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**INVESTIGATING THE STRUCTURAL RESPONSE OF MASONRY AND REINFORCED
CONCRETE BUILDINGS IN RELATION TO SOIL PROPERTIES AND LIFE CYCLE COST
ANALYSIS**

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GYŐR, January 2025

Declaration of originality

I, the undersigned **Benjamin Beeior Labar**, a student of Széchenyi István University, hereby declare that the thesis entitled "**INVESTIGATING THE SITE RESPONSE OF MASONRY AND REINFORCED CONCRETE BUILDINGS IN RELATION TO SOIL PROPERTIES AND ENVIRONMENTAL IMPACT ASSESSMENT**" is my original work. Information derived from the published and unpublished work of others has been acknowledged in the text, and a list of references is provided in the bibliography. The results presented in the thesis are based on my original work, calculations, research, and valid measurements, and are credible to the best of my knowledge.

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ABSTRACT

Structures built in seismic excitation zones are prone to earthquake forces, which can quickly cause vibrations that damage structures, leading to cracks and possible collapse. Buildings designed in accordance with the design response spectrum and site-specific response spectra, taking into account soil properties based on ground motion amplification data, are better at withstanding such forces and serving their design purposes. This study aims to investigate the site response of masonry and reinforced concrete buildings, considering the effect of soil properties on ground motion data amplification, and to compare the life cycle assessment of the buildings under consideration based on design and site-specific acceleration response spectrum design. In terms of soil properties and site-specific response spectra, STRATA, which is a 1D ground response analysis equivalent-linear site response program, is used to input motions based on vibratory theories. For structural analysis, the Axis-VM software, which is compatible with finite element analysis, is used for building design and analysis, generating comparative results based on the design and site-specific spectra. It is possible to identify potential casualties in an impending earthquake by using site-specific response spectra, thereby understanding the difference in horizontal deflection at two different instances (for Design and Site-specific spectra applied to the buildings). Following building design and analysis, a life cycle analysis in terms of environmental impact is performed using OpenLCA and IdematLightLCA. This is done to ascertain the additional expenses in terms of eco-costs and Carbon Footprints on some failed elements in the structure which are required to make the buildings more resilient when the site-specific response spectrum is applied and to compare the potential economic losses that may occur based on ecological costs. The findings of this study highlight the significance of soil property-based amplification in ground motion data for reducing potential economic losses and making the buildings more sustainable and better designed to withstand seismic and earthquake forces in suspected zones.

Keywords: Earthquakes, Seismic, Ground motion amplification, Axis-VM, OpenLCA, Buildings, STRATA, Soil properties, Design, and site-specific response spectra.

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CHAPTER ONE

1 INTRODUCTION

The amplification of ground motion from bedrock to the surface is influenced by soil properties (such as soil type and class, the shear wave velocity, and soil damping) can vibrate structures and generate forces such as base shear, moment, and torsion. These forces, resulting from vibratory motion, lead to displacements and rotations at the soil-foundation interface (NEHRP, 2003).

Seismic waves propagating through near-surface soil layers can cause ground vibrations on the soil surface that are substantially bigger and have distinct properties than those recorded at the rock base. Site impacts are the combined effects of earthquakes and local site soil conditions (Matinmanesh, 2011).

These forces enhance the flexibility of structures and prolong their period of vibration. Moreover, these displacements cause energy loss through soil damping (which is a non-linear property of the soil that varies on many parameters such as the cyclic strain amplitude) and radiation damping (which is a measure of the energy lost from the structure due to wave radiation away from the footing), both of which significantly affect the overall damping of the system (NEHRP, 2003).

According to Gervasio 2018, Life Cycle Analysis (LCA) in terms of environmental impact assessments when applied to buildings seeks to evaluate the potential environmental impact of structures throughout their entire life cycle, from materials production to the end-of-life and management of waste disposal. This also includes the assessment of environmental, economic (which can be in terms of ecological costs), and social elements of construction projects.

The built environment contributes significantly to global environmental, economic, and social implications. Huge efforts have been made in recent years to improve the efficiency of energy use in buildings during the use stage due to heating and cooling requirements. The European Union's goal is for all new projects to have almost zero-energy buildings by 2025 (Gervasio, 2018 and IEA, 2023).

The durability and life cycle assessment of structures are determined by factors such as building type, usage, and operation mode. The life cycle cost (LCC) is the total cost of a building or its components throughout its lifespan, including planning, design, purchase, operation, maintenance, and disposal expenses (Al-Hajj, 1999; Krstić et al., 2012). Considering the substantial contribution of buildings to global energy consumption and greenhouse gas emissions (approximately 36% and 39% respectively), it is crucial to promote robust and sustainable construction practices (Hasik et al., 2019).

To achieve sustainable construction (based on minimising environmental impacts), is imperative to account for uncertainties arising from various sources, including weather conditions, design uncertainties, and other random parameters (Dara et al., 2019). Dara et al. (2019) further emphasized the importance of selecting appropriate construction materials to significantly reduce building energy demands and minimize the environmental footprint throughout the life cycle.

In the building sector, there is growing attention towards reducing the environmental impact of products through life cycle analysis, which evaluates their effects from manufacturing to disposal or recycling. Reusing products and selectively deconstructing building components for retrofitting and redesigning purposes are gaining prominence as strategies for reducing life cycle effects (Asmone et al., 2019; Dara et al., 2019).

The assessment of natural hazard impacts, and seismic excitations on the structural integrity of buildings has become a popular approach for investigating resilience and the relationship between structures and the soil. Unlike free-field motion, soil motion includes translational and rotational components, representing

the seismic load on the building's structure. The ratio of foundation/free-field motion in the frequency domain, expressed by a transfer function, reveals the disparities between free-field and foundation input movements since soil is the only relationship that can quickly transfers seismic forces from the bedrock to the surface causing destructions of buildings (Hasik et al., 2019).

1.1 Objective and Scope

The main objective of this study is to comprehensively investigate the performance of two different types of buildings (unreinforced masonry and reinforced concrete structures), considering soil properties based on ground motion data amplification. The study will focus on how these buildings behave under various loading conditions, while also taking into account factors such as their Environmental Impact, and the potential for material use and reuse (e.g., using recycled and environmentally friendly materials like blocks, reinforcements, and concrete etc), and their interaction with the supporting soil. This analysis will be conducted using wave propagation analysis across the soil layers (site-specific spectrum from the soil conditions), which will allow for the propagation of ground motion data. Additionally, the study will examine the linear horizontal deformations, mode shapes in line with the propagation from the soil properties by utilizing finite elements software.

To accomplish these objectives, the research will utilize AxisVM, a robust finite element software and practical modeling tool incorporated with seismic and Earthquake recordings data, which is commonly employed in structural Design. The study will also make use of Strata Software, a one-dimensional response analysis software to assess geotechnical risk, determine stability, and analyze the deformation of the geotechnical engineering and rock mechanics of the soil layers, respectively by feeding it with the necessary soil parameters. By employing these software tools, the study aims to provide a comprehensive understanding of the behavior and performance of the buildings and their interaction with the underlying soil strata. **For the sake of clarity, these are hypothetical buildings with similar properties, such as layout, height, and number of stories, to be able to compare how different materials and technologies might work for the same building.**

The above will be achieved with the following objectives.

1. To design two different building types (unreinforced masonry and reinforced concrete structures) based on Eurocode using AxisVM software.
2. To obtain soil strata effect on ground motion data amplification for evaluating site-specific acceleration response spectrum by using Strata software.
3. Analyze the designed building by using the determined site-specific acceleration response spectrum.
4. To evaluate the behavioural models of the buildings (Masonry and RC) based on the designed and site-specific spectra using AxisVM.
5. To perform LCC analysis in terms of Environmental Impact Assessment (EIA) on the exceeded or failed members at site-specific spectrum design by understanding the impact on the supporting environment.

CHAPTER TWO

2 LITERATURE REVIEW

2.1 Structural Response of Reinforced Concrete and Masonry Buildings to Soil Properties

According to Brzev (2007), in the past centuries, confined masonry construction was a viable building technology that provided an alternative to both unreinforced masonry and reinforced concrete frame construction. The researcher Brzev further stated that confined masonry has features of both these technologies and its construction consists of masonry walls (made either of clay brick or concrete block

units) and horizontal and vertical RC confining members built on all four sides of a masonry wall panel. While Reinforced buildings consist of elements such as reinforced beams and columns, alternative terms horizontal ties and vertical ties could be used instead of tie beams and tie columns.

Civil structures are frequently utilized in areas prone to seismic activity. Nonetheless, due to the diverse range of geometric proportions, sectional shapes, and material properties associated with these structures, accurately assessing their dynamic behavior during earthquakes has proven to be challenging (Pettinga and Priestley, 2005).

The advancements made in the seismic design of reinforced concrete structures can be primarily attributed to research efforts focused on gaining a deeper understanding of the mechanics of concrete behavior. This improved understanding has led to the development of more refined and accurate design methods for both structural and non-structural elements. As a result of these advancements, some code requirements have been established to the extent that newly designed buildings can be deemed to have shown life safety performance levels in an impending earthquake (Pettinga, 2005 & Kegyes-Brassai, 2015).

Even under low levels of load, masonry walls exhibit a highly nonlinear response primarily due to their low tensile strength and the presence of weak head joints (Stavroulaki, 2013 and Senthil, 2023). While material strength plays a relatively smaller role, the response of the wall under blast loading is significantly influenced by boundary conditions and wall thickness.

When subjected to high-velocity and stress impact, it has been observed that the failure patterns of walls transition from global to local failure. This change occurs because, at high velocities or stresses, the contact time between the projectile and the target is minimal. As a result, stress waves generated by the impact do not propagate far from the point of impact, and the displacement of the target is also minimal (Tsompanakis, 2018, Stavroulaki, 2013 and Senthil, 2023).

Comparative Element	Confined masonry construction	RC frame construction
Gravity and lateral load-resisting system	Masonry walls are the main load-bearing elements and are expected to resist both gravity and lateral loads. Confining elements (tie beams and tie columns) are significantly smaller in size than RC beams and columns and cannot withstand heavy seismic vibration.	RC frames resist both gravity and lateral loads through their relatively large beams, columns, and their connections which have high resistance to earthquakes. Masonry infills are not load-bearing walls.
Foundation construction and rehabilitation.	Strip footing beneath the wall and the RC plinth band which are sometimes difficult to retrofit after earthquakes.	Isolated footing beneath each column is generally easy to retrofit after an earthquake.
Superstructure performance.	1) Masonry walls are prone to quick vertical and horizontal cracks after earthquake activities and subsequent collapse. 2) They are difficult to renovate after seismic actions.	1. The frame is constructed first and generally has a high ability to resist earthquake shaking. 2. Masonry walls are constructed at a later stage and are not bonded to the frame members; these walls are nonstructural, that is, non-load bearing walls which in most cases crack and can be easily renovated after earthquake activities.

Table 1 - Comparison of Performance between the confined masonry and RC frame construction in seismic prompt areas (Brzev, 2007).

2.2 Dynamic Amplification of Ground Motion Forces

The primary goal of earthquake risk analysis studies is to forecast the parameters needed for the design of earthquake-resistant structures. Among these parameters, the most crucial one is the lateral peak ground acceleration (PGA), which has an impact on structures during potential earthquakes. PGA is influenced by soil dynamic amplifications, representing the ratio of earthquake acceleration reaching the ground surface as it transitions from bedrock to the surface. This strong-motion station includes peak horizontal acceleration values (PGA) at the bedrock and the soil transfer function, representing the essential physical characteristics of the soil layers up to the bedrock, as outlined by Özkan et al. (2015).

Studies by Pettinga and Priestley, 2005 reviewed that force-based on dynamic amplification due to soil properties has been a fundamental approach in the seismic design process. It involves applying a set of external forces (Horizontal push and pull forces, shear forces, etc.) to the structure, which are considered equivalent to the inertial forces caused by ground accelerations. This design method primarily focuses on the first mode response of the structure, assuming that most of the structural behavior can be captured by this mode. However, in cases where certain limitations in structural behavior are not met, requirements for multi-modal analysis are imposed. This means that additional modes of vibration are considered to ensure a more comprehensive evaluation of the structure's response to seismic forces.

In the context of seismic risk assessment, Gatti (2020) and Bektaş (2022) highlighted the importance of examining the relationship between the height of the building, dynamic amplification, and building damage. To achieve this, it is necessary to establish a geometric correlation between the elastic period of vibration (T) and the height (H) or the number of stories above ground level. This correlation is derived from the behavior of the dynamic amplification factor (DAF), which is a function of the earthquake period (T) and determines the extent of resonance, frequency, or significant dynamic amplification.

The estimated fundamental structural period (T_a) of Reinforced and unreinforced masonry (URM) buildings is as shown in Equation (2.2) below.

$$T_a = C_t * h^{3/4} \quad 2.2$$

where, C_t is the coefficient of the building period and h is the height of the building.

