MASTER'S THESIS IN CIVIL ENGINEERING

Specialty: Structures

Topic

Isolation of vibration induced by shaking table of CGS

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ABSTRACTS

The reaction mass is a reinforced concrete block (also known as a seismic block) that holds the vibrating table and its accessories in place. In addition to serving as a foundation, the reaction mass must be capable of limiting acceleration transmission to the ground, although some vibration propagate in the surrounding soil.

These ground vibrations damage constructions, causing discomfort to nearby structures and sensitive equipment, and are a nuisance to occupants. It is critical to assess and analyze the amplitude and extent of the vibration waves propagated in the ground before minimizing these vibrations to avoid the risk of building rupture.

The aim of this research is to develop modeling methodologies for the entire reaction mass and the surrounding soil, as well as to examine certain isolation methods of vibration caused by the vibrating table. Following a bibliographical review of the many parts of this issue, a numerical model in FE was created using the multi-physics simulation program COMSOL. To understand the impact of specific factors on the isolation system, a parametric research is initially performed. Second, a dynamic investigation was performed by delivering a dynamic stimulation to the mass transmitted by the cylinders from the vibrating table. A fatigue study for three different structures was done using both static analysis and a combination of an empirical stress-life model and a damage rule for a good understanding of the reaction of each structure element.

Finally, the trench approach was used to isolate vibrations in various combinations. The result indicated the method's efficiency when the trenches were empty and in-filled with various materials.

Keywords: Vibrations; Shaking table; Fatigue phenomena; Isolation system.

RESUME

La masse de réaction est un bloc en béton armé (également appelé bloc sismique) qui maintient en place la table vibrante et ses accessoires. En plus de servir de fondation, la masse de réaction doit être capable de limiter la transmission des accélérations au sol, bien que certaines vibrations se propagent dans le sol environnant.

Ces vibrations du sol endommagent les constructions, provoquent une détérioration pour les structures voisines et les équipements sensibles, et constituent une nuisance pour les occupants. Il est essentiel d'évaluer et d'analyser l'amplitude et l'étendue des ondes vibratoires propagées dans le sol avant de minimiser ces vibrations pour éviter le risque de rupture des bâtiments.

L'objectif de cette recherche est de développer des méthodologies de modélisation de l'ensemble de la masse de réaction et du sol environnant, ainsi que d'examiner certaines méthodes d'isolation des vibrations provoquées par la table vibrante. Après une revue bibliographique des nombreuses parties de cette problématique, un modèle numérique en EF (éléments Finis) a été créé à l'aide du programme de simulation multi-physique COMSOL. Pour comprendre l'impact de facteurs spécifiques sur le système d'isolation, une recherche paramétrique est d'abord effectuée. Ensuite, une étude dynamique a été effectuée en délivrant une stimulation dynamique à la masse transmise par les cylindres à partir de la table vibrante. Une étude de fatigue pour trois structures différentes a été réalisée en utilisant à la fois une analyse statique et une combinaison d'un modèle empirique de contrainte-vie et d'une règle de dommage pour une bonne compréhension de la réaction de chaque élément de la structure.

Enfin, l'approche de la tranchée a été utilisée pour isoler les vibrations dans diverses combinaisons. Les résultats ont montré l'efficacité de la méthode lorsque les tranchées étaient vides et remplies de divers matériaux.

Mots clés: Vibrations ; Table vibrante ; Phénomènes de fatigue; Système d'isolation.

ملخص

كتلة التفاعل في طاولة الإهتزاز عبارة عن كتلة خرسانية مسلحة (تُعرف أيضًا باسم الكتلة الزلزالية) التي تثبت طاولة الاهتزاز وملحقاتها في مكانها. بالإضافة إلى العمل كأساس ، يجب أن تكون كتلة التفاعل قادرة على الحد من انتقال التسارع إلى الأرض ، على الرغم من انتشار بعض الاهتزازات في التربة المحيطة. تتسبب هذه الاهتزازات الأرضية في إتلاف المنشئات و في ضرر للهياكل المجاورة والمعدات الحساسة ، كما أنها مصدر إزعاج للسكان. من الضروري تقييم وتحليل شدة ومدى موجات الاهتزاز المنتشرة في الأرض قبل تقليل هذه الاهتزازات لتجنب أي خطر على المباني. و لفهم تأثير عوامل محددة على نظام العزل ، تم إجراء بحث حدودي في البداية وذلك بإنشاء نموذج رقمي بطريقة العناصر المنتهية "FE" وذلك باستخدام برنامج محاكاة الفيزياء المتعددة "COMSOL". ثانيًا ، تم إجراء تحقيق ديناميكي عن طريق تقديم تحفيز ديناميكي للكتلة التي تنقلها الأسطوانات من طاولة الاهتزاز. لدراسة إجهاد لثلاث بنايات مختلفة باستخدام كل من التحليل الساكن ومجموعة من نموذج الإجهاد والحياة التجريبية وقاعدة الضرر لفهم جيد لرد فعل كل عنصر من عناصر الهيكل.

أخيرًا، تم استخدام نهج الخندق لعزل الاهتزازات في مجموعات مختلفة. وأشارت النتائج على كفاءة الطريقة عندما كانت الخنادق فارغة ومملوءة بمواد مختلفة.

كلمات مفتاحية : الاهتزازات؛ طاولة الإهتزاز؛ ظاهرة التعب (الإجهاد)؛ نظام العزل.

ACKNOWLEDGEMENT

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LIST OF SYMBOLS AND ABBREVIATIONS

CHAPTER I. BIBLIOGRAPHIC RESEARCH

CGS   National earthquake engineering research center, Algiers.

MEMS   Micro-electromechanical systems.

P-waves   Primary waves or compressional–dilatational waves.

S-waves   Secondary waves or shear waves.

L-waves   Longitudinal waves or Love waves.

R-waves   Rayleigh waves.

V_P P-waves velocity.

V_S S-waves velocity.

V_R R-waves velocity.

n Coefficient of attenuation due to radiation damping.

D   Depth of the trench.

W   Width of the trench.

L   Distance of the trench.

F_0 Maximum value of transmitted force.

ω Excitation Frequency.

p Natural Frequency in rad/sec.

m Mass.

$\ddot{y}_g$ Ground Acceleration in Y-Direction.

SDOF   Single Degree-of-Freedom.

η Isolation Efficiency.

λ_R Wavelength of Rayleigh waves.

BEM   Boundary element method.

FEM   Finite element method.

SEM   Software Engineering and Middleware.

ANN   Artificial neural network.

CHAPTER II. PRELIMINARY DIGITAL INVESTIGATIONS

PDE	Partial differential equation.
∇	Standard derivative.
∂	Partial derivative.
S	Initial position.
Fv	Force.
u	Displacement of the degree of freedom.
α_{dM}, β_{dK}	Rayleigh damping coefficient .
σ_s	Calculation constraint.
F_v	Damping Force.
ζ_s	Stretching at breach.
ζ_1, ζ_2	Damping ratio.
f_1, f_2	Natural frequencies .
ISO	International Organization for Standardization.

CHAPTER III. DYNAMIC STUDY

σ_s	Steel stress.
f_e	Elastic limit stress.
γ_s	Safety coefficient of steel.
ζ_s	Relative deformation (elongation) of the steel
f_{c28}	Concrete Compressive Strength at the age of 28-Day.
f_{t28}	The tensile strength (in MPa) of concrete at the age of 28-Day.
ρ_b	Balanced reinforcement ratio.
E_{d28}	Delayed modulus of elasticity under long duration loading ($\geq$24 hours).
$FeE50$	"Fe" stands for steel and 50 means that the steel bar will have a yield strength of 500 N/mm²
Es	Young modulus of steel.
k	Horizontal displacement on each level.
δ_{ek}	Displacement due to seismic forces.
R	Global behavior coefficient of the structure.
Δ_k	Relative displacement of the level.

CHAPTER IV. FATIGUE DAMAGE

EN	The Eurocode series of European standards.
SIA	Swiss Society of Engineers and Architects
$\gamma_{F,fat}$	Partial factor for fatigue loading.
$\gamma_{s,fat}$	Partial factor for fatigue that takes the material uncertainties into account.
$\Delta\sigma_{Rsk(N*)}$	The reference resisting fatigue stress range at N^* cycles.
$\Delta\sigma_{Rsk}$	The resisting fatigue stress range.
$\Delta\sigma_{s,max}$	Maximum steel stress range.
D_{Ed}	The fatigue damage factor steel caused by the relevant fatigue loads
$n(\Delta\sigma i)$	The applied number of cycles for a stress range $\Delta\sigma i$.
$N(\Delta\sigma i)i$	The resisting number of cycles for a stress range $\Delta\sigma i$.
Δ	Stress range
N^*	The number of loading cycles
$\Delta\sigma_{s,max}$	The maximum steel stress range.
k_1	Constant that defines the slope of the first part of the stress-life curve
k_2	Constant that defines the slope of the second part of the stress-life curve
F_y	The yield strength of the reinforcement.
$\Delta\sigma i$	The corresponding ultimate number of cycles
t_0	The age of the concrete in days.
s	Coefficient which depends on the type of cement.
γ_c	The partial safety factor for concrete.
α_{cc}	The coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied.
$\sigma_{c.min}$	The minimum compressive stress at the same fibre where $\sigma_{c.max}$ occurs.
$\sigma_{c.max}$	The maximum compressive stress at a fibre under the frequent load combination.
α	The ratio between the two principal stresses.
f_{cd}	Design value of concrete compressive strength.
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days.
ATC-40	The Applied Technology Council.
FEMA-273	Federal Emergency Management Agency.

CONTENTS

GENERAL CONCLUSION

REFERENCES

GENERAL INTRODUCTION

The propagation of vibrations in soil is a fundamental problem in seismology and soil dynamics.It is this physical phenomenon, by which the dynamic energy is transferred through the soil from the source to the place of concern.

The substantial significance for the transportation of energy is not only the type of waves generated from vibrations but also the dynamic properties of the soil, the degree of inhomogeneity, vibrations in soils which are transferred to a building or other engineering structure can cause different degrees of damage. They can be natural in origin, such as during an earthquake, or mechanical (made by humans or machines).

Furthermore, by measuring wave propagation at the surface or in the subsurface soil, the engineer may be able to detect the dynamic behavior of the ground, which is necessary e.g. for the earthquake-resistant design of a structure. Special questions arise with topographic irregularities or in homogeneities in the ground. Among other things, they may have an isolating effect on travelling waves and can therefore be used as measures to protect sensitive objects against soil vibrations.

In this thesis we can be inquisitive about taking a look at the dynamic response of the reaction mass which supports the six axis 6 m × 6 m shaking table of the CGS Laboratory owned by the National Centre of Earthquake Engineering in Algiers and the method of attenuation the propagation of vibrations to the vicinity.

Our research is structured infive chapters plus an introduction and a widespread end that is prepared as follows:

The first chapter:**"bibliographic research"** Is carried out in order to acquire in-depth knowledge on the types of vibrations, wave propagation and isolation techniques in addition to a bibliographic review of researchers who have already studied these phenomena.

The second chapter: "Numerical modeling of the reaction mass"This chapter consists in developing the necessary analysis methods of the source of vibrations which is the reaction mass of the vibrating table of the CGS and the surrounding ground in order to study the impact of these vibrations.

The third chapter: "Dynamic study" This chapter consists of modeling and studying different structures to see the impact of the vibrations induced by the shaking table using SAP2000.

The fourth chapter:"Fatigue damage"This chapter consist of the verification and resistance of structures to fatigue phenomena.

The fifth chapter:"Conception of isolation systems" Focuses on modeling each proposed insulation method; Design, configuration and material procedures are presented to successfully achieve the insulation criterion by using COMSOL Multiphysics.

Finally, a **"conclusion"** Combining all parts of this study concludes this thesis with insights and recommendations.

CHAPTER I .BIBLIOGRAPHIC RESEARCH

I. Introduction

Soil vibrations can cause severe or moderate discomfort to users and have a negative impact on structures and sensitive equipment.

Vibrations from construction projects and traffic loads are significant since they can cause damage to neighboring structures as well as neighbor complaints. Structures can be damaged by both vibration-induced differential settling and vibrations transferred directly to structures. The complexities of these vibration-related issues make determining the sources of damage challenging. The combined influence of numerous aspects, such as the characteristics of vibration sources, the site properties, the propagation of surface and body waves in the ground, and the reaction of structures, must be considered for the study of vibration-related problems.

The construction of new buildings near vibration sources such as highways, rail lines, and tramways increases the possibility of disruptive vibrations. Some sources inevitably lead to pressure waves that travel through the earth and eventually reach the foundations of civil engineering structures, causing them to vibrate violently, as in earthquakes.

The environmental zone, which is effective to reduce the ground vibration amplitude, is often adopted to prevent the vibration damages. However, estimating how much the amplitude of vibration drops at a given distance is challenging. In general, vibration attenuation with distance is made up of two components: geometric damping and material damping. The geometric damping is affected by the type and position of the vibration source, whereas the material damping is affected by ground characteristics and vibration amplitude.

The majority of ground vibrations are now detected only at the ground surface, not in depth, and without taking the propagation path into consideration. The propagation characteristics of vibrations created by different sources may be affected by the kind of generated waves, which may be examined by measuring particle movements in three directions: vertical, longitudinal, and transverse. The monitoring of tridimensional particle mobility on the ground surface and in deep is critical for the characterization of propagating waves.

It is therefore critical to develop effective methods for reducing soil vibrations, particularly when these vibrations are problematic. Vibration reduction measures can be evaluated numerically to avoid costly and time-consuming scale models. To develop such measures, it is necessary to employ numerical models capable of predicting vibration phenomena that occur in built-up environments.

In the context of this research, it is worth mentioning that only vibrations from technological sources, such as moving machines, explosions, transportation vehicles, and construction works, are taken into account. We will be interested in identifying any signal of soil response induced by a vibratory source, particularly the vibrations generated by the mass of reaction from the CGS shaking table (national earthquake engineering research center, Algiers). , The proposed approach is based on a parametric study to determine soil vibrations at various distances from the source of vibrations, which is the mass of reaction of the CGS's shaking table[1], [2].

I.1.Definition of vibration

It is critical to understand what a vibration is and how it is characterized. A vibration is the oscillating movement of particles in an elastic medium (such as soil or a structure) from different sides of an equilibrium position. They are defined by the following parameters: the amplitude of the vibration and the frequency. The displacement, velocity (usually recorded in mm/s), or acceleration of an object's oscillatory movement describe the amplitude of the vibration, whereas the frequency (typically reported in Hz) is a function of the return time of one cycle of displacement. Vibrations that are more widely separated in time will have lower frequencies, and it is typically these vibrations that cause a problem.
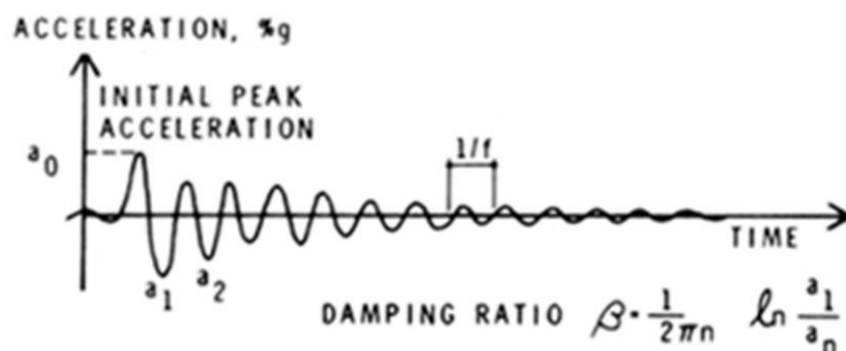
*FigureI.1Represent*the Damped Simple Harmonic Motion[3].

When a structure is subjected to significant vibrations, structural damage such as cracking of foundation walls or cracking in masonry sections, as well as excessive floor slope, can be detected. This damage shows in the same way as other structural problems that may harm a structure. To adequately determine the actual cause of the damage, it is required to collect all available data and information on the source of vibrations[3].

Vibration is defined as the movement of a mass constraint to an equilibrium point over a short distance. Because the direction of the force acting on the oscillator always points to the equilibrium point, no matter how the mass oscillates, it will always return to the equilibrium point. Only when the force direction (elastic force in a solid) is opposite to the movement can the vibration always be near the equilibrium point. As a result, vibration is a type of mass motion in which the working force and displacement are in opposite directions[4].

We might perceive motions in different directions if we watched a vibrating item in slow motion. There are two quantifiable quantities in any vibration. The object's vibrational properties are determined by how far (amplitude or intensity) and how fast (frequency) it travels. This movement is described using the words frequency, amplitude, and acceleration[5].
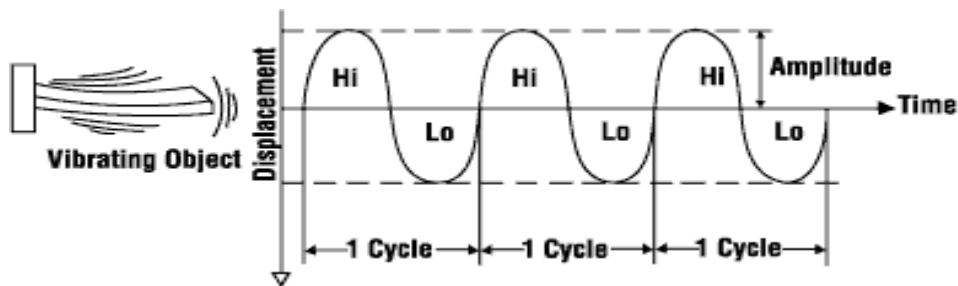
FigureI.2 *Representation of the Measures of Vibration Exposure*[5].

I.1.1. Frequency

A vibrating item oscillates from its typical stationary position. When an item goes from one extreme position to the other and back again, it completes a vibration cycle. The number of cycles completed by a vibrating item in one second is referred to as frequency. Hertz is the frequency unit (Hz)[5].

I.1.2. Amplitude

A vibrating item travels to a maximum distance on either side of its fixed location. Amplitude is measured in meters and is the distance from the stationary position to the extreme position on each side. The intensity of vibration is proportional to its amplitude[5].

I.1.3. Acceleration (measure of vibration intensity)

During each vibration cycle, the speed of a vibrating item fluctuates from zero to maximum. It travels the fastest when it transitions from its normal stationary posture to an extreme position. As it reaches the extreme, the vibrating item slows down, stops, and then proceeds in the opposite direction via the stationary position toward the other extreme. The speed of vibration is measured in meters per second (m/s).

Acceleration is the rate at which speed varies over time. Acceleration is measured in units of meters per second (m/s)/s or meters per second squared (m/s^2) and during each vibration cycle, the amplitude of acceleration varies from zero to maximum. It grows stronger as the vibrating item moves away from its typical fixed location[5].

I.1.4. Resonance

Everything vibrates at a certain frequency known as the natural frequency. Natural frequency is determined by the object's composition, size, structure, weight, and shape. A resonance state occurs when we apply a vibrating force to an item with a frequency equal to the object's inherent frequency. When a vibrating machine vibrates at the resonance frequency of an item, it transmits the most energy to it(at resonance the frequency of vibration reach the natural frequency of the element, which may causes the vibration of the studied element until rupture)[5].

I.1.5. Damped and undamped vibration

Undamped vibration occurs when no energy is lost or wasted during oscillation due to friction or other resistance. However, if any energy is lost in this manner, it is referred to as damped vibration. The quantity of damping in many physical systems is so minimal that it can be disregarded for most engineering applications. However, damping becomes extremely significant when examining a vibratory system approaches resonance[5].

I.2.Types of vibrations

The term "vibration" in physics refers to the movement of a system that remains near to its state of rest, or equilibrium.

> ➢ **Vibrations are classified into several types:**

- Forced or sustained vibrations arise from excitation. They last as long as the excitement continues.
- Free vibrations result from an action imposed at a given moment. They thus appear when the system is placed outside its rest position or when an initial impulse is transmitted to it. Due to the damping that physical systems generally undergo, free vibrations tend to decrease over time. We say that vibrations are "damped".
- Own vibrations appear in an un-damped system. They are periodic over time.
- Linear (satisfying the principle of superposition) or not;
- Deterministic (knowledge of the type of vibration makes it possible to predict the displacement at any time) or, on the contrary, random;
- Periodic (the movement is repeated identically over time) or transient (for example: a shock).
- Sinusoidal: this is the most frequently encountered type of periodic vibration. Other periodic vibrations can be crenellated, sawtooth, or even more irregular in shape (heartbeat is an example)[6].

I.3.Sources of vibrations

Ground vibration is an undulating movement spread from the detonation source, comparable to the ripple effect generated by throwing a stone into a pond. Shockwaves traveling through structures on the ground surface transfer vibration to these structures, causing them to resonate, for example if the vibration frequency of the terrain matches with the natural frequency of the structure. As a result, the amplitude of the vibration may increase to become greater than the amplitude of the initiating vibration[2].

Building vibrations can be caused by a variety of factors, the majority of which are sensed through the floor system. Vibrations can begin directly in the floor and then travel out from the source, or they can be transmitted through building parts from sources in the ground or outside the building[7].

The sources of vibrations that we can meet in our environment are numerous and diverse but they are classified in two distinct groups:

I.3.1 Internal sources include

- HVAC equipment.(heating equipment, ventilation and cooling or air-conditioning equipment)
- Lift and conveyance systems.
- Fluid pumping equipment.
- Human activity, e.g. walking, dancing, aerobic exercises, etc.
- Explosive fire etc.

I.3.2 External sources include

I.3.2.1 Vibrations caused by Seismic activity

Because the earth is an elastic mass, the high strain energy generated during an earthquake induces radial propagation of waves in all directions. These elastic waves, known as seismic waves, convey energy from one place of the earth to another via many layers, eventually carrying the energy to the surface, where it causes destruction. Elastic waves propagate through a nearly limitless, isotropic, and homogenous medium within the earth, forming what are known as body waves. These waves propagate as surface waves on the surface. Wave reflection and refraction occur near the earth's surface and at every layer within the earth[8].

As an example we have the Algerian paraseismic(Algerian Seismic Regulations RPA 99 - Version 2003 for Reinforced Concrete BuildingStructures) rules which are based on the principle that a building, faced with weak earthquakes but more frequent, must be able to control the damage of non-structural elements by a behavior essentially elastic (the cost of non-structural elements can be very considerable for some buildings). However, faced with strong and less frequent earthquakes, the building must have good ductility to be able to undergo large displacements with little or no capacity loss[9].

I.3.2.2Vibrations caused by road traffic

Mechanical vibrations such as noise, air pollution... are among the disadvantages caused by the large rise in road traffic density. The ground, on the other hand, serves two

functions: one as a support for the highway and the other as a medium for vibration transmission. In an urban setting, the stresses transmitted to structures near highways with heavy traffic can reach significant levels, the recurrence of which causes structural deterioration (rissuration of partitions for example).

(Bocquenet and le Houedecon 1979) investigated the dynamic behavior of a pavement-foundation assembly subjected to a load whose parameters are obtained from the vehicles and the road profile. The results demonstrate that the response modulus of the foundation has the greatest influence on the amplitude of the forces transmitted to the ground; the speed of the vehicle and the wavelength of the profile, on the other hand, have a considerably less impact[10].

I.3.2.3 Vibrations caused by industrial activities

Industrial equipment having impact loads, such as forging hammers, punch presses, and others, are used in plant production processes. Ignoring the vibration effects of impact machine foundations can cause issues for the outer walls of forging shops, people working in plant offices, and inhabitants in adjoining buildings.

Each construction or industrial site is unique, and vibration reduction techniques must be implemented appropriately. It is critical to establish performance criteria for vibrations and movement of nearby structures. Construction vibration control specifications should be developed for large-scale construction projects. In most situations, harmful soil movements and structural damage caused by vibrations induced by construction and industrial sources may be avoided.

