in exponential form will be

$$x(t) = \sum_{n=-\infty}^{\infty} \bar{H}_n(\bar{\omega}) \bar{P}_n e^{in\bar{\omega}t} \quad (43)$$

where $\bar{H}_n(\bar{\omega})$ represents the transfer function

$$\bar{H}_n(\bar{\omega}) = \frac{1/k}{1 - n^2 r^2 + 2nr\xi i} \quad (44)$$

The Fourier series studied so far, in trigonometric or exponential form, has as principal range of applicability, the arbitrary periodic loading of period T . However, after obtained the coefficients $\bar{P}_n$, by integrating with Eq. (40), over an arbitrary time interval $0 < t < T$, the resulting representation is a periodic function as Fig. 26 shows with the red and blue lines. The repetitions (red lines) which don't occur can be canceled by continuing $T \rightarrow \infty$.

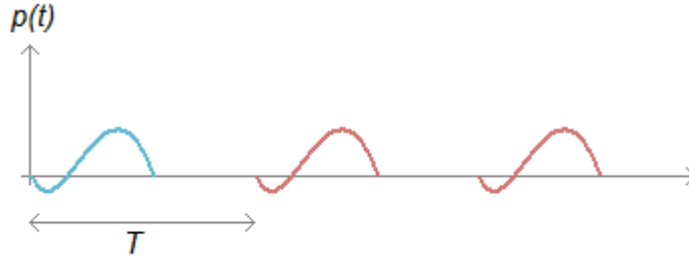


Fig. 26 – Transformation of a function into a periodic one

So the changes in the equations previously studied will be,

$$\frac{1}{T} = \frac{\bar{\omega}_1}{2\pi} \equiv \frac{\Delta\bar{\omega}}{2\pi} \quad (45)$$

$$n\bar{\omega}_1 = n\Delta\bar{\omega} \equiv \bar{\omega}_n \quad (46)$$

$$\bar{P}_n T \equiv \bar{P}(i\bar{\omega}_n) \quad (47)$$

Modifying the Eqs. (45), (46) and (47) in the Fourier series for periodic functions Eq. (39), we obtain, the Fourier series for non-periodic functions.

If the loading period is extended indefinitely, the frequency increment becomes infinitesimal ($\Delta\bar{\omega} \rightarrow d\bar{\omega}$) and the discrete frequencies become a continuous function of $\bar{\omega}$. Thus, the Fourier series take the integral form

$$p(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} \bar{P}(\bar{\omega}) \cdot e^{i\bar{\omega}t} d\bar{\omega} \quad (48)$$

$$\bar{P}(\bar{\omega}) = \int_{-\infty}^{\infty} p(t) \cdot e^{-i\bar{\omega}t} dt \quad (49)$$

where $\bar{P}(\bar{\omega})$ is the amplitude function. The Eqs (48) and (49) represent the pair of Fourier transforms (FT). Using the FT, the arbitrary loading $p(t)$ can be expressed as an infinite sum of harmonics having known complex amplitudes. The function $\bar{P}(\bar{\omega})/2\pi$ represents the complex amplitude intensity at frequency $\bar{\omega}$ per unit of $\bar{\omega}$. So the total response will be

$$x(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} \bar{H}(\bar{\omega}) \bar{P}(\bar{\omega}) e^{i\bar{\omega}t} d\bar{\omega} \quad (50)$$

where $\bar{H}(\bar{\omega})\bar{P}(\bar{\omega})/2\pi$ represents the $\bar{\omega}$ component of displacement amplitude per unit of $\bar{\omega}$, similar to the load excitation expression, Eq. (48).

The Fourier response integral, as described above, is limited to cases where the applied loading-expression is known, and even in these situations the evaluation can be a quite complicated. Thus, to make the Fourier transformation a practical tool to evaluate the response of a system under general vibration, approximations of the function that represent the applied loading should be used. This formulation should express the Fourier integral transforms in convenient approximate forms and develop an effective and efficient numerical technique for evaluating these forms. The following expressions were the result of that formulation. The approximation was made evaluating the coordinates of the function $p(t) \cdot e^{-i\bar{\omega}_n t}$ at discrete values of $t = t_m = m \cdot \Delta t$ ($m = 1, 2, \dots, N$) and then applying the trapezoidal rule of integration. The equivalent expressions of Eqs. (49) and (50) of DFT are respectively

$$p(t_m) \doteq \sum_{n=0}^{N-1} P_n \cdot e^{(i \frac{2\pi n m}{N})} \quad m = 0, 1, 2, \dots, N-1 \quad (51)$$

$$P_n \doteq \frac{1}{N} \sum_{m=0}^{N-1} p(t_m) \cdot e^{(-i \frac{2\pi n m}{N})} \quad n = 0, 1, 2, \dots, N-1 \quad (52)$$

where N is the number of intervals Δt in which the period T_p was divided ($T_p = N \cdot \Delta t$); $p(t_m)$ is the

ordinate to the function $p(t)$ at time $t = t_m$; while P_n is the complex amplitude of the discrete harmonic at the frequency $\bar{\omega}_n$.

3.5. SIGNAL ANALYSIS

The response of the soil due to an interaction between a moving train and the ground beneath the track line will generate different signals in the various degrees of freedom. The signal of the motions recorded (displacements, velocities and accelerations) are a function of time, although the study of the signal will be done with frequency functions, through the Fourier transformation, as explained in the preceding sub-chapter. The resulting *spectrum* of a signal can be interpreted as the set of frequency components that the original signal contains.

The signals measured or recorded have the time as independent variable and the signal is represented in the time domain. The following data, shown in Fig. 27, presents a measured signal, the y-velocity obtained in an assessment.

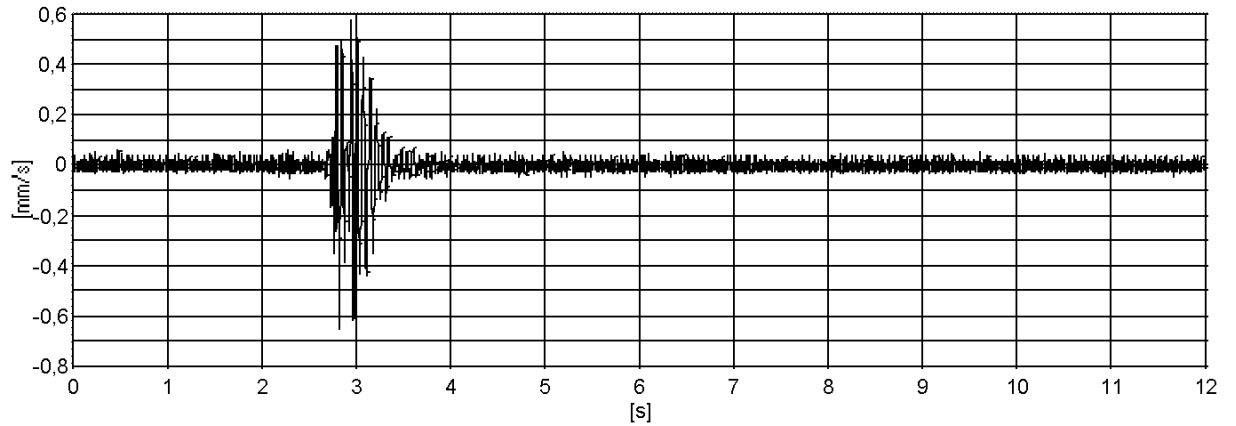


Fig. 27 – Measured signal: y-velocity

In the study of vibration signal it is possible to divide its analysis into two main groups: the random and the deterministic signal. The random signal corresponds to a signal formulated through statistics as it happens with earthquake engineering. For ground vibration the signal analysis will be done for deterministic signal, as it was said, for the signals recorded in the models and the signals measured in field experiments. For a signal, as it is represented in Fig. 27, it is complicated to track so many parameters and, furthermore, not all of them are equally important. Although, the fact that all parameters are directly or indirectly related either the amplitude or the frequency permit to do its analysis in the frequency domain through the Fourier transformations. This shows the importance of the frequency domain and how attractive the frequency-domain representations are. The Fig. 27 shows the amplitude *spectrum* of the measured velocity. The Fig. 28 shows the frequency *spectrum* according to the signal.

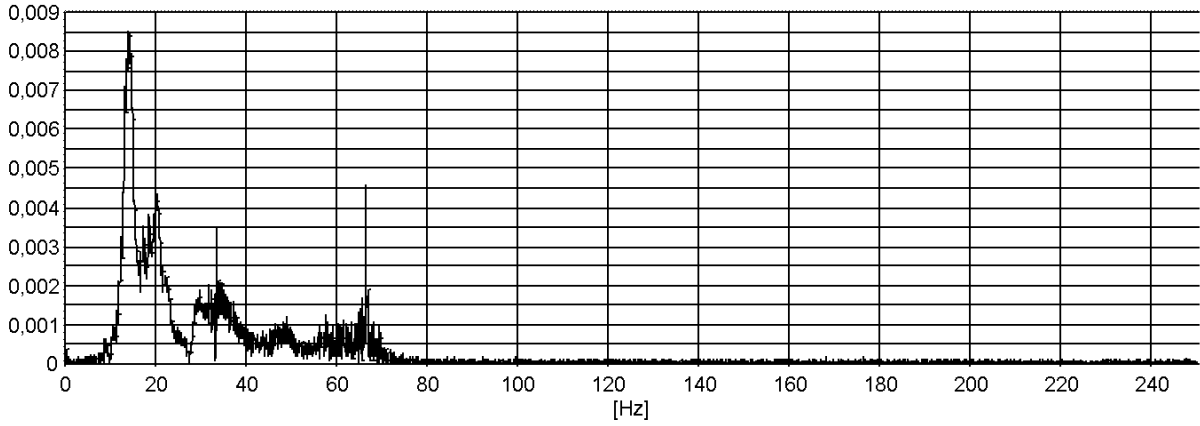


Fig. 28 – Frequency spectrum

The signal analysis will be done through the Discrete Fourier Transformation (DFT), as it was explained and demonstrated in the preceding subchapter, with the algorithm of Fast Fourier Transform (FFT).

Fast Fourier Transforms (FFT)

With the objective to reduce the computational effort and therefore the calculation time, Cooley and Turkey developed an algorithm based on the Discrete Fourier Transforms that permits evaluating both the direct and inverse FFTs. Studying only the direct FFT, since it is the same for evaluating the inverse, we have

$$A_n = P_n \cdot N = \sum_{m=0}^{N-1} p_m \cdot e^{-i \frac{2\pi n m}{N}} \quad n = 1, 2, \dots, N-1 \quad (53)$$

where p_m denotes $p(t = t_m)$. Mostly all of the software built to do the analysis of DFT's incorporates the algorithm FFTs, such as *Verknüpfung mit Videv*, developed in *iC-Consulenten*. This software is used for the analysis of the signals recorded in the FLAC models and measured signals.

A straightforward evaluation of this summation for all values of n requires N^2 complex multiplications. This number would be prohibitive for most practical solutions requiring large values of N .

The FFT algorithm is based on letting $N = 2^\gamma$, where γ is an integer. So, each value of n and m in their common range from zero to $N - 1$ can be expressed in

$$n = 2^{\gamma-1}n_{\gamma-1} + 2^{\gamma-2}n_{\gamma-2} + \dots + n_0 \quad (54)$$

$$m = 2^{\gamma-1}m_{\gamma-1} + 2^{\gamma-2}m_{\gamma-2} + \dots + m_0 \quad (55)$$

in which each binary coefficient is either +1 or 0 depending upon the particular value of n or m being represented. Using the relations, Eqs. (54) and (55), and letting $W_N = e^{-i\frac{2\pi}{N}}$, Eq. (53) can be expressed as

$$A(n_{\gamma-1}, n_{\gamma-2}, \dots, n_0) = \sum_{m_0=0}^1 \sum_{m_1=0}^1 \dots \sum_{m_{\gamma-1}=0}^1 p_0(m_{\gamma-1}, m_{\gamma-2}, \dots, m_0) \cdot W_N^{nm} \quad (56)$$

Note that each coefficient A_n for $n = 0, 1, \dots, N-1$ is represented by $A(n_{\gamma-1}, n_{\gamma-2}, \dots, n_0)$ and each load ordinate of p_m for $m = 1, 2, \dots, N-1$ is represented by $p_0(m_{\gamma-1}, m_{\gamma-2}, \dots, m_0)$. Simplifying the Eq. (56) we obtain

$$\begin{aligned} A(n_{\gamma-1}, n_{\gamma-2}, \dots, n_0) &= \\ &= \sum_{m_0=0}^1 \sum_{m_1=0}^1 \dots \sum_{m_{\gamma-1}=0}^1 [p_0(m_{\gamma-1}, m_{\gamma-2}, \dots, m_0) \times W_N^{2^{\gamma-1}(n_0 m_{\gamma-1})} \\ &\times W_N^{2^{\gamma-2}(2n_1+n_0)m_{\gamma-2}} \times \dots \times W_N^{(2^{\gamma-1}n_{\gamma-1}+(2^{\gamma-2}n_{\gamma-2})+\dots+n_0)m_0}] \quad (57) \end{aligned}$$

The result $A(n_{\gamma-1}, n_{\gamma-2}, \dots, n_0)$, corresponding to the sum of the recursive equations, i.e., each summation is used immediately in the next summation that represents the FFT analysis. It is very interesting that the reduction in computational effort occurred due to the use of recursive functions for a large number of intervals, without losing its accuracy. For an interval T_p divided into 1000 intervals the calculation by means of FFT is 200 times faster than the calculation through the Eqs. (54) and (55) [10]. Due to the calculation time saved, this algorithm is used in programs as MATLAB, OCTAVE, etc.

3.6. SEISMIC WAVES

Body waves can travel in any direction in the ground. They follow different ray paths influenced by the density variations (ρ) and shear modulus (G) they pass through, which also will change its velocity according to the stiffness of the materials. The most important body waves according to traffic induced vibrations are: compression and shear waves that constitute the body waves and Rayleigh waves which correspond to the surface waves. The Fig. 29 represents a plot obtained in FLAC of “Velocity vectors” from a model used wherein it is simple to distinguish the waves as its different propagation velocity.

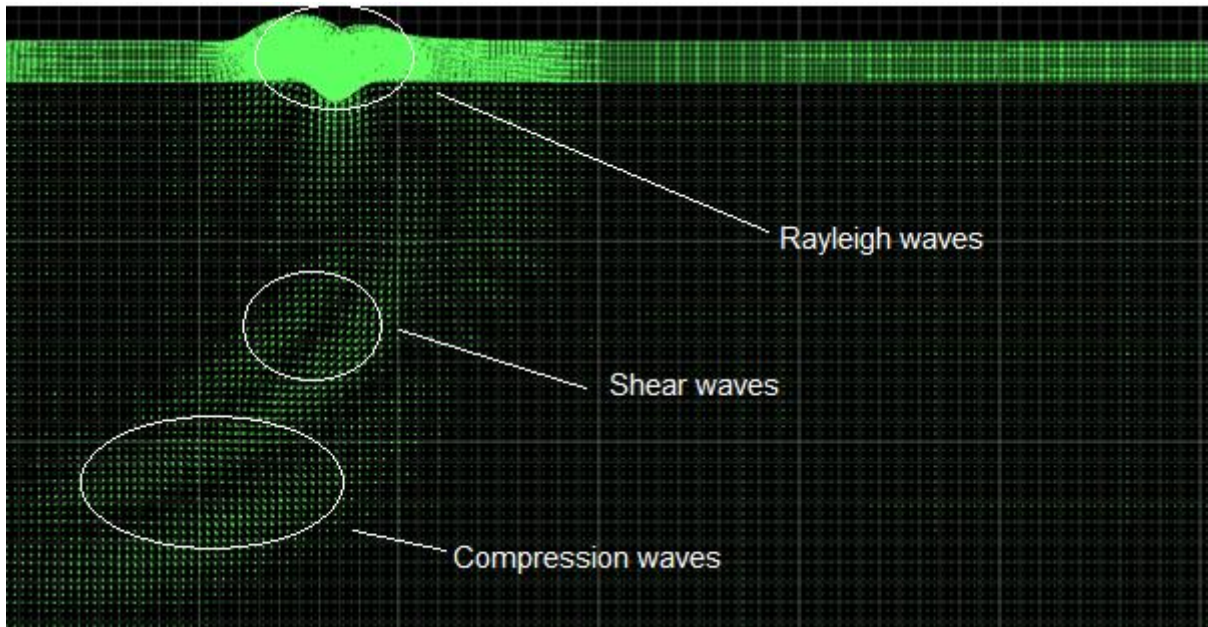


Fig. 29 – Propagation of seismic waves

The compression waves, also known as P-waves, primary and longitudinal waves, involve successive compression and rarefaction of the material through which they propagate, Fig. 29. They travel 1.7 times faster than the S-waves and can travel through any material, solid or liquid.

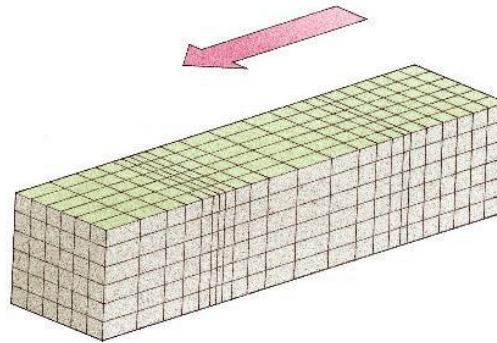


Fig. 30 – Scheme of compression wave

The propagation velocity (C_P) can be calculated as

$$C_P = \sqrt{\frac{G(2 - 2\nu)}{\rho(1 - 2\nu)}} \quad (58)$$

where G , ρ and ν are the shear modulus, density and Poisson's ratio, respectively, of the material where the waves are propagating.

The secondary or so-called transverse waves, also known as S-waves, cause shearing deformations as they travel through the material. The motion of a particle is perpendicular to the propagation direction as illustrated in Fig. 31. In concern of the frequency values shear waves have lower frequencies than P-waves.

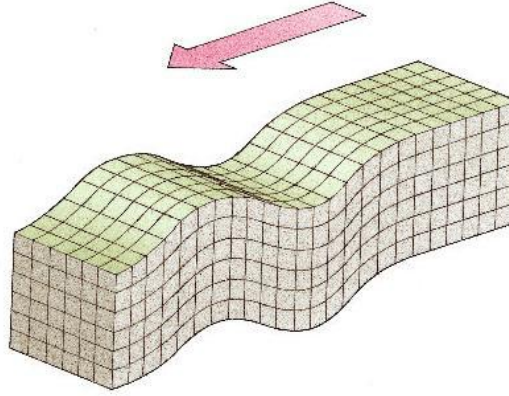


Fig. 31 – Scheme of shear wave

The shear wave velocity can be calculated as follows:

$$c_s = \sqrt{\frac{G}{\rho}} \quad (59)$$

As the name indicates surface waves only travel through the Earth's surface. They are a result of interaction between body waves, ground surface and upper layers in the ground. The surface waves travel slower than body waves, with low frequencies, long durations and higher amplitudes. The surface waves can be divided into Love and Rayleigh waves. The Love waves are generated from great amplitudes, thus the approach to such waves gains more importance when it comes to earthquake engineering [10].