2.3 Impact of Building Height on the Seismic Performance of Masonry and RC Buildings

The seismic performance and fragility analysis of buildings can be significantly affected by the characteristics of earthquakes (such as Magnitude, Depth, and Distance from the Epicenter, Ground Motion Characteristics, Soil Conditions and Duration of vibration). Among these factors, building height plays a crucial role. Alothman, 2023 noted that tall buildings, due to their longer natural periods and greater flexibility, are generally assumed to exhibit better seismic performance. Conversely, research by Alothman, 2023 and Pettinga, 2005 has emphasized that as the height of a building decreases, its seismic vulnerability tends to increase.

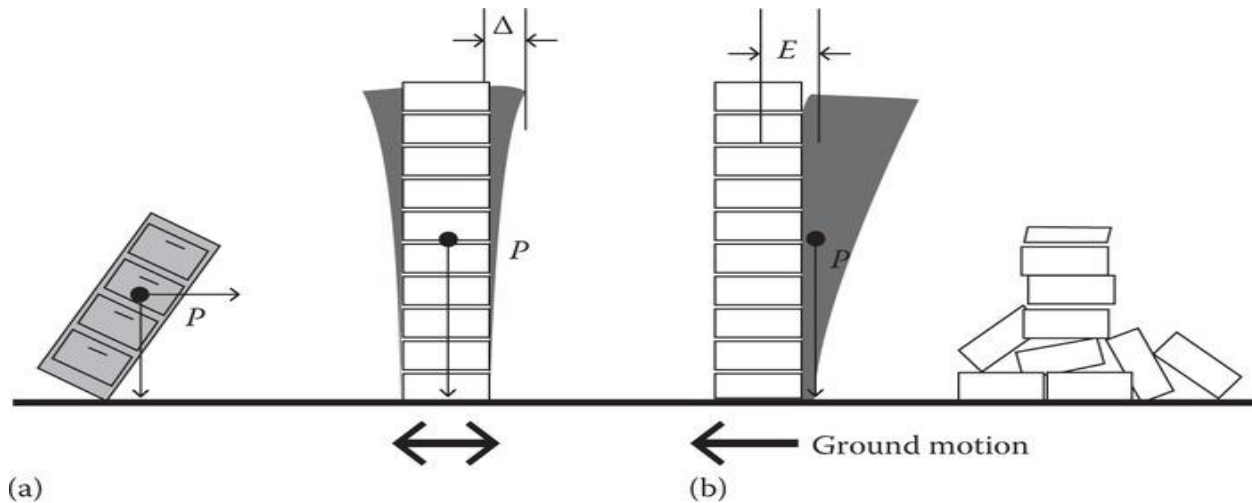


Figure 1 - Typical Displacement profile of tall buildings (Source: Taranath, 2016).

2.3.1 Dynamic Response of Masonry and Reinforced Concrete Structures

It is important to acknowledge that all structures experience gravity loads, such as self-weight (dead load) and occupancy load (live load), in addition to dynamic base motions. According to Anderson (2001), the responses to static and dynamic loadings in structures can be analyzed separately and then combined to determine the overall structural response. When subjected to strong earthquake motions, the structure is likely to exhibit nonlinear behavior, which can be attributed to either material nonlinearity or geometric nonlinearity. Material nonlinearity arises when stresses in certain critical regions of the structure surpass the elastic limit of the material which can result in failure or excessive deformations in the structure.

The number of dynamic degrees of freedom refers to the number of displacement components needed to define the positions of the mass points within the structure. The required number of degrees of freedom for an accurate solution depends on the complexity of the structural system. In some cases, a single degree of freedom may be sufficient (such as a vertical cantilever with the mass concentrated at its tip), while more complex structures may necessitate several hundred degrees of freedom such as high-rise buildings (Pan 2019, Lin 2018 and Anderson 2001).

Other studies by Chang, 2018 and Pan et.al., 2019 support the notion that the behavior of structures under dynamic loads is influenced by various factors. For instance, Chang and Lin (2018) examined the effect of structural damping on the dynamic response of buildings and found that damping can significantly reduce structural displacement and acceleration during seismic events. Additionally, Pan et.al., (2019) investigated the seismic response of steel structures with different configurations and concluded that the structural response is greatly influenced by factors such as stiffness, strength, and ductility.

2.3.2 Response to Dynamic Loadings and Degree of Freedom

Anderson (2001) examined the response characteristics of structures under two fundamental dynamic loadings, namely harmonic loading and impulse loading, valuable insights into their behavior can be obtained. Anderson (2001) further highlighted that harmonic loading involves representing the acceleration time history of an earthquake as a series of sinusoidal waves with a specified amplitude. These waves are intended to capture the accelerations induced by a remote and significant earthquake, with the understanding that the random waves generated at the seismic source have been modified by the soil conditions along their travel path.

Harmonic loading serves as a useful approximation for simulating the effects of earthquake motions on structures. It allows for the consideration of the filtered seismic waves resulting from the interaction between the earthquake source and the characteristics of the underlying soil. Through this approach, the response of structures to a realistic representation of earthquake or seismic accelerations can be studied, providing valuable information for assessing their performance and ensuring their resilience (Anderson 2001, Pan 2019, Lin 2018 and Smith 2016).

In support of this, studies in earthquake engineering have explored the dynamic response of structures under harmonic loading as submitted by Smith et al. (2016) investigated the behavior of high-rise buildings subjected to harmonic ground motions and examined their dynamic characteristics and structural performance. They found that the amplitude and frequency of the harmonic waves significantly influenced the response of the structures. Moreover, Li et al. (2019) conducted research on the seismic performance of bridges under harmonic loading and highlighted the importance of considering the frequency content and duration of the harmonic waves for accurate assessment.

2.3.3 Effect of building shape on the Response to Seismic Forces

According to Mohod (2015), the response of structures to seismic forces is influenced by factors such as plan irregularity, loads, and lateral displacements. Complex-shaped buildings were found to exhibit larger responses in terms of nodal displacement and storey drift under given loading conditions. To mitigate the effects of seismic actions and enhance structural performance during earthquakes, Mohod (2015); and Badry (2015) suggested adopting simpler shapes such as Rectangles, Square, Rounded shapes, etc. shown in figure 2.2.3a for the geometry of structures.

Similarly, Sazzad (2015) and Badry (2015) investigated the impact of building plan shape and size in relation to the analysis of the earthquake motions. Their study involved dynamic analysis to examine the effects, with particular emphasis on the influence of effective eccentricity. Moreover, the research considered the seismic behavior of structures with different plan configurations, categorized as regular, moderately irregular, and strongly irregular, using the response spectrum method as shown the Figure 2.2.3b below.

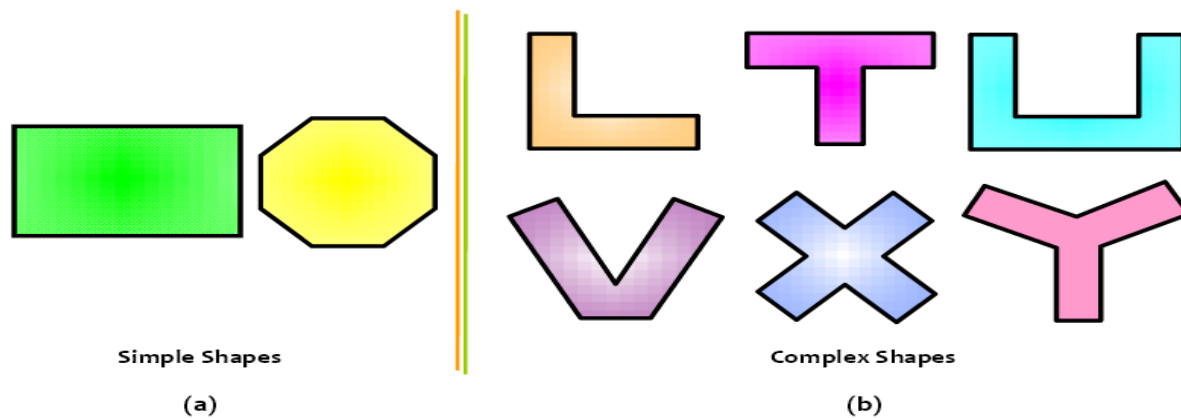


Figure 2 - Structural Plan Configurations recommended by SOAOC. (a) simple shapes undergo simple acceptable structural seismic behavior, while (b) those with complex shapes undergo complex unacceptable structural seismic behavior (Badry, 2015).

These studies collectively emphasize the significance of structure shape and configuration in determining its response to seismic forces. Mohod (2015), Badry (2015) and Sazzad (2015) underline the importance of adopting simpler shapes and reducing plan irregularities to enhance the seismic performance of buildings. By considering the effects of various factors such as loads, lateral displacements, and effective eccentricity, these studies contribute to a better understanding of the relationship between structure shape and seismic

behavior. Their findings can inform design practices aimed at optimizing structural response and improving the overall resilience of buildings in earthquake-prone areas.

Similarly, Kegyes-Brassai 2019 and Badry 2015, highlighted the performance of shape complexity in seismic damage surveys and analyses conducted on modes of failure of building structures during past earthquakes. The research later concluded that buildings that are not symmetrical are more likely to be at risk. Buildings with uneven stiffness and strength distribution tend to twist and move sideways during earthquakes because the force center and mass center don't line up in these structures.

2.4 Degree of Freedoms in Structural Composition

According to Paulay and Priestley (1992), the seismic design of building structures presents a significant challenge, as it involves conceptualizing and detailing a structural system that can withstand lateral ground shaking at a prescribed intensity while minimizing damage and reducing the risk of collapse. An important consideration in achieving this goal is the assessment of the statical degree of freedom (DOF) of the structure, which refers to the number of directions within the structure that allow free movement without any constraints.

Paulay and Priestley, 1992 noted that the higher the number of DOFs, the greater the flexibility of the structure to redistribute and dissipate seismic energy, thereby reducing the potential for damage accumulation and collapse. This check is crucial in enhancing the overall level of lateral movement resistance and ensuring the structural integrity under seismic loading conditions (Paulay and Priestley, 1992).

Liang et al. (2017) investigated the influence of DOF on the seismic performance of high-rise buildings and found that structures with a higher number of DOF exhibited improved energy dissipation and better overall response. Additionally, Wang et al. (2019) conducted research on the optimal distribution of DOF in the design of base-isolated structures and demonstrated that an appropriate arrangement of DOF can enhance the seismic resilience of the system.

2.4.1 Seismic Response to Single Degree of Freedom (SDOF)

Understanding the dynamics of SDOF systems and their equations of motion is crucial in analyzing and predicting the behavior of structures under different loading conditions. By considering the mass, damping, stiffness, and applied force, engineers can effectively assess the response and stability of these systems, ultimately contributing to the design and safety of various structures.

Sisemore and Babuška (2019) emphasize that a single-degree-of-freedom (SDOF) system is characterized by a second-order differential equation governing its motion. Such a system only requires two variables, position, and velocity, to describe its trajectory accurately. Many structures can be simplified and approximated as SDOF systems due to their behavior and response characteristics.

On the same note, Paz (1991) acknowledges that although structures are continuous systems with an infinite number of degrees of freedom, it is possible to reduce this number through idealization and model selection. In some cases, structures can be represented by a discrete number of degrees of freedom, and in simpler cases, they can be approximated as having just a SDOF as shown in Figure 2.3.1.

According to Sisemore (2019) the equation of motion for a one-degree-of-freedom system corresponds to a simple oscillator and can be derived using Newton's law by setting the sum of forces to zero. This equation is a second-order differential equation as shown in equation 2.3.1.

$$m * \ddot{x}(t) + c * \dot{x}(t) + k * x(t) = p(t) \quad (2.3.1)$$

where m represents the mass, c denotes the coefficient of viscous damping, k represents the spring stiffness, and $p(t)$ represents the applied force. The initial conditions of the system are given by:

$$x(0) = x_0; \dot{x}(0) = \dot{x}_0 \quad (2.3.2)$$

Where, $x(0)$ denotes the initial position, and $\dot{x}(0)$ represents the initial velocity of the system. It is important to note that the units on both sides of the equation are in force, with c having units of force per velocity and k having units of force per distance.