(Dowdingon 1996), (Woods on 1997), and (Svinkinon 2004, 2005b) found that monitoring and controlling ground and structural vibrations provides the justification for selecting solutions for vibration prevention or reduction, as well as settlement/damage threats. The basis for assessing the contribution of various elements to building vibrations is accumulated expertise in vibration measurements. Some of these characteristics can be employed to reduce vibration impacts while also ensuring the safety and serviceability of surrounding structures[11].

I.3.2.4Vibrations caused by a Vibrating machine

A foundation is intended to deliver charges from a structure to the ground under suitable conditions. Machines installed within industrial buildings transmit vibrations to the ground via foundations. The control of these vibrations is required to ensure the stability of the system in dynamic mode, as well as to minimize vibrations in the surrounding environment.

Machine vibrations, for example, have been shown to cause the formation of elastic waves in the soil, which propagate in the vertical direction of the massif, resulting in significant inertia forces. Seism and explosions are immediate phenomena that occur over time, while the machine continues to operate. Structures may withstand strong but brief excitations; however this is no longer the case when vibrations are prolonged.

(Leonard, G.A on 1940) To avoid excessive vibrations, heavy foundations are built; machine manufacturers frequently recommend a foundation weight based on their equipment. **(Couzens, W. J, and IEE, J, on 1938)** established empirical relationships relating the weight of the foundation to the weight of the machine based on the kind and power of the machine[12].

I.3.2.5Vibrations caused by explosion

The number of explosive attacks on civilian buildings has grown in recent years, and the pattern of damage caused on structures when the explosion occurs at altitude remains difficult to forecast.

(S. Trélat and al on 2007);the nature of reflection shock wave (regular, Mach) on the ground or on a structure consecutive to an explosion is investigated. Several dimensionallaws are expressed as function of reduced radial distance and the angle of incident shock[13].

I.4.Possible effects

From their source, the elastic waves radiate into the ground, penetrate the foundations of the structures, then propagate through the structure, and reach the occupants. To assess the possible effects, it is necessary to examine all the stages of the journey traveled:

- In the ground: vibrations can create settlements, or even liquefaction, ruptures of slopes, introducesfatigue, open pre-existing cracks, etc. . . .

- In constructions, the consequences are very variable in severity: surface cracking, displacement or fall of elements of precarious stability (plaster, cornices, suspended objects, etc.)

- In localized or general resonance of the framework, cracking of the load-bearing elements. For people, vibrations can constitute a gene for comfort, or even create physiological disturbances; in almost all cases, it is the cause of more or less well controlled or justified concerns[14].

I.5.Vibration measurement devices

- Transducers based on seismic mass displacement measurement,capacitive, linear variable differential transformer (LVDT), piezo-resistive, strain-gauges etc.

- Frequency compensated electro-dynamic sensor, specially designed for measuring low intensity, low frequency acceleration.

- Servo accelerometers, working on force compensation principle

Today, all these devices are neatly integrated in the form of MEMS – a revolutionary semiconductor based technology[15].

I.6. Body and Surface Waves

The refraction or reflection of seismic waves is used in geophysics to explore the structure of the Earth's interior, and man-made vibrations are frequently created to investigate shallow, subsurface structures, there are two types of waves:

- The body waves that cross the earth.
- Surface waves that propagate parallel to the surface.

I.6.1. Body waves

Body waves propagate inside the globe. Their speed of propagation depends on the material crossed and in general it increases with depth, Body waves are divided into two categories:

I.6.1.1P-waves

The first form of body wave is known as the primary wave or pressure wave, and it is also known as P-waves. This type of seismic body wave has the maximum velocity as it goes through the soil. P-waves move in the same way as sound waves do as a longitudinal compressional waveform. They travel through the soil by pushing (compressing) and pulling (expanding) it as they spread out.

P-waves may pass through solid rock as well as liquid material, such as volcanic lava or oceans. They move at velocities ranging from 1,600 to 8,000 m/s, depending on the material they're travelling through. They are the first type of wave to be felt and recorded on a seismograph during an earthquake due to their speed,propagating at speeds[16] :

$$V_p = \sqrt{\frac{\lambda + 2G}{\rho}} = \sqrt{\frac{E(1-\nu)}{(1+\nu)(1-2\nu)\rho}} \qquad\qquad (1.1)$$

Where E is the Young's modulus, ρ is the mass density and ν is the Poisson's ratio. The constants G (shear modulus) and λ are called Lame's constants are given by:

$$G = \frac{E}{2(1+\nu)} \qquad ; \qquad \lambda = \frac{\nu E}{(1+\nu)(1-2\nu)} \qquad\qquad (1.2)$$

I.6.1.2 S-waves

The secondary wave, shear wave, or shaking wave is the second form of body wave and is usually referred to as S-waves. S-waves are a type of transverse wave that shears the ground at right angles to the direction of motion.Depending on their polarization and direction of motion, S-waves have distinct impacts on the ground surface.

- Horizontally polarized S-waves will move the ground from side to side (left to right) in the direction they are traveling.
- Vertically polarized S-waves will cause the earth to move up and down in the direction of motion. Because liquids cannot be sheared or twisted, S-waves cannot travel through bodies of water such as oceans and lakes.

S-waves are generally 40% slower than P-waves in any given material, with velocities ranging from 900–4,500 m/s. During an earthquake, these waves are the second to register on a seismograph. Despite their slower pace, S-waves are frequently more

harmful than P-waves due to their bigger amplitudes and higher degrees of ground shaking,propagating at speeds[16] :

$$V_S = \sqrt{\frac{G}{\rho}} \qquad (1.3)$$

I.6.2. Surface waves

Surface waves are typically generated when the source of the earthquake is close to the Earth's surface. As their name suggests, surface waves travel just below the surface of the ground. Although they move even more slowly than S-waves, they can be much larger in amplitude and are often the most destructive type of seismic wave. There are several types of surface wave, but the two most common varieties are Rayleigh waves and Love waves.

Surface waves (Rayleigh and Love waves) can only travel along a free surface or a boundary between two dissimilar solid media[16].

I.6.2.1 Rayleigh waves

Rayleigh waves, also known as ground roll, spread as ripples across the ground, comparable to rolling ocean waves. Rayleigh waves, like rolling ocean waves, move both vertically and horizontally on a vertical plane pointing in the direction of movement.

Eyewitnesses have claimed to have seen Rayleigh waves in big open places such as car parks, where the vehicles were described as swaying up and down like corks floating on the ocean.

Rayleigh waves are slower than body waves and generally travel at a 10% slower speed than S-waves[16].

The ground vibration is mainly affected by Rayleigh wave inother definition they arethe surface acoustic waves that travel along the surface of solids. They are frequently used in non-destructive testing to detect defects and can be produced in materials in a variety of ways, such as by localized impact or piezoelectric transduction. Rayleigh waves are a type of seismic wave produced on Earth by earthquakes. They are known as Lamb waves, Rayleigh–Lamb waves, or generalized Rayleigh waves when guided in layers[17].

I.6.2.2 Love waves

Love waves move similarly to S-waves, but without the vertical displacement. They move the ground from side to side on a horizontal plane, but at an angle to the propagation direction. Because of the horizontal ground motion they induce, love waves are extremely harmful to structural foundations. Love waves can also produce horizontal ground shearing. They travel somewhat quicker than Rayleigh waves, with a pace that is about 10% slower than S-waves, although they, like S-waves, cannot spread over water[16].
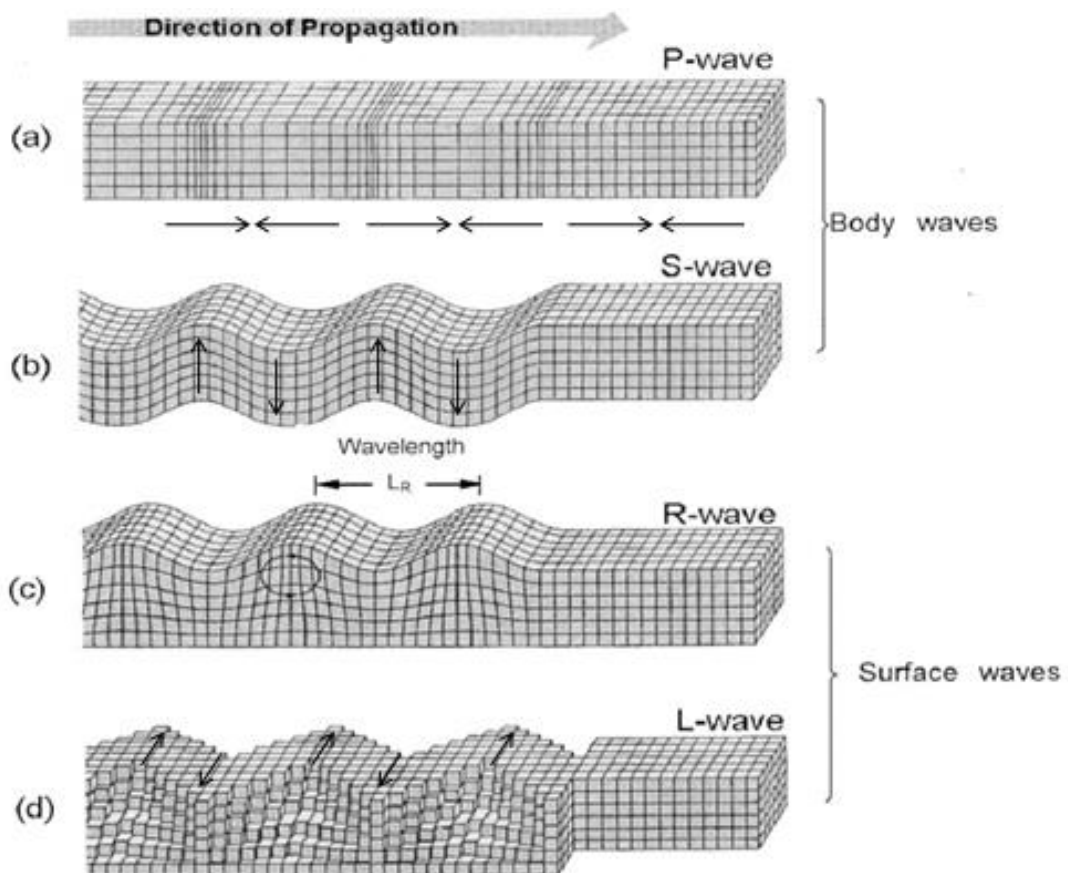
FigureI.3 *Types of seismic waves propagating in the ground :(a) compressional– dilatational waves. (b) Shear waves. (c) Rayleigh waves. (d) Love waves*[18].

I.7.Vibration propagation

A significant amount of research effort has been devoted to the subject of the propagation of man-made seismic waves and the motion attenuation caused by radiation and material damping. The majority of the studies involve field measurements of man-made vibrations and the development of empirical attenuation relationships that take into account the amount of radiated energy, distance from the source, vibration frequency, and

soil type. Some recent studies on the subject have also included the creation of numerical models that incorporate the source of vibration as well as the path of propagation[18].
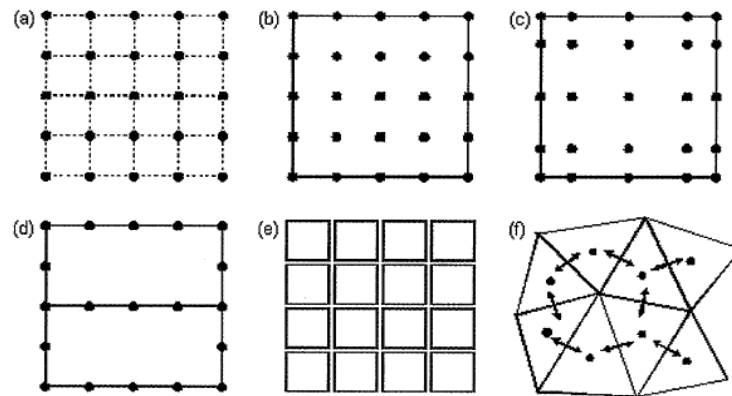
FigureI.4 *Different numerical methods for modeling the propagation of waves and vibrations: (a) finite difference method, (b) finite element method, (c) spectral element method, (d) boundary element method, (e) discrete element method, (f) finite volume method*[19].

Several numerical methods are available to simulate the phenomena of waves propagation (FigureI.4): finite-difference method (Moczoand al., on 2002); Virieux, on 1986)), finite element method (Joly, on 1982) ; Semblat, on 1998)), boundary element method (Bonnet, on 1999); Danglaand al., on 2005)), spectral element method (Faccioliand al., on 1997) ; Komatitschand al., on 1999))
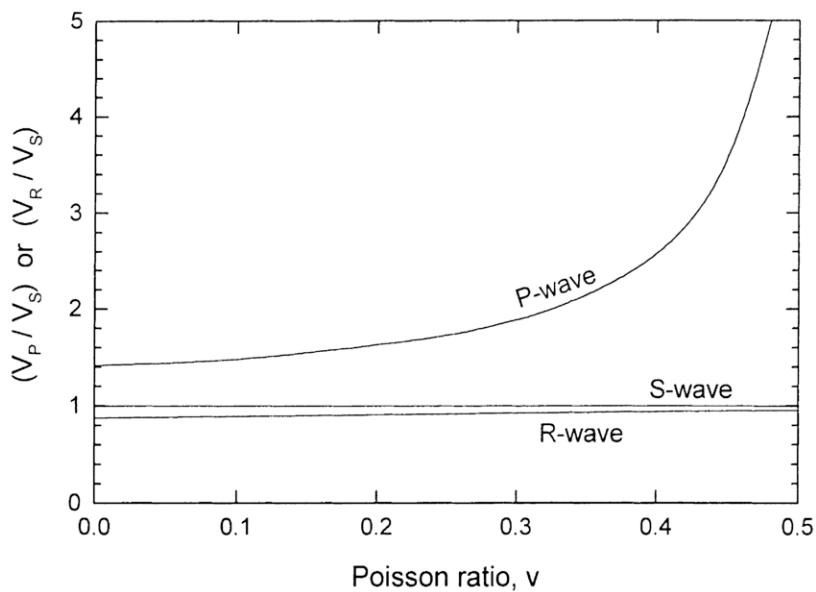
FigureI.5 *Relationships between propagation velocities of seismic waves*[18].

The surface waves can travel only in the vicinity of the ground surface and may be either out-of-plane Love waves (L-waves) or in-plane Rayleigh waves (R-waves). The L-waves are horizontally polarized shear waves and they exist only when there is a surface low-velocity layer on top of a higher velocity layer. Their velocity of propagation does not differ appreciably from that of S-waves. The propagation velocity of R-waves, V_R, is slightly lower than V_S, according to Figure 5, and their particle motion has both vertical and horizontal components. The diagram of Figure 6 depicts the rapid decay of vibration amplitude of R-waves with depth from the ground surface. From the diagram, it may be concluded that the R-waves propagate along a surficial layer having a thickness equal to one wavelength. It should be noted that in a homogeneous half-space the R-waves are non-dispersive, i.e. the value of propagation velocity, V_R, is independent of the frequency of vibration, f (or the corresponding wavelength, L_R). The R-wave wavelength, L_R, is defined in Figure 3c and is related to the propagation velocity, V_R, (also termed phase velocity) through the frequency of vibration, f, following this Eq[18].

$$V_R\, = f L_R \qquad\qquad (1.4)$$
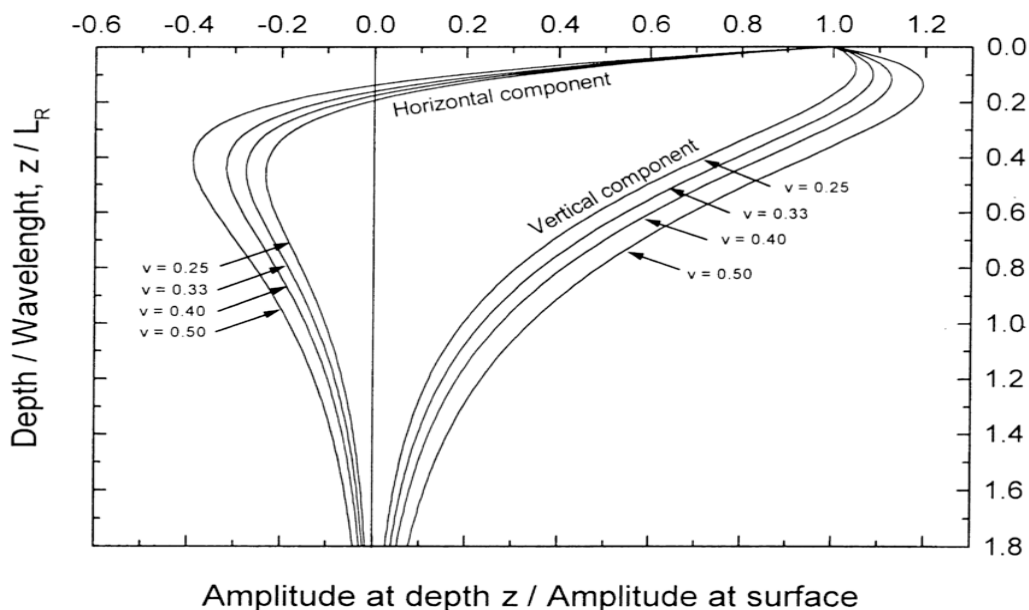
FigureI.6 *Decay of amplitude of Rayleigh waves with depth from the ground surface*[18].

R-waves become the dominant type of wave at increasing distances from the source of vibration due to their much lower rate of attenuation and the high percentage of vibration energy (approximately 70% of total energy) conveyed by Richart Jr and al.[20],Wolf[21] has recently presented convincing evidence that the partitioning of energy

carried by P-, S-, and R-waves is frequency dependent. In addition, he demonstrated that for vibration frequencies encountered in engineering practice, the majority of energy is carried by P-waves, in contrast to previous findings that were valid for very low (quasi-static) frequencies.Rayleigh wave propagation becomes more complicated in the case of a layered elastic halfspace.

Higher modes of vibration contribute to particle motion, and the R-wave velocity varies with frequency (or wavelength, L_R).In this case, the R-waves are dispersive, and a dispersion curve describes the V_R vs. L_R relationship for each mode of vibration.Furthermore, for the second and higher modes, there are cutoff frequencies below which the development of the particular mode is impossible.As the distance from the source of vibration increases, the amplitude of particle motion in all seismic waves decreases. A portion of this attenuation is caused by the distribution of a constant amount of vibration energy across the wavefront's continuously increasing area. This type of damping is known as radiation damping, and it is typically described by the equation:

$$w_2 = w_1 \left(\frac{r_1}{r_2}\right)^n \tag{1.5}$$

Where w1 is the amplitude of vibration at distance r1 from the source, w2 the amplitude of vibration at distance r2 from the source and n the attenuation due to radiation damping. The value of the attenuation coefficient, n, depends on the type of seismic wave and the position and size of the seismic[18].

Table I.1Values of attenuation coefficient due to radiation damping for various combinations of source location and type[18].

Source location	Source type	Induced wave	*n*
Surface	Point	Body wave	2.0
		Surface wave	0.5
	Infinite line	Body wave	1
		Surface wave	0
In-depth	Point	Body wave	1.0
	Infinite line		0.5

I.8. Vibration Hazards

The vibration wave spreads swiftly from near to far on the ground's surface. Environmental vibration not only causes vibration damage to engineering buildings, but it

also has a negative impact on production and the lives of inhabitants around the construction site. If adequate safety precautions are not taken, they may result in cracking of subgrade retaining walls, culverts, and bridge abutments, disruption of normal life for surrounding residents, a threat to the safe production of neighboring industrial and mining enterprises, and damage to the normal use and safety of surrounding buildings[22].

I.9. Vibration Criteria

For the construction of a foundation that takes vibrations into consideration, criteria must be established to give a way of determining if the design is adequate. The following requirements must be met[23]:

- No vibration damage is done to the structure in which the machine is housed and also to the adjacent structures.
- No damage is done to the machine itself.
- The performance of machine or the adjacentmachines is not impaired.
- Excessive maintenance cost for the machinesand structures is not generated.
- The health of workers in the vicinity is notimpaired.
- The health and comfort of the people in thesurrounding area is not adversely affected.

I.10. Vibration isolation

Vibration isolation is concerned with reducing the vibratory effect.In its most basic form, a vibration isolator may be thought of as a robust part linking the equipment and the foundation. An isolator's role is to minimize the amplitude of motion communicated to equipment from a vibrating foundation.[24]

Geotechnical engineers face a problem in mitigating the ground vibration caused by railway passing in order to protect the adjacent buildings. Open trenches are one of the most prevalent approaches for limiting the transmission of such ground vibrations among the different extant technologies. Although this technology has various advantages in isolating ground vibrations, the fundamental worry with such tunnels is their instability under applied dynamic stress. **(Qingsheng Chen and al., on 2021)**propose a new approach for isolating ground vibrations utilizing horizontally buried hollow pipes to address this issue. Under plane strain circumstances, the performance of such pipes in minimizing

ground vibrations caused by harmonic stress excitation has been examined using finite-element modeling. When compared to open trench barriers, these pipe assemblages have been proven to be highly successful in lowering ground vibration transmission while removing instability issues.In-depth research was conducted to determine the effect of numerous essential parameters related to pipe installation, such as barrier angle, depth, diameter, and diverse subsoil conditions, on vibration mitigation efficiency. The new technique's efficiency was maximized by adjusting the inclination angle, depth, and diameter of the pipe installation. Despite the fact that the type of the surface soil has a considerable impact on the performance of this approach, ground vibration mitigation works best in soft soils. The numerical results were analyzed and interpreted to arrive at specific conclusions[23].

I.10.1 Isolation using trenches (open and in-filled)

Waves produced by machine foundations, traffic, or blasting may cause damage to nearby structures and sensitive gear. The majority of the vibratory energy influencing buildings is conveyed by a surface wave (Rayleigh-wave) that propagates near the ground surface. It is feasible to greatly minimize ground vibrations by installing an appropriate wave barrier before the construction. The vibration waves would be absorbed or reflected by the barrier. In theory, the most efficient isolation approach is an open trench .However, for practical reasons, it is sometimes difficult or impossible to build and maintain an open trench to a sufficient depth. Effective wave barriers include infilled (concrete or bentonite) trenches, vibration isolation screens, concrete core walls, sheet pile walls, and rows of piles

The notion of wave barriers is based on the reflection, scattering, and diffraction of wave energy. Wave barriers can be made out of a solid, fluid, or empty zone in the earth. Both P and S waves are communicated at the solid-to-solid contact, only P waves are transmitted at the solid-to-fluid interface, and no waves are transmitted at the solid-to-void interface.The use of barriers adjacent to or surrounding the source of vibrations to minimize the quantity of wave energy radiated away from the source is known as active isolation. The use of barriers at a place away from the source of vibrations but near a spot where the amplitude of vibrations must be minimized is known as passive isolation. Rayleigh wave isolation is represented by passive isolation.The functional difference between the two types of trenches is principally due to an in-filled trench's capacity to

effect passage (transmission) of incident waves into the trench material and subsequently to the zone of screening, which open trenches cannot do.

As a result, the quantity of wave reflection and mode conversion varies greatly.As a function of the minimal transfer of wave energy, open trenches are more effective wave barriers. However, for practical reasons, it is sometimes difficult and impossible to create and maintain open trenches to adequate depths, necessitating the use of in-filled (concrete or bentonite) trenches, concrete core walls, or piles as wave barriers.

The vibration isolation screen consists of gas-filled panels (gas cushions), whichform stable vertical underground screen. This screenhas a vibration isolation effect which is very similarto that of an open trench[23].