The Rayleigh wave is the most important surface wave. This type of wave is very similar with the water surface wave. The Rayleigh wave moves ground up and down and back and forth in the direction of propagation. It travels along a cylindrical wave front way from the source (Fig. 32). Most of the shaking felt from an earthquake is caused by Rayleigh waves. Also, in the subject of ground-borne vibration due to the train traffic, the most energy is carried by Rayleigh waves, because the rail embankment is generally small compared with the wavelengths of shear waves expected to arise in the soil [2], which makes this as the most important type of waves.

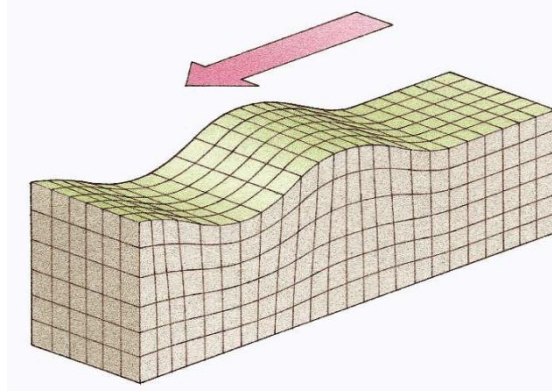


Fig. 32 – Scheme of Rayleigh wave

The Rayleigh wave is the slowest wave of the half-space, having a speed between 87% and 95% of the shear wave speed (depending on the Poisson's ratio of the material). The relationship between the Rayleigh wave velocity (C_R) and the shear wave velocity can be obtained through

$$C_R \approx \frac{0.87 + 1.12\nu}{1 + \nu} \cdot C_S \quad (60)$$

Poisson's ratio for soils is admitted to be around 0.3, so the velocity of Rayleigh waves is approximately 0.90 of the shear velocity.

According Miller & Pursey (1954) 67% of the energy from a circular oscillating vertically on the surface of an isotropic homogeneous half-space was transported as Rayleigh waves, 26% as shear waves and 7% as compression waves.

4

STUDY OF GROUND-BORNE VIBRATION DUE TO THE TRAIN TRAFFIC

4.1. INTRODUCTION

The achievement of good results in a software code is strongly dependent to how the physical processes are modeled. The numerical modeling should represent the motion of the materials, energy transfers, with the nearest approach possible, in reasonable time, which makes FLAC interesting software to do the soil analysis. Furthermore, this software has several algorithms for various issues related with the soil analysis, for example to stand out the different types of damping, the different constitutive models, absorbing boundaries and attached boundaries.

To achieve the results intended different models will be tested with different conditions related with ground borne vibration and its propagation through the half-space and its influence to how the foundation absorbs.

4.2. FLAC SOFTWARE

The study of dynamic motions in structures reveals to be a complex and hard process with or without informatic support. To be able to study of propagation of ground vibration in the soil, structure considered infinitely hiperestatic, with good accuracy reveals to be a hard task to be done with no informatic resources. The computer code has to have the ability to incorporate key factors and the phenomena that influences the behavior of the ground, when subjected to a dynamic loading, the passage of the train. As a guarantee of successs it is also important that the computer code is accepted by the geotechnical engineering community.

The computer code FLAC 5.0 (ITASCA, 2005) – Fast Lagrangian Analysis of Continua – was selected because it meets the criteria mentioned above. It was first developed by Peter Cundall in 1986. The code is explicit two-dimensional finite difference program that preforms a langrangian analysis. Explicit means it uses a time stepping procedure to solve the problem without forming the stiffness matrix. The lagrangian formulation enables the grid to move and deform with the material it represents since the incremental displacements are added to the coordinates.

This software is being used in the company *Ic – Consulenten*, since 1996, in allmost all complex projects that involve ground-vibration and the interaction soil-structure. The knowledge and the experience acquired with the use of FLAC are important factors to continue the bind.

One of the software problems regarding the process time has to do with the proportionality of the ratio between the major and the minor natural period of the system. For this reason the results obtained may not yield a right prediction in certain models that present materials stiffness modulus very distinct. In this case, the analysis of the vibration that affects a building due to the train traffic should be done separately from the analysis of the ground-borne vibration, i. e., the analysis of the ground vibration can be done with FLAC, recording the approximate signal in the foundation and continue the vibration study of the building with another software.

Despite of the fact that the program presents some “built-in” algorithms it is also possible to create new ones through the powerful computational code FISH, which presents extension *.fis*.

The fact that the program interface is not user-optimized makes it difficult to start dealing with dynamic complex problems without support.

4.2.1. TERMS AND CONCEPTS

FLAC is a finite difference numerical method that solves the differential equations in an explicit, time-stepping scheme. The equations of motion are solved for dynamic problems but are also solved when are the static equilibrium intended, which allows the software to follow the processes of dynamic equilibrium. This aspect has major importance when it comes to studying structure load which can exhibit nonlinear effects as for example the case of a slender column under axial compression or a soil subjected to an intense earthquake – structures physically unstable. This section will explain the concepts of finite differences and the “explicit, time-stepping” scheme and its importance for the success of the program.

4.2.1.1. Finite Differences

The finite difference method (FDM) is perhaps the oldest numerical technique used for the solution sets of differential equations. However, over the years, the finite element methods (FEM) have been favored. The FEM can combine the element matrices into a larger global stiffness matrix; in the case of FDM it is relatively efficient to regenerate the differential equations at each step. This is the main difference between the two methods.

With given initial and/or boundary conditions, the FDM replaces every derivative in the set of governing equations by an algebraic expression written in terms of the field variables (e.g. stress or displacement) at discrete points in space, these equations are called finite difference approximations. In contrast, the FEM has a central requirement where the field quantities (stress and displacement) vary throughout each element in a prescribed fashion, using specific functions controlled by parameters, in FDM there is no shape function since a linear variation between two points is assumed. The formulation involves the adjustment of these parameters to minimize error terms or energy terms. Both methods produce a set of algebraic equations to solve. Even though these equations are derived in quite different ways, in some cases it is easy to show that the resulting equations are identical in both methods [2].

4.2.1.2. Explicit, Time-Stepping Scheme

The FLAC software uses an “explicit, time-stepping” method to solve the algebraic equations. The procedure used to solve dynamic problems is also applied to solve static problems wherein a large

amount of damping is used, converting it into a quasi-static problem. The main reason is to ensure that the numerical scheme is stable when the physical system being modeled is unstable, as it can happen with nonlinear behavior.

The Fig. 33 represents a calculation cycle performed by FLAC. The procedure consists of reaching the dynamic equilibrium, invoking the equations of motion with new displacements and velocities, obtained from precede dynamic calculation cycle. With the new velocities found, the next step is to calculate the new stresses and strain and consequently the system will present a new arrangement of its stresses and forces. One timestep takes one cycle around the loop.

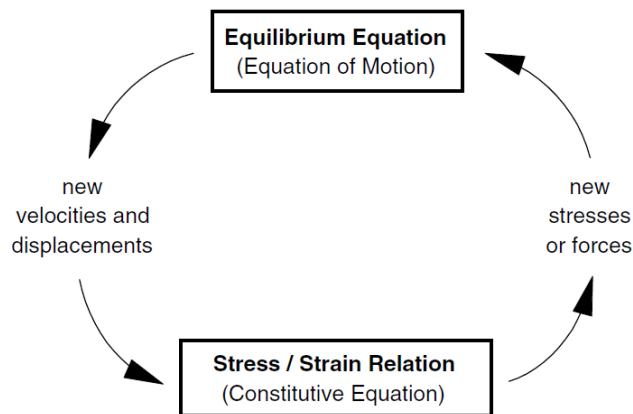


Fig. 33 – Calculation cycle performed by FLAC

It is important to realize that each box in Fig. 33 updates all of its grid variables from known values that remain fixed while the control is within the box. For example: the lower box takes the set of velocities already calculated and computes new stresses for each element, which will be used in the new equations of motion in the next step. The velocities are assumed to be “frozen” for the operation in the box. This may seem unreasonable because it is known that if a stress changes somewhere, it will influence the neighbors and change their velocities. However, the timestep chosen is so small that information cannot physically pass from one element to another in that interval. (All materials have a maximum speed at which information can propagate). Since one loop of the cycle occupies one timestep, the assumption of “frozen” velocities is justified. The central concept is that the calculation “wave speed” always keeps ahead of the physical wave speed, so that the equations always operate on known values that are fixed for the duration of the calculation. In the sub-chapter 4.3 the mathematical version will be presented. The most important characteristic of this method is that no iteration process is necessary when computing stresses from strains in an element (Itasca, 2005).

4.2.2. NUMERICAL FORMULATION

The obtained velocities and displacements from each element are then sent to the constitutive model to calculate the stresses and strains. These stresses and strains will be used to create the new equations of motion. Because there is no iteration the assumption that the neighboring elements are not affected within a time-step enables each element to be evaluated independently. This assumption is valid if the time-step used is small enough to ensure that there is no variation within a time-step. In the following

sections the numerical formulation will be presented, to understand its advantages as the inconveniences.

4.2.2.1. The Grid

To perform a calculation, FLAC divide the solid into a difference mesh composed of quadrilateral elements, however, internally the solid is subdivided into two overlaid sets of constants-strain triangular elements, as it is shown in the Fig. 34.

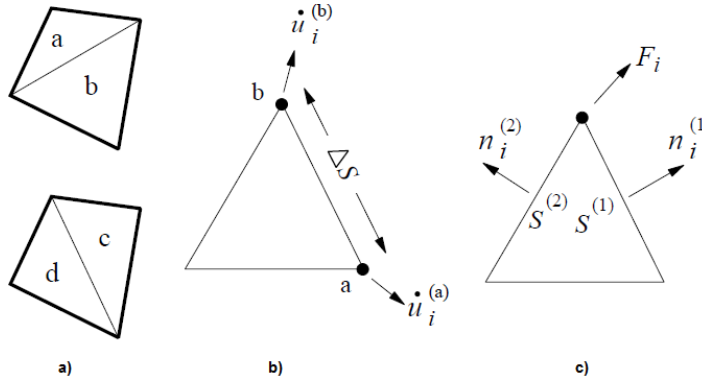


Fig. 34 – a) Overlaid quadrilateral elements in FLAC; b) Typical triangular element with velocity vectors; c) Nodal force vector (Itasca, 2005)

In the preceding figure two quadrilateral elements *a*) are represented, and each one is subdivided into two triangles *a*, *b*, *c* and *d*. Should be noted that the two quadrilateral elements represented relate to the same element, i.e. these two quadrilaterals are overlaid. It will be seen forward that FLAC use the average values calculated in these sub-triangles to find the properties of the element.

The deviatoric stress components of each triangle are maintained independently, requiring twelve stress components to be stored for each quadrilateral $4 \times (\sigma_{xx}, \sigma_{yy}, \sigma_{xy})$. The force vector exerted on each node is taken to be mean of the two vectors exerted by the two overlaid quadrilaterals. This way, the response of the composite elements is symmetric, for symmetric loading.

The use of triangular elements avoids the problem of hourglass deformations which may occur with constant-strain finite difference quadrilaterals. The phenomena “hourglassing”, is an effect that can occur in polygons with more than three nodes and is a result of some combinations of nodal displacements which produce no strain and result in no opposing forces.

4.2.2.2. Finite Difference Equations

For a continuous function f defined over the boundary of a closed surface or element the difference equations can be derived from the generalized form of Gauss’s divergence theorem (e.g. Malvern 1969).

$$\int_S n_i f ds = \int_A \frac{\partial f}{\partial x_i} dA \quad (61)$$

where $\int_S$ is the unit around the boundary of a closed surface; n_i is the unit normal to the surface, s ; f is a scalar tensor, vector or tensor; x_i are the position vectors; ds is the incremental arc length; and $\int_A$ is the integral over the surface.

Defining the average value of the gradient of f over the area A as

$$\left\langle \frac{\partial f}{\partial x_i} \right\rangle = \frac{1}{A} \int_A \frac{\partial f}{\partial x_i} dA \quad (62)$$

Substituting into Eq. (61) the Eq. (62), the finite difference form for a triangle sub-element is

$$\left\langle \frac{\partial f}{\partial x_i} \right\rangle = \frac{1}{A} \sum_s \left\langle f \right\rangle n_i \Delta s \quad (63)$$

Where Δs is the length of a side of the triangle, and the summation occurs over the three sides of the triangle. The value of $\left\langle f \right\rangle$ is taken to be the average over the side. With the Eq. (61) it is possible to write the strain rate $\dot{e}_{ij}$, in terms of nodal velocities $\dot{u}_i$ for a triangular sub-zone by substituting the average velocity of each side for f . The strain rate for the zone is average for the values of the triangular sub-zones. So, the equations are

$$\frac{\partial \dot{u}_i}{\partial x_j} \cong \frac{1}{2A} \sum_s \left(\dot{u}_i^{(a)} + \dot{u}_i^{(b)} \right) n_j \Delta s \quad (64)$$

$$\dot{e}_{ij} = \frac{1}{2} \left[\frac{\partial \dot{u}_i}{\partial x_j} + \frac{\partial \dot{u}_j}{\partial x_i} \right] \quad (65)$$

the summation is realized over the sides of the triangular sub-zone, and (a) and (b) are two consecutive nodes on a side. This expression is equivalent to a linear shape function between the two consecutive nodes.

With the strain rates calculated for each node, FLAC proceeds to the calculation of the stresses, through the constitutive model assigned. In this thesis, as was justified in the sub-chapter 3.1, it is admitted that the soil presents an elastic and isotropic behavior, so the relation of stress to strain in its incremental form is expressed by Hooke's law in plane strain as

$$\begin{aligned} \Delta \sigma_{11} &= \alpha_1 \Delta e_{11} + \alpha_2 \Delta e_{22} \\ \Delta \sigma_{22} &= \alpha_2 \Delta e_{11} + \alpha_1 \Delta e_{22} \\ \Delta \sigma_{12} &= 2G \Delta e_{12} \quad (\Delta \sigma_{12} = \Delta \sigma_{21}) \end{aligned} \quad (66)$$

where $\alpha_1 = K + (4/3).G$; $\alpha_2 = K - (4/3).G$; K and G are the bulk and shear modulus, respectively, and incremental strain tensor is

$$\Delta e_{ij} = \frac{1}{2} \left[\frac{\partial \dot{u}_i}{\partial x_j} + \frac{\partial \dot{u}_j}{\partial x_i} \right] \Delta t \quad (67)$$

where Δt is the time-step.

Once the stresses have been calculated, the equivalent forces applied to each nodal point are determined. The stresses acting in each triangular sub-zone act as tractions on the sides of the triangle, so each side of the triangle is composed by two equal forces. The corners receive two force contributions from each adjoining side, as it is showed in Fig. 34. The nodal force expression for a triangle corner is

$$F_i = \frac{1}{2} \sigma_{ij} \left(n_j^{(1)} \cdot S(1) + n_j^{(2)} \cdot S(2) \right) + \rho \cdot g_i \quad (68)$$

With the nodal forces from each triangle node, FLAC sums the forces from the two triangle sub-elements that compose each quadrilateral element. The forces of each quadrilateral set is then summed and averaged to provide the force contribution of the quadrilateral.

To determine the new velocity the central difference scheme to the equations of motion of the quadrilateral nodes is applied, resulting

$$\dot{u}_i^{(t+\Delta t/2)} = \dot{u}_i^{(t-\Delta t/2)} + \sum F_i^{(t)} \cdot \frac{\Delta t}{m} \quad (69)$$

where $\sum F_i$ corresponds to the net nodal vector force which results from the sum of all surrounding quadrilaterals forces; and the superscripts denote the time at which the corresponding variable is evaluated (Itasca, 2005). Resumed the FLAC algorithm follows the process shown in Fig. 35.

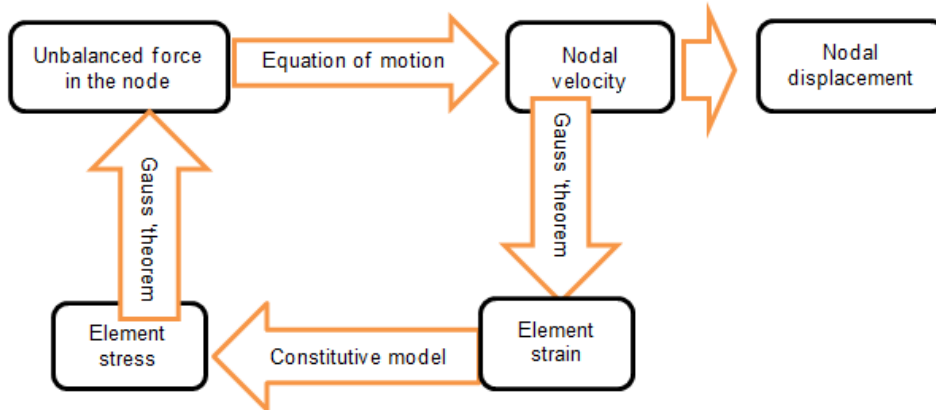


Fig. 35 – FLAC analysis

4.2.2.3. Time-Step

FLAC uses the explicit integration scheme which uses no iterations to find equilibrium among the elements. This means that changes in stress within a single element will not affect the calculated displacements, and therefore the neighboring elements, until the next step. This assumption is valid as long as the time-step is small enough to ensure that the calculation wave moves faster than the physical wave of the loading. The calculation of critical time-step involves contributions of stiffness and mass at each degree of freedom. For each triangular sub-zone, as it is shown in Fig. 34 b), the stiffness contribution (in units force/distance) is made from each three grid points of the sub-zone

$$k = \left(K + \frac{4}{3} G \right) \cdot \left(\frac{L^{max2}}{6 \cdot A_{\Delta}} \right) \quad (70)$$

where K and G are the bulk and shear modulus, respectively; L^{max} corresponds to the maximum edge-length of the triangle with area A_{Δ} . Thus for the full quadrilateral zone, the total contribution of stiffness from each of the four grid points is the summation of those for the three triangles meeting at the grid point

$$k_{nw} = \frac{\left(K + \frac{4}{3} G \right)}{6} \cdot \left(\frac{L_a^{max2}}{A_a} + \frac{L_b^{max2}}{A_b} + \frac{L_c^{max2}}{A_c} \right) \quad (71)$$

where A_n is the area of triangle n and L_n^{max} is the maximum edge length of triangle n . The same happens with the masses, which are also accumulated at zone grid points from each triangular sub-zone

$$M_{nw} = \frac{m_a + m_b + m_c}{6} \quad (72)$$

where m_n corresponds to the mass of triangle n .

In every grid point the stiffness and the mass are evaluated through the Eqs.(71) and (72), giving the total stiffness (k) and the total mass (M), respectively. The critical time-step for a single mass-spring system is obtained through the following Eq.(7).

$$\Delta t_{crit} = 2 \sqrt{\frac{M}{k}} \quad (73)$$

The critical time step is then calculated as the minimum (over all grid points) of the Eq.(73). However, the more general form, based on Eq.(73), is used in deriving the dynamic time step, Δt_d , using a safety factor of 0.5 (the calculation of time step an estimation). Thus

$$\Delta t_d = \min \left\{ \sqrt{\frac{\sum M}{\sum k}} \right\} \cdot \frac{1}{2} \quad (74)$$

where $\sum M$ and $\sum k$ corresponds to the summation of the masses and stiffness's that contribute for the grid point. The fact the model contains different element sizes, different materials connected in common grid points will contribute for the expression of Δt_d , and the resulting time step will be a combination of several conditions. Due to the fact that the models used contain a mesh that represent a concrete slab (small element size and high stiffness) the time step is substantially reduced.