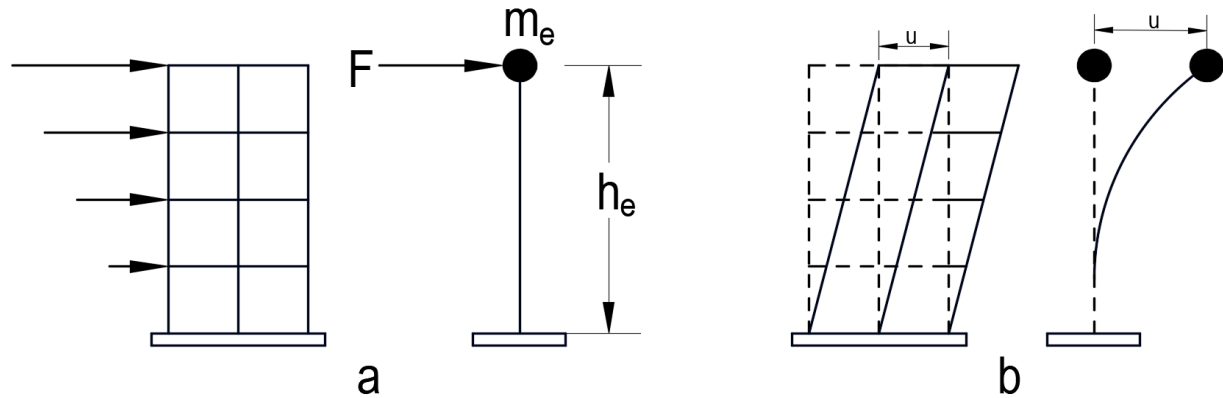


Figure 3 - Seismic Single Degree of freedom. (a) Initial building position, (b) Displaced building position of the mass with respect to its initial position.

2.4.2 Seismic Response to Multiple Degree of Freedom (MDF)

In the context of building structures, simplification techniques are often employed. According to Anderson (1992) and Priestley et al. (2007), it is common practice to assume that the mass of the structure is concentrated or lumped at the center of mass of each individual story level. This approximation allows for a simplified analysis and solution of the system's dynamics as shown the Figure 4 below.

Hence, MDF systems are characterized by the need for two or more coordinates to fully describe their motion. Unlike SDOF systems, which only require two variables, MDF systems necessitate multiple independent displacement coordinates to describe the position of the structure's mass at any given time (Anderson, 1992).

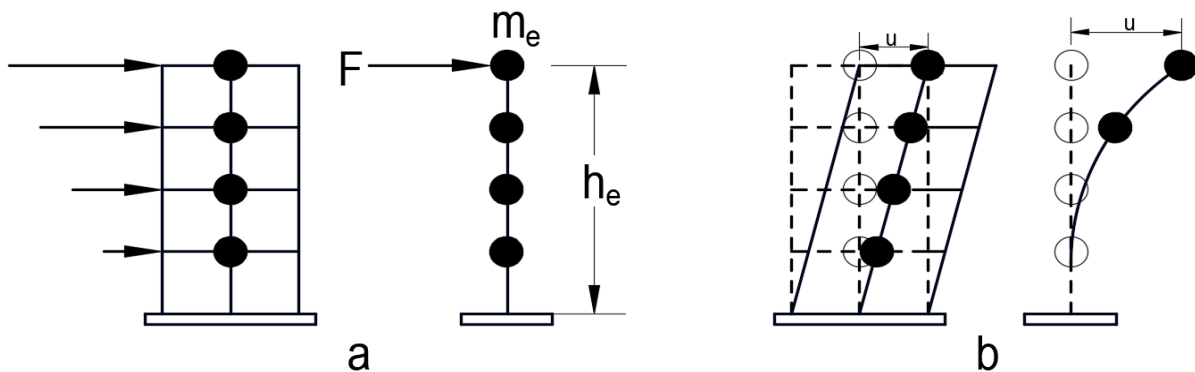


Figure 4 - Seismic Multi Degree of freedoms. (a) Initial building position, (b) Displaced Building position of the mass with respect to its initial position.

2.5 Impact of Soil Interactions on the Dynamic Response of Masonry and Reinforced Concrete Building

Investigations into the soil structure interactions (SSI) effect can offer valuable technical support for improving conventional seismic design practices and methods for urban earthquake damage evaluation.

The evaluation of building structures traditionally assumes that the building's foundation is rigid, disregarding the influence of local soil conditions on its response (Wani et al., 2022). However, the destructive impact of earthquakes has drawn attention to the seismic soil-structure interaction (SSI) in multi-story buildings (Wani et al., 2022).

Interestingly, in evaluating earthquake damage in cities or regions, the SSI effect on structural responses has often been overlooked to enhance computational efficiency (Wang and Yang, 2022). Nevertheless, recent studies by scholars such as Wani (2022), Awlla (2020), and Wang (2022) have emphasized the significance of considering SSI. They have observed that foundations' horizontal movement and rotation can amplify the lateral deformation and story drift of superstructures, leading to adverse conditions for structural seismic vulnerability.

2.5.1 Ground Motion Data

It is important to recognize that seismic waves can undergo amplification when traveling through certain types of soil or unconsolidated sediments. Consequently, an earthquake that may appear relatively harmless when experienced on solid ground can pose a significant threat in areas with soft or unstable soils. This amplification phenomenon can lead to higher levels of ground shaking, and amplifications which in turn can result in severe damage to infrastructure (Bijukchhen et al., 2017; D'Amico, 2018).

The impact of an earthquake on infrastructure is influenced by various factors, including source characteristics, path conditions, and site conditions. While earthquake magnitude is a key parameter, the significance of site conditions should not be overlooked. In fact, the nature of the ground on which a structure is built can greatly influence the extent of earthquake damage it sustains (Bijukchhen et al., 2017; D'Amico, 2018).

2.5.2 Classification of Ground Motion Data

Ground motion data classification is an essential process in earthquake engineering as it allows for the categorization of earthquake records based on their spectral characteristics and other attributes. This classification serves as a valuable tool in the development of ground motion models used for seismic hazard analysis and engineering design. The Building Seismic Safety Council (BSSC) classification is a widely recognized system that classifies ground motion records based on their frequency content and peak acceleration. This classification system, outlined by the Building Seismic Safety Council in 2014, is extensively utilized in the United States for seismic hazard analysis and building design.

In preliminary studies, Sucuoglu (1998) identified several key characteristics for the evaluation of ground motions, including peak ground acceleration, peak ground velocity, strong motion duration, pulses and pulse sequences, energy content, and spectral response forms specific to regions and building types. Similarly, Kamal (2021) investigated the parameters for characterizing strong motion data and categorized them into four major groups: velocity, acceleration, displacement, and frequency.

In line with the aforementioned perspectives, Derakhshani (2019) emphasized that various seismic variables contribute to the determination of strong ground motion parameters. The source, path, and site effects are crucial seismological factors that significantly impact the values of Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), and Peak Ground Displacement (PGD). Each earthquake event's unique source characteristics necessitate consideration of different parameters associated with the source effect.

2.5.3 Peak Ground Acceleration

Peak Ground Acceleration (PGA) represents the maximum acceleration experienced at a particular location during an earthquake. It is a fundamental metric used in earthquake engineering to assess ground shaking intensity. Douglas (2003) defines PGA as the maximum absolute acceleration measured on an accelerogram during a specific earthquake.

Both Douglas (2003) and Houqun (2016) emphasize that earthquake shaking occurs in all three directions, but PGA is often divided into horizontal and vertical components. Generally, horizontal PGAs tend to be larger than vertical PGAs, although there can be exceptions, particularly near powerful earthquakes. PGA is a significant parameter in earthquake engineering and is commonly used as a design basis for earthquake ground motion. It is considered a major intensity measure.

Peak horizontal acceleration is the form of ground acceleration most commonly used in engineering applications. It is frequently represented on seismic hazard maps and plays a crucial role in seismic building codes and earthquake engineering practices (Woessner et al., 2013).

According to Crowley et al. (2020) and Woessner et al. (2013), the extent of damage to buildings and infrastructure during an earthquake is more directly related to ground motion, which PGA measures, rather than solely relying on the earthquake magnitude. In severe earthquakes, the damage is often associated with Peak Ground Velocity (PGV), while PGA serves as a good indicator of damage in moderate earthquakes.

2.5.4 Peak Ground Velocity

Peak Ground Velocity (PGV) is a seismic parameter that quantifies the maximum velocity of ground motion induced by an earthquake. Its application extends to various fields, including engineering seismology and earthquake engineering. Bommer (2008) explains that PGV serves as a basis for assessing non-structural damage, particularly to sensitive systems like piping networks, electrical wiring, and equipment vulnerable to vibrations.

The significance of PGV goes beyond evaluating non-structural damage. Bommer (2008) highlights its role in assessing structure damage and macroseismic intensity in earthquake engineering and seismology. Additionally, due to its relationship with ground stresses, PGV finds application in assessing liquefaction potentials and plays a crucial role in the seismic design and evaluation of buried pipelines.

2.5.5 Peak Ground Displacement

According to Douglas (2018), Peak Ground Displacement (PGD) comprises two distinct components: a transient displacement and a permanent displacement. The transient displacement refers to the immediate displacement during the earthquake, which ideally returns to zero at the end of the recording. On the other hand, permanent displacement arises from long-term movements, such as fault slips or foundation failures, which persist over time.

It is important to note that the magnitudes of permanent displacements near large earthquakes can far exceed those of transient displacements. However, conventional processing techniques that involve filtering may not PGV and PGD for significant earthquakes could be underestimated. This underestimation has the potential to greatly impact research and projects that rely on these parameters for their analyses and assessments (Bommer, 2008).

2.6 Transfer Function

The transfer function, which quantifies the relationship between motion amplitudes of different layers at varying frequencies, is a crucial parameter in seismic analysis. This function is defined as the ratio of motion amplitudes, such as displacement, velocity, or acceleration, between two layers of interest. To compute transfer functions accurately, signal processing techniques are employed, including windowing to extract

the S-wave component of the recorded data, smoothing to reduce scatter, and standard filtering and baseline correction (Mikami, 2008; Kottke, 2019).

As transfer functions involve complex values, they can be expressed using both amplitude and phase. The ratio of Fourier amplitudes of the two records is commonly used to represent the amplitude component. However, in practice, power spectral density functions are often preferred over Fourier amplitudes (Mikami, 2008).

The amplitudes of motions at free surfaces, known as outcrop motions, can be characterized by twice the amplitude of the upward wave (2A), which signifies the transition from an outcrop motion to a within-motion response (Mikami, 2008).

2.7 Acceleration and Displacement Response Spectra

The earthquake ground motion response spectrum, commonly used in seismic analysis, is a representation of the peak responses of SDOF systems across a range of period lengths (Trifunac, 2012).

Trifunac (2012) and Hachem (2014) stressed that the acceleration response spectrum characterizes the relationship between the natural period of vibration of an SDOF system and the maximum absolute acceleration it can experience due to ground motion. Similarly, the displacement response spectrum illustrates the peak displacement, relative to the ground, of various SDOF systems with different periods.

To construct the response spectrum, an analysis involving multiple SDOF systems is conducted. Each point on the spectrum corresponds to the peak response of an SDOF system at a specific period length (Hachem, 2014).

2.8 Standard Shapes for the Linear Response Spectra

The design procedures outlined in Eurocode 8 adopt two standard shapes, Type 1 and Type 2, for the linear response spectra. Type 1 spectra are intended for areas with substantial seismic activity and represent higher energy levels in long-period ground motions. On the other hand, Type 2 spectra are designed for regions with low to moderate seismicity and exhibit larger amplitudes in short-period motions while having less energy in long-period motions compared to Type 1 spectra (Eurocode 8). These response spectra are determined using Equations (1), (2), (3), and (4).

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2,5 - 1) \right] \quad (1)$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \quad (2)$$

$$T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \cdot \frac{T}{T_B} \quad (3)$$

$$T_D \leq T \leq 4(s) : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \cdot \frac{T_C \cdot T_D}{T} \quad (4)$$

The amplitudes of the linear response spectra are determined by the scaling factors S1 and S2, while the shape of the spectra is defined by specific functions within different intervals. These intervals include the

range between 0 and T_B , T_B and T_C , T_C and T_D , and beyond T_D . The specific shape of the response spectra as defined by these functions can be visualized in Figure 1, which represents the generating shape for the linear response spectra in EC8 (Trifunac, 2012).

The use of scaling factors and interval-specific functions allows for a comprehensive representation of the response spectra, capturing both the amplitudes and shape characteristics.

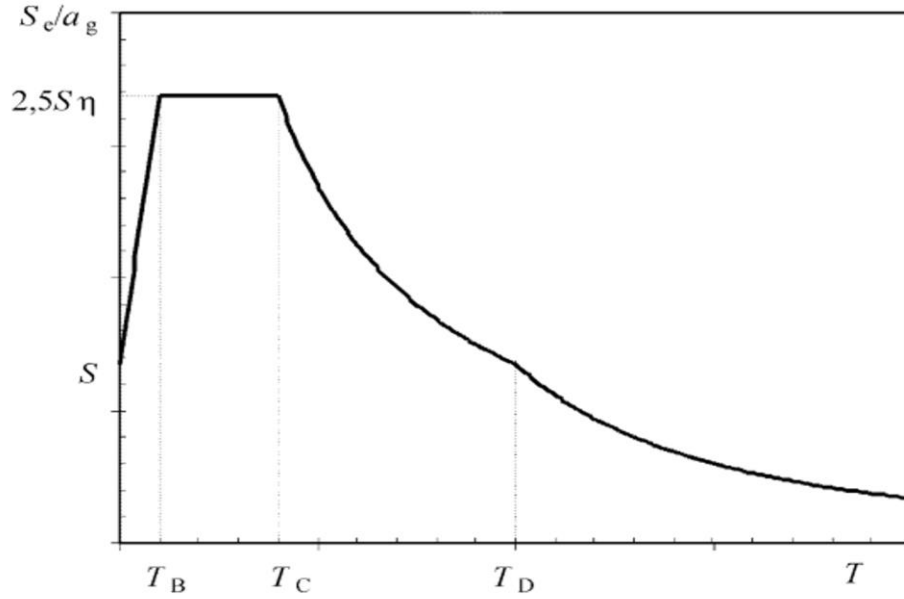


Figure 5 - The shape of the elastic response spectrum specified in (Eurocode 8).