(S. Ahmad and T. M. Al-Hussainion 1991),provides straightforward design expressions for estimating the vibration screening effectiveness of rectangular wave barriers in homogeneous soil deposits. The design formulas are produced after a thorough numerical examination into the effect of different geometrical and material properties on the vibration screening effectiveness of the barriers. The impact of numerous characteristics such as frequency, aerial range of the reduction zone, trench dimensions, trench location, Poisson's ratio of soil, and in-fill material properties such as wave velocity, density, and damping has been studied. The most important observations are given:

1. For practical purposes, the influence of the Poisson's ratio of the soil canbe ignored.

2. The effect of the frequency is taken into account by normalizing the dimensions of the trench with respect to the Rayleigh wavelength.

3. For open trenches, the normalized depth D is the governing factor; thenormalized width W is not important, except for shallow depths (D < 0.8 m).

4. For in-filled trenches, both depth and width are equally important etc.…

The issue of passive isolation under plane-strain conditions is discussed utilizing rectangular open and in-filled trenches. As illustrated in Figure I.7, a trench of depth **d** and width **w** is located at a distance **l** from a rigid surface footing exposed to a vertical time-harmonic stress. Because vibration isolation by a trench is usually realized by the filtering of surface (Rayleigh) waves.
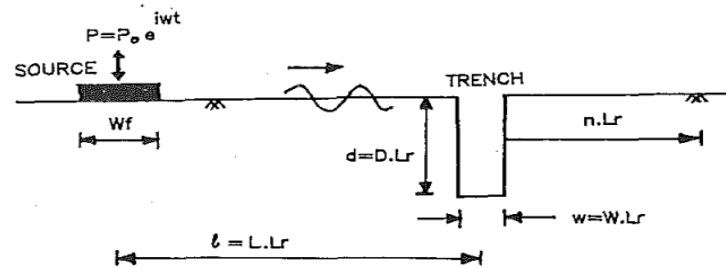
FigureI.7 *Schematic Diagram of Passive Vibration Isolation byRectangular Trenches in Viscoelastic Half-Space*[25].

The depth, width, and distance of the trench are normalized with respect tothe Rayleigh wavelength ($D = d/L_r$, $W = w/L_r$, $L = l/L_r$;where L_r= Rayleigh wavelength). The presence of a trench causes reduction in the vibration amplitude in an area behind the trench[25].

I.10.2 Isolator materials

Vibration isolation may be achieved by using materials that have a mix of extremely elastic behavior and dampening characteristics. In commercial vibration isolation applications, pneumatic, hydraulic, elastic metal, and elastomeric systems are often used. Elastomeric materials are likely the most prevalent and widely utilized in the industry, with a popular design consisting of elastomeric material attached to metal plates or a metal core. These isolators are commonly referred to as elastomeric mounts. Some of the most prevalent elastomers utilized in commercial vibration isolators include natural rubber, neoprene, and butyl rubber[26].

(Zhou and al. on 2006)concluded that rubber is considered to be favorite material for isolators or damper because of its flexibility and endurance to extremely large deformations.

I.10.3 Metal springs

(Rivin, on 2003) Metal springs have widely been utilized for vibration isolation because they may be designed to give a variation of stiffness characteristics in heavy equipment applications. Because most metal springs give relatively low material damping, most of these designs do not allow for major damping flexibility. Metal springs widely

utilized in vibration isolation applications include coil springs, disc springs, slotted springs, etc.…

I.10.4 hydraulic mount

Another type of vibration isolator utilized in automotive applications is a hydraulic mount, typically known as a hydro mount. **(Truong andAhn, on 2010)**An isolator of this type has characteristics that are both amplitude and frequency dependent. The isolator is usually composed of 2 chambers linked by a channel that permits fluid to flow from one chamber to the other. This design enables the vibration isolator to display low stiffness and high damping for dynamic excitations of big amplitude and low frequency, while exhibiting poor damping for vibrations at small amplitude and high frequency in some automotive applications, various hydro mount designs have been performed to establish dynamic features that may be modified to produce frequency-dependent behavior.[26]

I.11. Principle of Isolation

Installing a wave barrier (to mitigate vibration energy) nearthe vibration source is known as active isolation, whereas passiveisolation is located either at a location distant from the source in thesurrounding area or in the immediate vicinity of the structure to beprotected.

(Bhatia KG's on 2008) research covers the fundamentals required for understanding and evaluating the dynamic response of a machine-learning system. The researcher has also conducted extensive testing on machine foundation models as well as models. The author's goal in this work was to provide a more comprehensive evaluation of the site's pedological data, as well as a better understanding of machine data and its use in foundation design (modeling, analysis, structural design process, and construction technology).

Let us consider a damped spring mass system, having mass m, stiffness k and damping c,subjected to dynamic excitation. Consider following two cases:

a) Dynamic Excitation Force F_E (t) is applied at the mass and the Transmitted Force at the base (foundation) is F_T (t) as shown in Figure 8- (a)

b) Dynamic Excitation Force F_E(t) is applied at the base (foundation) and the TransmittedForce at the mass is F_T (t) as shown in Figure 8- (b)

In either case, the interest is thatthe transmitted force from the mass to the foundation (as in case (a)) or from the foundation to the mass (as in case (b)) should be least[27].

(a) Excitation on Mass   (b) Excitation on Base
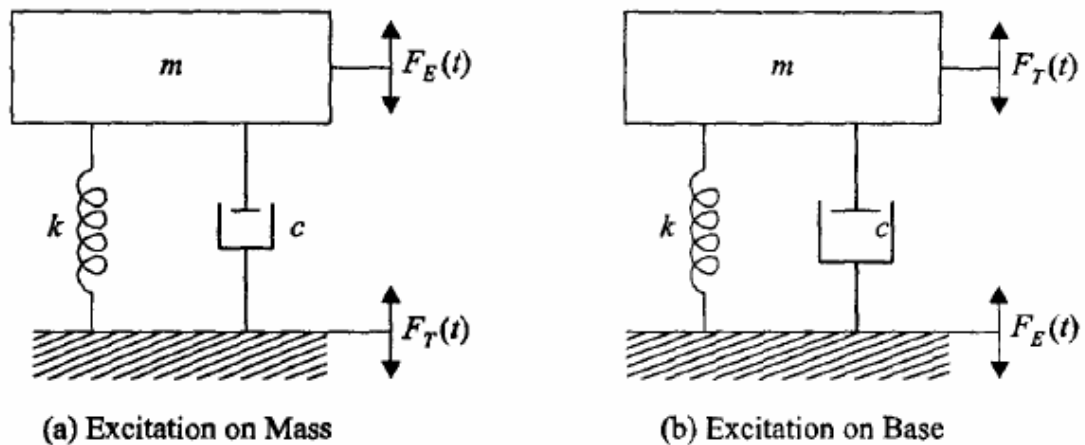
FigureI.8 *SDOF Spring Mass System, (a) Excitation on mass, (b) Excitation at base*[27].

I.11.1 Transmissibility Ratio

Let us denote the Transmissibility Ratio as TR that is defined as the ratio of transmitted force toexcitation force.

$$TR = \frac{F_T(t)}{F_E(t)} \tag{1.6}$$

Consider the two systems as shown in case (a) and case (b) in Figure I.8. For vibration isolationthe interest is that the transmitted force should be minimum in either case i.e. TR should beminimum and it is also true that TR depends upon the dynamic response of SDOF system (Single-Degree-of-Freedom Linear Oscillator) .Let us consider dynamic response of each case:

Case (a) Dynamic Excitation Force F_E (t) is applied at the mass and the TransmittedForce at the base (foundation) is F_T(t). The dynamic force could either be externally applied orinternally generated by the machine itself.

1. Let us first consider that the dynamic force is **externally applied**

Let this excitation force be$F_E(t) = F_0 \sin \omega t$ $\tag{1.7}$

Maximum transmitted force F_T (t) to the support is given as :

$$F_T(t) = F_0 \frac{\sqrt{1 + (2\beta\zeta)^2}}{\sqrt{(1 - \beta^2)^2 + (2\beta\zeta)^2}}$$

(1.8)

Where ζ is the damping constant $\& \beta = \omega/p$ is the frequency ratio

Thus, we get Transmissibility Ratio TR as

$$TR = \frac{F_T}{F_E} = \frac{F_T}{F_0} = \frac{\sqrt{1 + (2\beta\zeta)^2}}{\sqrt{(1 - \beta^2)^2 + (2\beta\zeta)^2}}$$

(1.9)

1.1 Let us now consider that the dynamic force is **internally generated**

Maximum value of transmitted force is given by $F_E = F_0 = m_r e\omega^2$
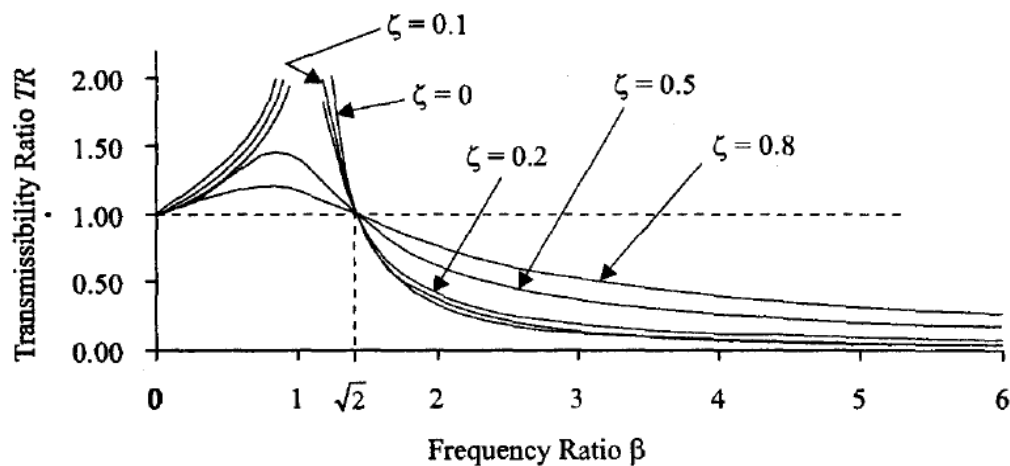
Figure I.9 *Transmissibility Ratio TR vs. Frequency Ratio β*[27].

Case (b) Dynamic Excitation Force F_E (t) is applied at the base (foundation). For this case, maximum value of transmitted force is given by $F_E = F_0 = -m\ddot{y}_g$

$$F_T = -m\ddot{y}_g \frac{\sqrt{1 + (2\beta\zeta)^2}}{\sqrt{(1 - \beta^2)^2 + (2\beta\zeta)^2}}$$

(1.10)

Thus it is clear that irrespective of whether the dynamic force is applied on the mass or applied at the base, the transmitted force remains the same for same system characteristics of SDOF system. Plot of equation (1.9) giving Transmissibility Ratio TR vs. Frequency ratio is shown in Figure I.9.[27]

I.11.2 Isolation Efficiency

Let us denote Isolation Efficiency as,isolation efficiency is thus defined as:

$$\eta = (1 - TR) \qquad\qquad (1.10)$$

(Generally it is convenient to represent isolation efficiency (η x 100) in percentage).

It is clear from this equation that lesser the **Transmissibility RatioTR** better is the **Isolation Efficiency.** Having determined Transmissibility Ratio TR (equation 1.9), we work out the**Isolation Efficiency**.

$$\eta = (1 - TR) = \left(1 - \frac{\sqrt{1 + (2\beta\zeta)^2}}{\sqrt{(1 - \beta^2)^2 + (2\beta\zeta)^2}}\right) \qquad\qquad (1.11)$$

Table I.2 *Isolation Efficiencyη vs. Frequency Ratio β for different*

Damping Ratiosζ [27]*.*

Frequency Ratio	Damping			Frequency Ratio	Damping		
β	$\zeta = 0.0$	$\zeta = 0.1$	$\zeta = 0.2$	β	$\zeta = 0.0$	$\zeta = 0.1$	$\zeta = 0.2$
2	0.67	0.64	0.59	4.2	0.94	0.92	0.88
2.2	0.74	0.72	0.66	4.4	0.95	0.93	0.89
2.4	0.79	0.77	0.71	4.6	0.95	0.93	0.90
2.6	0.83	0.81	0.75	4.8	0.95	0.94	0.91
2.8	0.85	0.83	0.78	5	0.96	0.94	0.91
3	0.88	0.85	0.81	5.2	0.96	0.94	0.91
3.2	0.89	0.87	0.83	5.4	0.96	0.95	0.92
3.4	0.91	0.89	0.84	5.6	0.97	0.95	0.92
3.6	0.92	0.90	0.85	5.8	0.97	0.95	0.92
3.8	0.93	0.91	0.87	6	0.97	0.96	0.93
4	0.93	0.91	0.87				

Isolation efficiency values for frequency ratio $\beta \geq 2$ are tabulated and given in **Table I.2**fordifferent values of isolator damping.

From the **FigureI.9**and **FigureI.10**, following observations are made:

1. Transmissibility Ratio TR is less than unity i.e. TR < 1 only for frequency ratiogreater than$\sqrt{2}$ i.e. $\boldsymbol{\beta} > \boldsymbol{\sqrt{2}}$.

2. for frequency ratio greater than.$\sqrt{2}$. TR decreases with decrease in damping value.In other words, TR is lower for zero damping compared to 10% damping i.e.**damping is not desirable for Isolation**(according to Bhatia)[27]**.**
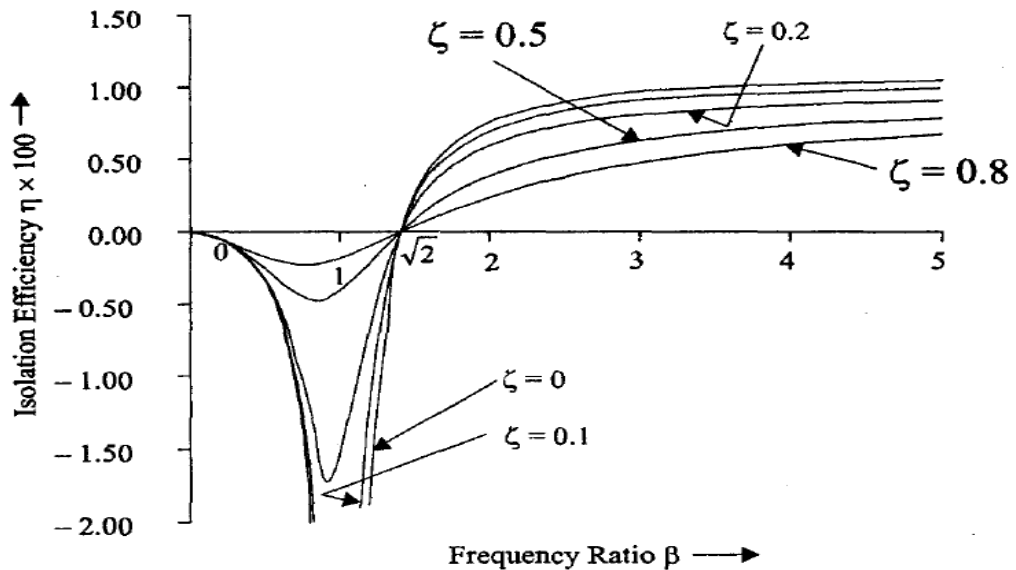
FigureI.10 *Isolation Efficiencyη vs. Frequency Ratio β for different*

Damping Ratios ζ [27].
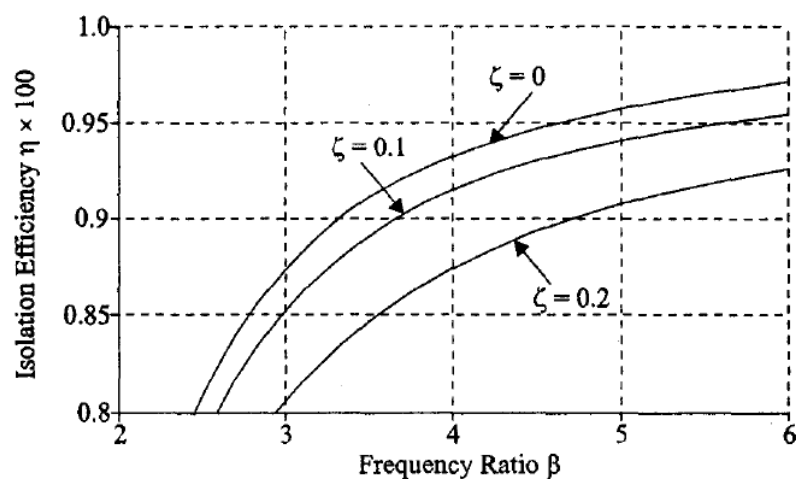
FigureI.11 *IsolationEfficiency η> 80% vs. Frequency Ratio β> 2*[27].

I.11.3 Isolation Requirements

Generally speaking, for machine foundation applications, one would be interested in isolationabove 85 % otherwise the very purpose of isolation gets defeated. In view of this, let us view the isolation plot for $\eta > 80\%$, which obviously means $\beta > 2$as shown in **FigureI.10**. It is noticedfrom the plot that even for zero damping it requires $\beta = 3$ for $\eta = 88\%$ and $\beta = 5$ for $\eta = 96\%$. Itgives an impression that one can achieve as high isolation as desired by just increasing frequencyratio. In reality, this impression, however, does not hold any ground. It is evident from **FigureI.9**that there is hardly any appreciable gain in η for $\beta > 6$ which corresponds to $\eta = 97\%$.

This implies that one can at best target for isolation efficiency of about $\eta = 97$ %.Knowing thatpresence of damping in isolators, if any, shall reflect in reduction ofη[27].

I.12.Shaking table

The shaking table consists of a moving platform on which the tested structure is fixed and designed to simulate an earthquake movement along one or more axis and properly simulate the dynamic forces involved.

In this thesis we are studying the vibrations induced by the shaking table of CGS Algiers, it is made out of a 6.1m × 6.1m steel platform that weighs 40 tons and has six degrees of freedom controlled by twelve actuators.The device can simulate earthquakes and other ground vibrations with displacements of 150 mm and 250 mm in the horizontal and vertical axis, respectively. With maximum test specimens of 60 tons, accelerations of 1.0 g in horizontal directions and 0.8 g in vertical directions .

This table is made up of three parts: a mechanical system, a hydraulic system, and an electronic system (control and acquisition). A mechanical system is made up of a platform and a reaction mass (seismic block). This latter is a reinforced concrete mass designed for the installation of the platform and these cylinders. It is distinguished primarily by its extremely large size, which ranges between 30 and 50 times the total mass (platform and specification). Its function is to absorb the table vibrations.

The reaction mass of the CGS's shaking table is a massive concrete reinforced block with an 18m x 18m x 4m foundation above a 16m x 16m x 6m block containing equipment reservations (Figure II.1)[28].
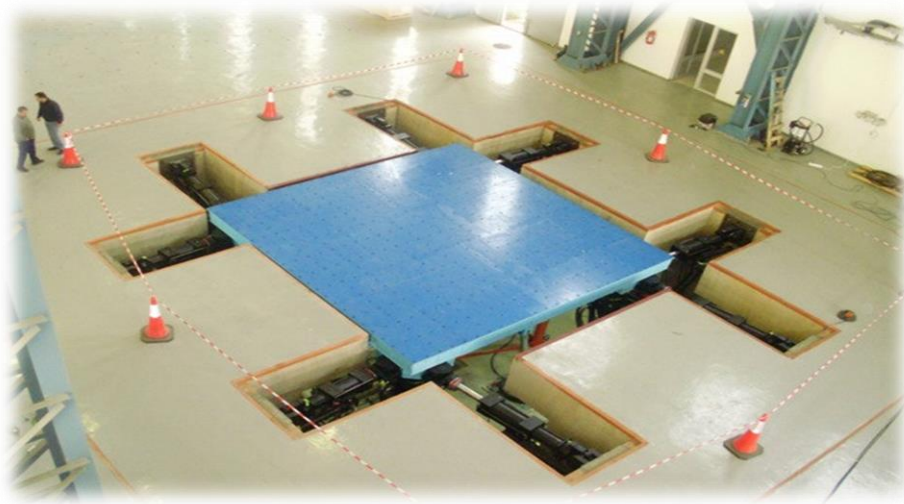
FigureI.12The shaking table and reaction mass of CGS[28].

I.13.Previous studies of vibrations isolation

(Barkanon 1962, Dolling on 1965, Neumeueron 1963 and McNeill and al. on 1965) were the first to report a number of practical case histories of vibration isolation.

(Barkanon 1962) reported on an attempt to isolate a building from traffic induced vibrationusing open and sheet-wall barriers , This installation was unsuccessful and vibration from the sheet-wall continued to affect the building adversely , However, he was first to recognize that theeffectiveness of the barrier does not depend so much on its physical dimensions, butrather on the normalized dimensions with respect to the wavelength of incident wave .He concluded that a sheet-pile barrier withsufficiently large dimensions compared to the wavelength of the surface waves is requiredto achieve a suitable reduction in the vibration amplitude as a result of the presence ofthat barrier[29].

(Neumeueron 1963andDolling on 1965) reported on the isolation of a printing plant in Berlin from vibration induced by subway trains using a bentonite-slurry-filled trench,was considered successful because only onehalf of the vibration amplitude before trench installation was observed at the printing plant after the trench installation[30][31].

(Dolling on 1965) performed a theoretical analysis of energy partitioning for Rayleigh waves across a trench. He proposed an isolation factor as a function of the normalized trench depth, given by trench depth divided by the Rayleigh wavelength. He concluded that soil Poisson's ratio does not appear to have a major influence on the isolation effect[30].

(McNeill and al. on 1965) another successful application in which trench and sheet-wall barriers were used to isolate a sensitive dimensional-standards laboratory this isolation system effectively limited the acceleration of the slab to the owner's specifications[32].

(Woods on 1968) conducted a series of scaled-field experiments to evaluate the screening effect of open trenches as wave barriers for both active and passive isolation cases. Based on the findings from these scaled-field experiments, **(Woods on 1968)**presented some guidelines for dimensioning open trenches to achieve a remarkable ground amplitude reduction. He suggested that the minimum trench depth should be $0.6\lambda_R$ for active isolation and $1.33\lambda_R$ for passive isolation to achieve an average reduction of 75% in vertical ground vibrations, where λ_R is the wavelength of Rayleigh waves. **(Woods and al. on 1974)** conducted some model tests utilizing the principle of holographic interferometry in order to study the effectiveness of rows of void cylindrical obstacles as passive isolation barriers. The applications were limited due to the wave reflections from the boundaries of the model[33].

(Haupton 1981) carried out a series of scaled-model tests in uniform, artificially densified sand on the vibration isolation of various types of barriers in a laboratory setup. The barriers investigated include solid barriers (concrete walls), light weight barriers such as rows of bore holes, and open trenches. The results showed that the screening performance of these barriers was a function of characteristic parameters in terms of wavelength-normalized dimensions. Therefore, all the results were presented as a function of characteristic parameters in terms of wavelength-normalized. He concluded that: for stiffer barriers, the ground amplitude reduction, in general, is related to the cross-sectional area normalized with respect to the square of Rayleigh wavelength rather than the actual shape of the barrier. On the other hand, for softer barriers, it depends on the shape; however, a satisfactory screening is not achieved except for some specific dimensions[34].

(Aviles and Sanchez-Sesma on1988) the wave-barrier problems for underground explosions have been numericallyand theoretically investigated[35].

(S. Ahmad, and T. M. Al-Hussainion 1991) studies a Simplified Design for Vibration Screening by Open and In-Filled Trenches[25].

(Baker on 1994) conducted a series of field model tests to investigate the effectiveness of barriers made of bentonite (i.e. soft barrier) and concrete (i.e. stiff barrier) installed near and far from the source of disturbance, which are known as active and passive vibration screening, respectively. He compared the experimental findings with the available numerical results in the literature obtained using the boundary element method (BEM) and the empirical design equations developed by **(Al-Hussainion 1992)**[36].

(Andersen and Nielsen on 2005) developed a coupled FEM–BEM model to investigate the reduction of ground vibrations by means of barriers or soil improvement along a railway track[37].