The Rayleigh damping (stiffness-proportional damping) will also contribute for the definition of the dynamic time step. According Belytschko (1983), in order to guarantee the stability the dynamic critical time step, Δt_β , should be reduced due to the effect of stiffness-proportional damping

$$\Delta t_\beta = \left(\frac{2}{\omega_{max}} \right) \cdot (\sqrt{1 + \lambda^2} - \lambda) \quad (75)$$

where ω_{max} is the highest eigen-frequency of the system, and λ is the fraction of critical damping at this frequency. FLAC will determine ω_{max} and λ according the Eqs. (76) and (77), respectively.

$$\omega_{max} = \frac{2}{\Delta t_d} \quad (76)$$

$$\lambda = \frac{0.4\beta}{\Delta t_d} \quad (77)$$

where

$$\beta = \xi_{min}/\omega_{min} \quad (78)$$

where ξ_{min} and ω_{min} are the damping fraction and angular frequency specified for Rayleigh damping (Itasca, 2005).

4.2.2.4. Mechanical Damping

Natural dynamic systems contain some degree of damping of the vibration energy within the systems; otherwise, the system would oscillate indefinitely when subjected to driving forces. Damping occurs because of energy loss as a result of internal friction in the intact material. The energy loss is gradually converted to heat or sound. Due to the reduction of energy, the response, such as the displacement of the system, gradually decreases determining the attenuation of the vibration. Although, the amount of energy losses is relatively small so the determination of damping becomes important for an accurate

prediction of the vibration response of a system. A damper is assumed to have neither mass nor elasticity, and damping force exists only if there is relative velocity between the two ends of the idealized damper. Hence damping arises from many sources, it is difficult to describe analytically in a simple and comprehensive way so, damping assumptions are adopted to represent the damping existent in the ground.

The damping used in soil models encompasses two different mechanisms of energy dissipation, intrinsic damping and radiation damping. The intrinsic damping refers to the many distributed dissipative mechanisms in a structural system for the dissipation of the mechanical energy, being represented through Rayleigh damping; the radiation damping refers to the energy dissipation from a system via wave propagation to the surrounding area [11].

4.2.2.4.1. Rayleigh Damping

Viscous damping constitutes the most commonly used damping mechanism in vibration analysis. The amount of dissipated energy depends on many factors, such as the size and shape of the vibration body, the frequency of vibration, the velocity of the vibrating body, etc. In a system with viscous damping, its motion will be resisted by a force whose magnitude is proportional to that the velocity but in the opposite direction.

The simplest way to formulate a proportional damping is to make it proportional to either the mass or the stiffness because the undamped mode shapes are orthogonal with respect to each of these. Thus the damping matrix might be given by

$$c_v = \alpha.m \quad \text{or} \quad c_v = \beta.k \quad (79)$$

In which the proportionality constants α and β have units sec^{-1} and sec , respectively. Substituting the previous Eqs. (79), into the Eq. (25) results

$$\xi = \frac{\alpha}{2.\omega_n} \quad \text{or} \quad \xi = \frac{\beta.\omega_n}{2} \quad (80)$$

For mass proportional damping the damping ratio is inversely proportional to the frequency while for the stiffness proportional damping the frequency is directly in proportional, as it shows in data of Fig. 36. Once the dynamic response generally includes contributions from all N modes, these expressions won't be possible to use with MDOF systems these expressions because the frequencies of the significant modes span a wide range, which will distort the relative amplitudes of the different modes.

So Lord Rayleigh proposed that, the damping matrices of the elements could be expressed through a linear combination of mass and stiffness matrix,

$$c_v = \alpha.m + \beta.k \quad (81)$$

Substituting the Eqs.(80) in the expression (81) leads to the following expression which relates the damping ratio and frequency

$$\xi = \frac{\alpha}{2 \cdot \omega_n} + \frac{\beta \cdot \omega_n}{2} \quad (82)$$

Graphically this relation is showed in the Fig. 33, in the combined curve.

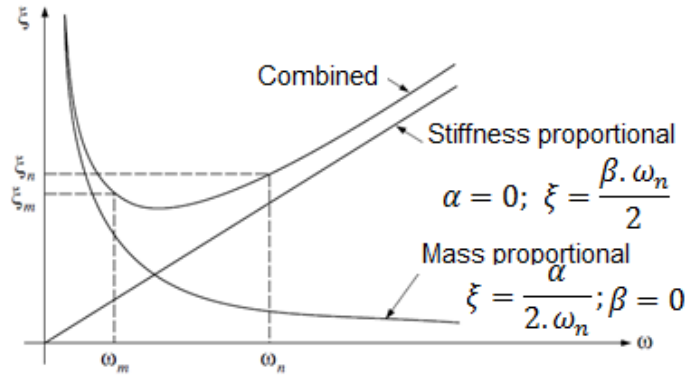


Fig. 36 – Relation between the damping ratio and the frequency in the Rayleigh damping

It is possible to find the constants α and β which can output a damping ratio for two different frequencies, approximately constant, through the following Eq. (83)

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2 \cdot \xi}{\omega_n + \omega_m} \cdot \begin{Bmatrix} \omega_n \cdot \omega_m \\ 1 \end{Bmatrix} \quad (83)$$

where ω_n and ω_m correspond to two specific frequencies (modes) known. When these factors have been evaluated, the proportional damping matrix that will give the required values of damping ratio at specified frequencies is given by Rayleigh damping Eq.(81), as shown in Fig. 36.

The halfspace is considered to be homogeneous, regarding the fact that the first layer presents a higher depth. As the system presents large degrees of freedom, the values of Rayleigh damping α and β may not be clear to represent it. So the minimum damping ratio of $\xi_{min} = 2\%$ is adopted with the correspondent frequency of 20Hz. The local damping present in the foundation slab is not being considered, since it can be considered that its influence can be neglected due to its high stiffness don't permit a significant damping of the vibration. Thus it is possible save computational effort, particularly in the time-step duration.

A shortcoming about the use of Rayleigh damping has to do with explicit scheme, i.e., as it was studied in the previous section 4.2.2.3 the existence of damping that depends from the frequency will shorten the time-step. The reduction of the time-step will increase the computation time of the models the higher the damping ratio is. Rafael Pascuali [103] tested the computation time of a model with different amounts of damping and summarized in the Table 2. With $\xi = 5\%$, the calculation can be

approximately 35 times longer than a model without damping. Even small variations in the value of Rayleigh damping can dramatically influence the overall solution time of a dynamic problem.

Table 2 – Time increase due to the utilization of Rayleigh damping in FLAC

Rayleigh damping, ξ	Initial time-steps (s)	Approximate relative time for solution
0.00E00	1.83E-05	1.0
1.00E-03	1.30E-05	1.4
1.00E-02	2.59E-06	7.1
5.00E-02	5.28E-07	34.7

4.2.2.4.2. Radiation Damping

The modeling of geotechnical problems involves media which at the scale of the analysis, are better represented as unbounded. So, the radiation damping is an important aspect when modeling the dynamic soil-structure interaction is made. In finite element analysis the region where the study of soil-structure interaction problems are often surrounded by boundary conditions that effectively mimic the dissipative effect of infinity, ideally absorbing all energy propagating outwards and allowing no energy back into the examined system. These boundary conditions are known as *transmitting boundaries* [11].

The boundaries which present conditions of zero-stress and zero displacement are called elementary boundaries, shown in Fig. 37 b). The impinging waves are not absorbed but reflected into the model, keeping the wave energy, which will be source of errors. To avoid significant errors the boundaries should be placed far enough so that the damping material will absorb most of the energy, although the computational effort will arise significantly.

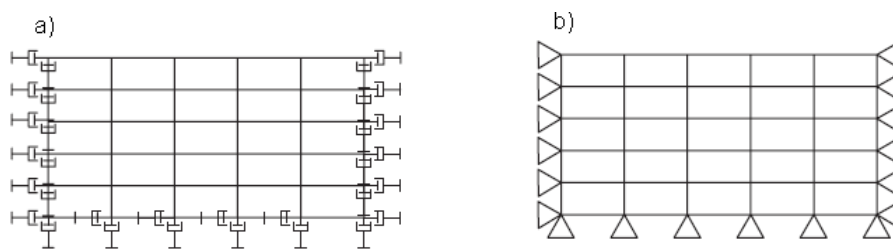


Fig. 37 – Transmitting boundaries: a) viscous boundary and b) elementary boundary

An alternative consists in the formulations of viscous boundaries, Fig. 37 a). One of the most common is the standard viscous boundary proposed by Lysmer and Kuhlemeyer. It consists of discrete dashpots attached to all degrees-of-freedom on the boundaries.

The coefficients for these dashpots are constant quantities only depending on the compressional and shear wave velocities. The evaluation of the viscous normal, t_n , and shear tractions, t_s , are given by

$$t_n = -\rho \times C_p \times v_n \quad (84)$$

$$t_s = -\rho \times C_s \times v_s \quad (85)$$

where ρ corresponds to the mass density; C_p and C_s are the p – and s – wave velocities; and v_n and v_s are the velocity component at the boundary, normal and shear direction respectively.

In FLAC this method is almost completely effective at absorbing body waves approaching the boundary at angles greater than 30° . For lower angles of incidence, or for surface waves, there is still energy absorption, but it is not perfect (Itasca, 2005).

The quiet boundaries, named by FLAC authors, are an important algorithm to solve the dynamic soil problems. It permits to model the portion of soil avoiding the wave reflections on the model limits, without it being necessary to use a large model which would consume much more calculation time.

4.3. NUMERICAL MODELING

For the analysis of ground-borne vibration due to the train traffic absorbed by the foundation slab, the interest of the modeling will be on the geometry of the slab, the weight of the construction, its location along the propagation path and on the type of concrete, especially its mass density. Therefore, the characteristics of the materials that constitute the half-space, the characteristics of the train (wheel force amplitude, shape function, etc.) and other aspects that influence the generation, propagation and reception won't be considered despite its importance in this type of study. As it will be studied, to do the parametric study of the several factors enunciated, it is necessary to calculate the static model due to the loads applied in the slab and the own weight of the half-space soil. The results from static models will be used as input of the dynamic models together with the dynamic load which simulates the train passage. In the Table 3 the models are presented which are of interest for this study.

Table 3 – Study models

Model Nr.	Slab characteristics			Distance of the slab from the source (m)
	Thickness (m)	Length (m)	Weight (kPa)	
1	0.50	10	100	30
2				40
3				50
4				60
5	1.00	10	100	30
6				40
7				50
8				60
9	1.50	10	100	30

10				40
11				50
12				60
13				30
14	2.00	10	100	40
15				50
16				60
17	0.50			
18	1.00			
19	1.50	20	100	30
20	2.00			
21	0.50			
22	1.00			
23	1.50	10	200	40
24	2.00			

4.3.1. ELEMENT SIZE

The element size, in numerical problems, reveals to be of great importance to calculate the ground-borne vibration. In order to save computation time, the element size should be as large as possible. However, the element size should be small enough to provide wave transmission between the elements and to present a good accuracy of the solution. The accuracy of the solution is governed by the frequency content of the input wave and the wave-speed characteristics of the system. Kuhlemeyer and Lysmer (1973) show that for accurate representation of wave transmission through a model, the spatial element size Δl , must be smaller than approximately one-tenth to one-eighth of the wavelength associated to the highest frequency component of the input wave

$$\Delta l \approx \frac{\lambda}{10} \quad (86)$$

where λ is the wavelength associated with the highest frequency component that contains appreciable energy. The wavelength in Eq(86), can be determined through the following expression

$$\lambda = \frac{c_s}{f_{max}} \quad (87)$$

where C_s is the S-wave velocity and f_{max} the highest frequency of interest. P-waves do not need to be considered in the determination of the element size, since their frequency is smaller than S-waves. The Fig. 38 shows the same model with two different element sizes. The mesh a), with fewer elements per wavelength presents an incorrect representation of the wave motion, whilst the second mesh b) presents its wave motion with more accuracy.

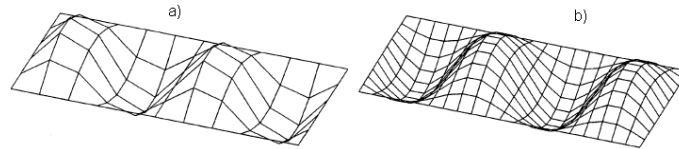


Fig. 38 – Representation accuracy

According [1], the ground vibrations induced by train traffic are mainly of low frequency character (0-20Hz), generated by the response of the track structure to the passage of the train. The higher frequencies, explained in the “Vibration Source” section, are generated by imperfections of the components of the train and the track, deflections between sleepers, etc. They have a frequency range between 70-110Hz [2] inside of the railway embankment. Notwithstanding high frequencies is not common outside of the railway embankment, once the embankment works as a filter of high frequencies.

In this thesis, the ground-borne vibration will be generated through the application of dynamic load directly on the surface, and thus the models won't contemplate the modeling of the ballast or the embankment.

Many authors suggest a frequency range up to about 200 Hz as relevant for railway ground vibrations; however, the range up to 100 Hz is the most important part, since resonances of walls, floors, and windows occur there. For the definition of the grid size a maximum frequency of 100Hz will be admitted. The soil properties used in the models and to determine the element dimension are resumed in the following Table 4.

Table 4 – Soil properties (*iC – Consulente*)

Soil Properties	
Soil density, ρ (Kg/m^3)	1814
Bulk Modulus, K (Pa)	5.12E8
Shear Modulus, G (Pa)	2.00E8
Shear velocity, C_s (m/s)	332
Wavelength, λ (m)	3.32
Element size, Δl (m)	0.33

The size of the elements that represent the soil will be 0,40 m. In the upper part of the model the grid elements have a dimension of 0,10 m, because the slab of the foundation, made by concrete, presents

higher stiffness. Therefore the decision was made to reduce the size element 4 times of the element size of the soil.

4.3.2. DIMENSIONS OF THE MODELS

To define the model dimensions the experience in similar cases of ground-borne vibration due to the train traffic should be taken into account. The model dimensions should not interfere in the propagation of vibrations however it should be a suitable problem in accordance with the informatics resources available.

Supported by field experience concerned ground-borne vibration due to the train traffic, it is known that the ground-borne vibrations, in very soft soils, can be felt till 200 m from the source, i.e., the railway line. However, the aim of this thesis is the study of the ground-borne vibration in a viennese soil type. The soil in Vienna basin which is located at the end of the northern Alps mainly consists of the following soil types:

- quaternary soil;
- neogene soil;
- rockmass of flysch.

In these soils and according the experience there, the ground-borne vibration can be considered as a problem for habitation up to distance of 40m away from the railway. As such the dimensions of the model should permit the study the vibrations felt in relevant points. To have a full description of the ground-borne vibration behavior over the surface a model with 70m length a record of vibration signals every each 10m from the source seems to be the most reasonable.

Due to the fact that the stress pattern over the static model will vary in depth, it is important to provide the model with dimensions that permits a good accuracy of the results presented by dynamic calculation. Thus, a 50m depth for all models were adopted.

4.3.3. IDEALIZED MODELS

The dynamic modeling is perhaps the most important phase of this study, once the success of it lies on the effectiveness of the dynamic modeling of the site conditions which are intended to be studied and the software used to do the calculation. The representation of the site conditions which it is intended to study will be done setting the initial conditions and choosing a model that simulates the dynamic behavior of the soil, i.e. the constitutive model. As explained in the sub-chapter 3.2, the constitutive model used to reproduce the deformations that occur due to the ground-borne vibrations caused by train traffic is the elastic model. With the initial conditions the model aims to assign the conditions present in the site. The best-suited initial conditions that could use are obtained by field measurement. However, the performance of measurements to achieve the properties of the soil, damping decreasing, etc. would be very expensive, so the modeling parameters have to be supported in similar projects or/and scientific information which needs to be available.

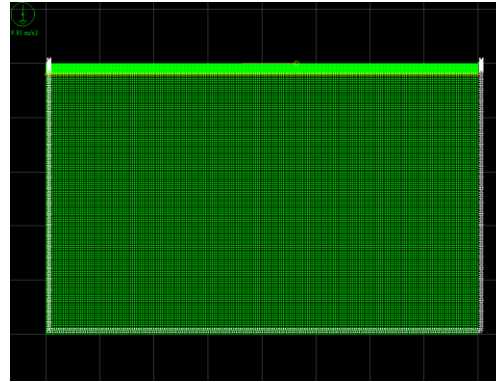


Fig. 39 – Static model

Each model used to make a parametric study about the weight and slab geometry is in fact constituted by two sub-models.

Static Calculation

The first sub-model respects a static calculation. In this calculation the stress gradients all over the model are obtained and have as initial conditions the applied loads, the load of the soil itself due to the gravity acceleration and the weight of the construction, idealized as a distributed stress over foundation slab. The support conditions were performed to idealize the portion of soil as having vertical deformations along the vertical limits, however the horizontal displacements are null, once the portion of soil analyzed is idealized as confined by the remaining soil. Perhaps, because the horizontal deformations are null it can introduces some error in the stress pattern, calculated by the static model, however the error introduced is not relevant and the points with interest to measure the vibrations are located away from the limits. In the Fig. 39 the mesh of the two sub-models is represented.

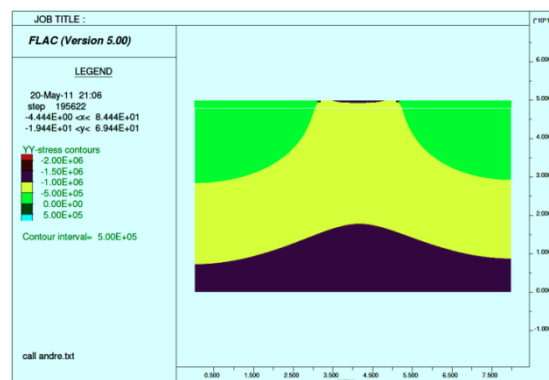


Fig. 40 – Stress pattern

Dynamic Calculation

Finished the static sub-model and obtained the stress gradients (Fig. 40) over the model, this result will be used as an input on the second sub-model, the dynamic model. Jointly with the stress gradients a dynamic loading will be applied corresponding to the wheel force, shown in the data of Fig. 41.

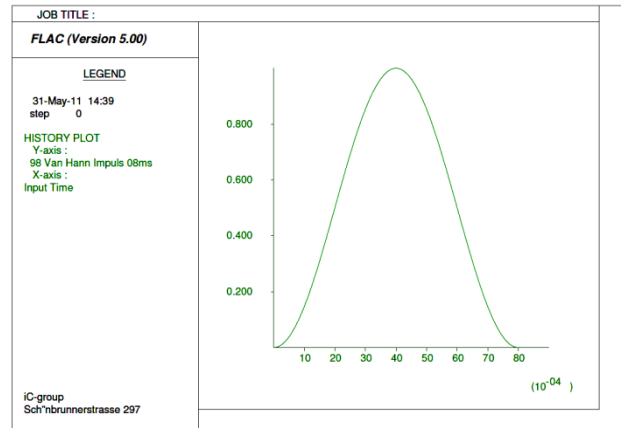


Fig. 41 – Dynamic loading

With the dynamic load, presented in the data of Fig. 37, a simulation of the excitation source produced by the passage of trains in the ground is intended, wherein the maximum amplitude represents the high load transmitted by the wheels of the train to the track. To represent the variation of the amplitude the *von Hann* function was chosen, because the deformation imposed by the wheel of the train to the rail is higher in the position of the axle of the wheel (middle) and decreases in direction to both sides, symmetrically. Its abrupt application, 8 milliseconds, together with the shape function used permits the generation of vibration frequencies within a wide span range, approximately from 1Hz to 100Hz. Thus, it is obtained a transfer function for a wide span of vibration frequencies. The amplitude of the dynamic load was considered 2000 kg. This value doesn't correspond to the real loads transferred to the ground by conventional trains at conventional velocities. As the objective of this thesis is to obtain the transfer functions in elastic scheme for the several studied characteristics, this aspect won't object of full study.