2.9 Ground Motion Scaling

Ground Motion Scaling is an essential process in seismology that aims to forecast the ground motion parameters, such as peak ground acceleration (PGA), peak ground velocity (PGV), and peak ground displacement (PGD), expected at a specific location during an earthquake. The relationship between earthquake magnitude and ground motion parameters is a fundamental concept in earthquake engineering, and it has received extensive attention in research (Boore and Atkinson, 2008).

In this study, the magnitude scaling methodology implemented in the REXEL software package (Iervolino et al., 2010) is utilized to determine the base motions, which are subsequently compared to the criteria outlined in Eurocode 8.

Among the various scaling relationships proposed, the Gutenberg-Richter relationship stands out as one of the earliest and most widely used. It associates the frequency of earthquakes with their respective magnitudes. Another significant scaling relationship is the empirical ground motion scaling law introduced by Boore and Atkinson (2008), which relates peak ground acceleration to earthquake magnitude and distance. According to Boore and Atkinson (2008), this empirical scaling law can be expressed as:

$$\log(A) = b_0 + b_1 M - b_2 \log(r) - b_3 R + e \quad (5)$$

Here, A represents the peak ground acceleration, M denotes the earthquake magnitude, R signifies the distance from the earthquake source to the site, and b_0 , b_1 , b_2 , and b_3 are empirical coefficients. The residual term, e , accounts for the variability in ground motion characteristics that cannot be explained solely by the scaling relationship.

Numerous other scaling relationships have been proposed, each with its own strengths and limitations. However, due to its simplicity and accuracy in predicting ground motion characteristics, the empirical scaling law proposed by Boore and Atkinson (2008) has gained widespread acceptance within the earthquake engineering community. It serves as a valuable tool for assessing and designing structures to withstand seismic events.

2.10 Damping in Seismic Motion

Damping, a phenomenon widely observed in solid materials and structures, refers to the gradual dissipation of vibratory energy over time and space. It occurs when various forms of friction impede motion and disperse energy, similar to how air absorbs sound waves (Kim et al., 2004).

Furthermore, as highlighted by Kim et al. (2004) and Belash (2020), damping plays a crucial role in determining the system's energy retention and energy loss. It significantly influences the dynamic response and transmissibility of the system, ultimately shaping its overall behavior. The level of damping directly impacts the system's ability to absorb and dissipate energy, thereby affecting its stability, response to external forces, and overall performance.

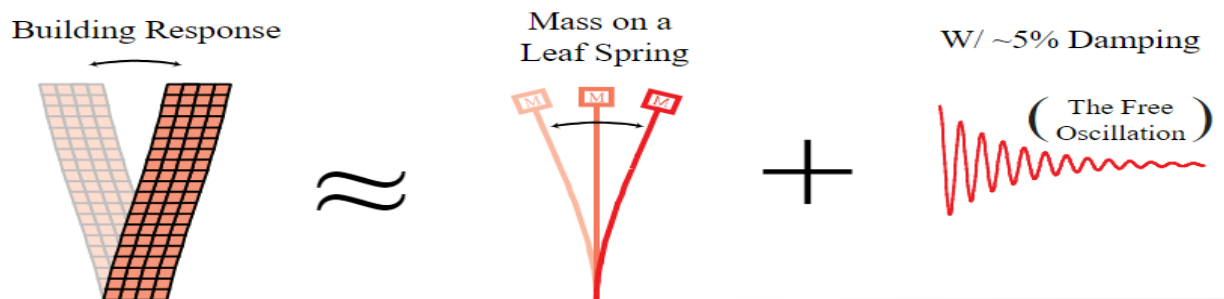


Figure 6 - Response-spectrum Oscillation of Building caused by Damping (Porter, 2016).

2.10.1 Classification of Damping Types

Damping, an essential aspect of structural behavior, can be classified into two broad categories: internal damping and external damping. Internal damping primarily arises from microstructural mechanisms inherent to the material, resulting in energy dissipation. In the case of soils, this can be attributed to factors such as the viscosity of pore fluids, inter-particle sliding, structural rearrangement, and friction. Internal damping, often referred to as "material damping," is an inherent characteristic of the materials themselves (Ashmawy, 1995).

Furthermore, Ashmawy (1995) emphasized that energy losses occurring within a structure or its components due to sources other than internal friction are classified as external damping. Unlike internal damping, external damping, also known as "system damping," does not originate from the material's intrinsic properties. Instead, it arises from various factors external to the material, such as added damping devices or energy dissipation mechanisms incorporated into the structural system.

2.11 Characteristics of Soil Profiles

Geotechnical engineering heavily relies on geological knowledge, as soil characteristics can exhibit significant variations even within small distances. Therefore, a thorough understanding of soil properties at a specific site is essential for geotechnical engineers. Soil originates from the weathering of rocks and is a complex material with inherent variability (Budhu, 2011).

Budhu (2011) emphasizes that due to soil's intricate and variable nature, geotechnical engineers must possess deep insights into soil properties to ensure the success of engineering projects. Failures in

geotechnical systems and increased construction costs can often be attributed to an insufficient understanding of geological formations and groundwater conditions.

According to Eurocode 8 (EN 1998-1:2004 E), the seismic classification of soils primarily relies on the average shear wave velocity of the upper 30 meters of the site, known as V_{s30} . V_{s30} serves as an indicator of soil stiffness under moderate deformations in the upper soil layers. Eurocode 8 establishes a total of five primary soil types and one unique soil type. EC8 defines five distinct ground types to account for local site conditions, as illustrated in Table 1. This classification framework allows for the consideration of site-specific soil behavior in seismic design and analysis.

Ground Type	General description	Average shear wave velocity to 30 m (m/s)
A	Rock or other rock-like geological formations, including at most 5 m of weaker material at the surface.	> 800
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, are characterized by a gradual increase in mechanical properties with depth.	360 - 800
C	Deep deposits of dense or medium-dense sand, gravel, or stiff clay with thickness from several tens to many hundreds of meters.	180 - 360
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $v_s > 800$ m/s.	
S_1	Deposits consisting, of or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index ($PI > 40$) and high-water content.	< 100 (indicative)
S_2	Deposits of liquefiable soils, sensitive clays, or any other soil profile not included in types A – E or S_1	

Table 2 - Soil profile type classification for seismic amplification based on Eurocode 8 (EN 1998-1:2004 E).

2.11.1 Soil Type

Soil exhibits diverse characteristics that can be classified using commonly known terms such as gravel, sands, silts, and clays. These terms correspond to the soil's texture, which provides insights into its visual and tactile properties. Coarse-grained soils, including sand and gravel, consist of larger particles, whereas fine-grained soils, such as clays and silts, comprise smaller particles. The distribution of particle sizes in coarse-grained soils serves as an indicator of their coarseness. Conversely, the mechanical behavior of fine-grained soils is influenced by the dominant mineral types present. Hence, in geotechnical engineering practice, comprehending soil texture and particle distribution plays a vital role in predicting soil behavior under different loads and ensuring the design of stable and secure structures (Budhu, 2011; Thitimakorn, 2012).

2.11.2 Life Cycle Cost Analysis of Seismic Sustainability of Masonry and Reinforced Concrete

Kumar (2018) and AlHamaydeh et.al. (2017) highlight the significance of LCC analysis as a framework for supporting resource allocation decisions throughout the design, construction, and operational phases of structures, including inspection and maintenance. This analysis aids in considering the long-term costs associated with a structure's lifecycle.

However, Asadi (2019) emphasizes that economic assessment should encompass not only the initial structural costs but also the anticipated damage costs over the effective lifetime of the structures. Drawing a comparison between Asadi (2019) and Kumar (2018), it becomes evident that the initial structural costs encompass aspects such as structural materials, construction quality programs, and construction expenses. On the other hand, damage costs primarily consist of non-structural elements, economic losses, human injuries and fatalities, indirect costs, and potential social losses in the event of an earthquake.

To address these concerns, Asadi (2019) proposes a framework for reliability and performance-based design for structures exposed to natural hazards. This framework takes into account the structural performance over the entire lifespan of the structure. In the design process, various uncertainties are considered, including hazard demand, structural capacity, nonlinear structural response behavior, redundancy, cost-benefit balance, and target reliability for single and multiple hazards.

Furthermore, Gencturk et al. (2015) identified and discussed the three key components of sustainability, namely economic, environmental, and social impacts, specifically in the context of reinforced concrete (RC) buildings subjected to earthquakes. Their framework encompassed these impacts throughout the entire lifespan of the buildings. In their comprehensive study, Gencturk et al. (2015) developed a framework that effectively quantified the sustainability of buildings, specifically focusing on the aspects directly associated with seismic hazards. This framework aligns with the findings of Asadi (2019), who similarly emphasized the importance of considering seismic hazards when evaluating sustainability.

The proposed framework by Gencturk et al. (2015) incorporated the Life Cycle Sustainability Assessment (LCSA) approach, which utilized the Performance-Based Earthquake Engineering (PBEE) methodology similar work was also presented by (Taflanidis, 2009). This allowed for the probabilistic evaluation of the structural damage and its subsequent conversion into economic, environmental, and social impacts. The authors also employed a set of algorithms, as illustrated below (Figure 2.11), to enhance the comprehension and application of their method.

Similar to the algorithm depicted in Figure 2.11, as presented in Micheli et al. (2019), the study focused on quantifying the structural response of the building using numerical models. In their research, Micheli et al. (2019) incorporated the concept of Performance-based design (PBD), which involves establishing specific performance levels for the structural system based on different hazard intensities. Micheli et al. (2019), models were utilized to simulate the building's behavior in response to varying wind speeds. By analyzing the expected acceleration costs, the research aimed to provide insights into the economic implications associated with different wind load intensities.

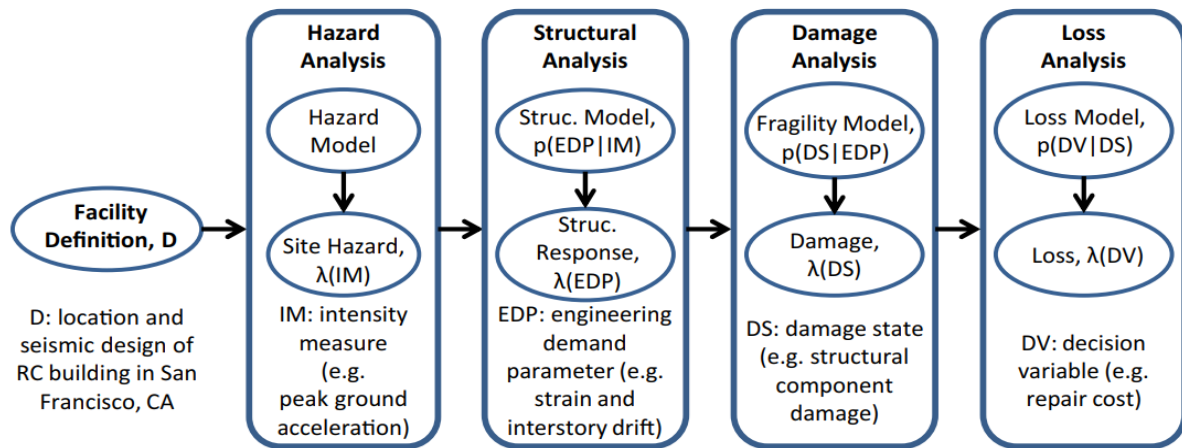


Figure 7 - PBEE methodology adapted by (Gencturk et al. 2015).

Unlike others, Loa et al. (2021) in their study, conducted research utilizing the methodologies developed by the Pacific Earthquake Engineering Research (PEER) and FEMA P58 (2012). With a targeted objective to estimate the potential seismic losses of both existing and retrofitted buildings, with a specific focus on evaluating the life-cycle costs associated with these structures.

To assess the cost-benefit aspect, Loa et al. (2021) established a defined indicator that took into account the rehabilitation costs and expected losses. It is important to note that their analysis assumed that structural aging and deterioration were not considered in the life-cycle cost analysis.