(Ashwani Jain, D.K.Sonion 2007) proposes several vibration isolation strategies have been conducted. The majority of active isolation strategies are available[23].

(Xu and al. on 2008) indicate that the traffic- or machine-induced vibration can be reduced due to the absorption of vibration by soilbags[38].

(Bhatia KG on 2008) describes the design aids/methodologies for foundation design. Various issues related to mathematical modeling and interpretations of results are discussed at length. Intricacies of designing vibration isolation system for heavy-duty machines are also discussed. Influences of dynamic characteristics of foundation elements, viz., beams, columns, and pedestals etc. on the response of machine[27].

(J.-F. Semblat on 2012) discussed various numerical algorithms may be used to model soil waves and vibrations (FEM, BEM, SEM, etc.)[19].

(I. Derbal and al. on 2017) offer an artificial neural network (ANN)-based strategy for predicting time history responses at any place on the ground near a shaking table reaction mass (Earthquake Engineering Research Centre, Algiers)[1].

I.14.Conclusion

In recent years, research in the field of foundation analysis under machines has evolved to an advanced state of development. This chapter demonstrated that there are several sorts of foundations beneath machines, as well as various modeling and computation approaches. Nonetheless, the approach is the same for all types of foundations under machines; it consists in restricting the reaction to amplitudes that do not interfere with the machine's normal operation and do not disturb the persons working in the near area. Several formulations and computer programs have been created to objectively determine the reaction of foundations under machines. We have also defined the origin and nature of wave propagation, as well as the isolation systems caused by machines (shaking table), which have been theoretically and analytically explored in the literature.

CHAPTER II. NUMERICAL MODELING OF THE REACTION MASS

II. Introduction

The mechanics of soils uses the mechanics of continuous media and the mechanics of solids combined with material resistance, however in general, the structures or geotechnical work, whether made of concrete, steel, or wood, are more rigid than the surrounding soil.

Civil engineering constructions are calculated in a linear elastic manner using a variety of methods, the most responsive being those of finite element[28].

General-purpose industrial software must be capable of resolving problems of varying magnitudes (from thousands to hundreds of thousands of variables). These sophisticated programs need a thorough approach before they can be expected to solve a real-world problem correctly. Consider the following software names: NASTRAN, ANSYS, ADINA, ABAQUS, CASTEM 2000, CESAR, SAMCEF, FLAC, ABACUS 3D, PLAXIS 2D, PLAXIS 3D, etc.…

The possibilities provided by such programs are numerous[39]:

- Linear or nonlinear analysis of a continuous physical system.
- Static ordynamicanalysis.

The analysis of finite elements was carried out using the COMSOL software in this study. The approach taken consists in digitally determining whether or not the desired effects are there using the isolation system.

II.1. COMSOL Multiphysics Software Presentation

COMSOL Multiphysics (previously FEMLAB) is a finite element analysis, solver, and simulation software / FEA Software package for physics and engineering applications, particularly coupled phenomena or Multiphysics. Also provides a comprehensive interface to MATLAB and its toolboxes allowing a wide choice of programming, preprocessing, and post-processing options. COMSOL Multiphysics supports coupled systems of partial differential equations in addition to traditional physics-based user interfaces (PDEs).

The Model Builder integrates all of the modeling workflow phases, such as establishing geometry, material characteristics, and the physics that define specific phenomena, as well as solving and post-processing models to get required result[40].

II.2.Analysis Procedure

In the beginning when we open the software, the module browser appears. This is where we defined the physical model(s) that will be used, and where the dimension of the space is chosen (2D, 2D axisymmetric, 3D.....). For each of the models, it is specified which are the variables and which is the suffix specific to this model for creating a project, one should go through the menu from top to bottom, starting with the geometry of the problem. it is the main window for entering all of the specifications of the model, including the dimensions of the geometry, properties of the materials, boundary conditions and initial conditions, and any other information that the solver will need to carry out the simulation. It is possible to display them in the post-processor window. The user can choose the variables and the shape he wants to represent[1].

II.3.Soil Behavior

Soil behavior and the stability of structures lies in the ability of civil engineers to perform a good design of buried structures while facing the difficulties of modeling soil-structure interaction systems. The goal of each study or analysis is to determine the actual behavior of structures (displacement, bending moment, shear force, etc.) in order to ensure an economical and safe design.

The soil is a continuous medium that depends on the constraints. It is made up of several layers and each one has its physical characteristics as well as different constitutive relations and this is what pushed the researchers to simplify the hypotheses which allow the analysis of the phenomenon of soil-structure interaction[27].

II.4.Models Studied

II.4.1Model 1 (designing the reaction mass and the surrounding soil)

Using the finite element method, a detailed numerical model of the soil-structure interaction of the reaction mass of the CGS Algiers shaking table and the surrounding soil was developed using the COMSOL Multiphysics software.

The following table shows the characteristics of the concrete used in the reaction mass blocks:

TableII.1 The characteristics of the concrete.

Material	Elasticmodulus(Mpa)	Density(Kg/m^3)	Poisson's ratio
Concrete	3.2×10^7	2500	0.2

II.4.1.1 The extent of the soil domain

When soil is treated as a continuum, it becomes important to constrain it to a finite domain for analytical reasons.

• What should be the extent of the soil domain to be considered for modeling?

•Whether to consider the soil domain merely below the foundation base (in which case the foundation becomes un-embedded) or to consider the foundation embedded in the soil domain (in which case the foundation becomes embedded in the soil)?; in our case soil domain is modeled right from the ground level encompassing the foundation. This makes the foundation embedded in the soil, which is a realistic situation.

It is generally understood in FE modeling that a restricted domain with fixed borders is unlikely to replicate true soil behavior, whereas a sufficiently vast domain would result in increasing issue size. As a result, an ideal value that reflects true soil behavior without severe accuracy loss is enabled.

(Bhatia KGon 2008) research said that the specific choice on the extent of the soil domain is yet unknown. Even from an academic standpoint, there is no definitive solution to this question. It is also true that, due to time constraints, a practicing engineer cannot afford to perform Research and development or ignore the problem. In the author's perspective, a soil domain equivalent to 3 to 5 times the lateral dimensions in plan on either side of the foundation and 5 times along the depth should be enough.

FE idealization is used to simulate the finite soil domain as well as the foundation block. The soil is given appropriate soil parameters such as Elastic Modulus/Shear Modulus and Poisson's Ratio. When a soil profile reveals the presence of layered media,

relevant soil attributes are allocated to the different soil layers, with variations in soil properties throughout the length, width, and depth of the soil domain[27].

II.4.1.2 Determination of the soil extended

(N.Bourahla on 2013) research said thatthe geotechnical and geophysical data provided by the field survey and confirmed by excavation during the construction phase shows a soil profile with a sandy clay layer from the surface to 2.5 m depth, a hard gravel layer approximately 3.5 m thick below, and a bottom layer of marl with varying characteristics. As shown in Figure II.2, the soil profile in the zone of interest for our research is quite uniform[41].

The following characteristics of each soil layer are shown in Table II.2:E (Elastic modulus), γ (Density ratio), and n (Poisson's ratio).

TableII.2The geotechnical characteristics of the soil[41].

Depth (m)	Elasticmodulus(Mpa)	Density ratio (Kn/m^3)	Poisson's ratio
0-1.5	400.0	17.6	0.32
1.5-2.5	1200.0	17.6	0.32
2.5-4.5	3000.0	15.9	0.34
4.5-6.0	5000.0	15.9	0.28
6.0-8.0	2700.0	16.2	0.22
below 8.0	1000.0	16.2	0.41

To study the response of this system we start modeling with two blocks of reinforced concrete the first block with a base of **18m x 18m x 4m** topped by another block of **16m x 16m x 6m** (width, depth, height) , then we add another one which represent the area of the ground chosen for the experiment with a dimension of **150m x 150m x 50m .**

The ground chosen have 6 different layers with a different geotechnical characteristics like mentioned in TableauII.2.
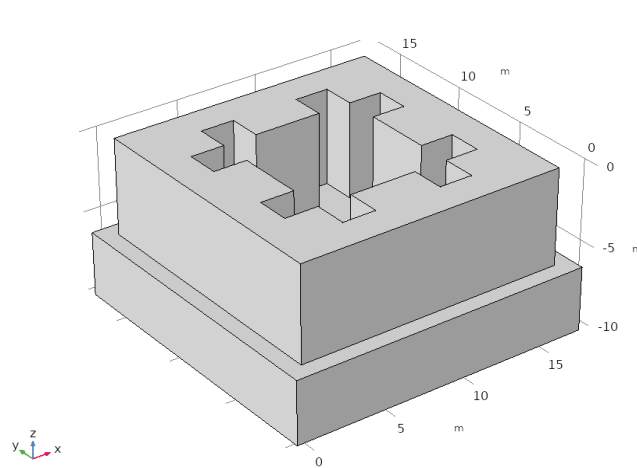
FigureII.1 *3-D view of the reaction mass.*
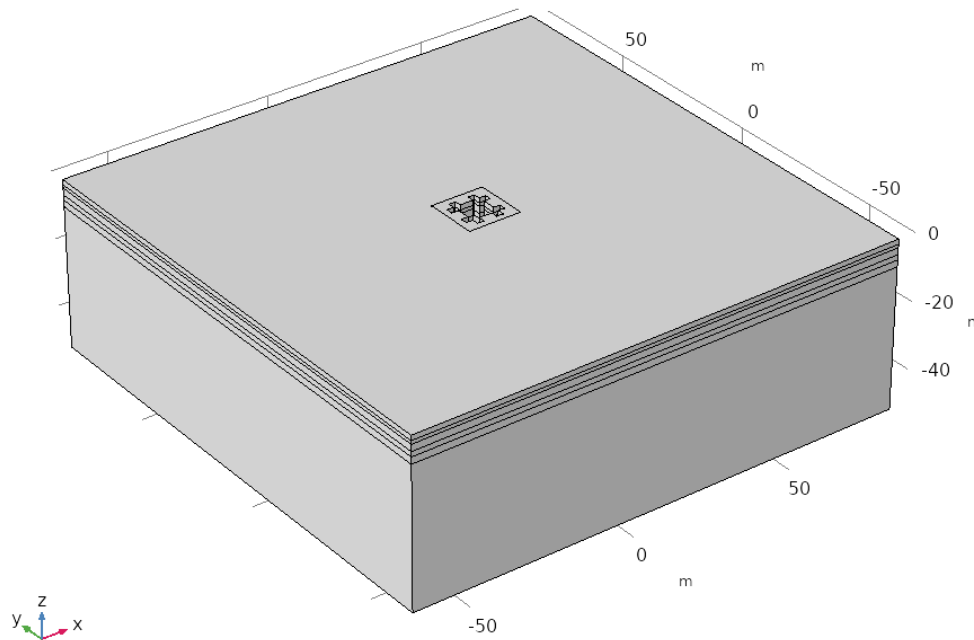
FigureII.2 *The extent of the soil chosen*

The reaction mass and the surrounding soil are represented by solid (volumetric) elements that consist of 119900 domain elements, 40290 boundary elements, and 1768 edge elements.
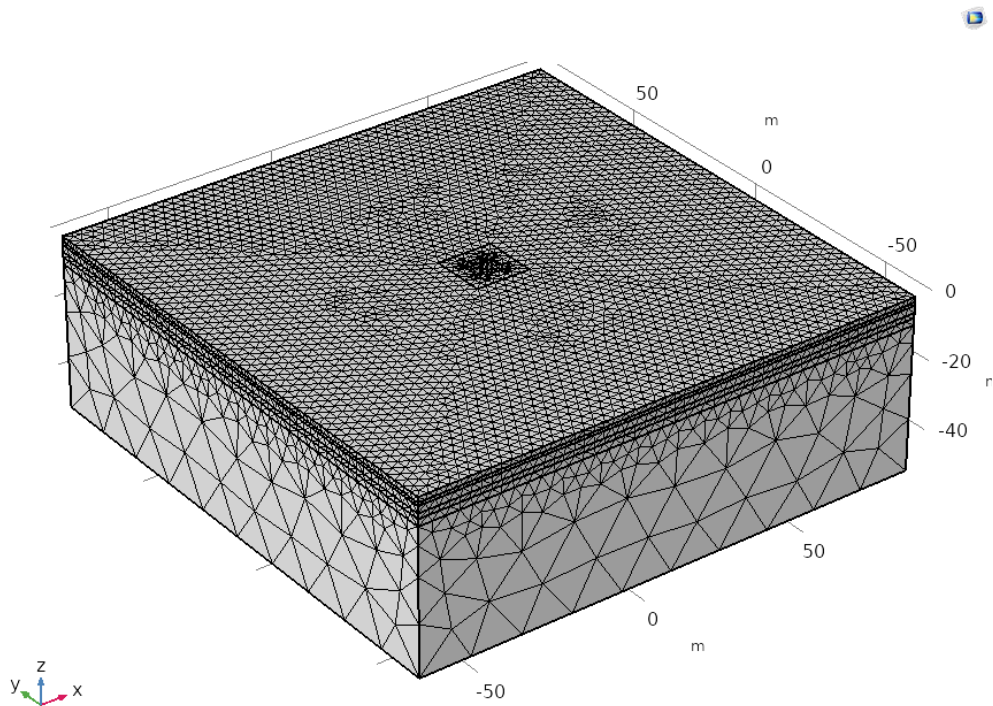
FigureII.3 *Schematic of the reaction mass and the surrounding soil and their chosen mesh size.*

The principle is to model the reaction mass and surrounding soil in finite elements, a mesh is chosen to distinguish the concrete mass from the surrounding soil and the different layers of soil, to take into consideration for the voids reserved in the concrete block for the vibrating table equipment.

The principle of this work is to introduce vibrations at the points of application of hydraulic cylinders by using point forces that represent the weight of the vibrating table and that of the specimen; we introduced these vibrations in the form of excitations of 10 m / s²(1g).
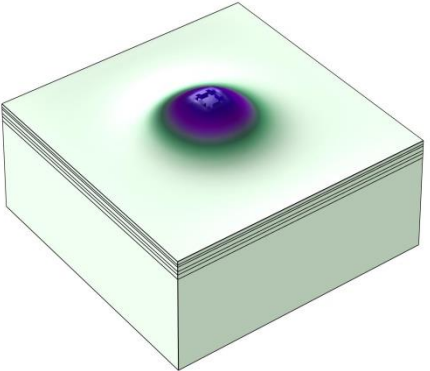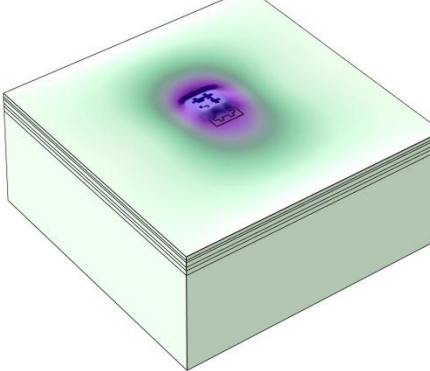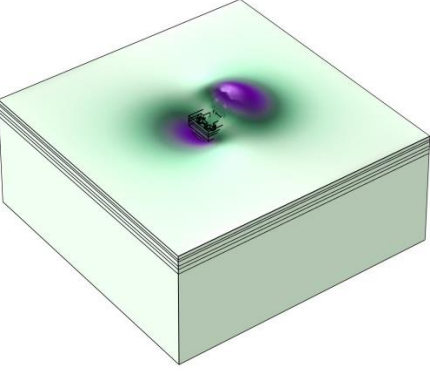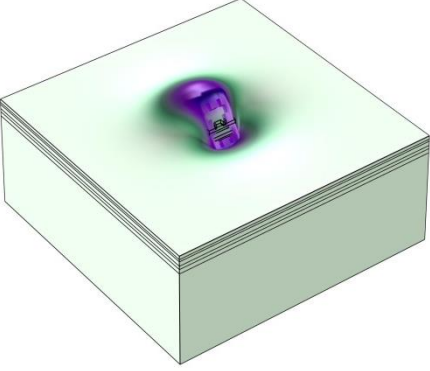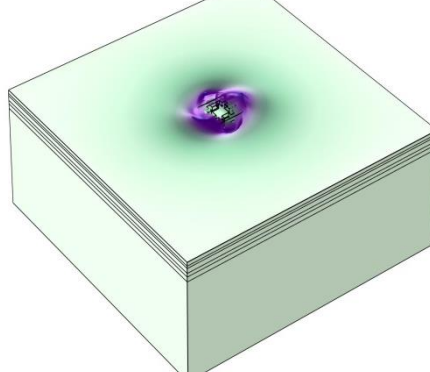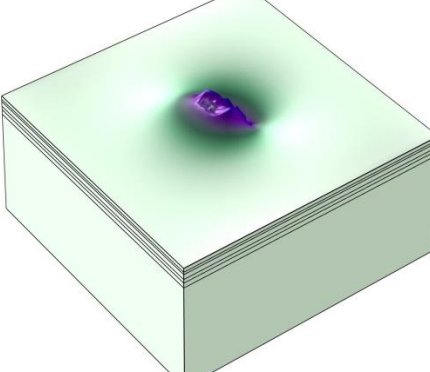
II.4.2Model 2 (Determination of Eigenfrequencies)

Eigenfrequencies, often known as natural frequencies, are discrete frequencies at which a system is prone to vibrating. Natural frequencies appear in various types of systems.

When a structure vibrates at a specific eigenfrequency, it deforms into the corresponding shape, the eigenmode. The amplitude of any physical vibration cannot be

determined by an eigenfrequency analysis. The true magnitude of the deformation can only be established if the actual excitation and damping parameters are known[42].

TableII.3Eigenfrequencies of the system.

Mode 1 (12.772 Hz)	Mode 2(14.822 Hz)
Mode 3 (21.322 Hz)	Mode 4 (24.383 Hz)
Mode 5 (31.206 Hz)	Mode 6 (35.559 Hz)

For our study we take into consideration the 1 & 2 modes to determinate the damping using the frequency and damping ratio.

(Bhatia on 2008) In the absence of any specified data for damping value of a site, the damping coefficient equal to 8 to 10% i.e. $\zeta = 0.08$ to 0.1 could safely be considered for computing response at resonance[27].

$$\begin{cases} f_1 = 12.772 \ Hz \\ \ f_2 = 14.822 \ Hz \\ \quad\ \zeta_1 = 0.1 \\ \quad\ \zeta_2 = 0.1 \end{cases}$$

By adding those parameters to the COMSOL model,it will calculate automatically the damping starts by checking equation (2.1) then move to calculate the damping using equation (2.2).

$$\frac{f_1}{f_2} < \frac{\zeta_2}{\zeta_1} < \frac{f_2}{f_1} \tag{2.1}$$

$$\rho \frac{\partial^2 u}{\partial t^2} + \alpha_{dM} \rho \frac{\partial u}{\partial t} = \nabla \left(s + \beta_{dK} \frac{\partial s}{\partial t} \right) + Fv \tag{2.2}$$

Where:

$$\alpha_{dM} = 4\pi f_1 f_2 \frac{\zeta_1 f_2 - \zeta_2 f_1}{f_2{}^2 - f_1{}^2} \tag{2.3}$$

$$\beta_{dK} = \frac{1}{\pi} \frac{\zeta_2 f_2 - \zeta_1 f_1}{f_2{}^2 - f_1{}^2} \tag{2.4}$$

In this chapter, we want to aim at the accelerations induced in the soil near the reaction mass. We assume that the reaction mass is isolated on a large area of soil, even though this implies the presence of other equipment and structures nearby , so it is required to know the influence of these vibrations on the surroundings of the vibrating table.

Using our model ,we did a "time dependent study" with an interval of time from [0 to 3s] with 0.1s step andfor the tolerance which is the convergence criteria value in COMSOL, the value entered here will be used as the default for all equations,the default value is 0.01 in COMSOL.

While doing an evaluation on the vicinity buildings knowing that the nearest one from the source is at 30 m.

The following (figure II.4) represents the acceleration of the reaction mass in relation to time at a distance of 30m:
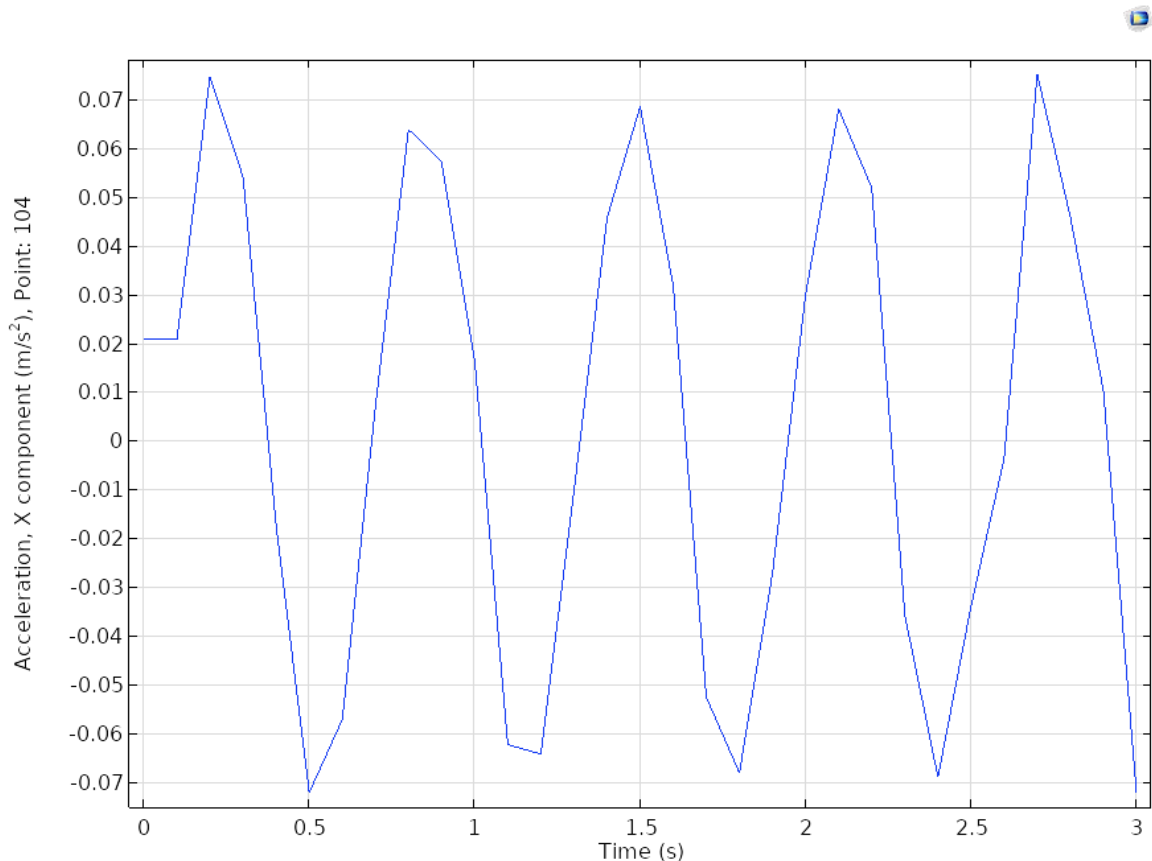
FigureII.4 *The acceleration of the reaction mass in relation to the time.*

II.5.Norms

For the intended use, there are no satisfying norms. The closest standard would be **ISO 10137**, which provides recommendations for vibration in buildings, whether the source is internal or external, in order to protect the human being who is subjected to it in terms of comfort, fatigue, and safety. Although it is not the same condition, it can be used as a reference. Human excitations, machinery, and construction or demolition operations are all considered internal causes. Furthermore, there are two types of vibration problems:

- Class A, where the source of vibration changes in time and position.
- Class B, where the source changes in time but is constant in position.