5

EQUIPMENT DESCRIPTION AND MEASUREMENTS

5.1. INTRODUCTION

In this chapter a description of the equipment employed by *iC-Consulteren* on vibration measurements will be done. Also, a summary of results obtained from assessments made will be present in order to help in the interpretation of the model results obtained.

In order to have an overview of the results obtained from the 50 vibration assessments done, the data is presented in function of amplification factor which is related with the vibration frequency and slab thickness.

5.2. DESCRIPTION OF THE INSTRUMENTS USED FOR THE MEASUREMENTS USED WITHIN THE OBJECTIVE WORK

The measurements done for the different structures consist into obtain the transfer function of the vibration received by the slab using the system called “*VibroStar*”. This measurement system was developed by *iC-Consulteren* during the last 4 years, especially for vibration measurements.

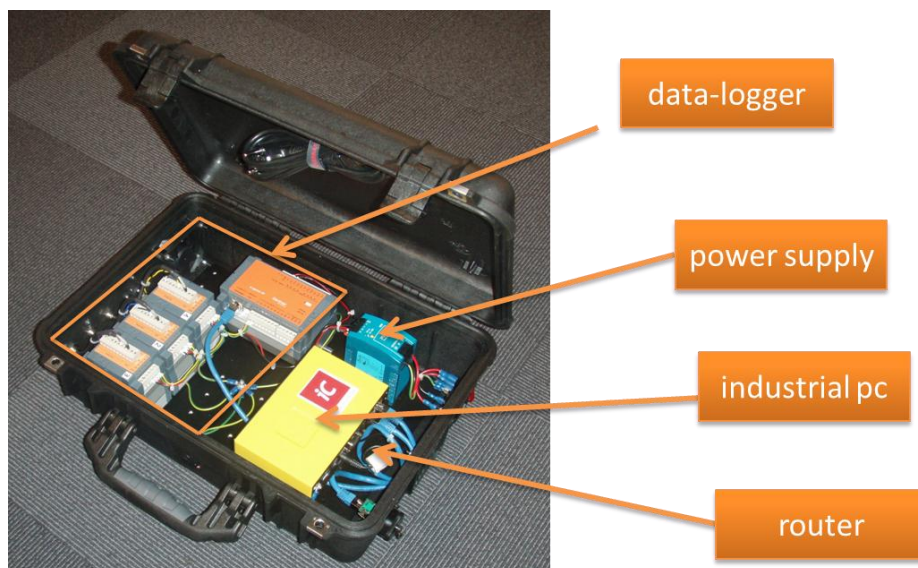


Fig. 42 – Measurement system *VibroStar*

This measurement system mainly consists of a 6-channel data-logger (*VibroStar 6000*) or a 3-channel data-logger (*VibroStar 3000*), an industrial PC and a GSM/UMTS router to provide internet access that enables communication with the instrument. The following Fig. 42 shows a picture of *VibroStar 3000* system. The main parts of this system are marked and described respectively.

The used data-logger provides an input measurement range of ± 10 V with selectable amplification factors (1, 10, 100, and 1000). Additionally, this type of data-logger contains a buffered memory, so the data can be stored for about 5 min, depending on the sampling frequency and the amount of channels. The industrial PC in the *VibroStar* System is a general purpose PC with enlarged temperature range, so that this type of PC can be used outside as well as inside buildings.

A geophone (vibration velocity transducer) of type *MST-1005* is used as vibration transducer. It was developed by *Walesch Electronic* (Switzerland). This type of transducer is a 3-component transducer, which means that the vibration velocity of all 3 directions (X, Y, Z) can be recorded simultaneously. The pictures in Fig. 43 show the vibration-velocity transducer as well as a showcase of a calibration data sheet according to this type of vibration transducer.

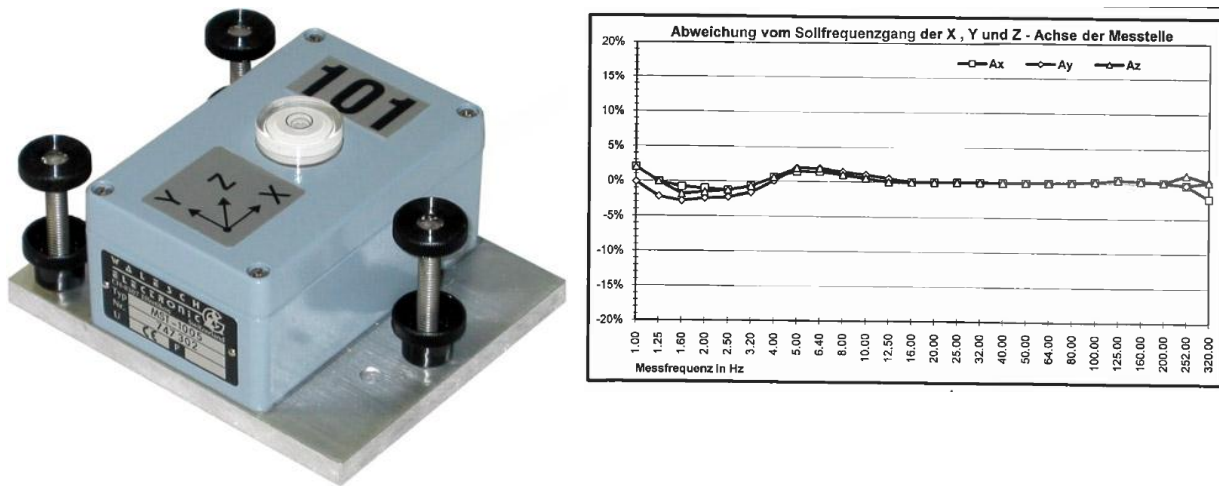


Fig. 43 – The 3-component vibration transducer

As visible in Fig. 43 the calibration data sheet of the transducer shows the frequency transfer function of the sensor. The behavior of the frequency transfer function shows a nearly linear behavior within the frequency range of this transducer that means that there is more or less no frequency dependent relationship between the input vibration and the recorded vibration velocity. The frequency range where the deviance from zero is a little bit higher is the frequency range where the natural frequency of this sensor (coil – spring – magnet) is located.

Basically the *VibroStar* system works as described in the following clause:

The data-logger records the measured data from the vibrations-transducer permanently. As described above this is realized by an internal memory circle buffer of 8 mb. Every 4 minutes the industrial PC reads out the data of the last 4 minutes, the circle buffer of the data-logger is deleted and restarted simultaneously. With the recorded data the industrial PC calculates the vector of the vibration velocity. This means that the 3-orthogonal components are combined to the resulting vector due to the following equation:

$$V = \sqrt{x^2 + y^2 + z^2} \quad (88)$$

The maximum of the calculated vector is transmitted over the internet to a server, so that the data values can be observed in “real-time” in the internet.

Additionally the raw data is saved on the industrial PC, so that all types of signal processing algorithms can be applied afterwards. The afterwards plotted Fig. 44 shows a time history and the frequency analysis which belongs to this time history.

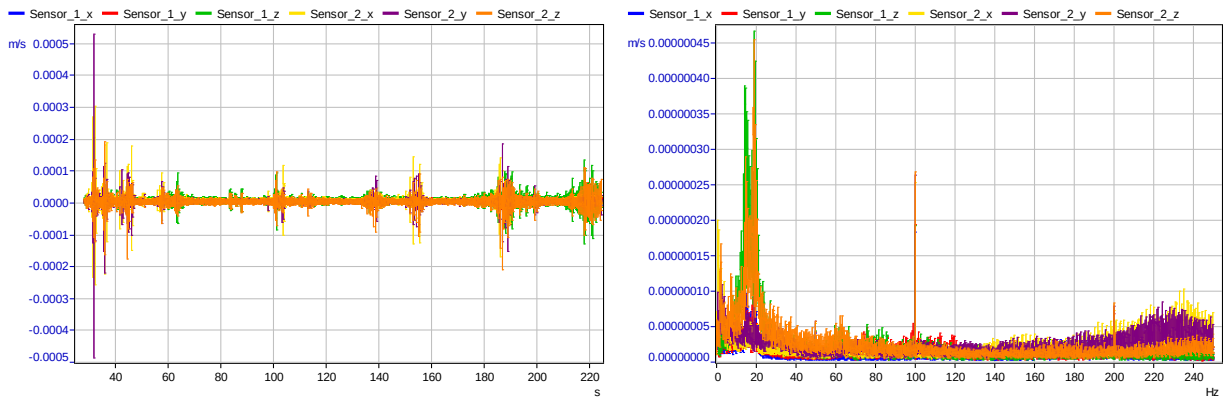


Fig. 44 – An exemplarily time history and frequency analysis recorded by *VibroStar*

5.3. MEASUREMENTS CARRIED OUT FOR THE OBJECTIVE WORK

To get the behavior of the vibration transmission from soil to the foundation slab a considerable amount of measurements were carried out. The general execution of the measurements was the following:

One vibration sensor (geophone) was situated in front of the building, 5 m away from the building. Another sensor was located on the foundation of the building. As excitation for the vibrations on the one hand trains from a nearby railway line were used and on the other hand small blastings was used.

For the evaluation of the transfer function, the frequency spectrum for both sensors was calculated. Afterwards by subtracting (logarithmic rules) the results of the sensor on the foundation from the sensor outside, the frequency depending transfer function was calculated.

5.4. RESULTS OF THE MEASUREMENTS

The following charts show the results of the measurements done for the objective work. The first chart shows an overview of the results of all measurements, Fig. 45. Plotted is a chart which shows the amplification factor in relation to the frequency (third band octave spectrum).

The overview of all the results show, at the very low frequencies (below approximately 10 Hz), the values of amplification factor within range 0.7 and 1.5. As all results here presented were measured in the foundation slab, the amplification factor which presents values superior to 1.0 are caused by the fact that the frequencies of the vibration generated reach the natural frequency of vibration of the

building, causing resonance. The results presented in this chapter concern to slabs with thicknesses which vary between 25 cm and 65 cm, Fig. 47.

In the frequency range between 12.5 Hz and 100 Hz the amplification factor shows values between approximately 0.1 and 0.6. This means that the vibrations in this frequency range are affected by the structure as it will be analyzed in following figures. The reduction occurred varies between 40 % and 90 % from outside the structure to the structure itself.

Above the frequency range of 100 Hz the amplification factor increases again. The reason for this has to do with the noise and other disturbing factors.

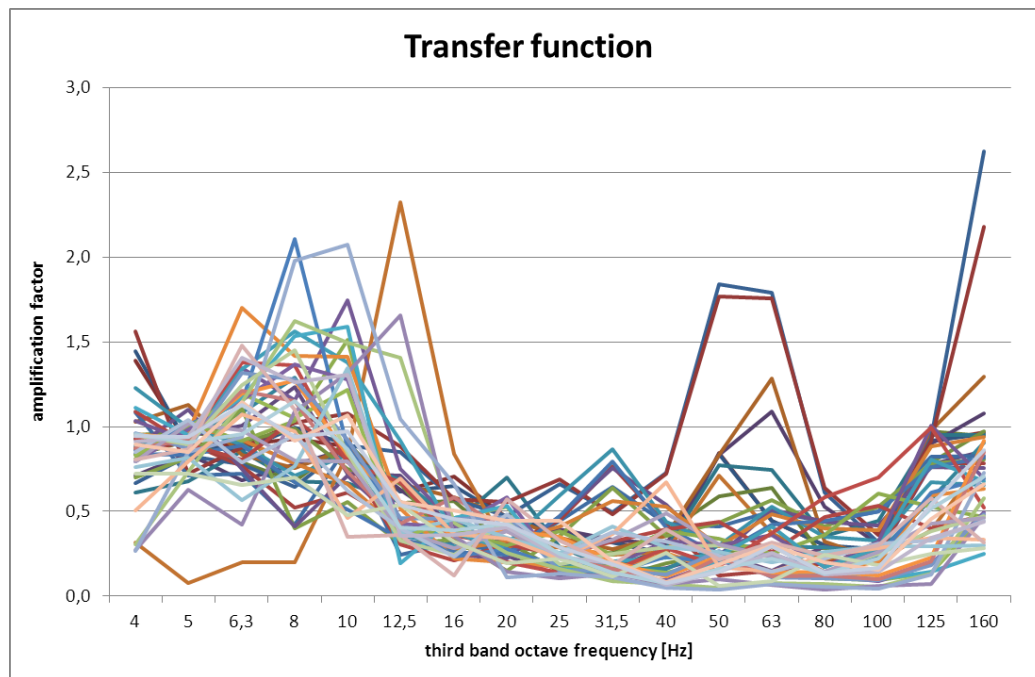


Fig. 45 – Summary plot of all measurements

The following chart, Fig. 46, shows an overview of the averaged amplification factors of all measurements. For every single measurement, the amplification factor was averaged for all frequencies observed.

As the results from the chart show the averaged amplification factors, Fig. 46, are within the range of about 0.4 and 1.0, but more than 90 % of the measured amplification factors are within the range of 0.4 and 0.7.

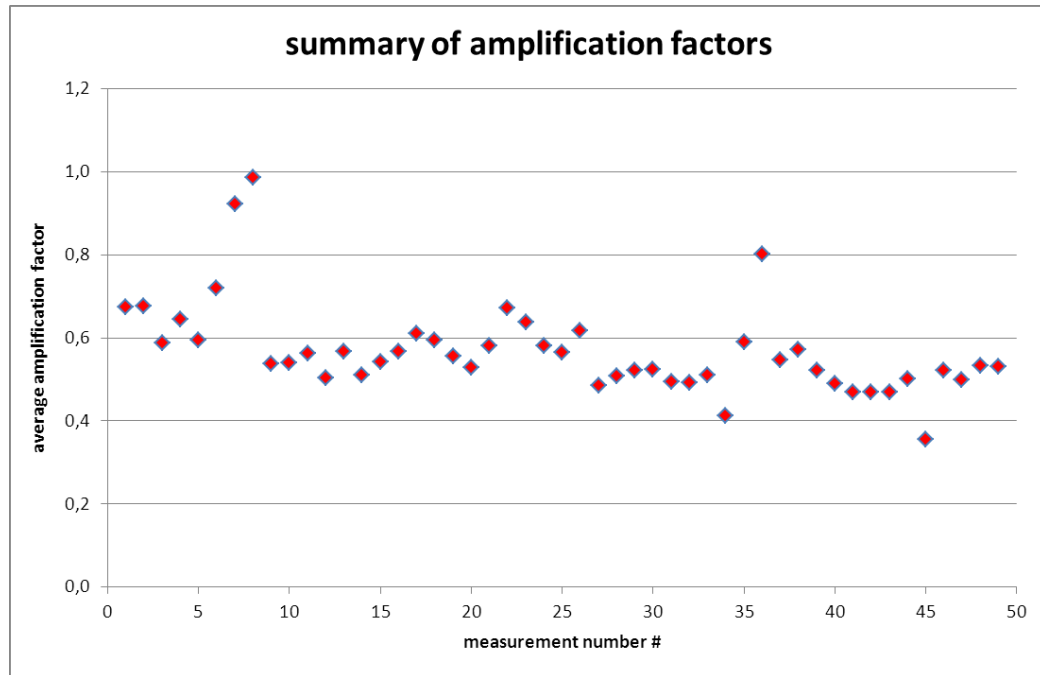


Fig. 46 – Summary plot of all measurements – average of amplification factors

To get a relationship between the amplification factor and the foundation thickness the following plot shows an overview. Additionally, a regression line including the equation and the correlation value is presented.

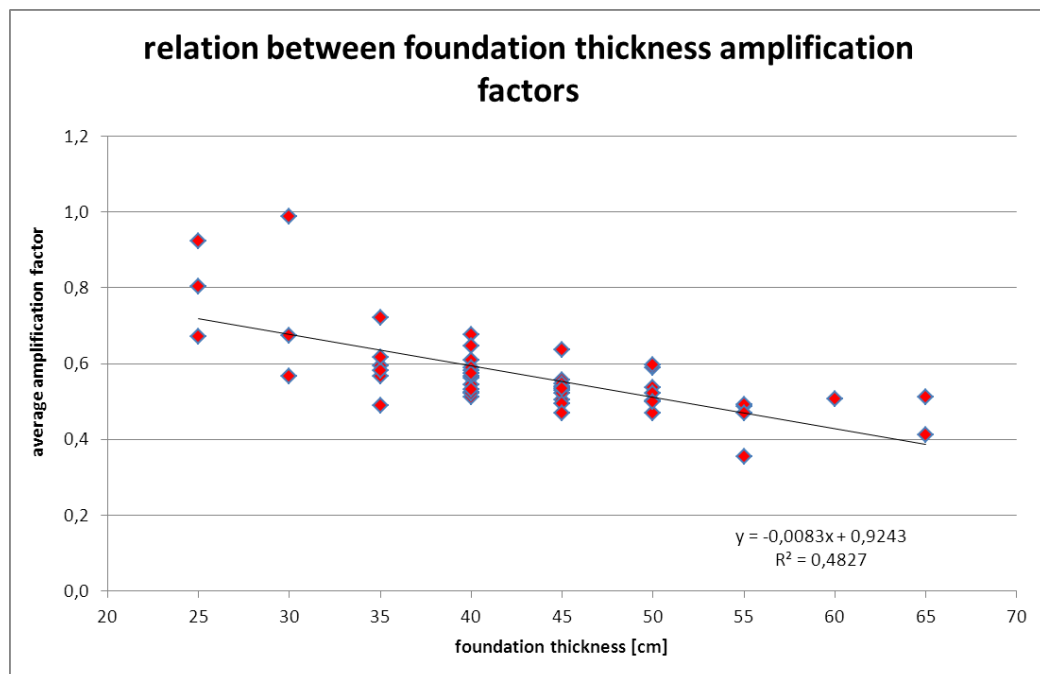


Fig. 47 – Relation between foundation thickness amplification factors

As the figure above shows, there is a relationship between the foundation thickness and the amplification factor. From the Fig. 47, one can recognize that with increasing foundation thickness the amplification factor decreases. That means that with increasing foundation thickness the vibrations coming from the surrounding soil are reduced. However, as the chart shows the relationship can't be described by the correlation coefficient R^2 . The correlation coefficient shows a value of about 0.5.