The findings of Padgett et al. (2019) contribute to the existing body of knowledge on cost estimation methods in seismic risk analysis. The incorporation of fragility curves and post-event functionality assessments enhances the accuracy and applicability of cost estimates, enabling stakeholders to make informed decisions regarding risk reduction strategies and resource allocation. The fragility curves established in the research were associated with different damage states, including slight, moderate, extensive, and complete damage. Each damage state was then linked to an anticipated level of post-event functionality, which ranged from temporary restrictions lasting one day to complete closure lasting beyond 30 days.

CHAPTER THREE

3 PRESENTATION OF METHODOLOGIES

Multichannel Analysis of Surface Waves (MASW) is employed in collecting the data from three randomly selected sites in Gyor. These collected data were then analyzed using a computer software STRATA performs equivalent linear site response analysis in the frequency domain and permits the randomization of the site characteristics such as the peak ground acceleration, transfer function, and the local site acceleration spectra which represents the maximum ground acceleration experienced at a spot during the shaking of an earthquake. These parameters are important in ascertaining the behaviors of buildings withing fault zones. Two considerably buildings with different structural systems (Masonry and reinforced Concrete) building are modeled and analyzed using AxisVM finite element software considering the results obtained from the 1-dimensional strata software in evaluating the behavioral characteristics of the buildings such as vulnerability and seismic risks based on the soil conditions. After this a decisional making tool process using Life Cycle Cost Analysis is employed to ascertain the initial and rehabilitation in terms of economic losses site specific spectrum is applied.

3.1 Techniques of multi-channel surface wave seismic Analysis data

The methodology employed in this thesis involved the utilization of techniques for analyzing multi-channel surface wave seismic data. One such technique was the Multichannel Analysis of Surface Waves (MASW), which proved to be a powerful approach for seismic analysis, enabling the determination of the shear wave velocity profile of the subsurface. The MASW method involved measuring surface waves generated by a seismic source and utilizing the dispersion characteristics of these waves to estimate the shear wave velocity structure of the subsurface (Park et.al., 1999), as illustrated in Figure 5.

Several techniques utilized in MASW analysis, as noted by Park et.al. (1999), were as follows:

- a) Data acquisition: The first step in MASW analysis was to acquire seismic data using an appropriate source and receiver configuration. The data was acquired using various sources, including a hammer, a weight drop, and a seismic vibrator.
- b) Data processing: The acquired data was then processed using a range of techniques to remove noise and enhance the signal. The processing steps typically involved filtering, stacking, and Fourier transformation.
- c) Dispersion analysis: The dispersion characteristics of the surface waves were then analyzed to estimate the shear wave velocity structure of the subsurface. This involved the identification of the dispersion curves, which represented the relationship between the wave velocity and frequency.
- d) Inversion: The final step in MASW analysis was the inversion process, which involved the calculation of the shear wave velocity profile of the subsurface. This was typically achieved using an iterative algorithm that compared the observed dispersion curves with the predicted curves based on the assumed velocity model.

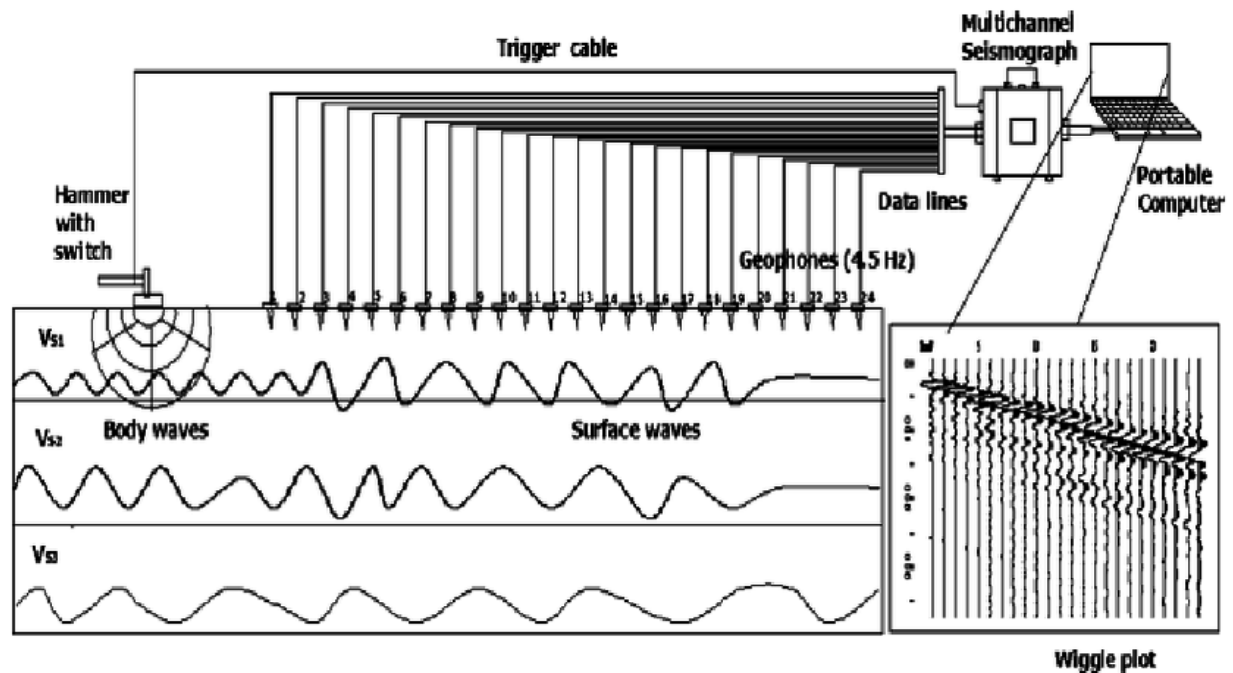


Figure 8 - Typical setup and operating technique for Multichannel analysis of surface waves (MASW) (Source: Jakka, 2014).

3.2 STRATA (Working Principles)

The second methodology employed in this thesis involved the utilization of the computer software STRATA, which operates based on time domain input motions or random vibration theory methodologies. STRATA perform equivalent linear site response analysis in the frequency domain and allows for the randomization of site characteristics. This software computed the dynamic site response of a one-dimensional soil column by employing linear wave propagation and accounting for strain-dependent dynamic soil properties (Kottke, 2019).

According to Kottke (2019), STRATA specifically focus on calculating the response of shear waves that propagate through a site characterized by horizontal layers and exhibit vertical propagation and horizontal polarization. The software utilizes recursive formulas to calculate the amplitudes of the propagated waves. These formulas ensure the compatibility of displacement and shear stress at the boundaries between layers. It should be noted that these formulas are applicable to layers with uniform soil characteristics and wave amplitudes (A and B), as depicted in Figure 4.

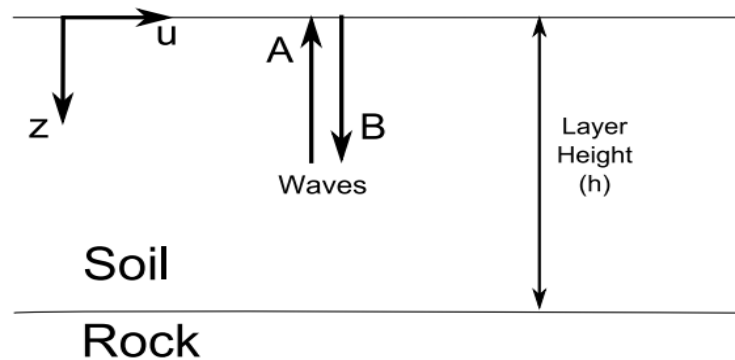


Figure 9 - Illustration of Wave Amplitudes (A and B) with soil Profiles (Kottke, 2019).

In order to compute the response at the surface of a site, STRATA utilize the wave amplitudes (A and B) present within the soil profile at each frequency. This computation relies on the assumption that the stiffness and damping properties within each layer are known (Kottke, 2019). By incorporating this information, the software calculated the response characteristics of the site's surface based on the given wave amplitudes and the properties of each layer.

3.2.1 STRATA Inputs

To define the soil types and profiles within STRATA, several parameters are considered. These include the name of the soil type, unit weight, initial damping, shear modulus reduction model, damping model, and any relevant remarks. The annotations are utilized to maintain clarity and organization within the software, while the name serves as an identifier for each soil type. For soil types that adopt the Darendeli (2001) model for nonlinear properties, additional features required by the empirical model need to be described (Kottke, 2019).

The soil profile is formed by combining the shear-wave velocity profile with different soil types. Determining the thickness, soil type, and shear-wave velocity of each soil layer within the profile is essential. The available list of soil types in the Soil Types tab is used to select the appropriate soil types for each layer (Kottke, 2019).

3.2.2 Defining the output

According to Kottke's (2019) technical manual for STRATA, there are four main forms of output that the software provides:

- a) Response location output: This refers to the response at a specific location, such as the acceleration response spectrum or a time series.
- b) Ratio output: This involves the ratio of a response at two different locations to the response at a given location, such as a transfer function or spectral ratio.
- c) Profile: This represents the variation of a specific property with depth, such as the maximum shear strain. Profiles are computed at different depths as the layering within the model may vary.
- d) Computation: To initiate the computation process, one needs to switch to the Compute tab and click the Compute button. The information related to the computation will be displayed in the window, and once the calculation is complete, the view will transition to the output widget.

3.3 AxisVM Finite Element Software

AxisVM, a finite-element program specifically designed for civil engineers, was utilized in this thesis for the static, vibration, and buckling analysis of structures. This software combines robust analysis capabilities with a user-friendly graphical interface, making it accessible for engineers in the field (User's Manual X5, 2015).

The modeling process within AxisVM involved the use of various tools and functionalities. Geometry tools such as points, lines, and surfaces were employed, along with automatic meshing, material and cross-section libraries, and element and load tools. Additionally, CAD geometry could be imported or exported using formats like DXF, and there was compatibility with architectural design software like Graphisoft's ArchiCAD via IFC, enabling the creation of the model framework directly from those software products (User's Manual X5, 2015).

Throughout the modeling process, graphical verification was provided at each step, ensuring accuracy and progress. The software offered multilevel undo/redo commands and online help for assistance (User's Manual X5, 2015).

AxisVM was capable of analyzing static behavior, vibrations, and buckling phenomena. It presented the results through various visual representations, including deformed and undeformed shape displays, diagrams, iso-line/surface plots, and animations. Additionally, customizable tabular reports could be generated to summarize the analysis outcomes (User's Manual X5, 2015).

Following the analysis, AxisVM offered powerful visualization tools for a comprehensive interpretation of the results. Numerical tools were also available to facilitate result search, reporting, and further calculations based on the obtained data. The software allowed for the display of deformed or animated shapes of the analyzed geometry, as well as the representation of isoline/surface plots. Furthermore, AxisVM enabled the linear combination or enveloping of results, providing additional analytical flexibility (User's Manual X5, 2015).

3.4 EN 15804 Framework Programme on Cost-effective improvement and Environmental Impact Assessments Using OpenLCA and IdematLCA

The methodology employed in this thesis involved utilizing the EN 15804 Framework Programme of "Cost-effective improvement on infrastructure." The framework is to conduct a lifecycle analysis in terms of EIA that goes beyond the initial capital cost of an asset. This comprehensive framework allowed the comparative alternatives of projects and determine the most favorable approach for addressing environmental impact and sustainable developments for carbon footprints and ecological costs.

The framework also enhances decision-makers with valuable insights into the economic and environmental consequences of different intervention options. This method used in this work will enhance the following:

- Enhancement of degradation and structural models to develop more realistic structures through environmental impacts and safety models.
- Exploration of innovative construction methods by improving the sustainable targets of net-zero negative emissions.
- To quantify key properties, when integrated into a building, for functional use, quantified performance characteristics or minimum performance of the construction product, considering the functional equivalent of the building in terms of carbon footprints and ecological costs.

By examining these aspects, the methodology aimed to provide insights into the assessment of carbon footprints and ecological costs of inadequate structural members resulting in the applications of site-specific spectra. Furthermore, it sought to identify sustainable approaches for repairing and enhancing the resilience of buildings, considering both economic considerations (Ecological costs) and environmental consequences (Carbon footprints). Hence this thesis will examine critically carbon footprint and ecological cost using OPENLCA and IdematLCA for the masonry and RC buildings, considering the total amount of greenhouse gases that will be generated in reconstruction of exceeded structural members using various indicators that contributes to the environmental hazards.