For machinery vibration measurement, the sensor should be located where the most intense vibrations are expected to occur, either on the platform or the support structure.The effect of vibration on people in buildings should be evaluated in a position where people are likely to feel it and in the direction that has the most impact on them. Human occupant characteristics are classified into three categories:

- Sensitive occupancy such as operating rooms;
- Typical occupancy such as offices and residential properties;
- Active occupancy such as industrial settings, industrial surroundings.

However, vibrations that influence humans may be divided into numerous categories:

- Class a: influences below human perception threshold;
- Class b: basic threshold effects;
- Class c: intrusion, alarm and fear (which may be associated with a range of adverse comments);
- Class d: interference with activities;
- Class e: possibility of injury or health risk.

In Class a, the criterion is based on interference when working with sensitive instruments where individualscome to realize that vibratory motion is present, although it is not directly perceived by any normal humansense, body function or component.

NOTE: commonly encountered ranges of measured quantities of building vibrations are the following:

- Frequency: 0,15 Hz to 100 Hz, except that for measuring impulsive responses and for buildings on rock it may be higher than 100 Hz;
- Accelerations: 10^{-3} m/s^2 to 10 m/s^2;
- Velocities: 10^{-5} m/s to 10^{-1} m/s; for measurements involving micro-electronics, optical and similar technologies (nanotechnology), lower limits may apply;
- Displacements: 10^{-7} m to 10^{-2} m.

II.6.Conclusion

We were able to analyze acceleration of the reaction mass at distance of 30 m,we see that the acceleration of the reaction mass goes from 10 m/s² to 0.07m/s² though one moves away from the reaction mass, the accelerations decrease to the point that they are neglected because of the geotechnical characteristics of the soil we had chosen.

The CGS experts says that the vibrations at 10 m should not exceed transcend 0.1 m/s² either it will be a danger to the surrounding structures, also in ISO 10137 which offers vibration estimates for structures which range from 10^{-3} m/s^2 to 10 m/s^2 for the regular vibration's acceleration. So we need to investigate the vibrationsif they have an influence on the structures for safety measurement.

CHAPTER III. Dynamic Study

III. Introduction

Building has always been one of the first concerns of man and one of his privileged occupations. Nowadays, construction is progressing in most of the countries and many professionals are engaged in the activity of building or public works. Every building project study has some goals:

- Safety: ensures the stability of the work.

- Economy: to reduce the costs of the project (expenses).

- The comfort.

- Aesthetics.

III.1. Presentation of the project

For our study we modelled 3 different types of building [G+3 , G+5 , G+8] to evaluate the displacements between floors which is caused by the shaking table .

The project consists in studying a building which includes a ground floor for commercial use and a current floor for residential use for each building , located in Algiers (group of use 2) which is classified as a zone of high seismicity (Zone III) according to the classification of zones established by the Algerian seismic regulations (RPA 99 version 2003).

Table III.1Geometric characteristics for each building.

Geometric characteristics		G+3 (m)	G+5 (m)	G+8 (m)
Dimensions in elevation	Total heightwithout an acroter	13.43	19.55	28.73
	Total heightwith an acroter	14.03	20.15	29.33
	Height of the ground floor	04.25	04.25	04.25
	Height of the current floor	03.06	03.06	03.06
Dimensions in plan	Total length in plan	23.50	23.50	23.50
	Total width in plan	08.57	08.57	08.57

III.2.Calculation Assumptions

The reinforced concrete design is based on the following assumptions at the ultimate limit state:

- The straight sections remain flat after deformation.
- There is no slip between the steel reinforcement and the concrete.
- The tensile concrete is neglected in the strength calculation because of its low tensile strength.
- The unit shortening of concrete is limited to 3.5 ‰ in simple or compound bending and 2‰ in simple compression
- The unit elongation in steels is limited to 10‰. (stretching at breach)
- The calculation constraint, noted "σ_s"and which is defined by the equation :

$$\sigma_s = \frac{f_e}{\gamma_s} \text{ is equal to :}$$

$$\text{High} - \text{adherence} \begin{cases} \sigma_s = 435 \ Mpa \quad Sustainable \ Situation \\ \ \sigma_s = 500 \ Mpa \quad Accidental \ Situation \end{cases}$$

- Stretching at breach $\zeta_s = 10\,‰$

- $$\begin{cases} f_{c28} = 25 \ Mpa \\ f_{t28} = 2.1 \ Mpa \\ \rho_b = 25 \ Kn/m^3 \\ E_{d28} = 10818.8 \ MPa. \\ FeE50 = 500 \ Mpa \\ Es \ = \ 2,1 * 10^5 \ MPa \end{cases}$$

Observation: The characteristics of the materials used in the construction of the building must be in accordance with the technical rules of construction and calculation of reinforced concrete works (BAEL91 modified 99) and all applicable regulations in Algeria (RPA 99 version 2003 and CBA93).

III.3.The pre-dimensioning

Given that the pre-dimensioning of the structural elements is done; and that all the regulatory requirements are met; the following dimensions are adopted that are both safe and economical.

Table III.2 The pre-dimensioning of the resistant elements.

Elements	G+3	G+5	G+8
	Dimensions		
Hollow-body floor	16 + 4 cm		
Solid slab floor	15 cm		
Principal Beams (P.B)	30 x 40 cm		
Secondary Beams (S.B)	30 x 35 cm		
Column	30 x 30 cm	45 x 45 cm	50 x 50 cm
Shear wall	20 cm		

III.4.Dynamic Study

The methodology adopted is based on the following points:

- Establishment of a numerical model of three-dimensional calculation in finite element of the structure with the software SAP2000v14.2.2.
- Definition of the various loads G, Q and the combinations.

III.4.1 Displacement Verification

The horizontal displacement at each level (k) of the structure is calculated by $\delta_k = R * \delta_{ek}$ (article 4.4.3 R.P.A99/v2003) with R = 4.

The relative displacement of level (k) is related to level (k-1) which is equal to:

$\Delta_k = \delta_k - \delta_{k-1}$, it is necessary that $\Delta_k < 1\% * hauteur_{étage}$ (article 5.10 R.P.A99/v2003)

Table III.3Inter-floor displacements across X axis G+8.

LEVELS	δ_{ek}(cm)	δ_k(cm)	δ_{k-1}(cm)	Δ_k(cm)	h_k(cm)	$\Delta_k/h_k\%$	Observation
	X-X						
Ground Floor	0,276	1,104	0	1,10	425,0	0,260	**Checked**
Floor 1	0,670	2,680	1,104	1,58	306,0	0,515	**Checked**
Floor 2	1,180	4,720	2,680	2,04	306,0	0,667	**Checked**
Floor 3	1,772	7,088	4,72	2,37	306,0	0,774	**Checked**
Floor 4	2,417	9,668	7,088	2,58	306,0	0,843	**Checked**
Floor 5	3,092	12,368	9,668	2,70	306,0	0,882	**Checked**
Floor 6	3,774	15,096	12,368	2,73	306,0	0,892	**Checked**
Floor 7	4,448	17,792	15,096	2,70	306,0	0,881	**Checked**
Floor 8	5,104	20,416	17,792	2,62	306,0	0,858	**Checked**

Table III.4Inter-floor displacements across X axis G+5.

LEVELS	δ_{ek}(cm)	δ_k(cm)	δ_{k-1}(cm)	Δ_k(cm)	h_k(cm)	$\Delta_k/h_k\%$	Observation
			X-X				
Ground Floor	0,394	1,379	0	1,38	425,0	0,324	**Checked**
Floor 1	0,959	3,357	1,379	1,98	306,0	0,646	**Checked**
Floor 2	1,665	5,828	3,357	2,47	306,0	0,808	**Checked**
Floor 3	2,444	8,554	5,8275	2,73	306,0	0,891	**Checked**
Floor 4	3,243	11,351	8,554	2,80	306,0	0,914	**Checked**
Floor 5	4,022	14,077	11,351	2,73	306,0	0,891	**Checked**

Table III.5 Inter-floor displacements across X axis G+3.

LEVELS	δ_{ek}(cm)	δ_k(cm)	δ_{k-1}(cm)	Δ_k(cm)	h_k(cm)	$\Delta_k/h_k\%$	Observation
			X-X				
Ground Floor	0,457	1,826	0	1,83	425,0	0,430	**Checked**
Floor 1	1,104	4,416	1,826	2,59	306,0	0,846	**Checked**
Floor 2	1,847	7,387	4,416	2,97	306,0	0,971	**Checked**
Floor 3	2,591	10,366	7,386508	2,98	306,0	0,974	**Checked**

After doing a study on the vibrations induced by the shaking table (taking into consideration only the X axis), we converted the **"FigureII.4, page 40"**to a table inserting it in sap2000 (timehistory) we got the displacements between floors which is caused by the shaking table:

Table III.6 Inter-floor displacements of the propagated waves across X axis G+8.

LEVELS	δ_{ek}(cm)	δ_{k-1}(cm)	Δ_k(cm)	h_k(cm)	$\Delta_k/h_k\%$	Observation
			X-X			
Ground Floor	0,049	0	0,05	425,0	0,011	**Checked**
Floor 1	0,120	0,049	0,07	306,0	0,023	**Checked**
Floor 2	0,212	0,120	0,09	306,0	0,030	**Checked**
Floor 3	0,318	0,212	0,11	306,0	0,035	**Checked**
Floor 4	0,432	0,318	0,11	306,0	0,037	**Checked**
Floor 5	0,550	0,432	0,12	306,0	0,039	**Checked**
Floor 6	0,668	0,550	0,12	306,0	0,039	**Checked**
Floor 7	0,784	0,668	0,12	306,0	0,038	**Checked**
Floor 8	0,897	0,784	0,11	306,0	0,037	**Checked**

Table III.7 Inter-floor displacements of the propagated waves across X axis G+5.

LEVELS	δ_{ek}(cm)	δ_{k-1}(cm)	Δ_k(cm)	h_k(cm)	$\Delta_k/h_k\%$	Observation
	X-X					
Ground Floor	0,023	0	0,023	425,0	0,005	**Checked**
Floor 1	0,055	0,023	0,032	306,0	0,010	**Checked**
Floor 2	0,093	0,055	0,038	306,0	0,013	**Checked**
Floor 3	0,134	0,093	0,041	306,0	0,013	**Checked**
Floor 4	0,175	0,134	0,041	306,0	0,013	**Checked**
Floor 5	0,214	0,175	0,039	306,0	0,013	**Checked**

Table III.8 Inter-floor displacements of the propagated waves across X axis G+3.

LEVELS	δ_{ek}(cm)	δ_{k-1}(cm)	Δ_k(cm)	h_k(cm)	$\Delta_k/h_k\%$	Observation
	X-X					
Ground Floor	0,003	0	0,003	425,0	0,001	**Checked**
Floor 1	0,006	0,003	0,004	306,0	0,001	**Checked**
Floor 2	0,010	0,006	0,003	306,0	0,001	**Checked**
Floor 3	0,013	0,010	0,003	306,0	0,001	**Checked**

Observation: Table III. (6& 7 &8)represent inter-floor displacements across X axis for three different structures we see that the displacementare small.

III.5.Conclusion

As a conclusion we see that the vibration has no influence on the structures but taking into account that the vibrations are continuous unlike thehighly unpredictable and instant seismic vibrations.

Those small vibrations cangenerate a discomfort for inhabitant and for the sensitive equipment, also may cause "the fatigue phenomena" which results in the appearance and development of cracks, damaging these structures and may lead to their ruin so we need to investigate the fatigue phenomena to negate those vibrations.

CHAPTER IV.FATIGUE DAMAGE

IV. Introduction

The phenomenon of fatigue designates the progressive degradation of structures subjected to fluctuating, alternating and repeated stresses, which results in the appearance and development of cracks, damaging these structures and may lead to their ruin. Fatigue cracks develop in three main stages: the initiation of the crack, its propagation and finally the sudden rupture of the remaining resistant section. The duration scale is very different between each stage, from a very long duration for the initiation of the crack to an almost instantaneous rupture.

Typically, a combination of an empirical stress-life model and a damage rule (e.g., the Palmgren-Miner rule) is used for such checks. However, due to the effects of progressive deformation, damage accumulation, and material crack initiation and propagation associated with fatigue loading on an overall structure, the implementation of time/cycle dependent models in fatigue analysis of structural elements is considered invaluable[43].

IV.1.Verification conditions(EN 1992-1-1: 6.8.1)

(1) The resistance of structures to fatigue shall be verified in special cases. This verification shall be performed separately for concrete and steel.

(2) Fatigue verification should be carried out for structures and structural components whichare subjected to regular load cycles (e.g. crane-rails, bridges exposed to high traffic loads).

IV.2.Fatigue Life of Steel Reinforcement

Stress-life models are usually used to predict the fatigue life of steel reinforcing bars. This means plotting the stress range versus the number of failure cycles. For predicting the number of cycles to failure for steel reinforcing bars under fatigue loading, many models have been presented and applied in various codes (Tilly, 1979; JSCE, 1986; Chinese Code, GB 50010-2002; CEB-FIP Model Code 1990; AASHTO (Specified in ACI-217R-74); EN 1992-1-1: 6.8; EN 1992-2; SIA262).

Eurocode (EN 1992-1-1: 6.8) specifies two main methods for estimating the fatigue life of steel reinforcement, the first method is used under constant fatigue loading which is

the damage of a single stress amplitude is determined using the corresponding S-N curves (FigureV.1), for the second method equation 4.2 (EN 1992-1-1: 6.8.5) is satisfied for an appropriate fatigue resistance capacity[43].

IV.2.1.Method I :

$$\gamma_{F,fat} \cdot \Delta\sigma_{s,max} \leq \frac{\Delta\sigma_{Rsk(N*)}}{\gamma_{s,fat}} \tag{4.1}$$

$\gamma_{F,fat}$ is a partial factor for fatigue loading.

$\Delta\sigma_{s,max}$ is the maximum steel stress range.

$\Delta\sigma_{Rsk(N*)}$ is the reference resisting fatigue stress range at N^*cycles (Table 6.3N in EN 1992-1-1:2004-2.4.2.4).

$\gamma_{s,fat}$ is a partial factor for fatigue that takes the material uncertainties into account (Table 2.1N in EN 1992-1-1:2004-6.8.4).

NOTE 1 the value of $\gamma_{F,fat}$ used by a given country can be provided from its National Annex. The recommended valueis $\gamma_{F,fat} = 1.0$.
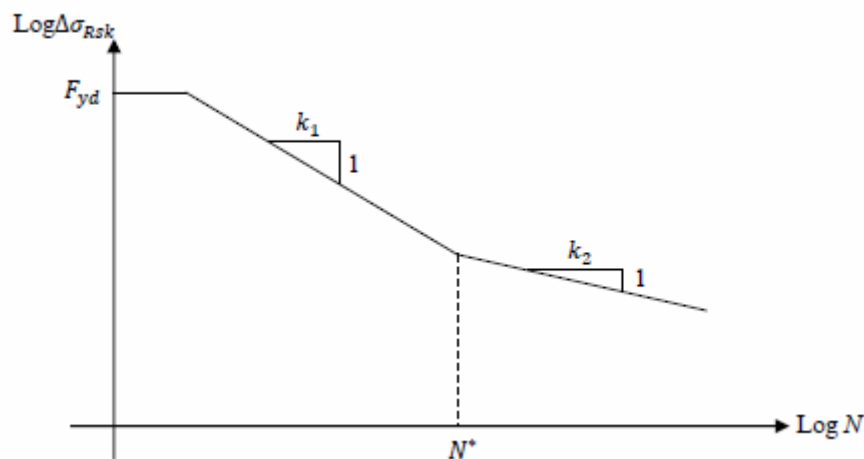
FigureIV.1Shape of the characteristic fatigue strength curve (S-N-curves for reinforcing and prestressing steel).

$k_1$and k_2 respectively, they are the constants that defines the slope of the first and second partof the stress-life curve (Table 6.3N in EN 1992-1-1:2004), F_{yd} is the yield strength of the reinforcement.

- **Numerical application**

$\gamma_{F,fat} = 1$

$\gamma_{S,fat} = 1.15$

$\Delta\sigma_{Rsk}(N*) = 162.5\ Mpa$

$\Delta\sigma_{s,max} \leq \dfrac{162.5 * 1.15}{1} = 186.875\ Mpa$

IV.2.2.Method II:

For the second technique for determining the fatigue life of steel reinforcement, we used the Palmgren-Miner variable damage accumulation rule.The fatigue damage factor D_{Ed} of steel caused by the relevantfatigue loads should satisfy the condition:

$$D_{Ed} = \sum \frac{n(\Delta\sigma i)}{N(\Delta\sigma i)i} < 1 \tag{4.2}$$

Where:

$n(\Delta\sigma i)$is the applied number of cycles for a stress range $\Delta\sigma i$.

$N(\Delta\sigma i)i$is the resisting number of cycles for a stress range $\Delta\sigma i$.

From the stress-life curve of reinforcing and prestressing steel, the corresponding ultimate number of cycles $(\Delta\sigma i)$ can be estimated thus:

$$N(\Delta\sigma i) = N * \left(\frac{\frac{(\Delta\sigma_{Rsk})}{\gamma_{s,fat}}}{\gamma_{F,fat}.\Delta\sigma i} \right)^{K_1} \quad if\ \gamma_{F,fat}.\Delta\sigma i \geq \frac{\Delta\sigma_{Rsk}}{\gamma_{s,fat}} \tag{4.3}$$

$$N(\Delta\sigma i) = N * \left(\frac{\frac{(\Delta\sigma_{Rsk})}{\gamma_{s,fat}}}{\gamma_{F,fat}.\Delta\sigma i} \right)^{K_2} \quad if\ \gamma_{F,fat}.\Delta\sigma i < \frac{\Delta\sigma_{Rsk}}{\gamma_{s,fat}} \tag{4.4}$$

Δ: stress range

$\Delta\sigma_{s,max}$: the maximum steel stress range.

$$\gamma_{F,fat} \cdot \Delta\sigma_i \geq \frac{\Delta\sigma_{Rsk}}{\gamma_{s,fat}} \tag{4.5}$$

- **Numerical application**

$$\gamma_{F,fat} \cdot \Delta\sigma_i = 1 * 186.875 = 186.875 \; Mpa$$

$$\frac{\Delta\sigma_{Rsk}}{\gamma_{s,fat}} = \frac{162.5}{1.15} = 141.30 \; Mpa$$

$$186.875 \; Mpa \geq 141.30 \; Mpa$$

$$N(\Delta\sigma i) = N * \left(\frac{\frac{(\Delta\sigma_{Rsk})}{\gamma_{s,fat}}}{\gamma_{F,fat} \cdot \Delta\sigma i}\right)^{K_1} = 10^6 * \left(\frac{\frac{162.5}{1.15}}{1 * 186.875}\right)^5 = 2.5 * 10^5$$

IV.3.Fatigue Life of Concrete

The fatigue life of concrete can be estimated using stress-life models similar to steel reinforcement. Stress-life models for measuring concrete fatigue life are often shown as graphs of normalized concrete stress levels besides the logarithm of the number of cycles resulting in failure.

According to previous research, several elements such as stress level, stress ratio, eccentricity of loading, frequency, shape of the waveform, and stress reversals impact the fatigue behavior of concrete.

Other ways that consider a comparison between the induced concrete stresses and a limiting value for suitable fatigue resistance capability are known in the literature. There are two methods in EN 1992-1-1 2004[43].

IV.3.1.Method I:

In the first method (EN 1992-1-1 2004 (6.8.7), a suitable fatigue resistance capacity for concrete under compression may be assumed if the following conditions are met:

$$E_{cd.max.equ} + 0.43 \sqrt{1 - R_{equ}} \leq 1 \tag{4.6}$$

Where:

$$R_{equ} = \frac{E_{cd.min.equ}}{E_{cd.max.equ}}$$

$$E_{cd.min.equ} = \frac{\sigma_{cd.min.equ}}{f_{cd.fat}}$$

$$E_{cd.min.equ} = \frac{\sigma_{cd.max.equ}}{f_{cd.fat}}$$

R_{equ}: stress ratio.

$E_{cd.min.equ}$: minimum compressive stress level.

$E_{cd.max.}$: maximum compressive stress level.

$f_{cd.fat}$: design fatigue strength of concrete.

$\sigma_{cd.max.equ}$: upper stress of the ultimate amplitude for N cycles.

$\sigma_{cd.min.equ}$: lower stress of the ultimate amplitude for N cycles.

IV.3.2.Method II :

In the second method (EN 1992-1-1 2004 (6.8.7)), the fatigue verification for concrete under compression may be assumed if the following condition is satisfied:

$$\frac{\sigma_{c.max}}{f_{cd.fat}} \le 0.5 + 0.45 \frac{\sigma_{c.min}}{f_{cd.fat}}$$

≤ 0.9 for $f_{ck} \le 50$ MPa

≤ 0.8 for $f_{ck} > 50$ MPa

$$\frac{\sigma_{c.max}}{f_{cd.fat}} \le 0.5 + 0.45 \frac{\sigma_{c.min}}{f_{cd.fat}} \le 0.9 \tag{4.7}$$

Where:

$\sigma_{c.max}$ is the maximum compressive stress at a fibre under the frequent load combination (Compression measured positive)(EN 1992-2:2005 (6.109)).

$$\sigma_{c.max} = 0.85 f_{cd} \frac{1 + 3.80\alpha}{(1 + \alpha)^2} \tag{4.8}$$

Where $\alpha \le 1$is the ratio between the two principal stresses.

$\sigma_{c.min}$is the minimum compressive stress at the same fibre where $\sigma_{c.max}$occurs. If $\sigma_{c.min}$

is a tensile stress, then $\sigma_{c.min}$ should be taken as 0.

Note: The value of N ($\leq 10^6$ cycles) for use in a Country may be found in its National Annex. Therecommended value is N = 10^6 cycles.

$$f_{cd.fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \frac{f_{ck}}{250}\right) \tag{4.9}$$

$\beta_{cc}(t_0)$ is a coefficient for concrete strength at first load application.

Note: The value of k_1 for use in a Country may be found in its National Annex.

$$\beta_{cc}(t_0) = exp\left[s\left(1 - \left(\frac{28}{t}\right)^{1/2}\right)\right] \tag{4.10}$$

t_0 is the age of the concrete in days.

s is a coefficient which depends on the type of cement

= 0.25 for cement of strength Classes CEM 32,5 R, CEM 42,5 N (Class N)

$$f_{cd} = \alpha_{cc} f_{ck}/\gamma_c \tag{4.11}$$

Where:

γ_c is the partial safety factor for concrete, see (EN 1992-1-1:2004(2.4.2.4)).

Note: The values of γ_c and γ_s in the serviceability limit state for use in a Country may be found in its National Annex. The recommended value for situations not covered by particular clauses of this Eurocode is 1.0.

α_{cc} is the coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied.

Note: The value of α_{cc} for use in a Country should lie between 0.8 and 1.0 and may be found in its National Annex. The recommended value is 1.

- **Numerical application**

$k_1 = 5$

$$\beta_{cc}(t_0) = exp\left[0.25 * \left(1 - \left(\frac{28}{28}\right)^{1/2}\right)\right] = e^0 = 1$$

$\alpha_{cc} = 1$

$$f_{ck} = 25 \ Mpa$$

$$\gamma_c = 1.5$$

$$f_{cd} = 1 * 25/1.5 = 16.67 \ Mpa$$

$$f_{cd.fat} = \beta_{cc}(t_0)f_{cd}\left(1 - \frac{f_{ck}}{250}\right) = 5 * 1 * 16.67 * \left(1 - \frac{25}{250}\right) = 75.015 \ Mpa$$

$$0.5 \ + \ 0.45 \ \frac{\sigma_{c.min}}{f_{cd.fat}} \leq \ 0.9 \Rightarrow \sigma_{c.min} = \frac{(0.9 - 0.5)f_{cd.fat}}{0.45}$$

$$\sigma_{c.min} = 66.68 \ Mpa$$

$$\alpha = 1$$

$$\sigma_{c.max} = 0.85 f_{cd} = 0.85 * 16.67 * 1.2 = 17 \ Mpa$$

$$\frac{\sigma_{c.max}}{f_{cd.fat}} = \frac{17}{75.015} = 0.23$$

$$\left(\frac{\sigma_{c.max}}{f_{cd.fat}} = 0.23\right) < \left(0.5 \ + \ 0.45 \ \frac{\sigma_{c.min}}{f_{cd.fat}} = 0.9\right) \leq 0.9 \qquad \textbf{Confirmed Condition}.$$

IV.4. Static Analysis

This method was designed to allow for the analysis of existing buildings and the investigation of the efficacy of plans for strengthening these structures and giving them greater ductility.