As the results from the measurements show, the foundation thickness has an influence to the vibration induced to a structure from outside. A comparison between the measured amplification factor and the foundation factor shows that with increasing foundation thickness the vibrations induced to a certain structure are reduced. However, as the results from the regression evaluation show, the amplification values are distributed within a considerable range. The results from the frequency evaluation show that the amplification factor generally decreases with increasing frequency up to a frequency of about 100 Hz. That means, from a practical view, that higher frequencies are reduced better than lower frequencies. That's important for evaluation of re-radiated noise.

Generally the results for the relationship between the foundation thickness and the amplification factor show that it is necessary to make a numerical calculation for a certain structure because the correlation factor for this relationship is rather low.

6

PARAMETRIC STUDY

6.1. INTRODUCTION

In a total of 24 models were calculated for different geometric situations slabs with different simulated weights, situated along the path. It was intended to evaluate the reception of ground vibration by several slab for the different thicknesses presented. The first recording point is located 10m away from the source, a dynamic load application point. The remaining points are separated from each other by 10m, Fig. 48. The recording point in the slab is located in its middle length. After the application of the dynamic load, the wheel force, the produced ground vibration will travel over the model until it reaches the silent boundaries, which work as radiation damping, as it is shown in Fig. 50. To reach the last history point of the model, i.e. the point located at 70m from the source, the slower waves (Rayleigh waves), will do it before 0.4sec. The algorithm for this model is presented in Appendix A.

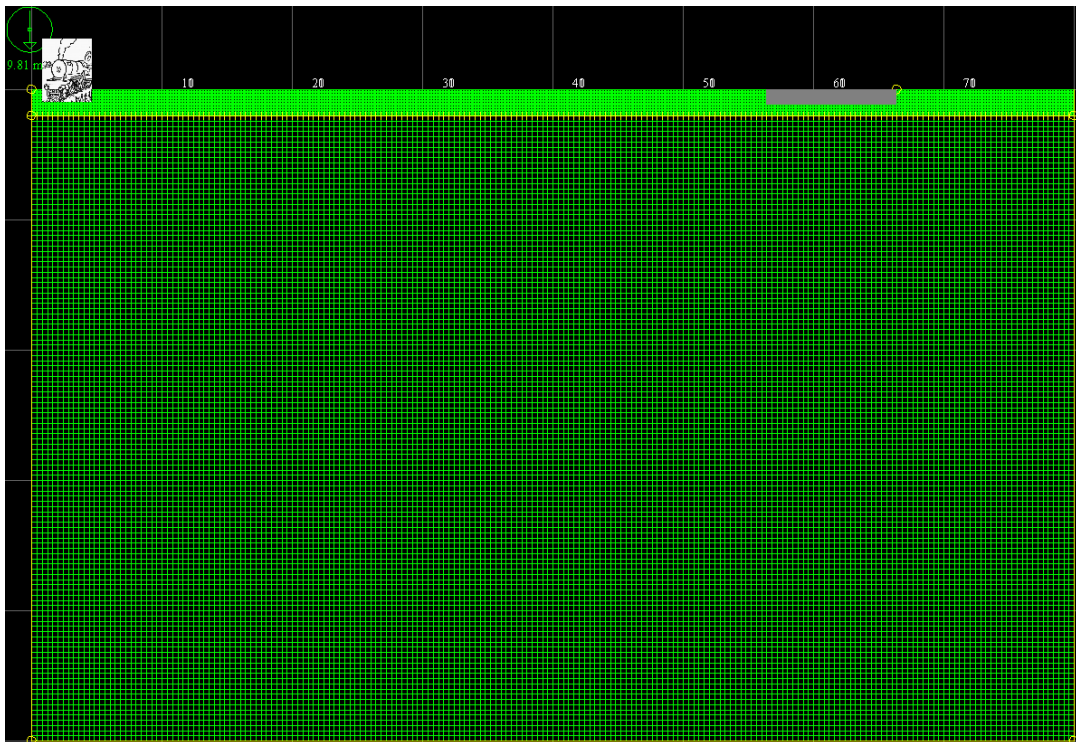


Fig. 48 – FLAC model with slab 1.0m thick located 60m away from the source

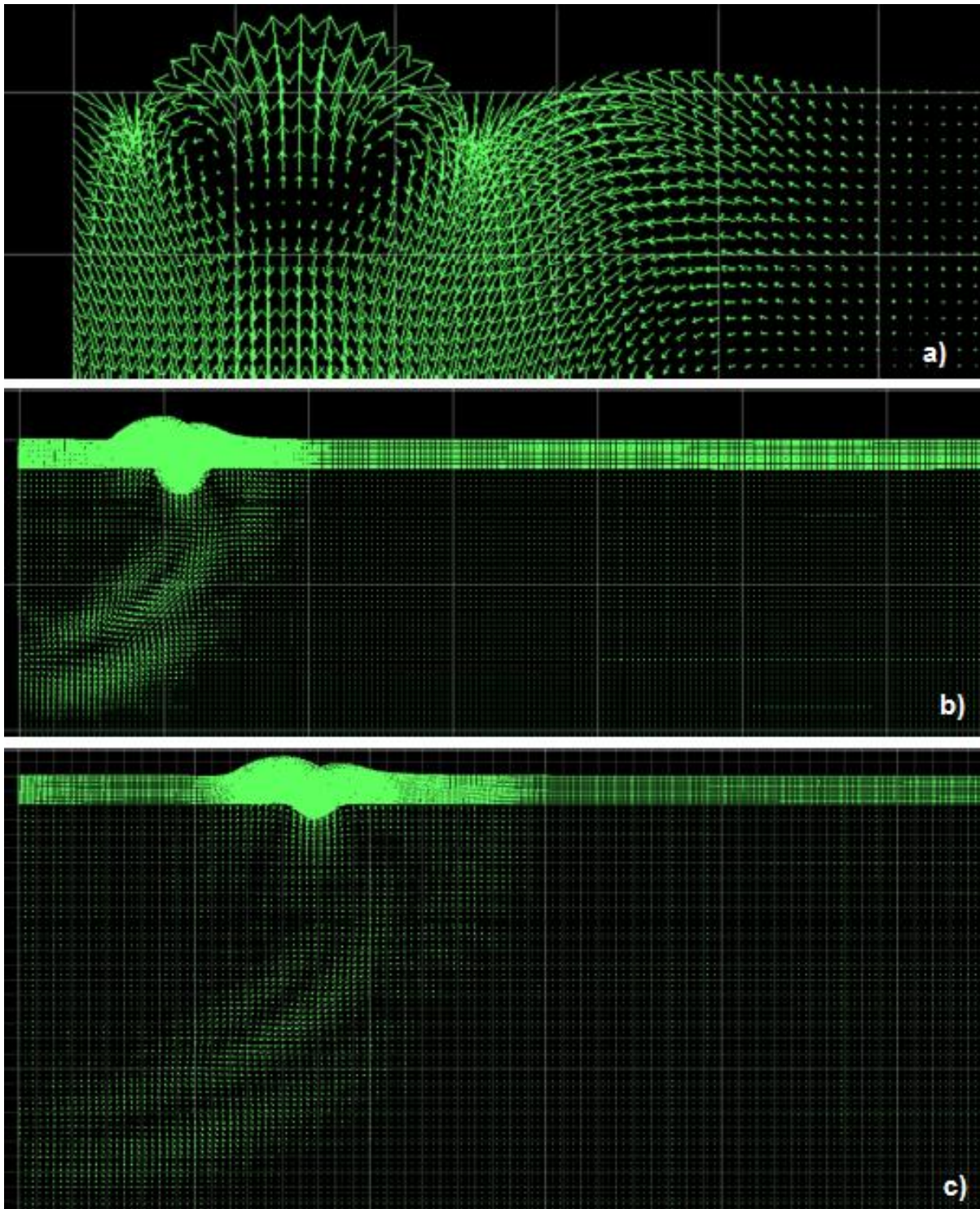


Fig. 49 – Generation and propagation of seismic waves

The Fig. 49 a) is shows the generation of ground vibration, as Hall (2000) said about, the moving train will produce a variation of stress patterns in the ground beneath and around the track caused by the train weight (Fig. 8), the so-called wheel forces. The others vibration sources, as the irregularities of the tracks, explained in the section 2.2.1 won't be considered for this study. In the Fig. 49 b) and c) are

represented the propagation of the seismic waves. As expectable the seismic waves as they move away from the source tend to distance themselves due to its different velocities.

The models used to calculate the ground vibration felt along the path, are summarized in Table 3. From these models a large quantity of information will be obtained, being necessary to analyze them carefully in order to resume it and extract the information that is of interest for this report, the transfer functions that can provide a relation between the model results. After the calculation of the dynamic models, the models will export the velocity history during the dynamic time considered, and with this plots will be performed all this study.

6.2. RESULTS

Due to the large quantity of data obtained from the FLAC models, once each model record the wave velocity in 7 points, as previously described. In each recording point the particle velocity in x-, y-direction and its vector plots will be produced, generating 21 velocity plots for each model, on total 168 velocity plots for this study. The Appendix B contains the absolute values of the ground vibration obtained from the models calculated. With these values the conclusions were obtained.

The information presented in this thesis and the conclusions are completely focused on the study of the influence of the slab thickness on the reception of ground vibration. Information as the particle velocity experimented along the way before and after the seismic waves hit the slab and its possible incidence for the following buildings won't object of study. However this information is an important means to detect eventual errors.

In the following Fig. 50 shows the evolution of ground vibration along the propagation path for the model represented in Fig. 49. As expected the ground vibration hits the slab presents a logarithmic decrement as result of the Rayleigh damping application, with similar values for the four models with different slab thicknesses.

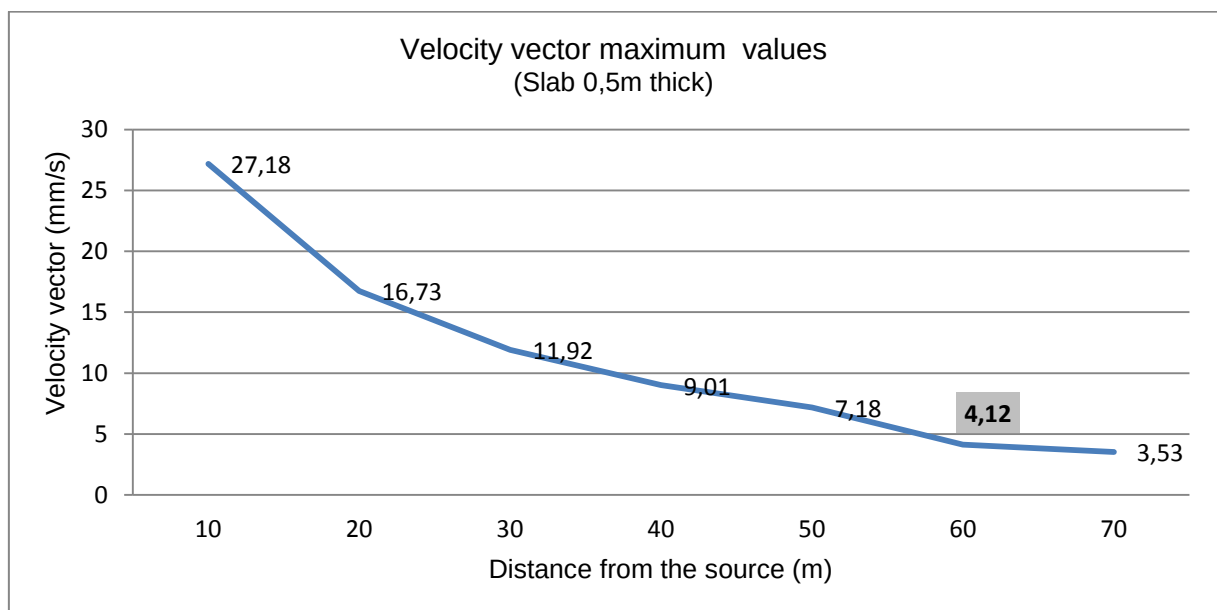


Fig. 50 – Velocity vector maximum values recorded from a model with a slab 1,0m thick located 60m from the source

The parametric study is focused in the point located in the middle of the slab wherein is recorded the ground vibration, marked in the data of Fig. 50 with gray color. For this parametric study, the most important results are the velocities recorded in the slabs. The velocity was recorded on top of the slab for the distances and conditions already enounced in Chapter 4.

The purpose of the evaluation that follows is the change in velocity received by the slab when increasing the slab thickness.

The following data present in the next figures correspond to the relative values of ground vibration relatively on the vibration recorded on the slab with 0,5m thick. With increasing the slab thickness, the vibration recorded at the top of the slab will decrease for the different locations of the slab.

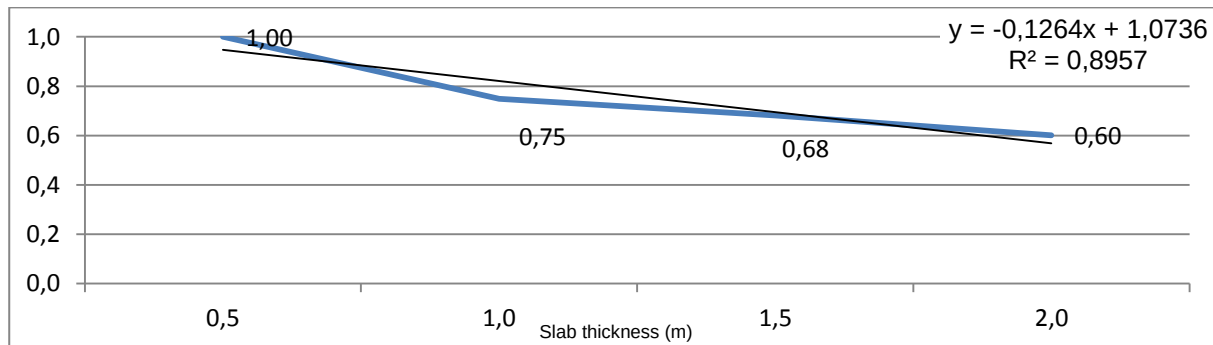


Fig. 51 – Variation of ground vibration felt in the slab located 30m away from the source relatively to a slab with 0,5m thick and the correspondent trend line

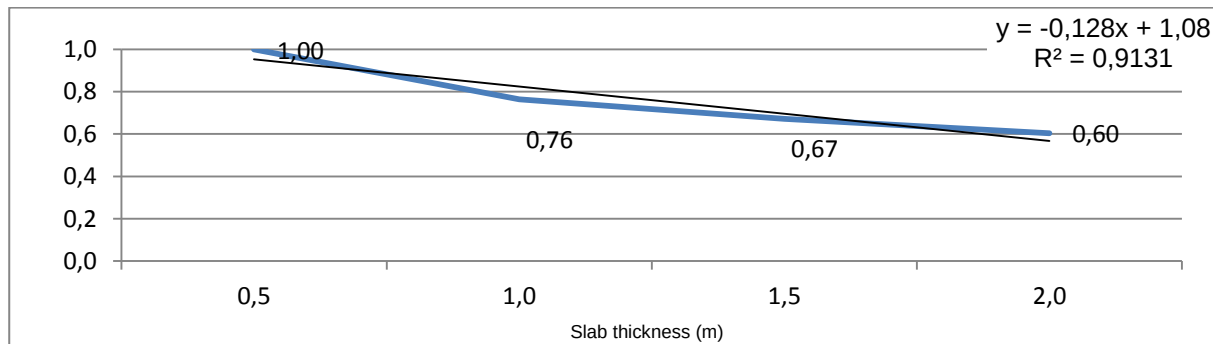


Fig. 52 – Variation of ground vibration felt in the slab located 40m away from the source relatively to a slab with 0,5m thick and the correspondent trend line

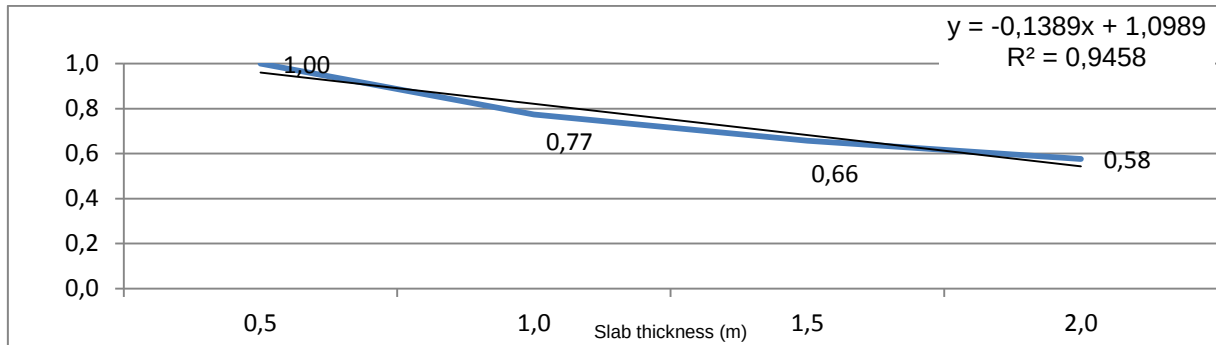


Fig. 53 - Variation of ground vibration felt in the slab located 50m away from the source relatively to a slab with 0,5m thick and the correspondent trend line

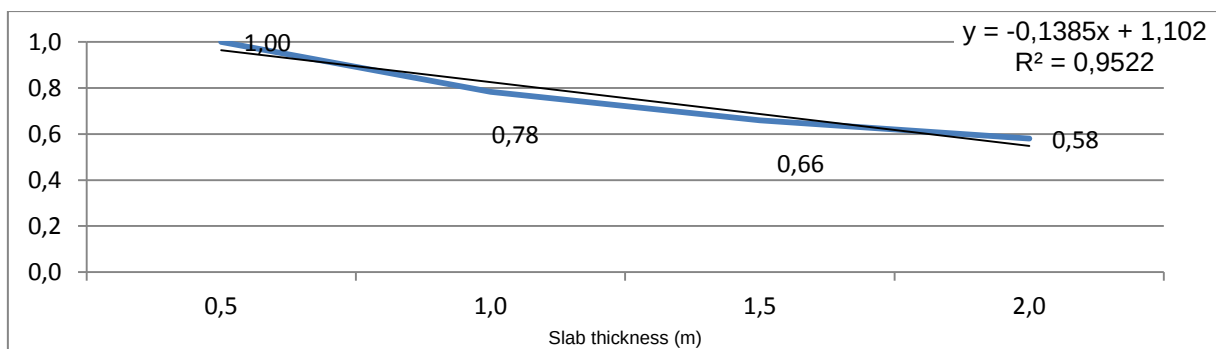


Fig. 54 - Variation of ground vibration felt in the slab located 60m away from the source relatively to a slab with 0,5m thick and the correspondent trend line

From the present data in Fig. 51, Fig. 52, Fig. 53 and Fig. 54 one can conclude that the variation of ground vibration received by a foundation slab follow approximately a linear law since all the models at distances of 30, 40, 50 and 60m present a R-squared value superior to 0.90 which reveals a good approximation. In the following Fig. 55 shows an average of the values obtained from the distances above enounced.