CHAPTER FOUR

4 DATA, RESULTS PRESENTATION, AND DISCUSSION

4.1 Determination of Local Spectral Acceleration Response Spectrum Based on Field Test Using Strata Software

This research utilized the Multi-Channel Surface Wave (MASW) method, a widely recognized non-destructive geophysical technique, to investigate the shear-wave velocity profile of the subsurface. The selected sites for this study were carefully chosen based on their specific geological and geophysical characteristics. The obtained data are summarized in Table 3.0

Soil Profile 1			Soil Profile 2			Soil Profile 3		
Dept h (m)	OBI mellet Soil	Vs	Dept h (m)	Keksgyár Soil	Vs	Dept h (m)	Víztükör ltp. Soil	Vs
0.5	Cl_mcalc_m stiff_pl	171.0	0.5	oFSa_calc_pl	165.8	0.5	siFSa_calc_ quartz_soft_ pl	131.4
3.5	clFSa_mcalc_ quartz_loos e_pl	171.0	3.5	mgrMSa_qua rtz_musc_loo se_pl	165.8	3.5	fgrMSa_qua rtz_loose_pl	131.4
7.5	clFSa_mcalc_ quartz_loos e_pl	218.3	6.5	mgrMSa_qua rtz_musc_loo se_pl	169.2	7.5	fgrMSa_qua rtz_loose_pl	194.2
12.5	clFSa_mcalc_ quartz_loos e_pl	324.5	11.5	mgrMSa_qua rtz_musc_loo se_pl	210.4	12.5	siCl_calc_sti ff_pl	266.2
17.5	MSa_mi	264.6	16.5	mgrMSa_qua rtz_musc_loo se_pl	252.5	17.5	siCl_calc_sti ff_pl	333.8
23.5	saCl_calc_m stiff_mi	443.0	22.5	saCl_mi	372.4	23.5	siCl_calc_sti ff_pl	378.5
30.5	saCl_calc_m stiff_mi	426.0	24.5	mgrMSa_qua rtz_musc_loo se_mi	461.3	25.5	siCl_calc_sti ff_pl	365.2
			26.5	fsaCl_mi	420.0	30.5	siCl_calc_sti ff_pl	370.0
			28.5	clFSa_mi	380.0			
			30.5	fsaCl_mi	430.0			

Cl	Si	clSa	siSa	Sa	saGr	grSa	siCl	saCl	stiffCl

Table 3 - Soil Profile Colour code.

The data above was utilized by inputting into the Strata software and the results of Peak ground acceleration, Shear wave velocity, and the Site-specific Spectrum were obtained as shown below.

The analysis of the presented results indicates that site location II (soil profile II) exhibits the highest Peak Ground Acceleration (PGA) and Shear Wave Velocity among the preselected areas. This suggests that this site will experience the strongest ground shaking during an earthquake. The recorded PGA at this site reached 0.32 g, representing the maximum level of ground motion acceleration that can occur during seismic events and will be used in the design of the buildings.

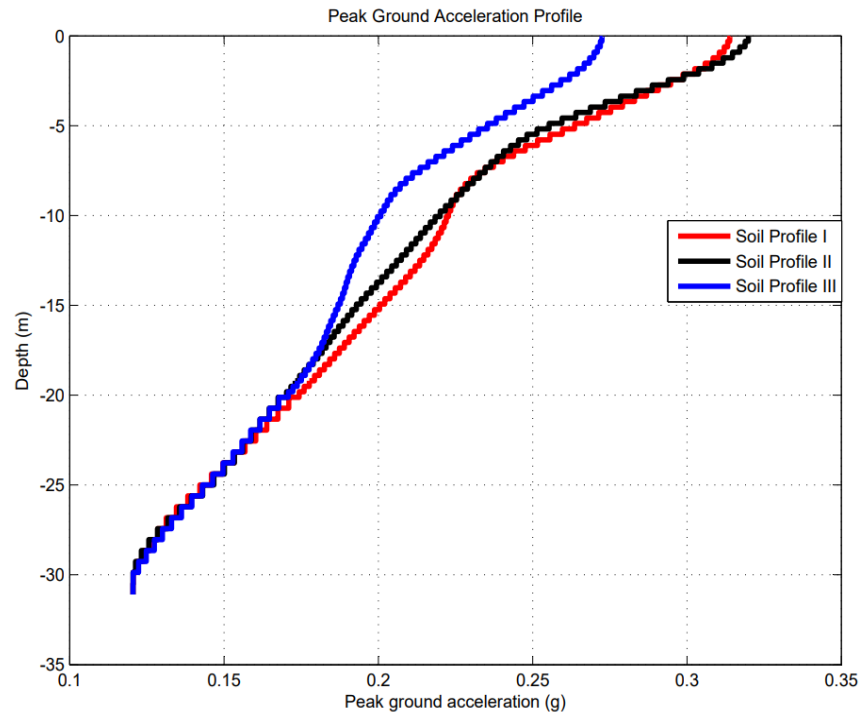


Figure 10 - Peak ground acceleration profile for the three considered sites.

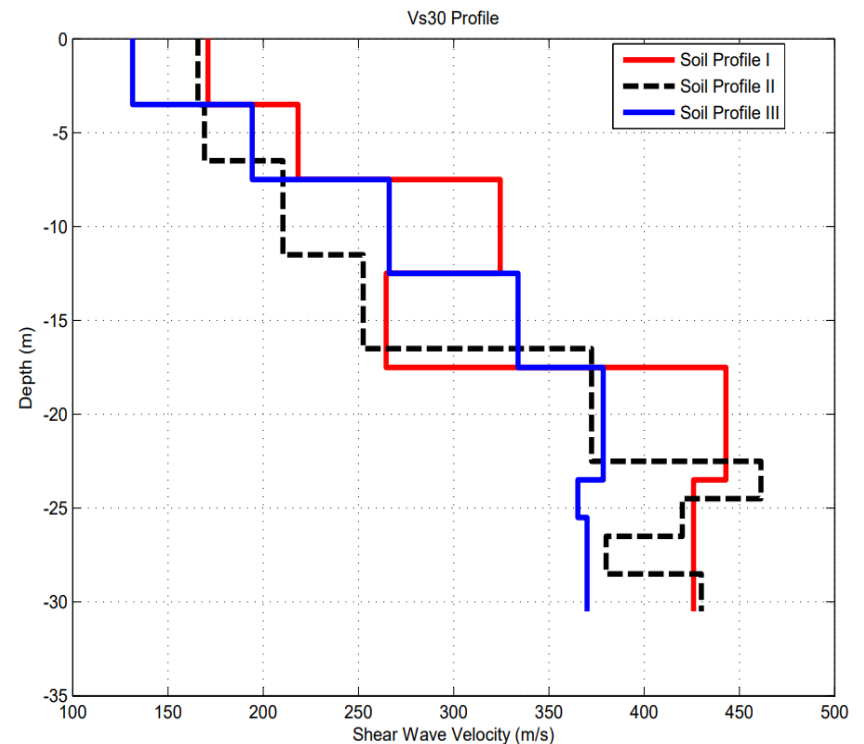


Figure 11 - Shear wave Velocity profile for the three considered sites.

The observed variations in soil profiles at different site locations likely contribute to these differences in ground shaking intensity. The geological and geotechnical characteristics of the subsurface play a significant role in determining the seismic behavior and amplification of ground motions.

4.1 Acceleration Response Spectrum

In order to assess the structural response to ground motion at the selected sites, the acceleration response spectrum was generated for each of the three locations. This involved plotting a graphical representation that illustrates the maximum and minimum structural responses to earthquake ground motion at various frequencies.

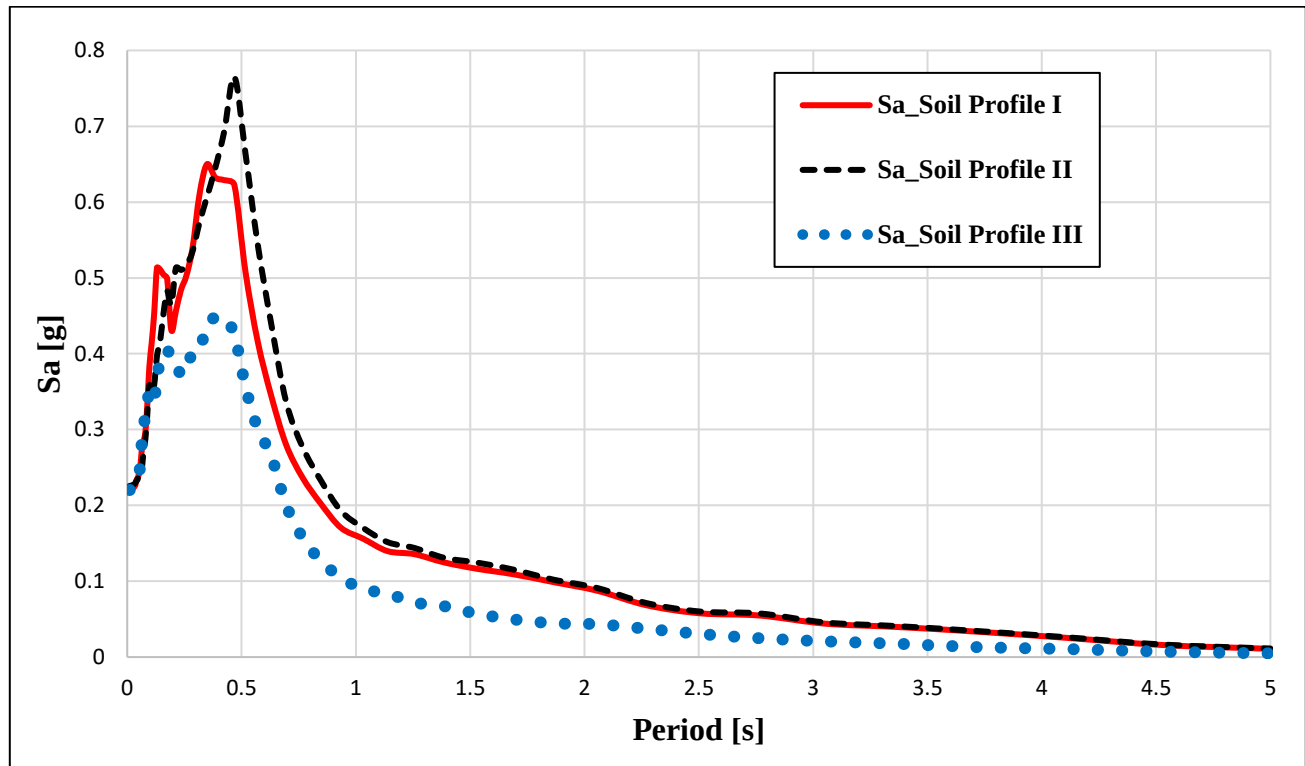


Figure 12 - Acceleration response spectrum profiles for the three considered sites.

The plot above (Figure 12) displays the relationship between the vibration frequency and the corresponding maximum and minimum accelerations experienced from the site data obtained using Strata software. It provides valuable information about the range of accelerations that a structure may encounter over a given period.

To evaluate the structural response against design standards, the resulting plot was compared with the Type 1 and Type 2 acceleration response spectra specified in Eurocode 8. These design spectra define the maximum allowable accelerations that structures should be designed to withstand for a given seismic hazard level for soil type C as shown below in Figure 13 and Figure 14 - Acceleration Time History for the soil profile II.

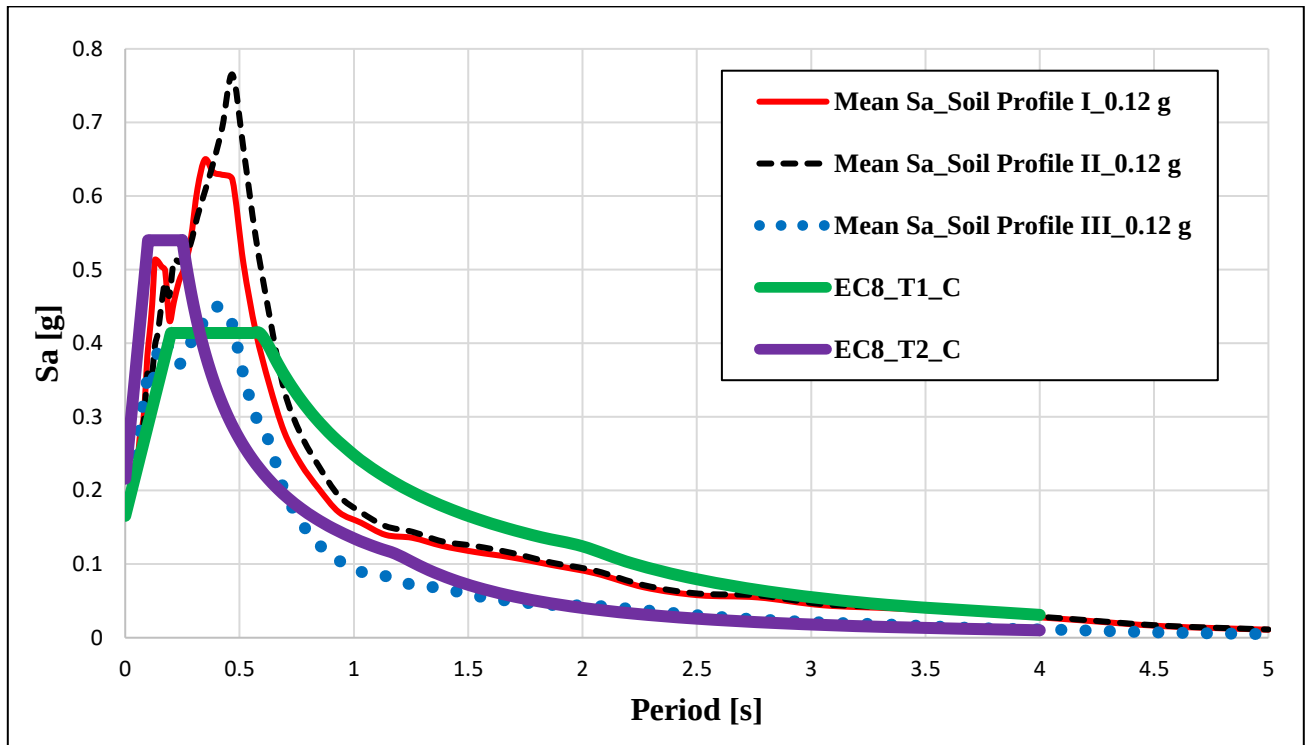


Figure 13 - Compared Type 1 & 2 Elastic Response Spectrum to Specific-site Acceleration Spectrum profiles for the Maximum and Minimum soil Type C profiles.