IV.4.1.Pushover Equivalent Static Analysis

Pushover analysis is a static study used to determine how far into the inelastic range a building may reach before collapsing completely or partially. On a computer, a model of the structure is created, including all load-resisting elements and their force-deformation connections both before and after yielding, as well as dead loads plus average live loads. Then, to model the impacts of ground movements, a limited set of horizontal forces is applied, and deformations are identified[44].

This analysis is used to assess the predicted performance of structural systems by predicting the strength and deformation demands in design earthquakes using static inelastic analysis and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, inter-story drift,inelastic element deformations (either

absolute or normalized with respect to a yield value),the inelastic static pushover analysis could also be considered as a method for estimating seismic force and deformation demands that accounts for the redistribution of internal forces that can no longer be resisted within the elastic range of structural behavior in an approximate way.

The distribution of mass and stiffness is generally recognized to be one of the primary concerns in the seismic design of moderate to high rise structures. Because these factors invariably introduce coupling effects and non-linearities in the system, it is necessary to use a non-linear static analysis approach with specialized programs such as SAP2000,STAADPRO2005,ETABS,IDARC,NISA-CIVIL, and others for cost-effective seismic evaluation and retrofitting of buildings.

Modeling methodologies, acceptance criteria, and analytical processes for pushover analysis have been defined in the ATC-40 and FEMA-273 documents. These documents define the forcedeformation criteria for hinges that are utilized in pushover analysis. As illustrated in "FigureIV.2", five points labeled A(unloaded component), B(initial yielding of the fixed-base structure point), C(abscissa value where component strength degradation begins), D(ultimate strengh point), and Beyond point D, the component responds with substantially reduced strength to point E. At deformations greater than point E, the component strength is essentially zero. Those points are used to describe the hinge's force deflection behavior, and three points labeled IO, LS, and CP (Immediate Occupancy, Life Safety, Collapse Prevention respectively) are applied to identify the hinge's acceptance requirements. The values given to each of these points differ based on the kind of member and a number of other characteristics stated in the ATC-40 and FEMA-273 documents[45].
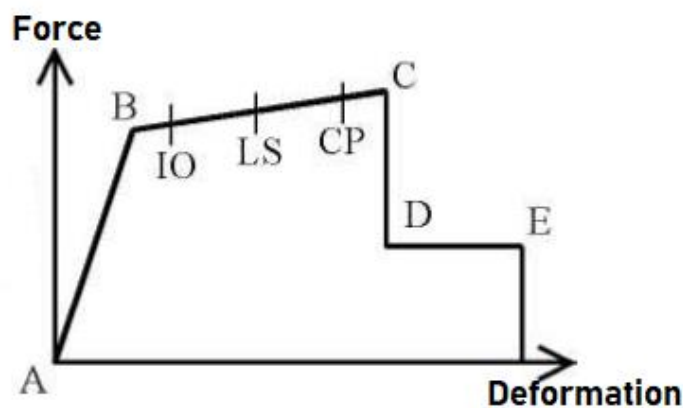

FigureIV.2 *forcedeformationcriteria for hinges used in pushover analysis*[45].

IV.4.1.1.Earthquake Software for Response Spectrum Matching

SeismoMatch is an application which can adapt earthquake accelerograms to match a certain target response spectrum using the wavelets approach given by Abrahamson [1992] and Hancock and al. [2006] or the Al Atik and Abrahamson [2010] algorithm.

The target response spectrum is created by following regulations from a selection of over 25 National Building Codes, including Eurocode 8 norms, ASCE 41-13 and numerous Regulations throughout the world, by computing the spectrum of a given accelerogram, or by simply loading a user-defined spectrum[46].
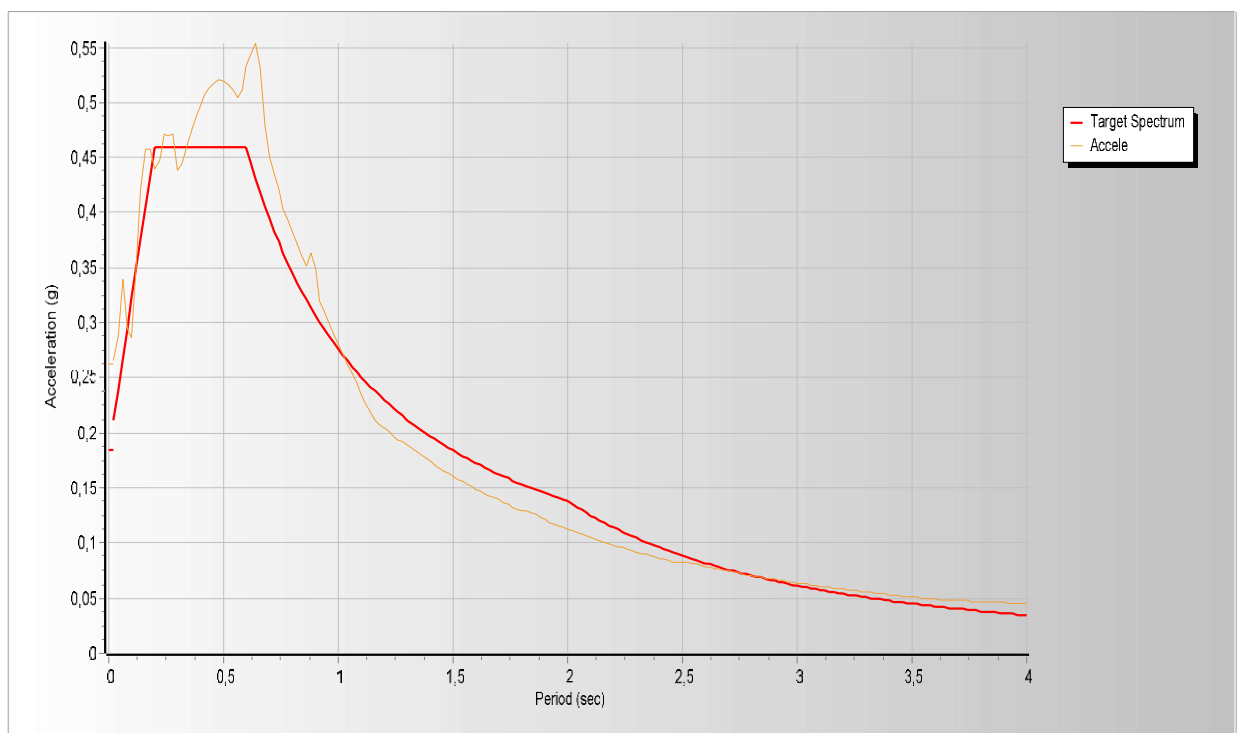
*Figure**IV.3** Evolution of the Matched accelerogramscurve (acceleration by period).*

We used the SeismoMatch to converge the accelerogram we had From COMSOL **"FigureII.4, page 40"** to get the aimed response spectrum since the pushover method in sap2000 needs a specified responsespectrum(acceleration by period).

IV.4.2. Sap2000 Analysis

The result of fatigue study we had using the non-linear approach of the Pushover analysis which can determine whether the building is safe or needs strengthening and its extent.

- **Deformation of a "G+3" building before isolation**
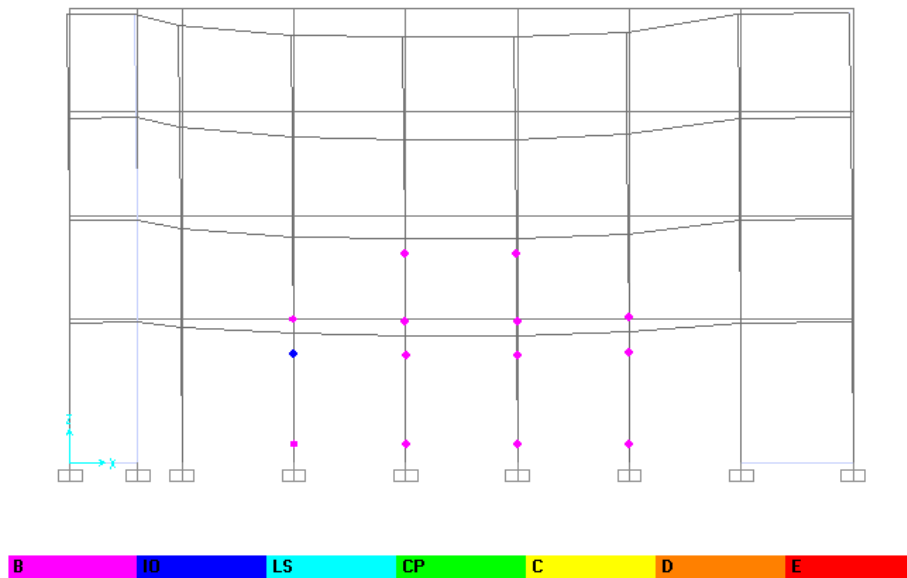
*Figure***IV.4** *Result of Pushover analysis (step2, G+3).*

*Figure***IV.5** *Result of Pushover analysis (step3, G+3).*
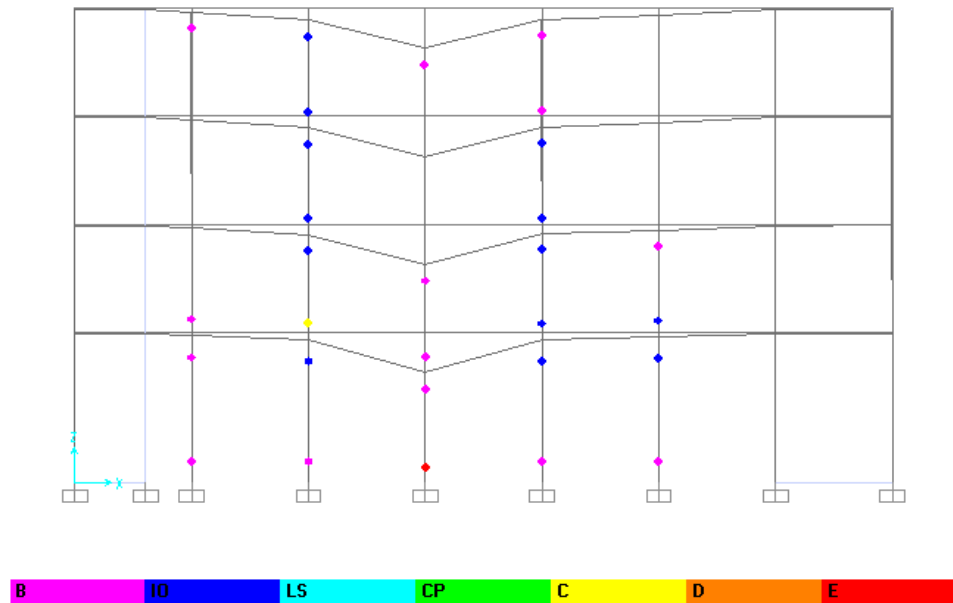
***Figure*IV.6** *Result of Pushover analysis (step4, G+3).*

- **Observation**

The figures (4, 5, 6) shows the steps of deformation"G+3" structure go through:
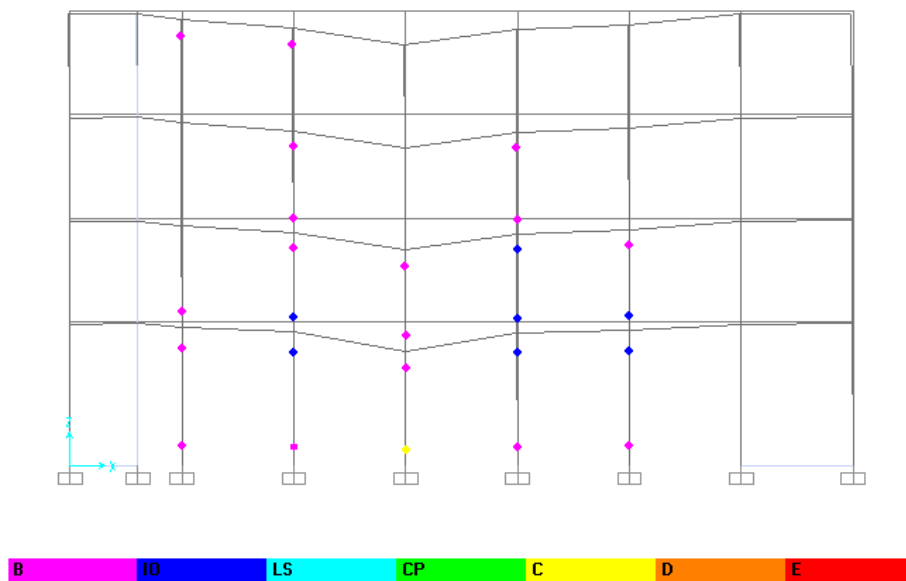
- Step2 is the first stage of deformation.
- Step 3 we see that the weak element is the middle column in the ground floor with the yellow point and show a high risk of rupture.
- Step 4 we see that same column pass the limit of resistance and start breaking down.
- **Deformation of a "G+5" building before isolation**

***Figure*IV.7** *Result of Pushover analysis (step2, G+5).*

***Figure*IV.8** *Result of Pushover analysis (step6, G+5).*
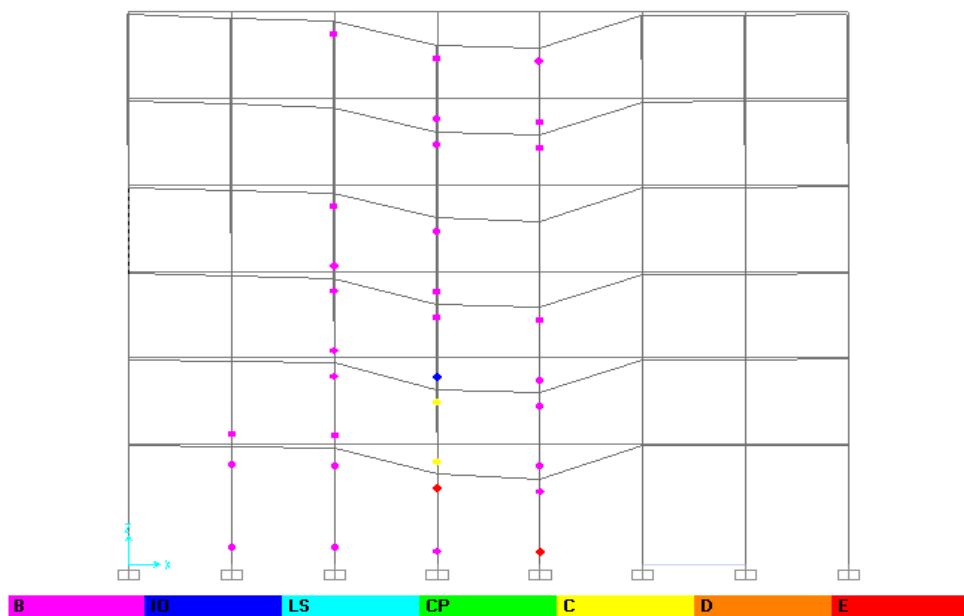
***Figure*IV.9** *Result of Pushover analysis (step7, G+5).*

- **Observation**

The figures (7, 8, 9) shows the steps of deformation "G+5" structure go through:

- Step2 is the first stage of deformation (the two central columns).
- Step (3, 4, 5) have the same deformation with the same level of danger as step 2.
- Step 6one of the two columns in the ground floor shows a high risk of rupture (yellow danger level).
- Step 7 we see that same column pass the limit of resistance and start breaking down, while the second have more aptitude of breaking down on a higher level.

- **Deformation of a "G+8" building before isolation**

***Figure*IV.10** *Result of Pushover analysis (step2, G+8).*

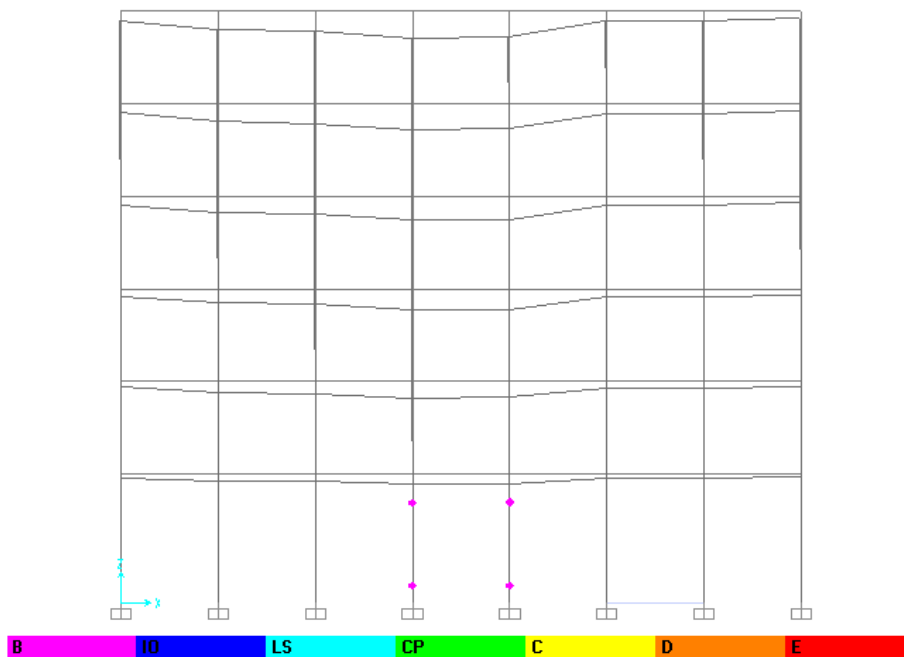***Figure*IV.11** *Result of Pushover analysis (step3, G+8).*

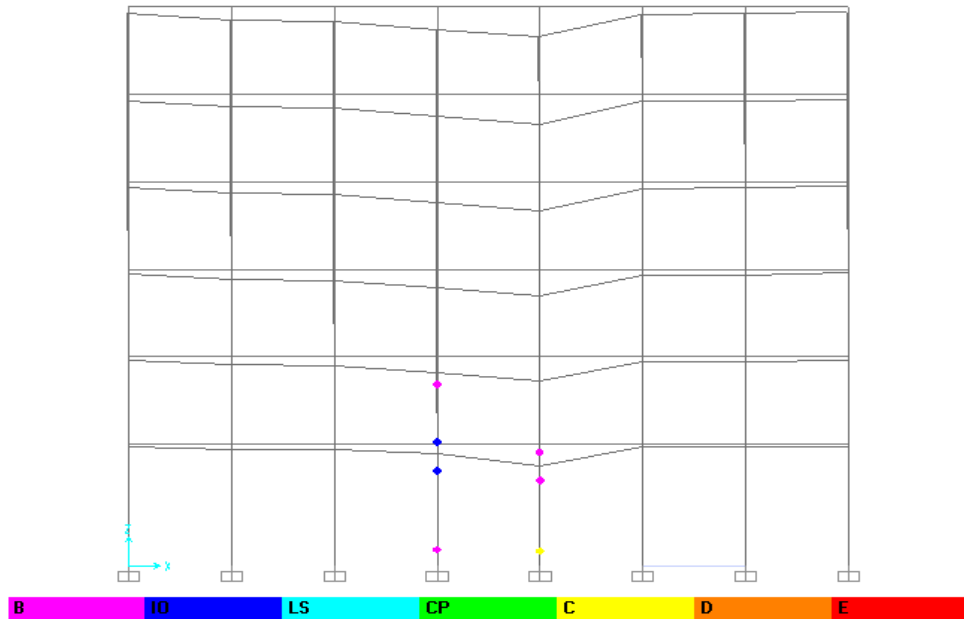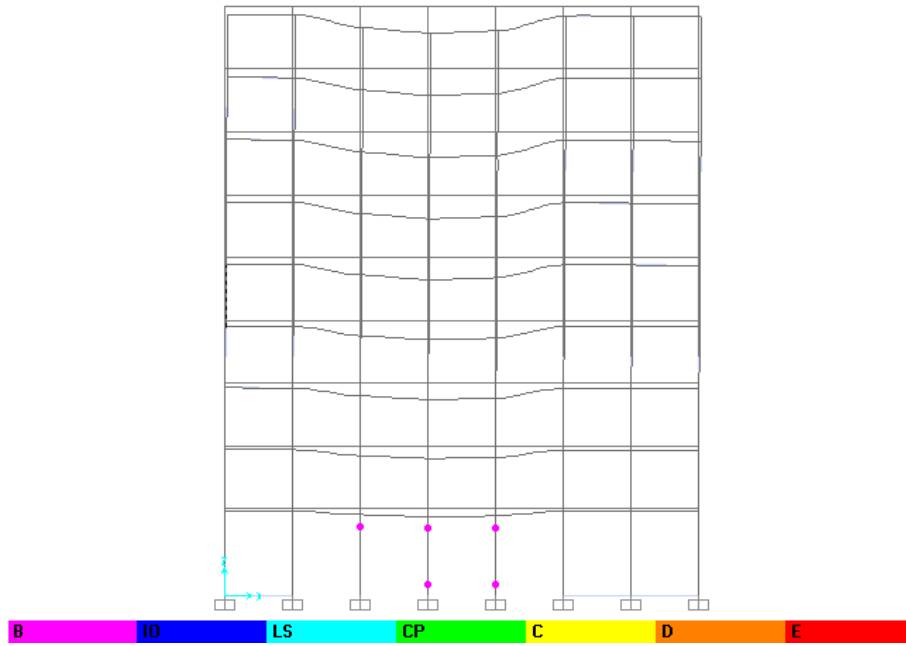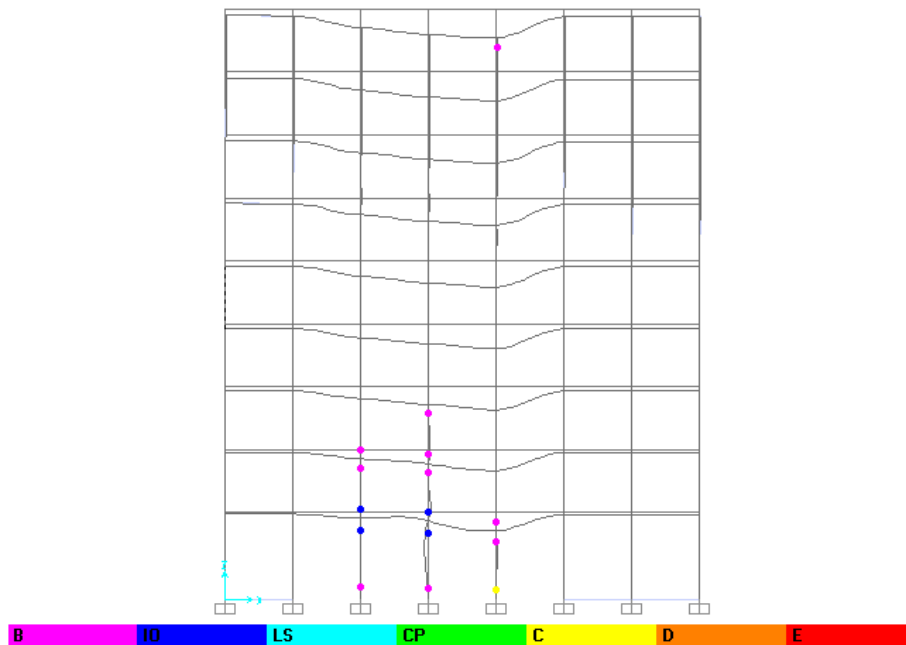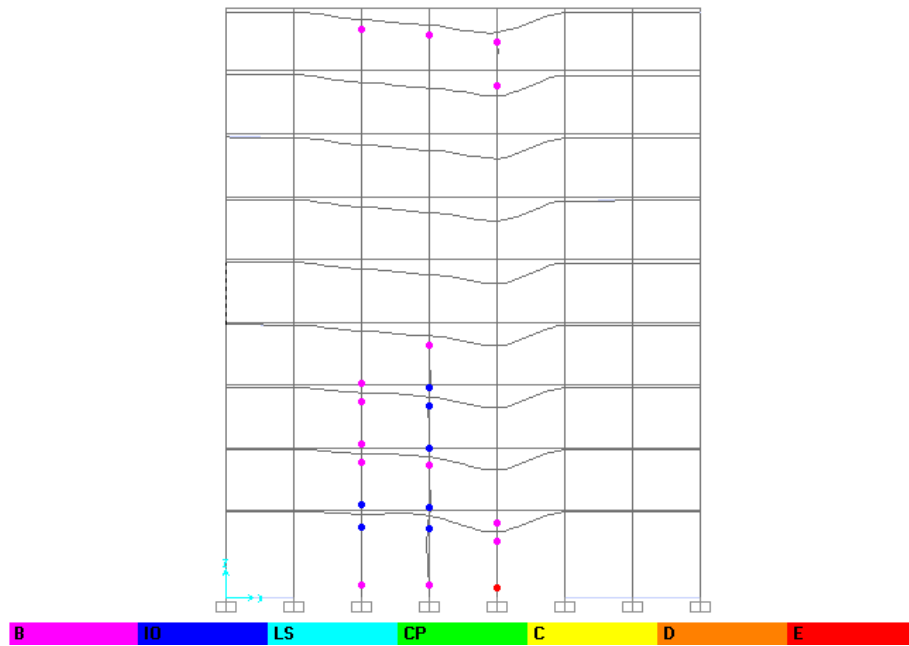***Figure*IV.*12*** *Result of Pushover analysis (step4, G+8).*

- **Observation**

The figures (10, 11, 12) shows the steps of deformation"G+8" structure go through:

- Step2 is the first stage of deformation (the central column).
- Step 3the first element which has a high risk of rupture is located in the middle column in the ground floor (yellow danger level).
- Step 4 we see that same column pass the limit of resistance and start breaking down.