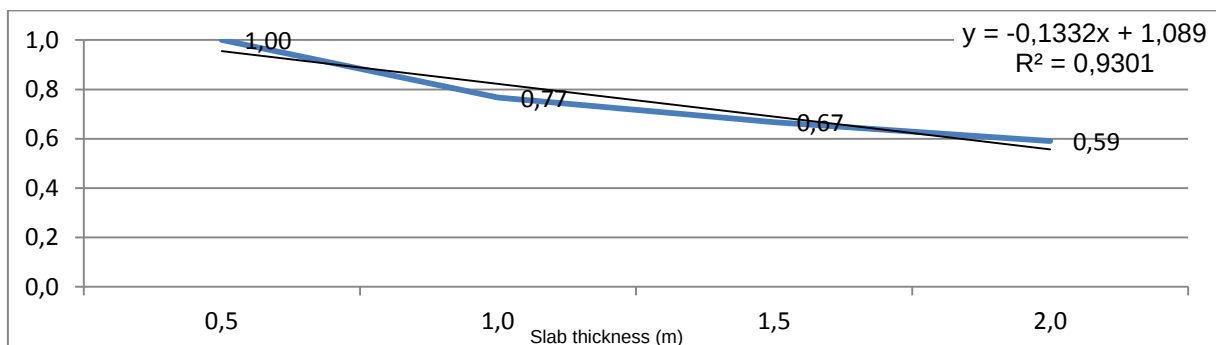


Fig. 55 – Average of variation of ground vibration felt in the slab relatively to a slab with 0,5m thick

From this last data, one can conclude that for a slab 10m length, a linear variation between the reception of ground vibration and the slab thickness will occur. The relation can be quantified as 0.21. This means that for each meter of slab thickness incremented the vibration in the top of the slab recorded decreases around 21% from the original value.

As it is expectable the building weight won't influence the level of ground vibration received by the slab since it is admitted that the wave propagation occurs in elastic scheme. In the following data, presented in Fig. 56, it is presented the results for a model with a slab, 10m length, located 40m away from the source subjected to uniform distributed load of 200kPa.

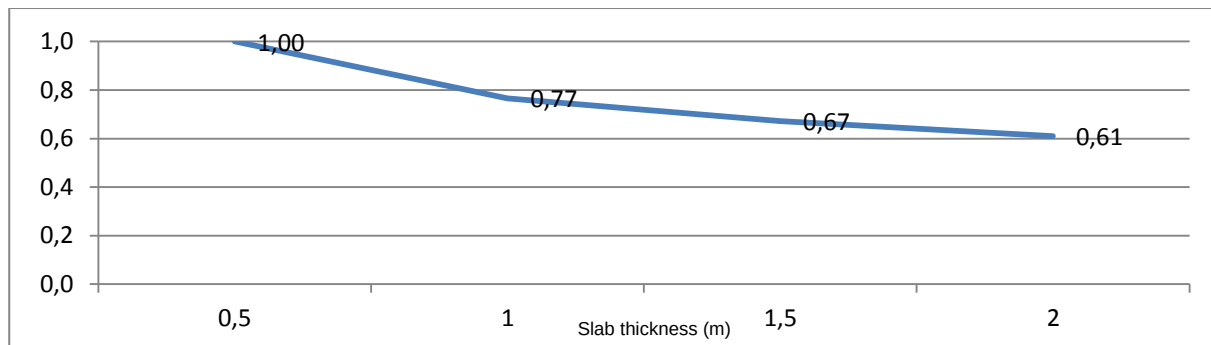


Fig. 56 - Variation of ground vibration felt in the slab located 40m away from the source relatively to a slab with 0,5m thick under 200kPa

The values obtained in this model are coincident with those obtained for a slab under 100kPa, showed in Fig. 52. These results prove that the weight transmitted to the ground won't influence the reception of ground vibration. This is true for points located far from the source wherein the wave propagation does not cause an alteration of the soil properties, especially the soil shear modulus, reason why it was admitted the elastic scheme for this study. However, in points near the source, i.e. near the railway line it is admitted that the ground present a plastic scheme as a result of higher shear strain. In these cases it is necessary to do a non-linear study admitting that the existing damping is not dependent of the vibration frequency but dependent on the shear strain, i.e. hysteretic damping.

Once obtained the variation law of vibration reception for different thicknesses of the foundation slab 10m length, the characterization of the vibration frequency that reaches the slab will be also analyzed. The study of the vibration frequency changes due to the increasing of slab thickness is important to have sensibility to know how the frequencies will be affected when the slab thickness is increased. The vibration frequency characterization will be done using the 1/3 octave bands.

The data presented in Fig. 57, Fig. 62, Fig. 67 and Fig. 68 shows that the variation of the vibration frequency between two thickness slabs: 0.5m vs. 1.0m, 0.5m vs. 2.0m, 1.0m vs. 1.5m and 1.0m vs. 2.0m, respectively; for slabs located 30, 40, 50 and 60m from the source. The Fig. 58, Fig. 59, Fig. 60 and Fig. 61 compare the absolute values recorded of the vibration frequencies for the slabs with 0.5m and 1.0m in the positions 30, 40, 50 and 60m, respectively. The Fig. 61, Fig. 64, Fig. 65 and Fig. 66 present data that compare the absolute values recorded of the vibration frequencies for the slabs with 0.5m and 1.0m in the positions 30, 40, 50 and 60m, respectively. The absolute values of the recorded vibration frequencies are shown in Appendix C.

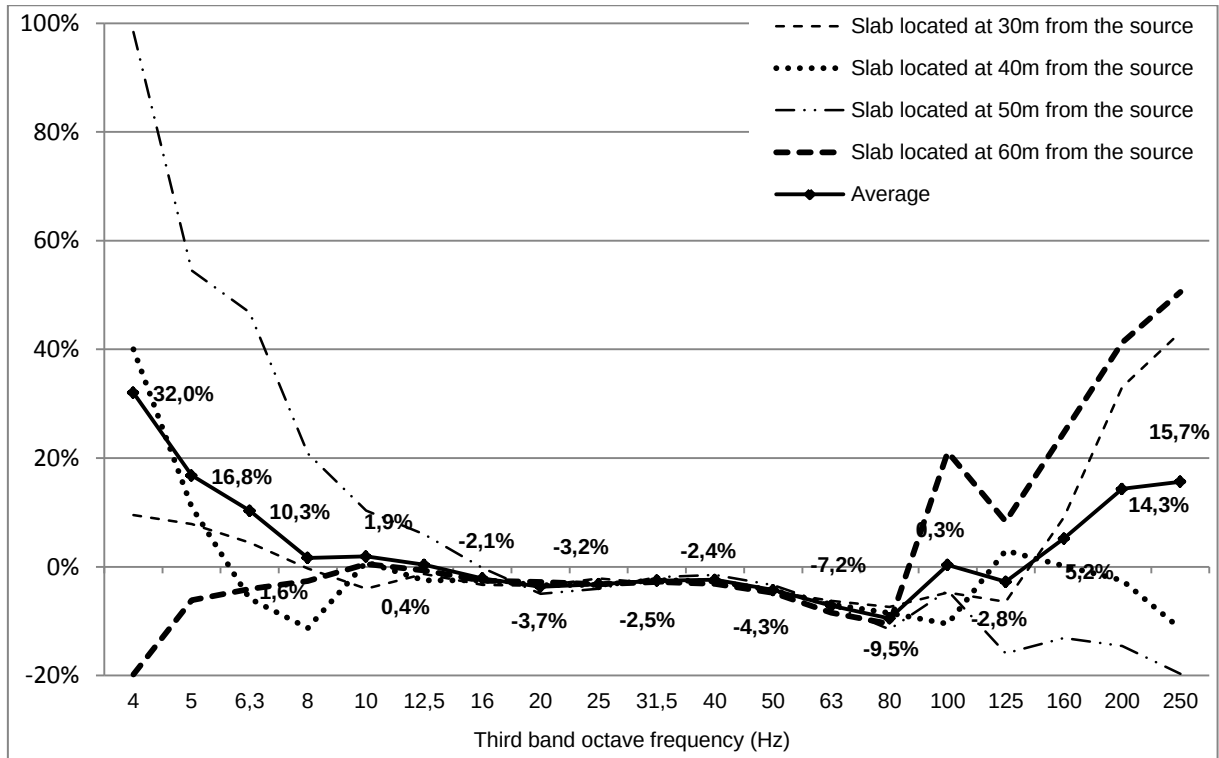


Fig. 57 – Variation of vibration frequencies between two slabs 0.5m and 1.0m thick for different distances

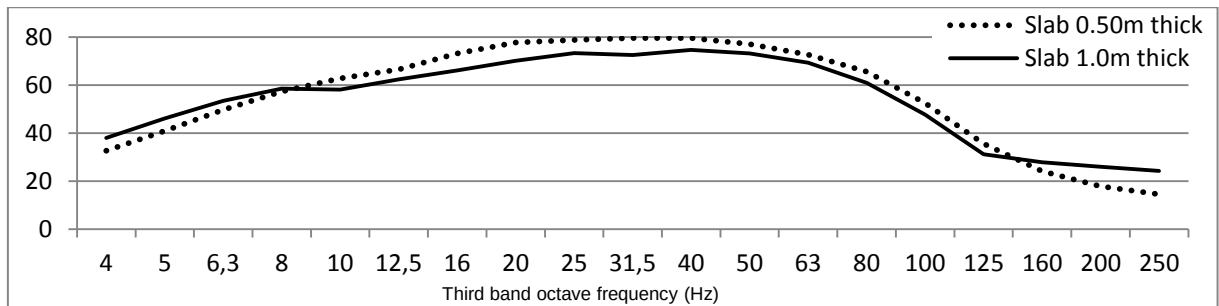


Fig. 58 – Absolute values of vibration frequencies recorded at the top of the slab, 30m away from the source

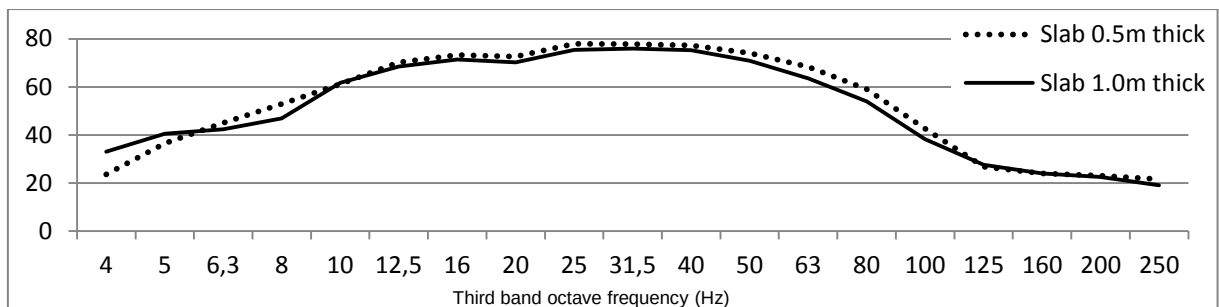


Fig. 59 – Absolute values of vibration frequencies recorded at the top of the slab, 40m away from the source

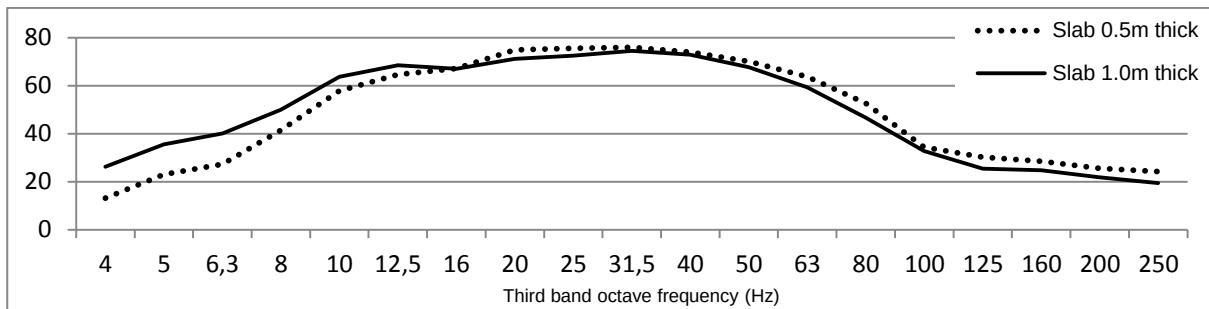


Fig. 60 – Absolute values of vibration frequencies recorded at the top of the slab, 50m away from the source

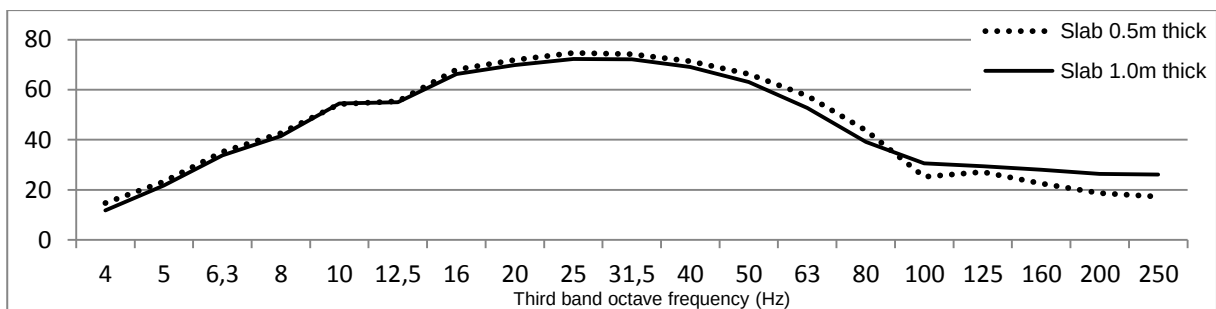


Fig. 61 - Absolute values of vibration frequencies recorded at the top of the slab, 60m away from the source

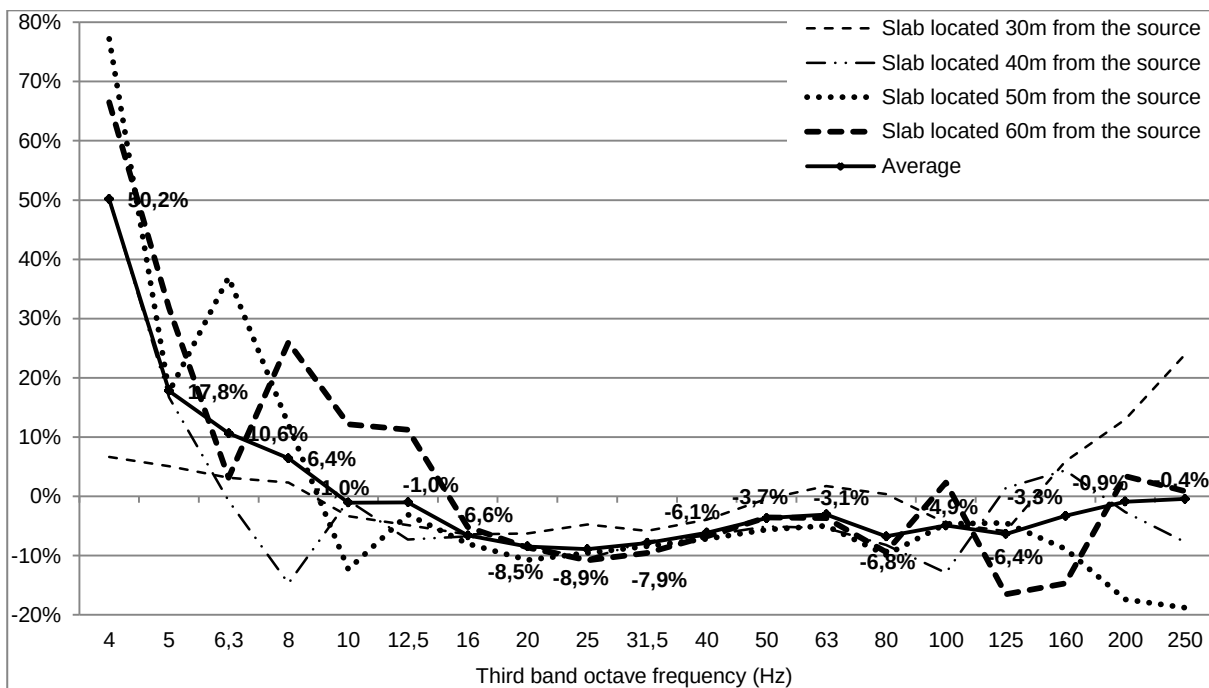


Fig. 62 – Variation of vibration frequencies between two slabs 0.5m and 2.0m thick for different distances

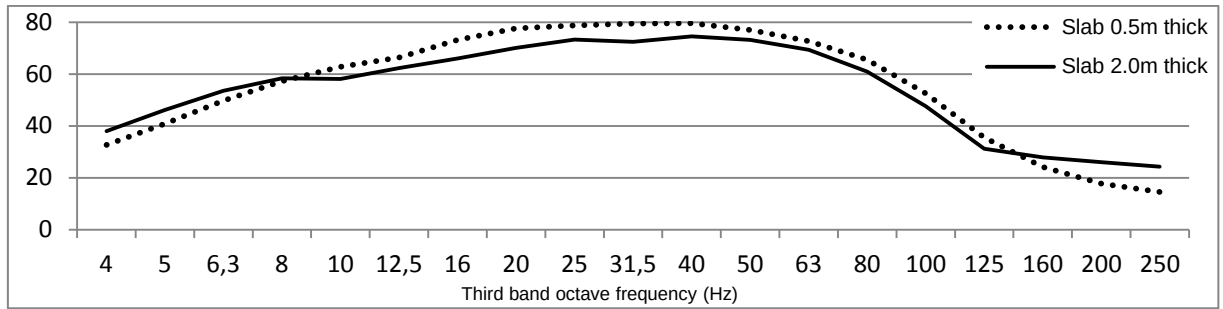


Fig. 63 – Absolute values of vibration frequencies recorded at the top of the slab, 30m away from the source

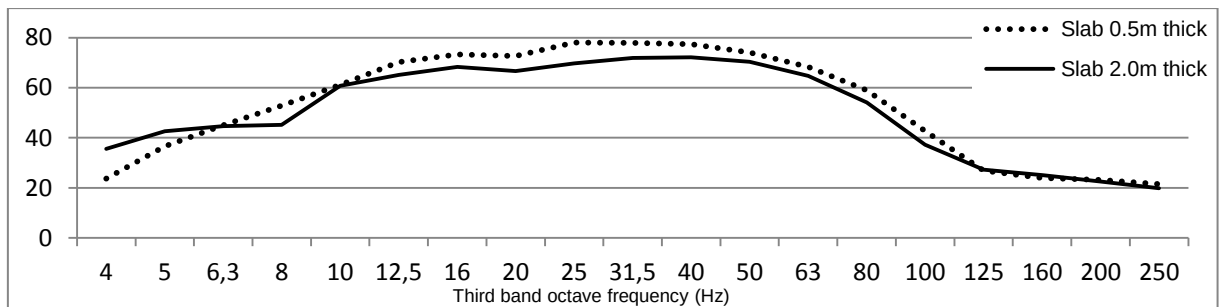


Fig. 64 - Absolute values of vibration frequencies recorded at the top of the slab, 40m away from the source