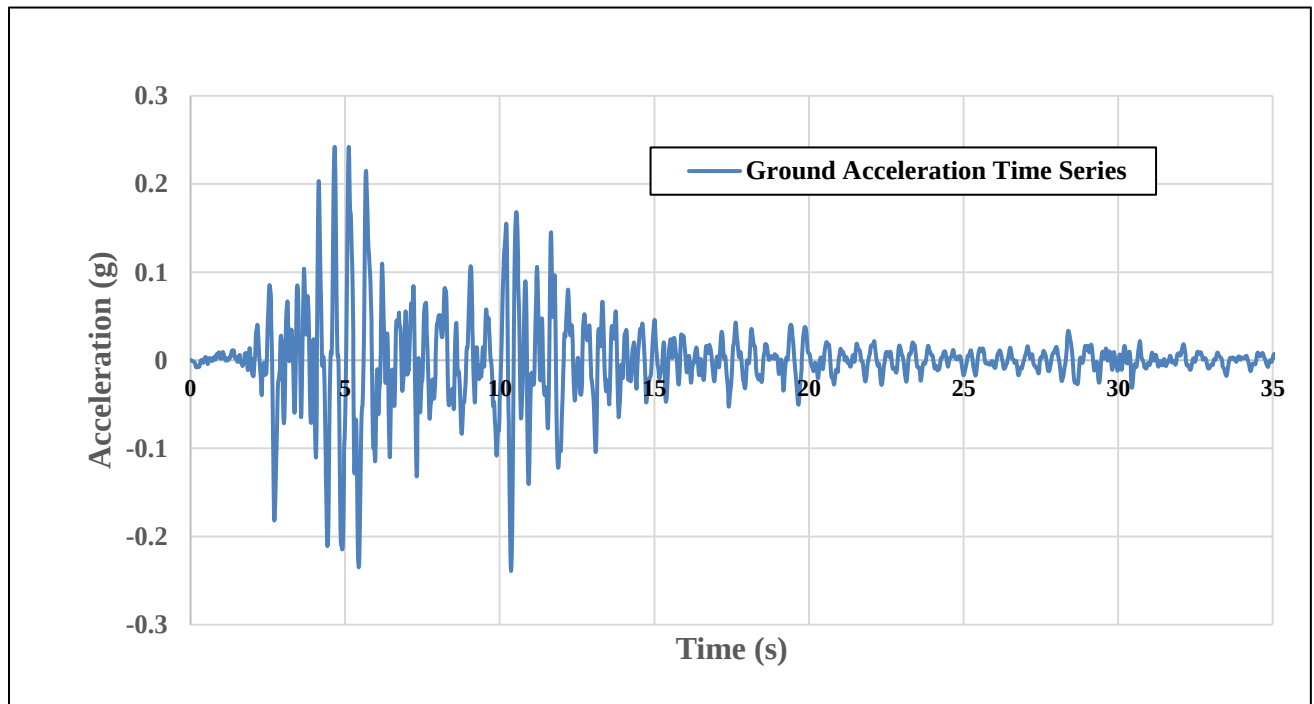


Figure 14 - Acceleration Time History for the soil profile II.

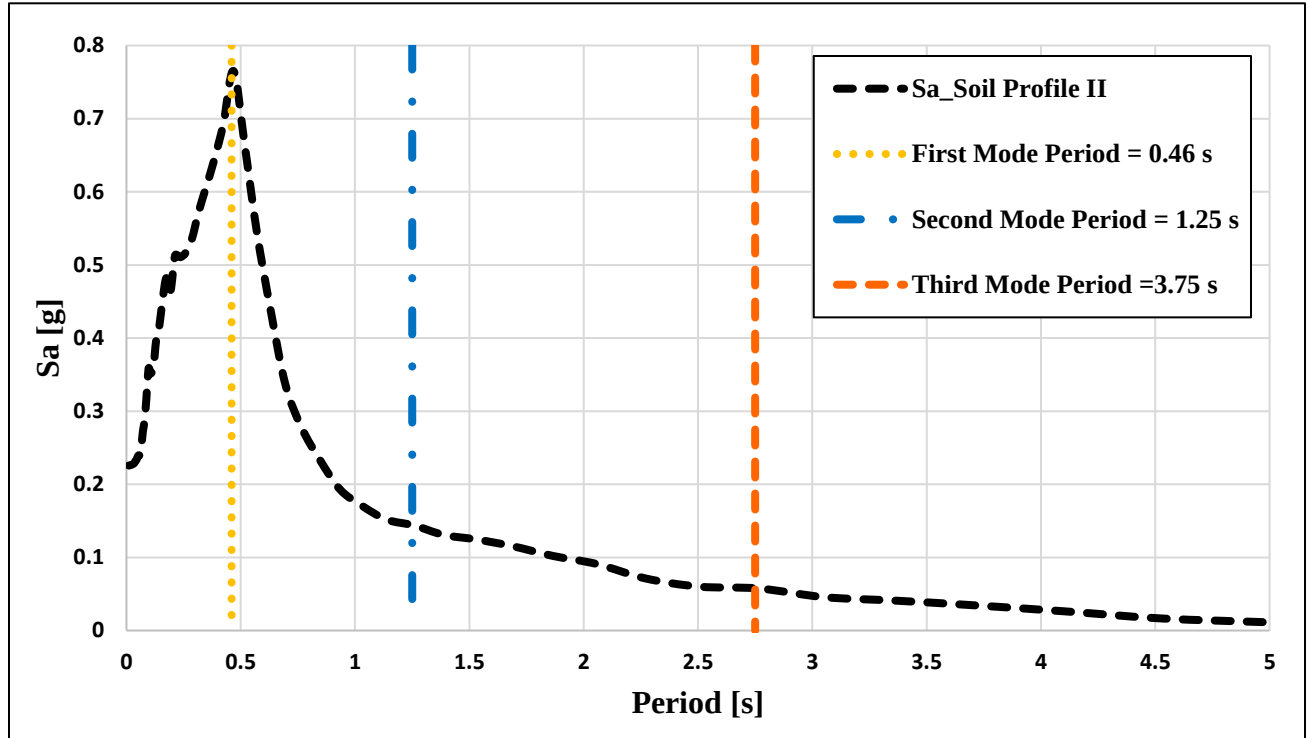


Figure 15 - Acceleration Time History for the soil profile II.

Based on the acceleration response spectrum graphs above, the three soil profiles are grouped and categorized into two groups according to EC8 design spectrum Types 1 and 2 (after running 100 realizations for each profile). These two groups are considered and compared with the design spectrum Type 1 and Type 2, as specified in Eurocode 8 respectively based on soil type C.

Notably, the design spectrum Type 1 falls within the mean class of 0.414 g of the soil profiles below soil profile I. Moreover, the design spectrum Type 1 covers the amplification range on the period axis. On the other hand, the design spectrum Type 2 falls at 0.54 g below the highest mean soil profiles, which are denoted by soil profile I and II.

For careful consideration and the fulfillment of the highest requirements, the two buildings (Masonry and Reinforced Concrete) have been modeled and inputted with soil profile type II parameters (which is the highest) to identify potential failures and deformations within these structures. Through this meticulous process, we can highlight any vulnerabilities that may exist, ensuring a comprehensive assessment of structural integrity.

4.1.1 Evaluating the Behavior of Modelled Buildings Using Axis-VM Finite Element Software

The process of assessing the performance of constructed buildings under the influence of seismic forces generated by soil properties is a meticulously structured endeavor. This systematic evaluation involves subjecting the buildings to seismic stresses, followed by a comprehensive analysis of standard seismic data and soil parameters. By meticulously collecting, analyzing, and comparing data against explicitly defined performance criteria, this approach equips designers with invaluable insights into how their designs perform during real-world usage.

High-rise residential buildings play a pivotal role in the assessment of seismic performance and resistance due to their ability to withstand longer periods and endure high stages of deflection. This prominence stems from

their distinctive characteristics, making them a primary focus when evaluating structural responses to seismic forces.

In this study, two distinct four-story buildings, each designed with differing structural systems - one from Masonry and the other from Reinforced Concrete. Both structures are designed using the parameters of soil II (Kekszgyár), characterized by the highest site-specific response spectrum measuring 0.76 g in order to determine the best fit and their behavior.

Moving forward, the next step is the application of diverse loads onto the constructed models. These loads fall into two fundamental categories:

- Live Loads:** These dynamic forces are imposed upon the model's surface as domain loads, with a specified value of 3.0 kN/m² for class C1, as outlined in EN 1991-1-1:2002 (E).
- Dead Loads:** Calculated from the geometry of the elements, dead loads pertain to the inherent self-weight of the structure. A dead load of 1.9 kN/m² is applied to the slab plate.

Additionally, the study encompasses a range of other dynamic and seismic Modal Response Spectrum Analysis (MRSa) loads considered based on EN 1998-1:2004 (EC 8-1).

- Static Load:** Displacement, internal force, stress, buckling analysis, and reinforcement are considered.
- Vibration:** Mode shape ordinate and the loads are considered as masses.
- Pushover:** The uniform load distribution option is generated as nodal forces proportional to the masses assigned to each node in the model for seismic determination.
- Seismic Load:** Parametric loads are chosen based on the spectrum recordings in the software.

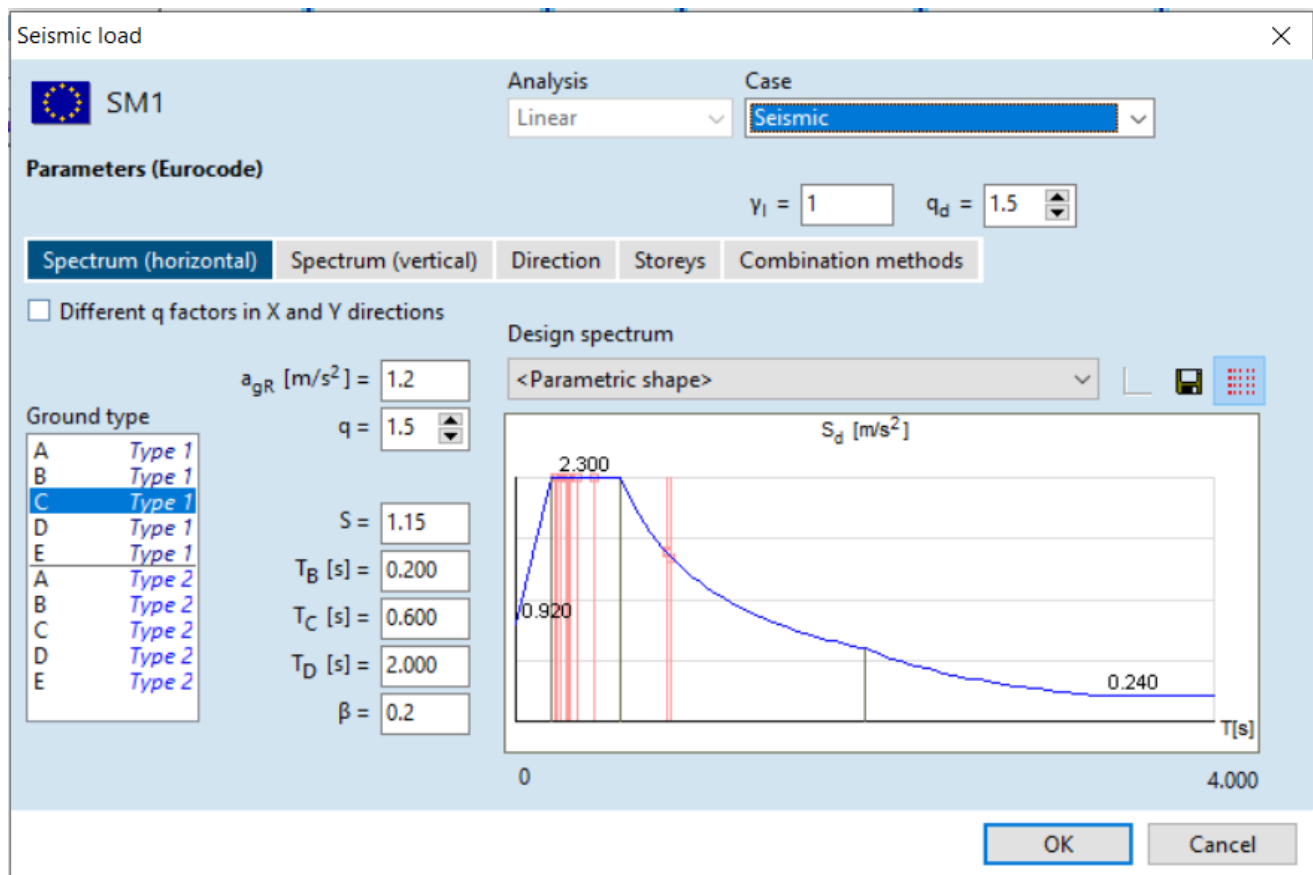


Figure 16 - Seismic Load indication in AXIS-VM software.