IV.5. Conclusion

As an improvement over the rudimentary use of stress-life models and the Palmgren-Miner rule, the literature has fatigue constitutive models and damage models for concrete that account for progressive damage and irreversible deformation accumulation.

As fatigue loads increase, irreversible deformation accumulates in concrete structures, this chapter provide an approach for the fatigue loads using the stress-life models and the Palmgren-Miner rule and the pushover no-linear study (to find out the weak elements of the three structures).

These small deformations (concrete cracking), in addition to the irreversible deformation buildup in concrete, may impact the larger structure's stability and

deformation. As a result, rather than estimating the number of cycles at which a material may fail at a locality, the progressive stability in terms of residual strength and the corresponding deformation evolution is more important, given that the global structure's residual strength or serviceability limits may be achieved even before a local material failure occurs under fatigue loading.

From the result of our study we could ascertain that the materials (concrete and steel) have the ability to withstand the fatigue loads using the analytical method, however the static analysis for the structures hinges we could identify the weak elements which may ruin the buildings (rupture) so we can reinforce it or isolating the structures from this vibrations by proposing an isolation system that can eliminate or negate those vibrations.

CHAPTER V .CONCEPTIONS OF ISOLATION SYSTEMS

V. Introduction

Isolation barriers are a technique of treatment that removes or decreases undesired vibration to a safer level. Different approaches are proposed for optimizing the transmission channel to lessen the intensity of vibration transmitted from source to receiver (between the vibration source and the structure). Among these there is the isolation barriers, the vibration waves would be absorbed or reflected by the barrier.

It is possible to reduce vertical displacements and acceleration by constructing an adequate wave barrier in the ground before the structure, using open or in-filled trenches.

V.1. Types of isolation

As we mentioned in the bibliographic study there are different types of isolation, we opted for open and in-filled trenches as a wave barrier.

Regarding the filled trenches, the following materials were used:

- Concrete (to study the conception of propagated vibration for a solid materiel)
- Water (An academic study of the conception of propagated vibration for a fluid material)

V.1.1. Model I : Open trench
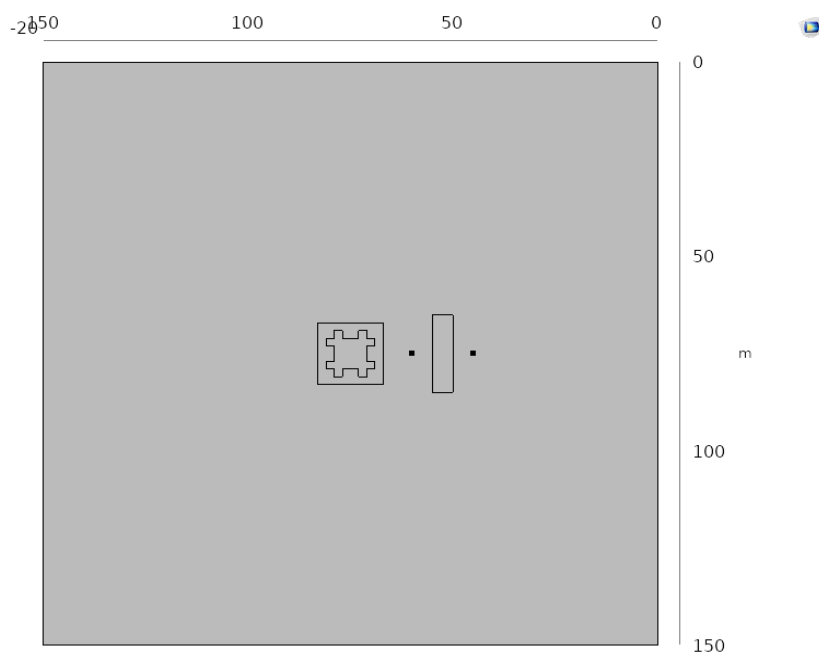
FigureV.1 *Conception of the open trench (view x-y).*

"FigureV.1" represent the x-y view of the first model which is the conception of the open trench with two points to evaluate the vibrations before and after the barrier.

For this model a parametric sweep analysis is launched to vary the dimensions of the trench including width, depth and height of the trench, furthermore evaluating vibrations before (vibration induced by the mass reaction with the reflected waves by the barrier) and after the trench (the propagated harmless waves).

The results are mentioned in the following tables:

- **Variation of the width**

Start = 0.5 m

Step = 0.5 m

End = 5 m

Depth = 10 m

Height = 5 m

Table V.1 *Variation of acceleration before and after the open trench with the width.*

W	D	H	Acceleration m/s²			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
0,5	10	5	5,398	163,477	0,571	74,252
1	10	5	5,365	162,477	0,555	72,172
1,5	10	5	5,428	164,385	0,542	70,481
2	10	5	5,430	164,446	0,527	68,531
2,5	10	5	5,402	163,598	0,513	66,710
3	10	5	5,451	165,082	0,500	65,020
3,5	10	5	5,431	164,476	0,486	63,199
4	10	5	5,444	164,870	0,472	61,378
4,5	10	5	5,463	165,445	0,458	59,558
5	10	5	5,453	165,142	0,444	57,737

FigureV.2 *Variation of acceleration before and after the open trench with the width.*

- **Variation of the Depth**

Start = 1 m

Step = 1 m

End = 10 m

Width = 2 m

Height = 5 m

Table V.2 *Variation of acceleration before and after the open trench with the Depth.*

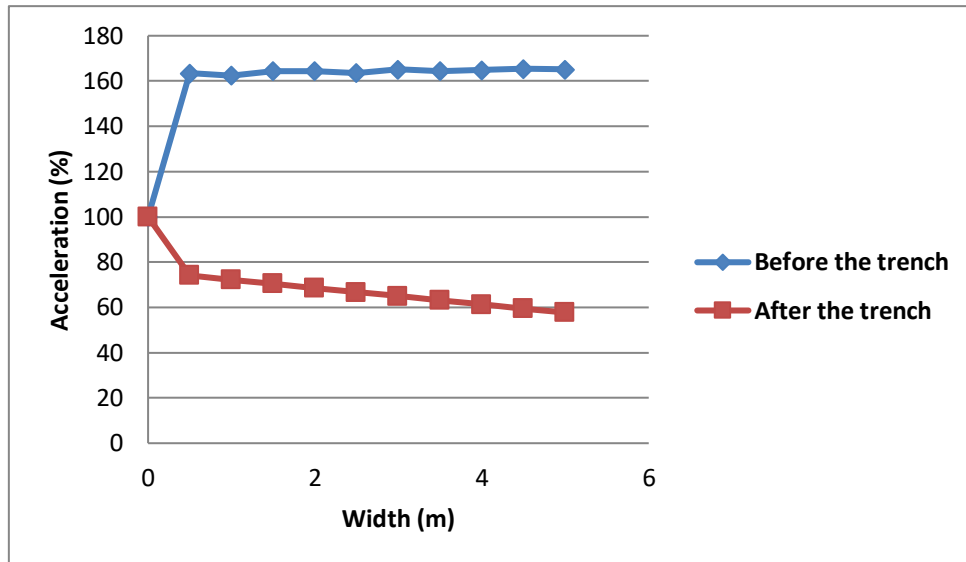| D	W	H	Acceleration m/s^2			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
1	2	5	3,338	101,090	0,769	100,000
2	2	5	3,387	102,574	0,763	99,220
3	2	5	3,472	105,148	0,749	97,399
4	2	5	3,673	111,236	0,722	93,888
5	2	5	3,990	120,836	0,681	88,557
6	2	5	4,418	133,798	0,642	83,485
7	2	5	4,883	147,880	0,598	77,763
8	2	5	5,168	156,511	0,559	72,692
9	2	5	5,411	163,870	0,528	68,661
10	2	5	5,624	170,321	0,506	65,800
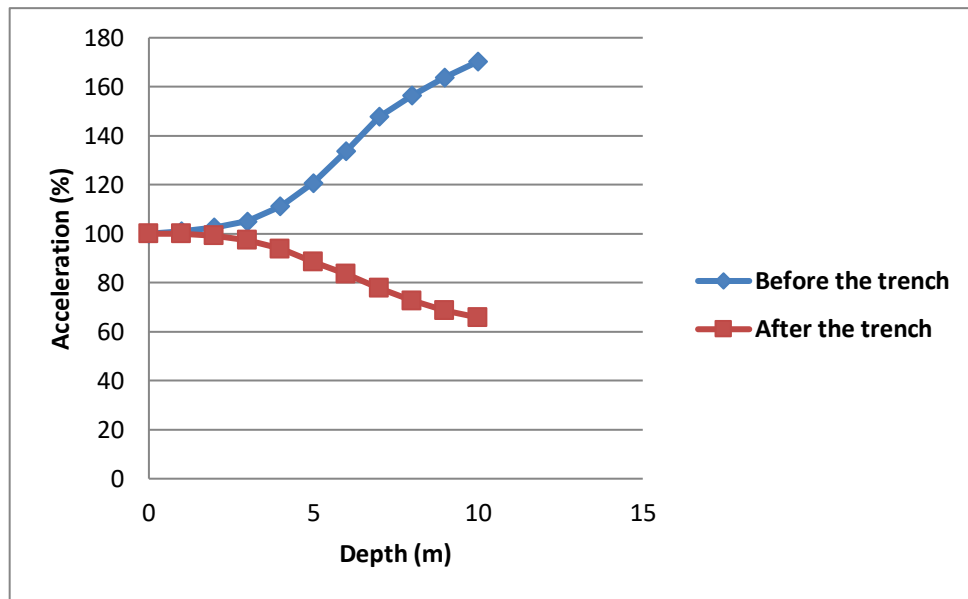
FigureV.3Variation of acceleration before and after the open trench with the Depth.*

- **Variation of the Height**

Start = 0.5 m

Step = 0.5 m

End = 5 m

Width = 2 m

Depth = 10 m

Table V.3 *Variation of acceleration before and after the open trench with the Height.*

H	D	W	Acceleration m/s²			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
0,5	10	2	3,389	102,635	0,760	98,830
1	10	2	3,556	107,692	0,749	97,399
1,5	10	2	3,713	112,447	0,74	96,229
2	10	2	3,914	118,534	0,721	93,758
2,5	10	2	4,102	124,228	0,704	91,547
3	10	2	4,332	131,193	0,669	86,996
3,5	10	2	4,631	140,248	0,630	81,925
4	10	2	4,904	148,516	0,593	77,113
4,5	10	2	5,141	155,694	0,563	73,212
5	10	2	5,429	164,416	0,527	68,531
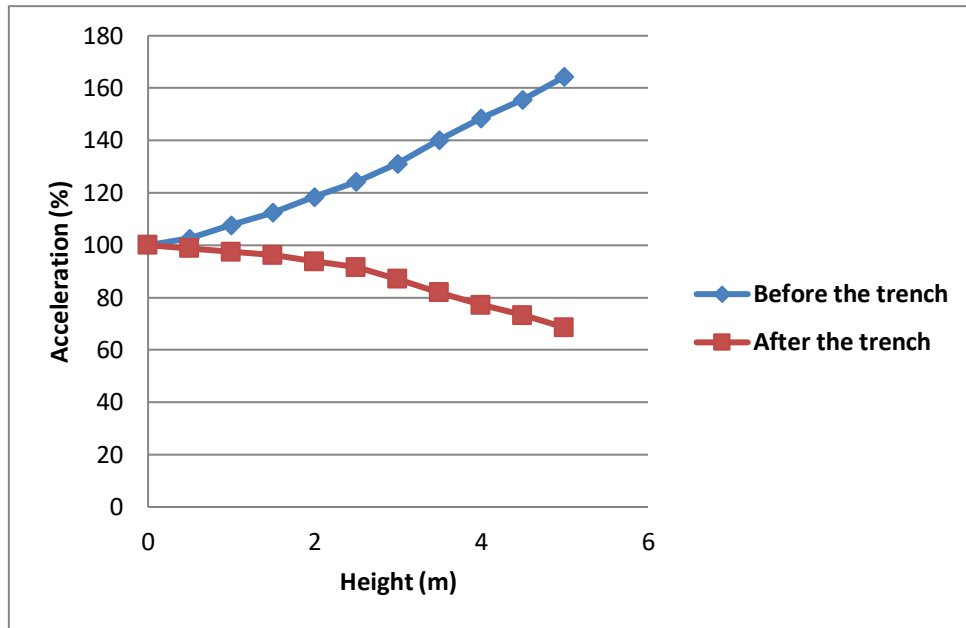
FigureV.4 *Variation of acceleration before and after the open trench with the Height.*

- **Observation**

After we did a variation of 3 different parameters (width, depth, height).We went from 0.769m/s² to 0.444m/s² with an isolation of 0.325 m/s² ,we can say that the barrier isolate 42% of the vibration induced by the shaking table. In addition the reflected waves had gone to 2.151m/s² (65,142%).

We conclude that the best dimensions of the open trench barrier to isolate the shaking table are 5m of width, 10m of depth and 5m for the height.

V.1.2. Model II : In-filled trenches (Concrete)
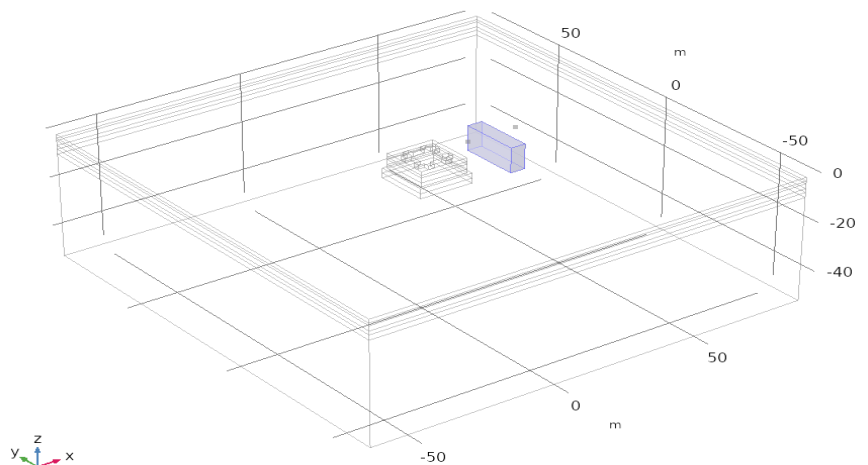
FigureV.5*Conception of the concrete in-filled trench.*

"FigureV.5" represent the 3D view of the second model which is the conception of the concrete in-filled trench with two points to evaluate the vibrations before and after the barrier.

For the second model a parametric sweep analysis is launched to vary the dimensions of the trench including width, depth and height of the trench, furthermore evaluating vibrations before (vibration induced by the mass reaction with the reflected waves by the barrier) and after the trench (the propagated harmless waves).

For the in-filled trench "Table II.1" shows the characteristics of the concrete used in this trench,results are mentioned in the following tables:

- **Variation of the width**

Start = 0.5 m

Step = 0.5 m

End = 5 m

Depth = 10 m

Height = 5 m

Table V.4 *Variation of acceleration before and after the concrete in-filled trench with the width.*

W	D	H	Acceleration m/s²			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
0,5	10	5	4,400	133,253	0,663	86,216
1	10	5	4,693	142,126	0,627	81,534
1,5	10	5	4,857	147,093	0,604	78,544
2	10	5	4,939	149,576	0,586	76,203
2,5	10	5	4,976	150,697	0,569	73,992
3	10	5	5,059	153,210	0,555	72,172
3,5	10	5	5,058	153,180	0,542	70,481
4	10	5	5,093	154,240	0,529	68,791
4,5	10	5	5,121	155,088	0,517	67,230
5	10	5	5,126	155,239	0,506	65,800

FigureV.6 *Variation of acceleration before and after the concrete in-filled trench with the width.*

- **Variation of the Depth**

Start = 1 m

Step = 1 m

End = 10 m

Width = 2 m

Height = 5 m

Table V.5 *Variation of acceleration before and after the concrete in-filled trench with the Depth.*

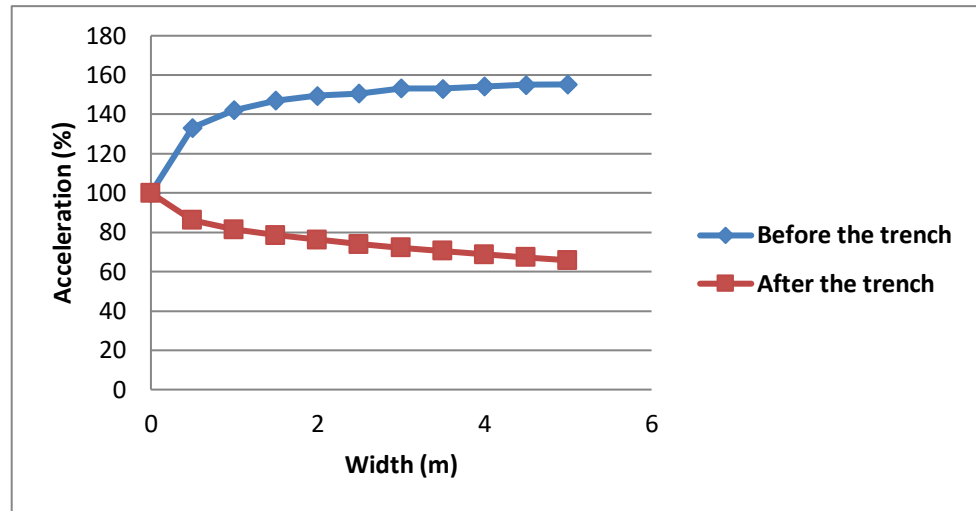| D	W	H	Acceleration m/s²			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
1	2	5	3,336	101,030	0,769	100,000
2	2	5	3,381	102,392	0,763	99,220
3	2	5	3,454	104,603	0,752	97,789
4	2	5	3,627	109,843	0,73	94,928
5	2	5	3,877	117,414	0,697	90,637
6	2	5	4,205	127,347	0,667	86,736
7	2	5	4,549	137,765	0,634	82,445
8	2	5	4,748	143,792	0,605	78,674
9	2	5	4,929	149,273	0,584	75,943
10	2	5	5,089	154,119	0,569	73,992
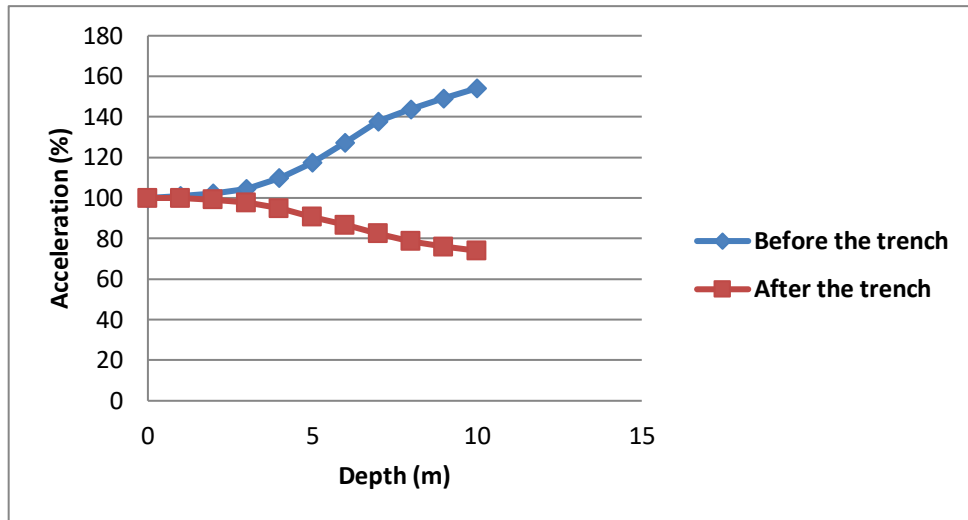
FigureV.7 *Variation of acceleration before and after the concrete in-filled trench with the Depth.*

- **Variation of the Height**

Start = 0.5 m

Step = 0.5 m

End = 5 m

Width = 2 m

Depth = 10 m

Table V.6 *Variation of acceleration before and after the concrete in-filled trench with the Height.*

H	D	W	Acceleration m/s^2			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
0,5	10	2	3,371	102,090	0,762	99,090
1	10	2	3,488	105,633	0,754	98,049
1,5	10	2	3,611	109,358	0,748	97,269
2	10	2	3,755	113,719	0,735	95,579
2,5	10	2	3,901	118,141	0,721	93,758
3	10	2	4,086	123,743	0,693	90,117
3,5	10	2	4,327	131,042	0,662	86,086
4	10	2	4,543	137,583	0,634	82,445
4,5	10	2	4,727	143,156	0,611	79,454
5	10	2	4,939	149,576	0,586	76,203

FigureV.8 *Variation of acceleration before and after the concrete in-filled trench with the Height.*

- **Observation**

After the procedure of variation for 3 different parameters (width, depth, height).We went from 0.769m/s² to 0.506m/s² with an isolation of 0.263 m/s² ,we can say that the barrier isolate 34.2% of the vibration induced by the shaking table. In addition the reflected waves had gone to 1.824m/s² (55,239%).

We conclude that the best dimensions of the concrete in-filled trench barrier to isolate the shaking table are 5m of width, 10m of depth and 5m for the height.

V.1.3. Model III: In-filled trenches (Water)

For the third model a parametric sweep analysis is launched to vary the dimensions of the trench including width, depth and height of the trench, furthermore evaluating vibrations before (vibration induced by the mass reaction with the reflected waves by the barrier) and after the trench (the propagated harmless waves).