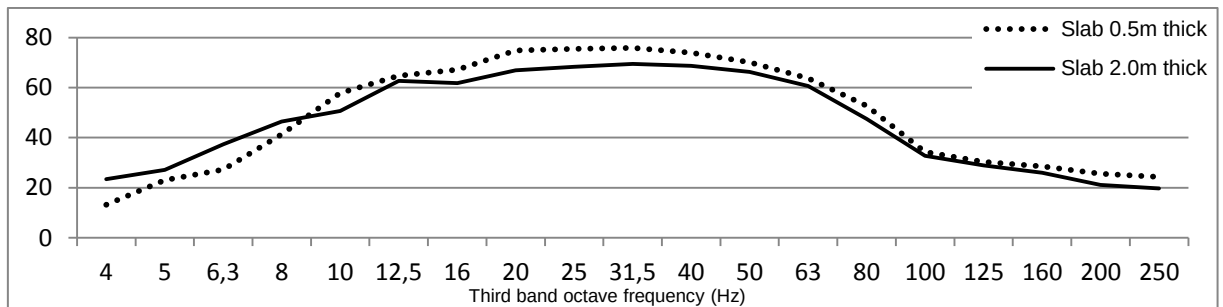


Fig. 65 - Absolute values of vibration frequencies recorded at the top of the slab, 50m away from the source

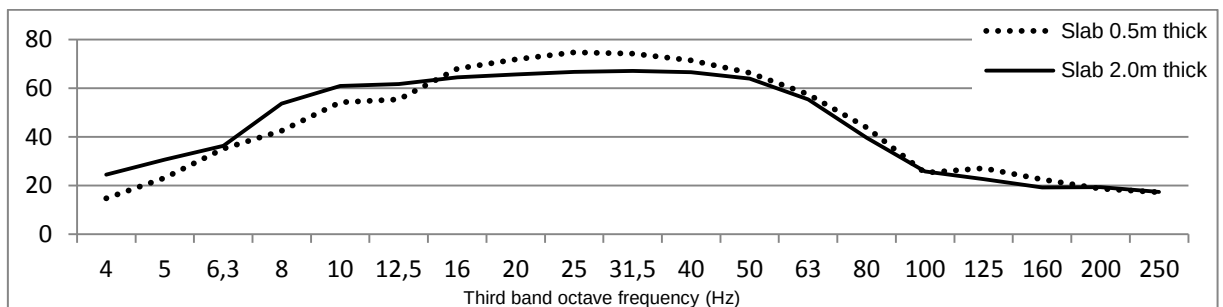


Fig. 66 - Absolute values of vibration frequencies recorded at the top of the slab, 60m away from the source

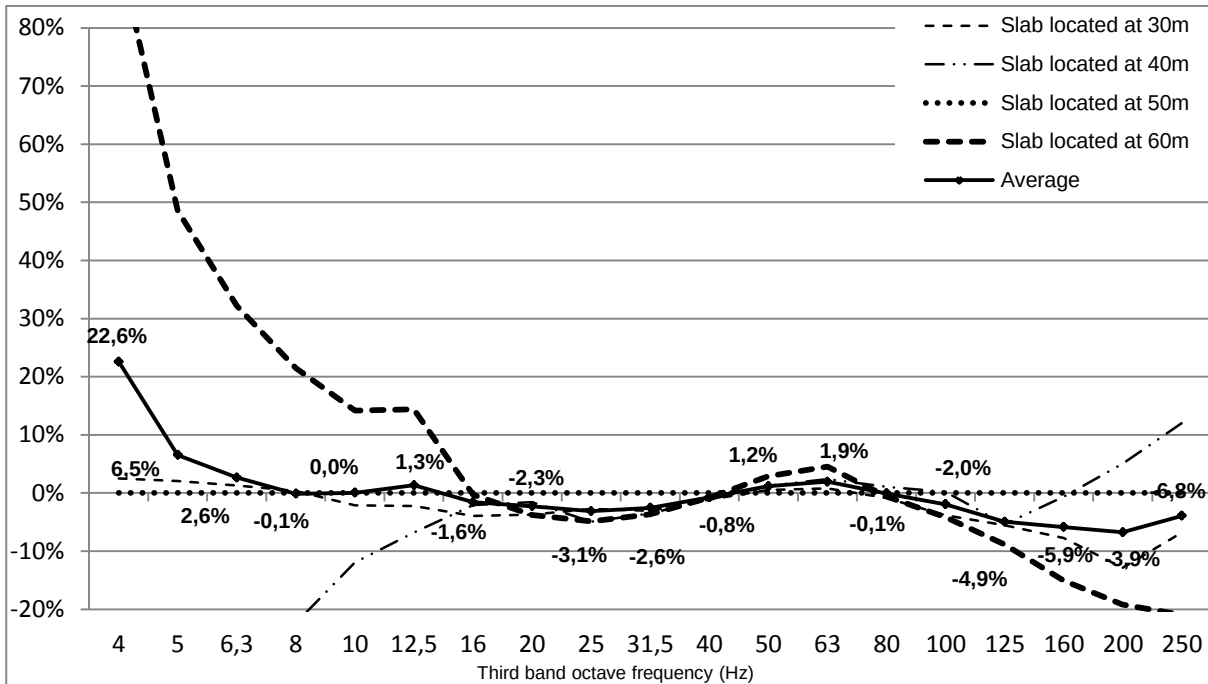


Fig. 67 - Variation of vibration frequencies between two slabs 1.0m and 1.5m thick for different distances

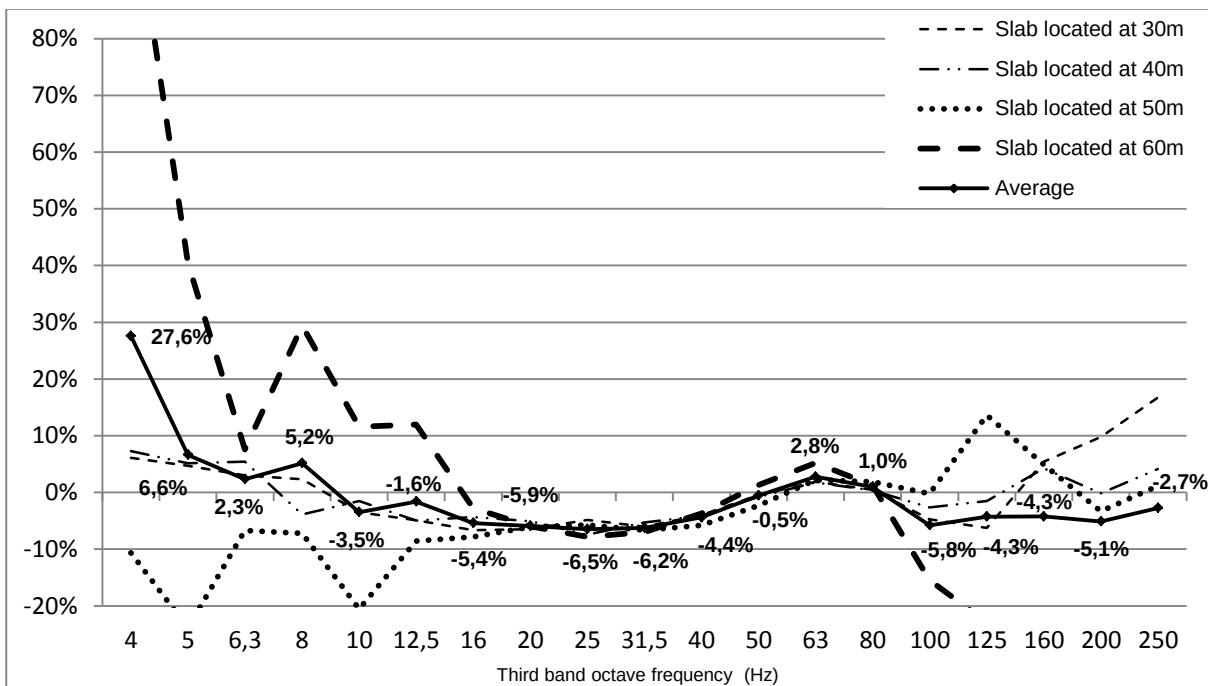


Fig. 68 - Variation of vibration frequencies between two slabs 1.0m and 2.0m thick for different distances

From the analysis of the data presented on previous figures related to the frequency it is possible to conclude that increasing the foundation slab thickness a reduction of the frequency levels within the range 16-80Hz will occur. For lower vibration frequencies, i.e. below 16Hz, in percentage values, the

frequency level for some models increases or decreases sharply, so is not possible establish a relation between these results and the foundation thickness.

When comparing two slab thicknesses 0.5m vs. 1.0m, Fig. 54, it is possible to assume that the frequency level for the range 10-100Hz presents a reduction of the vibrations around 3%. However, it is necessary to point out that the decreasing is more pronounced for the slab located 30m away from the source, Fig. 58. For the remaining distances the decrease presented is not so pronounced. There is no apparent reason for this is result, maybe is due to some reflections.

The difference between the frequencies recorded for slabs with 0.5m and 2.0m, presented in Fig. 62, has a reduction that can reach 8.9% for the frequency of 25Hz. For the range above 10Hz, the frequency level decreasing is around 6%. In the distances evaluated, the decreasing is approximately the same, in contrast with the first two pairs of slabs analyzed.

For frequencies higher than 100Hz the results should not be taken into account, since the grid size of all the models evaluated was admitted for a maximum frequency of 100Hz. Above this frequency, results obtained are not reliable. Also most of the energy carried by the vibration due to the train traffic present vibration frequencies below 100Hz, emptying the importance of vibrations with high frequencies. The importance of frequencies above 100Hz, in the issue of ground vibrations, is relevant for railway lines in a tunnel in stiff soils, where the ground-borne noise assumes higher importance.

In Fig. 67, where it is compared the frequencies received by two slabs 1.0m and 1.5m thick, shows a slight reduction of frequency levels, within the range 16-100Hz. By increasing the thickness 0.5m, the reduction occurred isn't similar as the first pair of slabs studied. For the range below 16Hz the models also present important difference between the values recorded for the different distances.

The Fig. 68 presents the frequencies recorded between two slab thicknesses, 1.0m vs. 2.0m. In this comparison it is showed that between the range 16Hz and 100Hz the frequencies absorbed by the slab decrease. The average of the frequency decrease is around 5%. Below 16Hz the frequencies recorded by the different thicknesses diverge significantly. The interest of this data has to do with the fact that, increasing the slab thickness twice, as it happens in Fig. 57, the frequency decrease is different, being around 3% for the range within 16-100Hz.

The facts enounced above and the analysis of the data wherein shows the average variation of the frequency levels for the four distances analyzed which compare different slab thicknesses, Fig. 57, Fig. 62, Fig. 67 and Fig. 68, doesn't permit establish an linear approximation between the slab thickness and frequency alterations.

However, it is perceived that the stiffness (parameter that has more influence on vibration in elastic scheme) presents an effect on frequency reduction more pronounced, within the range 10-100Hz, between two slender slabs when compared with two thick slabs, which present the same thickness variation. In the plot presented in Fig. 67, the variation of frequencies in the range between 10Hz and 100Hz is lower when compared two slabs 0.5 and 1.0m thick, Fig. 57.

Evaluated the relation between the slabs thicknesses with the ground vibration received by a slab 10m length, it of interest to see how the magnitude of the ground vibration will vary when the slab length is increased twice.

To do this evaluation, the model formulated has a slab on the model surface, 20m length, located 30m from the source. As the evaluation done before, a four thicknesses (0.5, 1.0, 1.5 and 2.0m) will be evaluated.

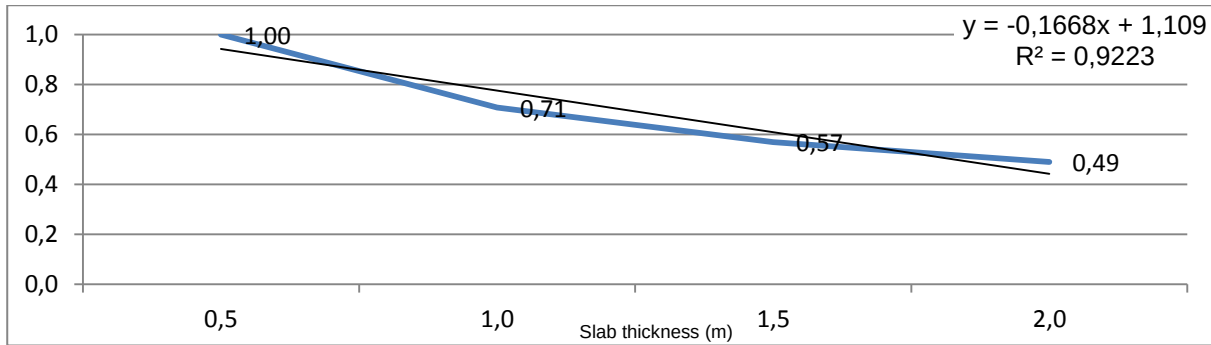


Fig. 69 – Variation of ground vibration felt in the slab, 20m length, located 30m away from the source

The results presented here show that the reduction of ground vibration transmitted to the foundation slab is more pronounced. In a 20m length slab the reduction of ground vibration is around 26% when increasing the thickness. Another interesting characteristic when the slab length is increased has to see with the absolute values recorded. The absolute values, presented in the Appendix A.3.1., are lower than the values recorded for a 10m length slab. This reduction can be explained by the stability that a higher length will offer. As the effect of the sea waves in a small boat are more pronounced than in a large boat which can reach a higher stability.

7

CONCLUSIONS

As it was expectable the train traffic nowadays presents some problems to the trackway neighborhood, especially when the transit of faster and freight trains crosses inhabited areas, as the ground-borne vibration and the noise borne vibration. Civil Engineering, together with other branches of engineering, looks for effective, practical and economic solutions to solve these problems. This report based its study in one component of the mechanism of generation, propagation and reception, the slab foundation. It can be a very interesting solution to solve the problem of ground-borne vibration increasing the slab thickness, solving a problem without application of expensive countermeasures, when it compared with the price of m^3 of concrete.

Ground-borne vibration is also a source of another problem perhaps with more complicated resolution, the re-radiated noise. Another source of discomfort due to the traffic train is the noise that propagates through air. All these factors should be taken into account when the train traffic effects are studied. For same levels of ground vibration the people have different reactions, the reason for the different acceptances has to do with the building elements that constitute the building (natural frequency), since it is an important aspect in the way the re-radiated noise is generated. Culture, age, sex and social condition of the population affected are also aspects that should be taken into account.

The software FLAC is a common software used in geotechnical analysis, statics or dynamic. To ensure that the study done follows an elastic law, it was tested different loads on the slab. The models evaluated were divided into two calculations, the static and dynamic calculation. With the static calculation was produced the stress pattern along the model, for the different thicknesses, lengths and weights. Then it was made a dynamic calculation using the stress pattern as input. The dynamic time considered was 0.40 sec for all models, the time necessary for the seismic waves generated by the application of the dynamic load to cross completely the point at 70m away from the source. FLAC presents an explicit, finite-difference algorithm that performs a “Lagrangian analysis”.

It should be pointed out that in the models studied it was decided to model only the slab, since this software is not able to study vibrations on models with materials that can present physical characteristics widely divergent, like soil and concrete. This occurs due to the time-stepping generated by the software. As explained in the section 4.2.2.3, the time-step depends on the bulk and shear modulus of the materials involved.

An interesting aspect provided by this software are the silent or quiet boundaries that permits to use models with reduced dimensions, once the radiated energy is absorbed by this boundaries, showed in Fig. 37.

A negative aspect relative to this software has to do with the fact that it is not a “friendly-user” program and its use for beginners can reveal to be a very complicated task without help of someone that feels comfortable with it.

The results obtained from the models calculated were a fundamental data to reach the results intended for this report. As expected the ground vibration presents a logarithmic decreasing along its propagation path. For a 10m length slab disposed at 30, 40, 50 and 60m away the ground vibration recorded on the slab decreases when the slab thickness is increased. When the slab thickness is increased linearly the ground vibration amplitude decreases around 21%. However, when the length of the slab foundation increases, the ground vibration recorded in the models is lower. This can be a result of higher stability offered by the slab. Also, when the slab thickness has a linear increase, the reduction of ground vibration is around 26%, however this value is not conclusive, since it was necessary to calculate the ground vibration for different slab lengths. For a question of time this topic wasn't studied so well.

To ensure that the weight of the building won't influence the values obtained, i.e. the stress pattern produced in the ground wouldn't affect the seismic wave propagation, since the scheme is elastic, was tested different models with 100kPa and 200kPa. As expected, the results of both models matched.

With these results it can be concluded that the reduction of ground vibration when the slab thickness is increased can be an interesting solution. The model results, and supported by the assessments done, summarized in Fig. 47, show that it is possible to increase the slab thickness in order to reduce the ground vibration.

The field of application of the mitigation measure "elastic bedding" is the same as the slab increase. As showed in Fig. 16, where it is shown the plot that relates the insulation factor with the vibration frequency, for higher frequencies the effect of the "elastic bedding" is more effective. For lower frequencies there's almost no effect.

In stiff soils, where the high frequencies are more relevant, an increase of slab thickness can be used to decrease the high frequencies. It is also in stiff soils where the mitigation measure "elastic bedding" is more effective. For this reason the increasing the slab thickness can be interesting as mitigation measure to reduce the ground vibration and reduce the high frequencies.

In soft soils, foundation increase is useless due to the large wavelength produced. Increasing the slab thickness will increase the parameter stiffness. This parameter is effective against vibrations with high frequencies.

The vibrations that carry most of the energy due to the train traffic have frequencies which usually are superior to the natural frequency of vibration of the building. In this thesis wasn't considered the natural frequency of the building. In a decision about the countermeasure which should be take the natural frequency of vibration can be a very important factor. Thereat this report can't answer definitely about the benefits of increasing the slab, since it wasn't made any structure modeling.

In next works would be interesting to know the influence of the slab thickness for different structure natural frequencies. With those results would be possible to have a more interesting comparison between the countermeasures applied in the receiver and the slab thickness influence.

Although, the results here obtained show that it is possible to reduce the ground-borne vibration due to the train traffic, its application should be admitted only for situations where the amplitude of vibrations makes it as an interesting solution, i.e. when the building isn't under high amplitudes of vibration. When comparing the costs of the countermeasure "elastic bedding" (around 150€/m²) and the costs of concrete (around 90€/m³), increasing the slab thickness can reveals to be an interesting solution to solve lower vibration amplitudes.