For the water in-filled trench we changed the material of the last model with water as shown in "FigureV.5",results are mentioned in the following tables:

- **Variation of the width**

Start = 0.5 m

Step = 0.5 m

End = 5 m

Depth = 10 m

Height = 5 m

Table V.7 *Variation of acceleration before and after the water in-filled trench with the width.*

W	D	H	Acceleration m/s²			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
0,5	10	5	3,665	110,993	0,712	92,588
1	10	5	3,659	110,812	0,708	92,068
1,5	10	5	3,663	110,933	0,711	92,458
2	10	5	3,684	111,569	0,72	93,628
2,5	10	5	3,67	111,145	0,731	95,059
3	10	5	3,723	112,750	0,746	97,009
3,5	10	5	3,696	111,932	0,766	99,610
4	10	5	3,713	112,447	0,793	103,121
4,5	10	5	3,721	112,689	0,823	107,022
5	10	5	3,723	112,750	0,866	112,614

FigureV.9 *Variation of acceleration before and after the water in-filled trench with the width.*

- **Variation of the Depth**

Start = 1 m

Step = 1 m

End = 10 m

Width = 2 m

Height = 5 m

Table V.8 *Variation of acceleration before and after the water in-filled trench with the Depth.*

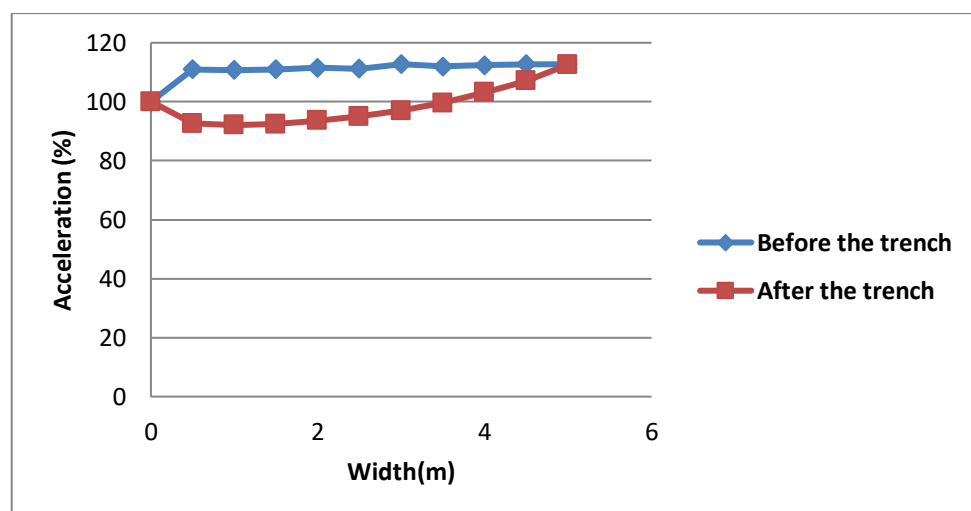| D	W	H	Acceleration m/s^2			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
1	2	5	3,325	100,697	0,769	100,000
2	2	5	3,347	101,363	0,767	99,740
3	2	5	3,365	101,908	0,764	99,350
4	2	5	3,418	103,513	0,757	98,440
5	2	5	3,446	104,361	0,744	96,749
6	2	5	3,518	106,541	0,738	95,969
7	2	5	3,597	108,934	0,729	94,798
8	2	5	3,633	110,024	0,721	93,758
9	2	5	3,697	111,962	0,717	93,238
10	2	5	3,736	113,144	0,713	92,718
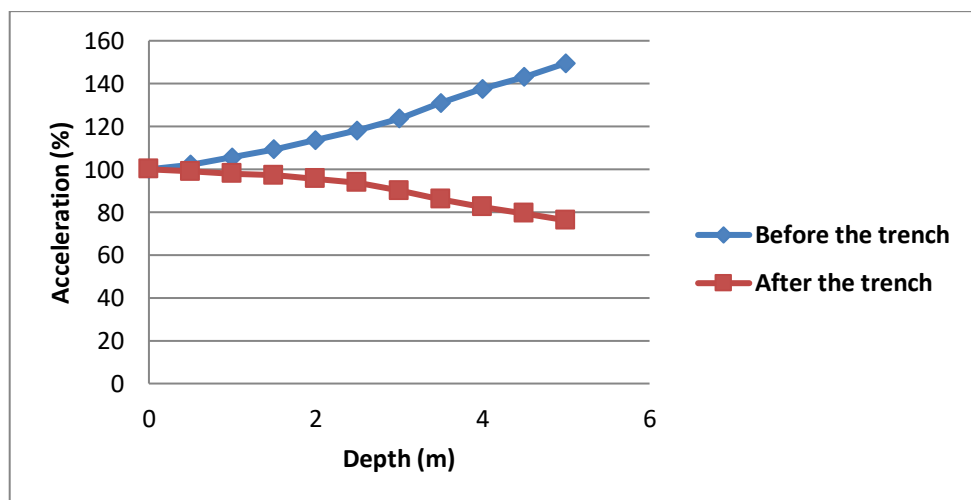
FigureV.10 *Variation of acceleration before and after the water in-filled trench with the Depth.*

- **Variation of the Height**

Start = 0.5 m

Step = 0.5 m

End = 5 m

Width = 2 m

Depth = 10 m

TableV.9 *Variation of acceleration before and after the water in-filled trench with the Height.*

H	D	W	Acceleration m/s^2			
			Before the trench	%	After the trench	%
0	0	0	3,302	100	0,769	100
0,5	10	2	3,371	102,090	0,762	99,090
1	10	2	3,488	105,633	0,754	98,049
1,5	10	2	3,611	109,358	0,748	97,269
2	10	2	3,755	113,719	0,735	95,579
2,5	10	2	3,901	118,141	0,721	93,758
3	10	2	4,086	123,743	0,693	90,117
3,5	10	2	4,327	131,042	0,662	86,086
4	10	2	4,543	137,583	0,634	82,445
4,5	10	2	4,727	143,156	0,611	79,454
5	10	2	4,939	149,576	0,586	76,203

FigureV.11 *Variation of acceleration before and after the water in-filled trench with the Height.*

- **Observation**

After the procedure of variation for 3 different parameters (width, depth, height).We went from 0.769m/s² to 0.72m/s² with an isolation of 0.049m/s² ,we can say that the barrier isolate 6.37% of the vibration induced by the shaking table. In addition the reflected waves had gone to 0.382 m/s² (11,569%).

We conclude that the best dimensions of the water in-filled trench barrier to isolate the shaking table are 2m of width, 10m of depth and 5m for the height.

V.2.Pushover Equivalent Static Analysis(After the isolation)

Furthermore, we should verify these vibrations using the fatigue study with different approaches to see if the need for reinforcement of weak elements exists. We adopt Pushover Equivalent Static Analysis using Sap2000.

As we mentioned perversely the best isolation system is the open trench with 5m of width, 10m of depth and 5m for the height.The following (FigureV.12) represents the acceleration of the reaction mass in relation to time at a distance of 30m after the trench:
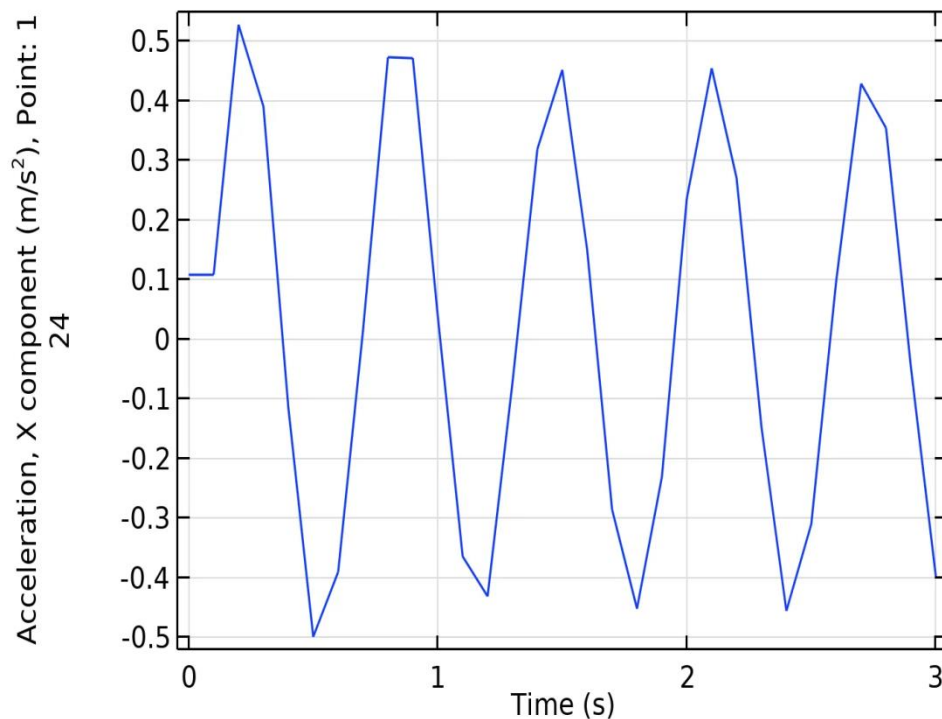
FigureV.12 *The acceleration of the reaction mass in relation to the time(after the isolation process)*

We used the SeismoMatch to converge the accelerogram we had From COMSOL to get the aimed response spectrum since the pushover method in sap2000 needs a specified responsespectrum(acceleration by period).
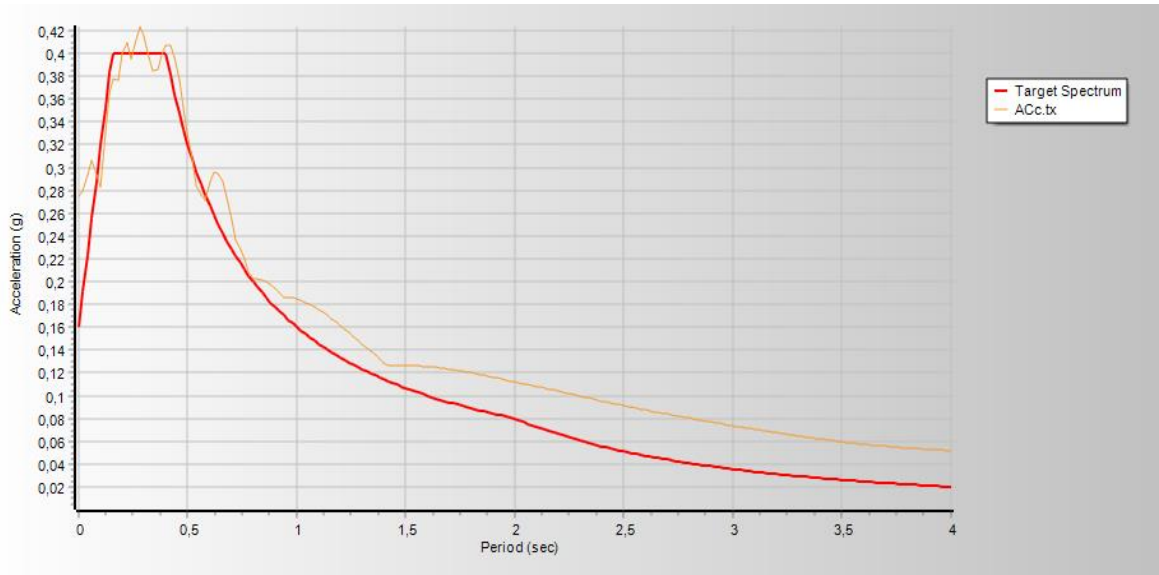
FigureV.13 Evolution of the Matched accelerograms curve (acceleration by period).

V.2.1.Sap2000 Analysis

The result of fatigue study we had using the non-linear approach of the Pushover analysis which can determine whether the building is safe after the isolation.

- **Deformation of a "G+3" building after isolation**
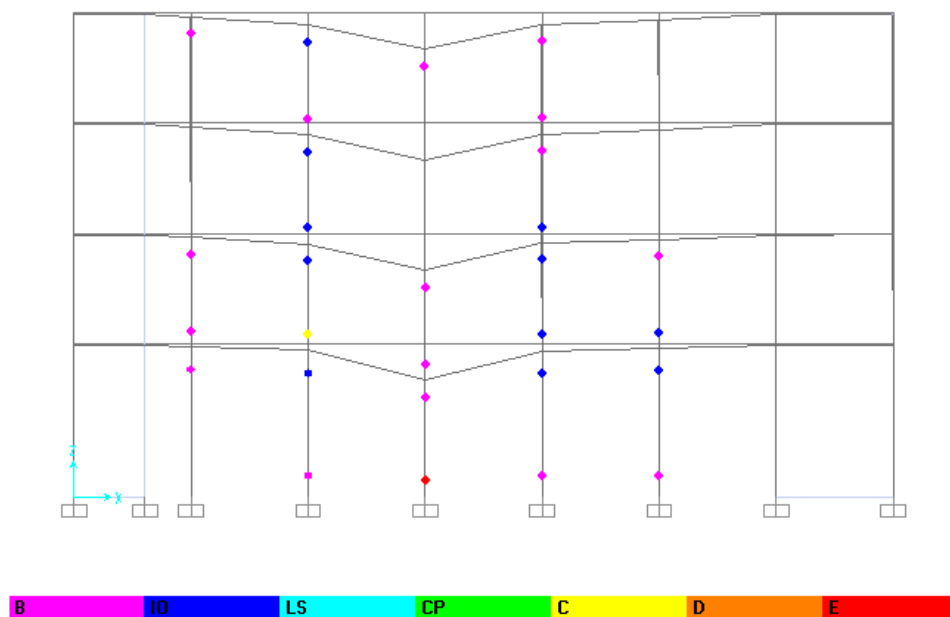
FigureV.14 Result of Pushover analysis (step6, G+3).

- **Observation**

In figureV.14 the deformation showing step 6 for "G+3" structure:

- Step 6 (goes from step 4 to 6)we see that same column pass the limit of resistance and start breaking down (the structure appear to resist the vibration propagated after the trench compared to **FigureIV.6, page 58**)

- **Deformation of a "G+3" building after isolation**
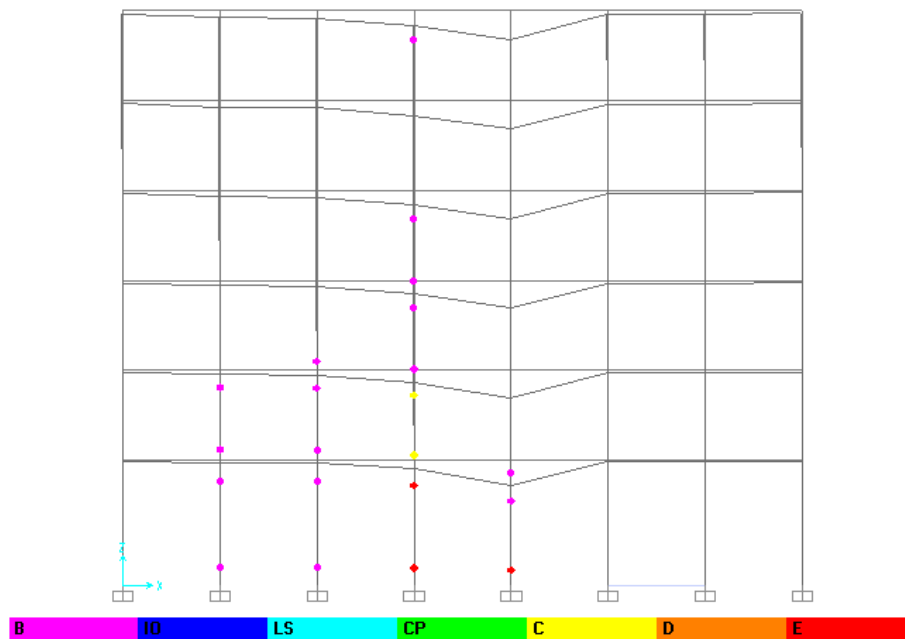
FigureV.15 Result of Pushover analysis (step16, G+5).

- **Observation**

In figure V.15the deformation shown in step 16 for "G+5" structure:

- Step 11 is the first stage of deformation, the same as (**FigureIV.7, page 58**)
- Step 16 (goes from step 7 to 16) we see that same column pass the limit of resistance and start breaking down,(The resistance of the building intensified because of diminished vibration propagated after using the isolation system compared to**FigureIV.9, page 59**)

- **Deformation of a "G+3" building after isolation**
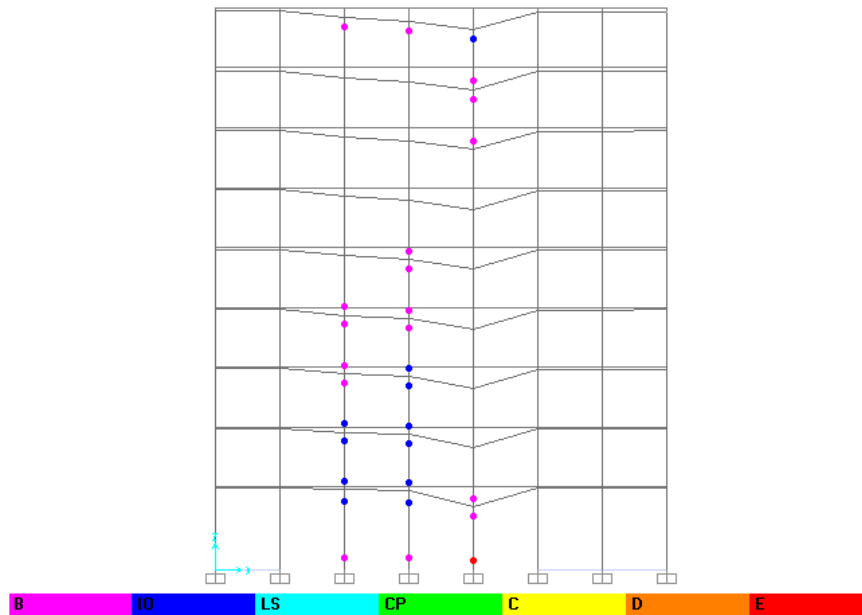
FigureV.16 Result of Pushover analysis (step7, G+8).

- **Observation**

In figure V.16the deformation shown in step 7 for "G+8" structure:

- Step 3 is the first stage of deformation, the same as (**FigureIV.10, page 60**)
- Step 7 (goes from step 4 to 7) we see that same column pass the limit of resistance and start breaking down, (The resistance of the building intensified because of diminished vibration propagated after using the isolation system compared to **FigureIV.12, page 61**)
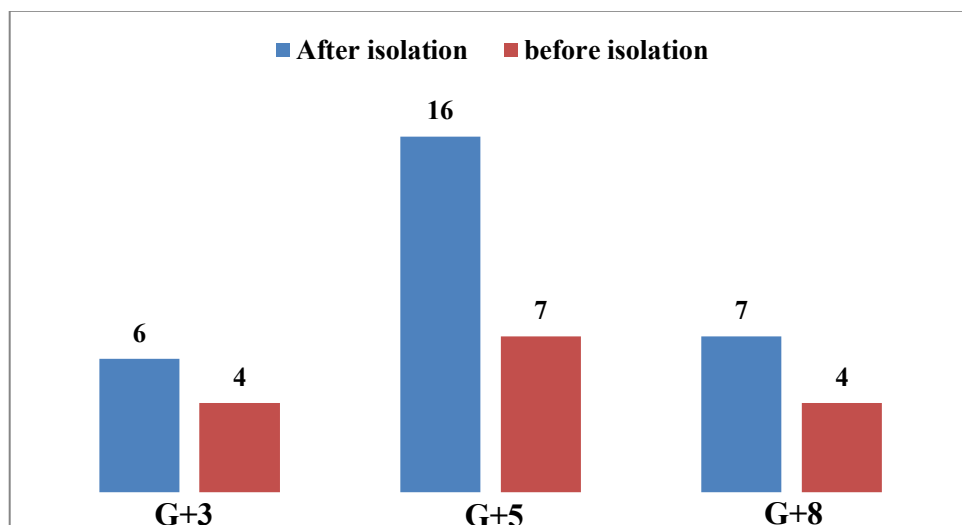
FigureV.17 Comparison between the steps before and after isolating the system.

V.3.Conclusion

After comparing the three models, we noticed that the dimensions have an effect on the vibration's acceleration. Each system has one or two acceleration-influencing parameters:

- In all systems, the height and the width are the most significant parameters.
- In the following isolation systems, width has a significant impact: concrete in-filled trench and open trench. It is rather different in the case of the water in-filled trench (height is the most influential).

In theory, an open trench is the most effective isolation method. However, for practical reasons, it is sometimes difficult or impossible to build and maintain an open trench to a sufficient depth. Stated by Ashwani jain, D.K.Soni.

Accordingly, it is possible to highly minimize ground vibrations by installing an appropriate wave barrier before the building.

After doing the fatigue study approach (Pushover Equivalent Static Analysis) for the propagated vibrations acceleration from the trench (to verify if the vibrations are harmless or the effect on structures is reduced).From the results, we conclude that the open trench isolation system (effective one) performed in improving the resistance of the studied structures to add for the average lifespan while achieving the users comfort.

GENERAL CONCLUSION

In this thesis the reaction mass of the shaking table at the National Centre of Earthquake Engineering in Algiers (CGS) is taken as a vibration sourcewith the purpose of isolating the produced waves from this source. This study necessitates a detailed understanding of the parameters impacting the features of the machine-mass-soil system; the type of modeling used is an essential consideration. Because each sort of modeling has its unique features, this option may be made based on the study's objectives. The modeling of the reaction mass was done in finite elements, which is an appealing option.Using the COMSOL multi-physics simulation program, the mass-foundation system and soil are represented by the same model. This modeling enabled us to accurately replicate the propagation of the translation waves with a simulation of a three structures to study the influence of vibrations on each structure. We found that the response of this modeling approach is dependent on numerous parameters:

- The soil extent must be thoroughly investigated in order to minimize wave refraction issues.
- The mesh must be carefully designed in order to accurately reflect soil heterogeneity and to save analysis time, since a complex mesh might increase computation time (we used a normal mesh).

In addition the fatigue approach that designates the progressive degradation of structures was done in order to evaluate the behavior of the structure under the same source of vibration using bothstatic analysis and a combination of an empirical stress-life model and a damage rule (the Palmgren-Miner rule). The results of this investigation revealed that:

- In special circumstances, the fatigue resistance of buildings must be verified. This verification must be carried out independently for concrete and steel.
- Fatigue verification was carried out for structures and structural components which are subjected to regular load cycles.
- The predicted performance of structural systems by predicting the strength and deformation using static inelastic analysis to find the weak elements of the three structures.

Then parametric research was carried out to assess the dynamic response of the ground surface and the efficacy of the open trench and thein-filled trench isolation systems in screening vibrations caused by the shaking table vibrations on the ground surface. The impacts of different factors such as trench depth, height, width, as well as filling materials, were investigated.The results of this investigation revealed that:

- The usage of open or in-filled trenches, particularly open trench, helps minimize structural vibration.
- The efficacy of vibration isolation rises as the trench's height and width increase.

In conclusion we can isolate the shaking table using the isolation system (open trench) or by reinforcing the weak element we found from the fatigue analysis.

The research has given us the opportunity to gain some experience in this subject and to broaden our ideas, which we have expressed in the form of recommendations:

- Parametric studies for other isolation approaches that can reduce vibrations.
- Consider various modeling approaches, such as the boundary element method or hybrid methods (finite element, boundary element), which can minimize the size of the ground and simplify the process.
- Compare the findings of the previously researched models to the experimental data acquired from reaction mass tests, and then attempt to calibrate the numerical models using these results.
- Fatigue study using different approach (cyclic loads, wöhler test, SIA261, DIN50100etc.) and correlate the results for better evaluation of structure behavior.

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