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Appendix A

FLAC MODEL DEVELOPED FOR A SLAB 1.0M THICK 60M AWAY FROM THE SOURCE

```

new
;-----DEFINE SLAB GEOMETRY & PROPRETIES-----
def slab
width=10
thickness=1
distance=60
slab_bulk_modulus=1.67e10
slab_shear_modulus=1.25e10
slab_density=2500
end
;-----DEFINE SOIL PROPERTIES-----
def soil
soil_bulk_modulus=5.12E8
soil_shear_modulus=2E8
soil_density=1814
damp_ratio=0.02
frequency_=20
end
;-----LOAD DEFINITION (TRAIN)-----
def load_
amplitude=-20000
space=1.2
distance_from_left_border=0.8
solving_time=0.4
end
;
conf dyn
g 800 500
;
gen 0,0 0,50 80,50 80,0 i 1 801 j 1 501
;
slab
soil
pp_
load_
;
def s1
a=width * 10
b=thickness*10
c=distance*10
d=c-a/2
e=c-a/2 +1
f=c+a/2
h=501-b
i=501-b-1
j1=f+1
inf_lim=distance_from_left_border/0.1+1
sup_lim=inf_lim+space/0.1
end
s1
;
m e i 1,200 j 1 120
ini x mult 4 y mult 4 i 1,201 j 1,121
m elas i=1,d j 481,501
m elas i=e,f j= h , 501 ;(foundation)
m elas i=e,f j=481,i
m elas i=j1,801 j=481,500
;
prop dens soil_density sh soil_shear_modulus bulk soil_bulk_modulus
;
group slab i e f j h 501
model elastic group slab
prop d slab_density b slab_bulk_modulus s slab_shear_modulus g slab
;
attach aside from 1,121 to 201,121 bside from 1,481 to 801,481
;-----
set dyn off

```

```

set gra 9.81
;
def kkk
n=f+1
end
kkk
apply yforce -0.1e6 i e n j 501
;
fix y j 1
fix x i 1
fix x i 801
fix x i 201 j 1 121
solve step 5000000
save static60.sav
;
rest static60.sav
set dyn on
;-----
apply nquiet squiet i 1 j 1,121
apply nquiet squiet i 1 j 481,501
apply nquiet squiet i 201 j 1,121
apply nquiet squiet i 801 j 481,501

apply nquiet squiet i 1,201 j 1
;
set dy_d ray damp_ratio frequency_
;
hist n 1
his 100 dytime
;
def pp_hist
ppp=(distance_pp+distance_from_left_border+space)*10
pp10=(distance_from_left_border+space)*10+100
pp20=(distance_from_left_border+space)*10+200
pp30=(distance_from_left_border+space)*10+300
pp40=(distance_from_left_border+space)*10+400
pp50=(distance_from_left_border+space)*10+500
pp60=(distance_from_left_border+space)*10+600
pp70=(distance_from_left_border+space)*10+700
end
pp_hist
;
his 10 yv i pp10 j 501 ; yvel at 10m from the source
his 20 yv i pp20 j 501 ; yvel at 20m from the source
his 30 yv i pp30 j 501 ; yvel at 30m from the source
his 40 yv i pp40 j 501 ; yvel at 40m from the source
his 50 yv i pp50 j 501 ; yvel at 50m from the source
his 60 yv i pp60 j 501 ; yvel at 60m from the source
his 70 yv i pp70 j 501 ; yvel at 70m from the source
;
his 11 xv i pp10 j 501 ; xvel at 10m from the source
his 21 xv i pp20 j 501 ; xvel at 20m from the source
his 31 xv i pp30 j 501 ; xvel at 30m from the source
his 41 xv i pp40 j 501 ; xvel at 40m from the source
his 51 xv i pp50 j 501 ; xvel at 50m from the source
his 61 xv i pp60 j 501 ; xvel at 60m from the source
his 71 xv i pp70 j 501 ; xvel at 70m from the source
;
his read 98 VH08.INP
apply yforce amplitude his 98 i inf_lim,sup_lim j 501
solve dytime solving_time

;
set hisfile velocity_10.y
his write 10 vs 100
set hisfile velocity_11.x
his write 11 vs 100

```

```
;
set hisfile velocity_20.y
his write 20 vs 100
set hisfile velocity_21.x
his write 21 vs 100
;
set hisfile velocity_30.y
his write 30 vs 100
set hisfile velocity_31.x
his write 31 vs 100
;
set hisfile velocity_40.y
his write 40 vs 100
set hisfile velocity_41.x
his write 41 vs 100
;
set hisfile velocity_50.y
his write 50 vs 100
set hisfile velocity_51.x
his write 51 vs 100
;
set hisfile velocity_60.y
his write 60 vs 100
set hisfile velocity_61.x
his write 61 vs 100
;
set hisfile velocity_70.y
his write 70 vs 100
set hisfile velocity_71.x
his write 71 vs 100
;
save dynamic60.sav
```

Appendix B

VIBRATION RECORDED IN THE MODELS

B.1.1. SLAB LOCATED 30M AWAY FROM THE SOURCE

Slab Thickness (m)	Distance of the recording point from the vibration source (m)	Particle velocity (mm/s)				
		X		Y		maxVector
		max	min	max	min	
0.5	10	14,035	-12,279	9,643	-27,148	27,164
	20	8,181	-7,459	6,329	-16,578	16,583
	30	1,351	-0,935	4,242	-6,845	6,856
	40	2,908	-4,119	3,210	-4,648	4,892
	50	2,128	-2,579	3,070	-4,557	4,557
	60	1,827	-2,361	2,598	-4,212	4,216
	70	1,588	-2,131	2,267	-3,734	3,741
1.0	10	14,078	-12,237	9,631	-27,166	27,184
	20	8,246	-7,528	6,235	-16,517	16,521
	30	1,304	-1,635	3,088	-5,101	5,132
	40	3,572	-3,378	4,078	-3,406	4,412
	50	1,370	-2,105	3,021	-2,649	3,113
	60	1,264	-1,822	2,562	-2,701	2,701
	70	1,131	-1,668	2,257	-2,577	2,580
1.5	10	14,003	-12,315	9,631	-27,147	27,162
	20	8,312	-7,616	6,102	-16,410	16,420
	30	1,138	-1,885	2,388	-4,651	4,668
	40	3,539	-3,453	4,751	-3,823	5,087
	50	0,935	-2,319	3,145	-2,121	3,262
	60	0,912	-1,829	2,650	-1,768	2,692
	70	0,788	-1,583	2,305	-1,640	2,348
2.0	10	-	-	-	-	27,222
	20	-	-	-	-	16,464
	30	-	-	-	-	4,123
	40	-	-	-	-	5,040
	50	-	-	-	-	3,126
	60	-	-	-	-	2,639
	70	-	-	-	-	2,249

B.1.2. SLAB LOCATED 40M AWAY FROM THE SOURCE

Slab Thickness (m)	Distance of the recording point from the vibration source (m)	Particle velocity (mm/s)				
		X		Y		maxVector
		max	min	max	min	
0.5	10	14,039	-12,273	9,634	-27,158	27,175
	20	8,157	-7,587	6,289	-16,678	16,680
	30	5,924	-5,454	4,609	-11,979	11,981
	40	0,859	-0,660	3,206	-5,688	5,702
	50	2,332	-3,557	2,616	-4,227	4,419
	60	1,812	-2,277	2,506	-4,048	4,050
	70	1,587	-2,089	2,209	-3,705	3,710
1.0	10	14,040	-12,274	9,631	-27,159	27,176
	20	8,085	-7,629	6,300	-16,645	16,646
	30	5,903	-5,486	4,640	-11,966	11,967
	40	0,798	-1,308	2,424	-4,339	4,356
	50	2,819	-2,894	3,292	-2,970	3,631
	60	1,180	-1,869	2,473	-2,566	2,581
	70	1,120	-1,686	2,199	-2,543	2,546
1.5	10	14,044	-12,273	9,628	-27,158	27,175
	20	8,047	-7,639	6,300	-16,621	16,621
	30	5,909	-5,513	4,638	-11,948	11,949
	40	0,764	-1,466	1,888	-3,813	3,830
	50	2,783	-2,805	3,831	-3,155	4,148
	60	0,832	-1,943	2,528	-1,861	2,644
	70	0,802	-1,600	2,237	-1,625	2,283
2.0	10	14,033	-12,279	9,633	-27,155	27,172
	20	7,974	-7,701	6,319	-16,598	16,598
	30	5,895	-5,553	4,629	-11,942	11,943
	40	0,693	-1,631	1,526	-3,262	3,443
	50	2,380	-2,526	4,047	-2,969	4,215
	60	0,734	-1,989	2,509	-1,713	2,629
	70	0,714	-1,537	2,232	-1,355	2,273

B.1.3. SLAB LOCATED 50M AWAY FROM THE SOURCE

Slab Thickness (m)	Distance of the recording point from the vibration source (m)	Particle velocity (mm/s)				
		X		Y		maxVector
		max	min	max	min	
0.5	10	14,047	-12,271	9,629	-27,159	27,176
	20	8,309	-7,554	6,300	-16,720	16,724
	30	6,064	-5,278	4,631	-12,022	12,029
	40	4,529	-4,126	3,572	-9,123	9,124
	50	0,684	-0,577	2,557	-4,816	4,832
	60	1,899	-3,067	2,116	-3,789	3,982
	70	1,599	-2,022	2,122	-3,569	3,570
1.0	10	14,046	-12,269	9,637	-27,152	27,169
	20	8,314	-7,550	6,309	-16,712	16,716
	30	6,083	-5,239	4,630	-12,053	12,062
	40	4,561	-4,120	3,561	-9,118	9,120
	50	0,584	-1,048	1,969	-3,725	3,738
	60	2,317	-2,487	2,690	-2,745	3,228
	70	1,110	-1,709	2,087	-2,433	2,433
1.5	10	14,043	-12,274	9,619	-27,167	27,184
	20	8,318	-7,549	6,306	-16,719	16,724
	30	6,099	-5,214	4,633	-12,063	12,073
	40	4,590	-4,115	3,566	-9,105	9,107
	50	0,576	-1,181	1,556	-3,160	3,176
	60	2,249	-2,313	3,128	-2,740	3,421
	70	0,777	-1,716	2,119	-1,655	2,248
2.0	10	14,038	-12,278	9,627	-27,160	27,177
	20	8,280	-7,573	6,278	-16,734	16,738
	30	6,081	-5,259	4,700	-11,983	11,992
	40	4,564	-4,169	3,582	-9,088	9,090
	50	0,554	-1,356	1,302	-2,617	2,782
	60	1,884	-2,044	3,280	-2,521	3,431
	70	0,639	-1,752	2,125	-1,373	2,263

B.1.4. SLAB LOCATED 60M AWAY FROM THE SOURCE

Slab Thickness (m)	Distance of the recording point from the vibration source (m)	Particle velocity (mm/s)				maxVector
		X		Y		
		max	min	max	min	
0.5	10	14,045	-12,272	9,628	-27,159	27,177
	20	8,329	-7,536	6,293	-16,729	16,734
	30	5,914	-5,417	4,713	-11,917	11,919
	40	4,490	-4,153	3,663	-9,008	9,009
	50	3,561	-3,263	2,764	-7,177	7,178
	60	0,540	-0,555	2,017	-4,110	4,123
	70	1,539	-2,642	1,786	-3,344	3,527
1.0	10	14,045	-12,272	9,628	-27,159	27,176
	20	8,329	-7,536	6,293	-16,729	16,729
	30	5,914	-5,417	4,713	-11,917	11,920
	40	4,490	-4,153	3,663	-9,008	8,992
	50	3,561	-3,263	2,764	-7,177	7,183
	60	0,540	-0,555	2,017	-4,110	3,231
	70	1,539	-2,642	1,786	-3,344	2,864
1.5	10	14,046	-12,271	9,628	-27,159	27,176
	20	8,356	-7,506	6,291	-16,738	16,743
	30	5,889	-5,442	4,751	-11,884	11,885
	40	4,499	-4,154	3,711	-8,954	8,954
	50	3,637	-3,236	2,749	-7,176	7,178
	60	0,449	-0,999	1,240	-2,697	2,716
	70	1,785	-1,946	2,629	-2,346	2,865
2.0	10	14,046	-12,269	9,636	-27,154	27,171
	20	8,311	-7,549	6,307	-16,716	16,720
	30	5,913	-5,409	4,714	-11,925	11,927
	40	4,496	-4,162	3,675	-8,978	8,978
	50	3,671	-3,243	2,732	-7,199	7,201
	60	0,502	-1,118	1,008	-2,256	2,391
	70	1,493	-1,662	2,697	-2,173	2,827

B.2.1 VIBRATION RECORDED IN THE MODELS WITH THE SLAB LOCATED 40M AWAY FROM THE SOURCE UNDER UNIFORM LOAD OF 200MPa

Slab Thickness (m)	Distance of the recording point from the vibration source (m)	Particle velocity (mm/s)				
		X		Y		maxVector
		max	min	max	min	
0.5	10	14,050	-12,263	9,628	-27,163	27,180
	20	8,162	-7,588	6,279	-16,675	16,677
	30	5,923	-5,423	4,627	-11,969	11,972
	40	0,871	-0,661	3,214	-5,669	5,683
	50	2,333	-3,553	2,608	-4,218	4,412
	60	1,812	-2,288	2,512	-4,042	4,043
	70	1,589	-2,071	2,205	-3,717	3,721
1.0	10	14,039	-12,275	9,628	-27,162	27,179
	20	8,097	-7,626	6,290	-16,639	16,640
	30	5,904	-5,473	4,637	-11,953	11,954
	40	0,814	-1,301	2,425	-4,334	4,351
	50	2,799	-2,883	3,284	-2,959	3,613
	60	1,179	-1,870	2,469	-2,557	2,579
	70	1,129	-1,678	2,191	-2,549	2,553
1.5	10	14,031	-12,279	9,636	-27,154	27,170
	20	8,012	-7,692	6,319	-16,598	16,598
	30	5,880	-5,524	4,644	-11,946	11,947
	40	0,759	-1,485	1,911	-3,796	3,816
	50	2,744	-2,796	3,836	-3,114	4,134
	60	0,810	-1,948	2,524	-1,833	2,644
	70	0,793	-1,613	2,238	-1,628	2,288
2.0	10	14,058	-12,271	9,607	-27,172	27,189
	20	8,081	-7,580	6,259	-16,639	16,640
	30	5,936	-5,484	4,558	-12,016	12,018
	40	0,798	-1,580	1,461	-3,306	3,469
	50	2,396	-2,462	3,968	-2,987	4,151
	60	0,769	-1,935	2,446	-1,740	2,553
	70	0,766	-1,495	2,156	-1,400	2,191

B.3.1. VIBRATION RECORDED IN THE MODELS WITH THE SLAB, 20M LENGTH, LOCATED 30M AWAY FROM THE SOURCE

Slab Thickness (m)	Distance of the recording point from the vibration source (m)	Particle velocity (mm/s)				
		X		Y		maxVector
		max	min	max	min	
0.5	10	13,994	-12,281	9,784	-27,153	27,169
	20	1,849	-3,180	6,209	-16,169	16,302
	30	0,932	-1,019	5,102	-5,210	5,212
	40	0,998	-1,015	4,197	-3,589	4,214
	50	1,365	-1,899	1,630	-2,561	2,564
	60	1,258	-1,401	1,815	-2,628	2,628
	70	1,091	-1,368	1,707	-2,464	2,465
1.0	10	14,059	-12,181	9,852	-27,165	27,185
	20	1,385	-3,028	4,157	-12,992	13,187
	30	0,996	-0,944	3,689	-3,314	3,690
	40	1,313	-0,843	5,066	-4,081	5,116
	50	1,848	-1,318	2,062	-1,803	2,125
	60	0,966	-0,922	1,722	-1,800	1,800
	70	0,932	-0,747	1,540	-1,348	1,542
1.5	10	14,087	-12,148	9,892	-27,138	27,160
	20	1,237	-2,864	3,204	-10,887	11,114
	30	1,142	-0,996	2,953	-2,610	2,970
	40	1,231	-1,026	4,486	-3,125	4,583
	50	1,851	-1,430	2,326	-1,667	2,333
	60	1,067	-1,073	1,882	-1,780	1,884
	70	0,929	-0,825	1,634	-1,412	1,634
2.0	10	14,038	-12,164	9,967	-27,110	27,132
	20	1,162	-2,743	2,742	-9,606	9,864
	30	1,128	-1,300	2,488	-2,180	2,554
	40	1,180	-0,923	3,372	-2,688	3,474
	50	1,851	-1,488	2,204	-1,457	2,205
	60	1,031	-1,194	1,789	-1,663	1,804
	70	0,862	-0,922	1,613	-1,336	1,615

Appendix C

FREQUENCY REGISTERED IN THE SLAB

C.1.1. FREQUENCY REGISTERED FOR SLABS LOCATED AT 30 AND 40M

Distance (m)	30m				40m			
Frequency (Hz)	Thickness (m)							
	0,5	1,0	1,5	2,0	0,5	1,0	1,5	2,0
4	32,7	35,8	36,7	38,0	23,6	33,1	23,9	35,5
5	40,9	44,1	45,0	46,1	36,6	40,6	30,6	42,7
6.3	49,7	51,9	52,6	53,5	45,0	42,4	32,7	44,6
8	57,3	57,1	57,3	58,4	52,9	46,9	36,4	45,1
10	62,8	60,2	59,0	58,1	61,2	61,7	54,4	60,8
12.5	66,4	65,5	64,1	62,3	70,2	68,5	63,8	65,1
16	73,2	70,7	67,9	66,0	73,3	71,5	70,0	68,4
20	77,6	74,9	72,1	70,1	72,7	70,2	69,1	66,7
25	78,8	77,0	74,9	73,3	78,0	75,4	71,7	69,8
31.5	79,5	77,1	74,7	72,5	77,9	76,0	73,3	71,9
40	79,6	77,7	76,8	74,6	77,3	75,3	74,4	72,2
50	77,0	73,5	73,9	73,2	74,1	70,9	71,7	70,3
63	72,6	68,1	68,6	69,3	68,4	63,6	65,1	64,7
80	65,5	60,7	60,1	60,9	59,0	53,9	54,5	54,1
100	52,5	50,1	48,1	47,7	42,7	38,2	38,3	37,2
125	35,6	33,3	31,5	31,2	26,8	27,6	26,1	27,2
160	24,2	26,4	24,3	27,8	24,0	24,0	23,9	25,0
200	17,8	23,7	20,6	26,0	23,1	22,5	23,7	22,5
250	14,5	20,8	19,4	24,2	21,5	19,1	21,4	19,9

C.1.2. FREQUENCY REGISTERED FOR SLABS LOCATED AT 50M AND 60M

Distance (m)	50m				60m			
Frequency (Hz)	Thickness (m)							
	0,5	1,0	1,5	2,0	0,5	1,0	1,5	2,0
4	13,2	26,2	17,0	23,4	14,7	11,8	22,8	24,5
5	23,1	35,6	26,1	27,1	23,2	21,8	32,4	30,6
6.3	27,3	40,1	32,1	37,4	35,2	33,8	44,6	36,3
8	41,4	50,0	41,3	46,4	42,6	41,4	50,3	53,6
10	57,8	63,7	57,3	50,6	54,2	54,5	62,2	60,8
12.5	64,7	68,6	62,4	62,7	55,4	55,0	63,0	61,6
16	67,2	67,1	63,6	61,8	68,0	66,3	66,1	64,5
20	74,9	71,2	68,9	66,8	71,8	69,8	67,2	65,6
25	75,5	72,5	70,3	68,3	74,7	72,3	68,8	66,6
31.5	75,9	74,5	71,8	69,5	74,2	72,1	69,5	67,1
40	74,0	72,9	70,7	68,7	71,4	69,1	68,6	66,6
50	70,1	67,8	67,2	66,2	66,2	63,0	64,8	63,9
63	63,7	59,3	60,2	60,6	57,5	52,7	55,1	55,4
80	52,7	46,6	47,2	47,5	43,8	39,2	39,0	39,7
100	34,3	32,8	31,3	32,7	25,2	30,6	29,3	25,8
125	30,3	25,5	27,9	29,0	27,2	29,4	26,8	22,7
160	28,5	24,8	24,1	26,0	22,5	28,0	23,8	19,2
200	25,6	21,8	20,1	21,1	18,7	26,4	21,3	19,3
250	24,3	19,5	20,3	19,7	17,3	26,1	20,6	17,5