Below is the general floor arrangement for the proposed buildings with different structural systems.

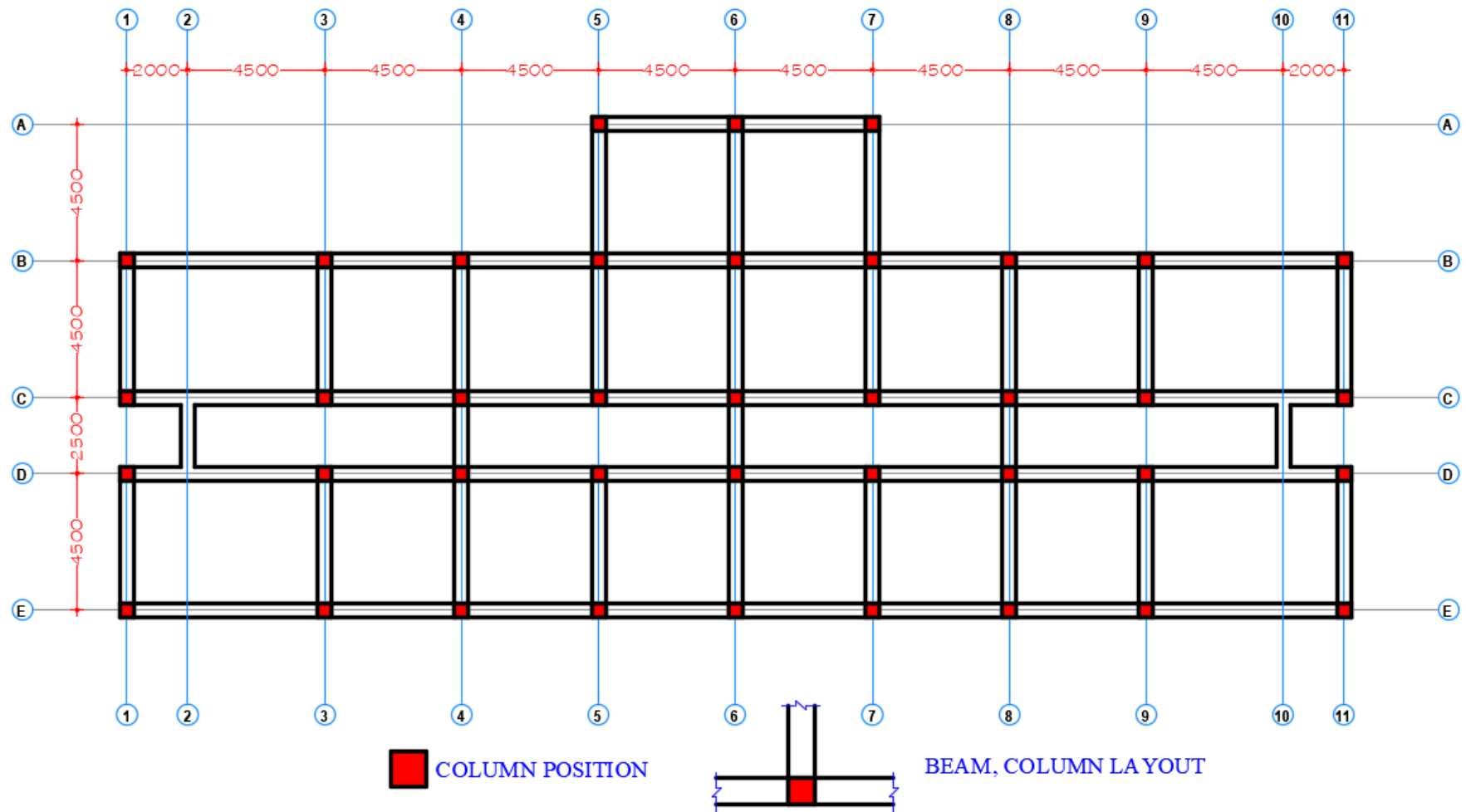


Figure 17 - Proposed Layout of Reinforced Concrete Building.

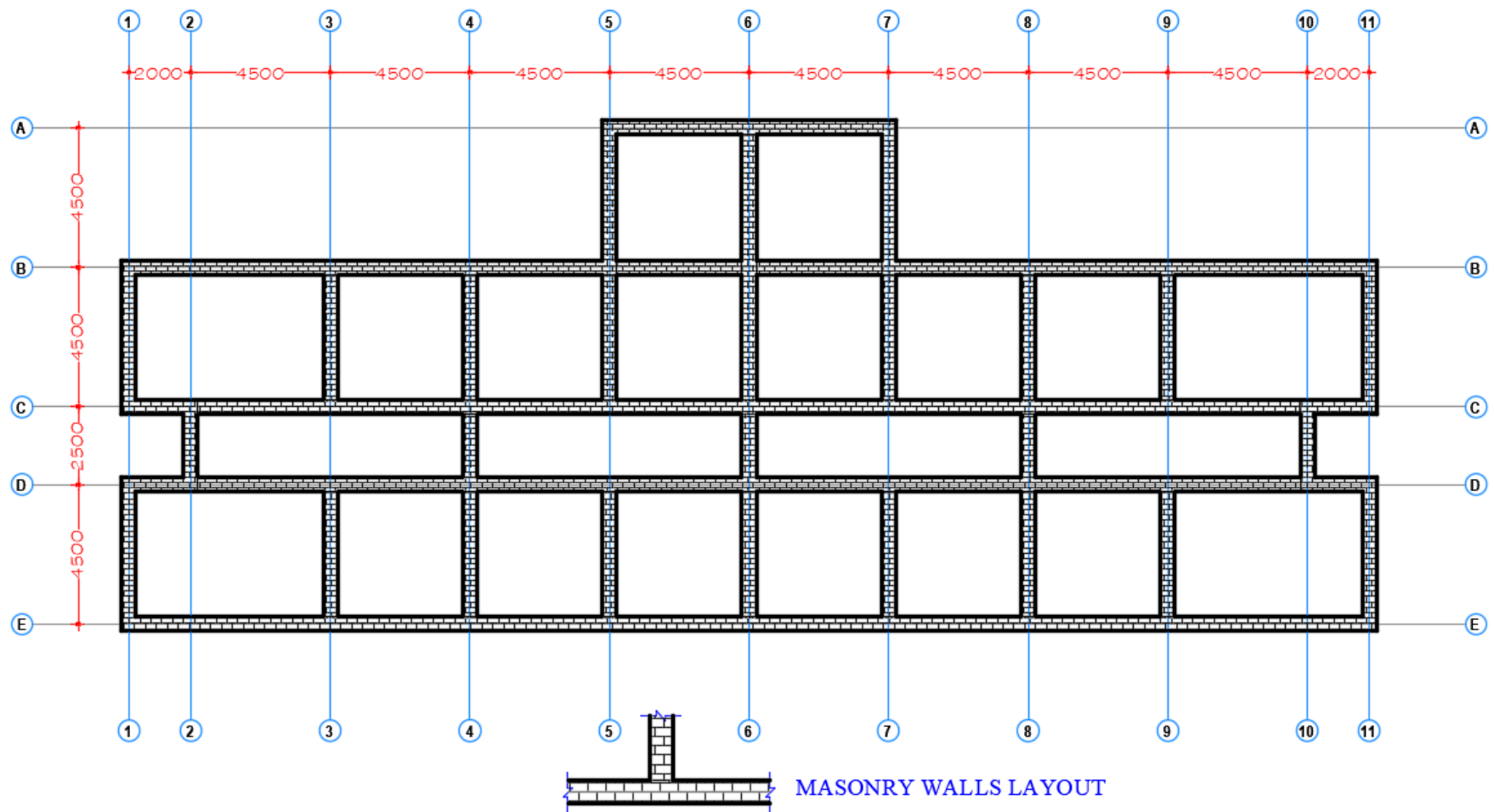


Figure 18 - Proposed Layout of Masonry wall Building.

4.2 Static And Dynamic Seismic Analysis

Dynamic analysis was employed to compute displacements and stresses arising from specified dynamic loading encompassing Design Spectral and Site-specific Response Accelerations based on the soil data. Both linear and nonlinear properties were taken into account during the analysis. The structural model's earthquake resistance was assessed through the response spectrum (Design Spectral and Site-Specific Response Accelerations). Seismic analysis and design incorporated the utilization of seismic isolators and dampers to address structural response in the weakest direction.

In the latter scenario, non-linear material properties and contact elements were employed, along with consideration for geometrical non-linearity, which encompasses large displacements and results of the analysis as are shown in **Error! Reference source not found.** below.

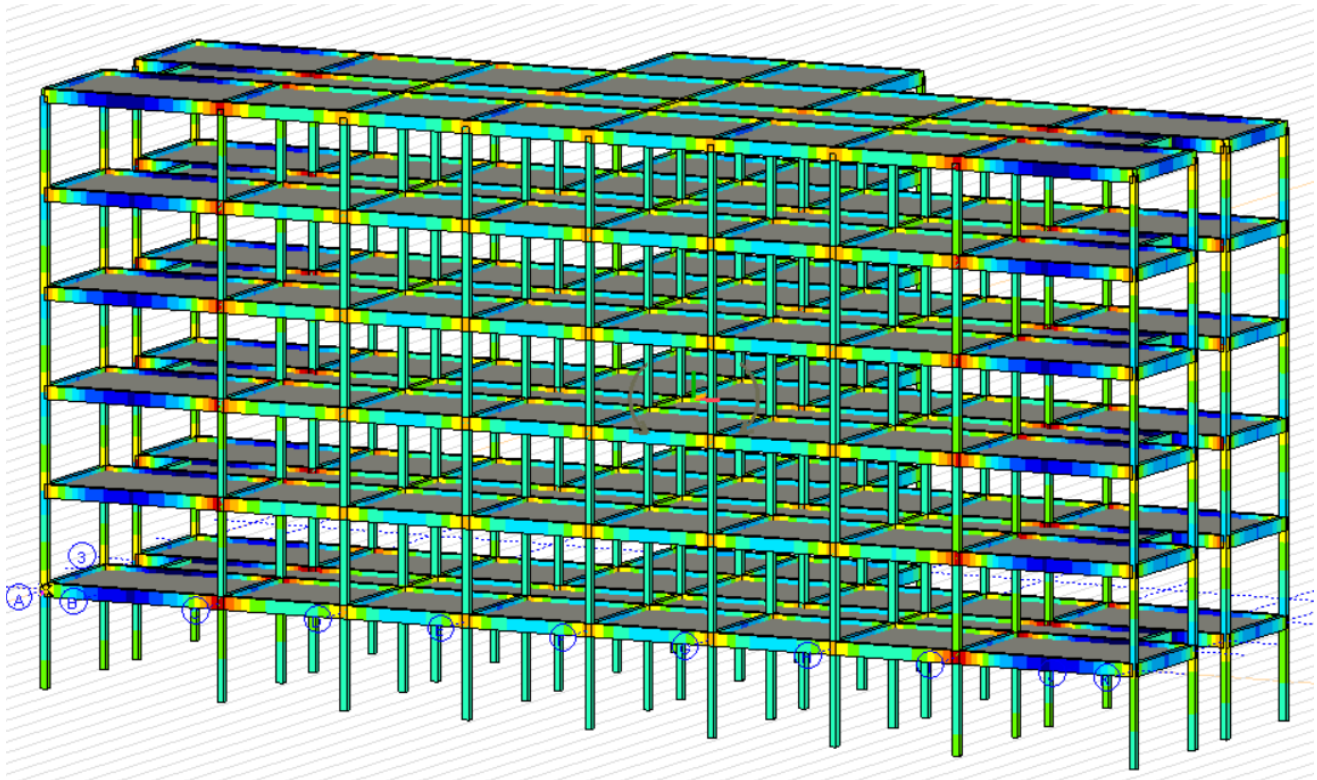


Figure 19 - 3-D physical model showing stresses arising from dynamic loadings.

Considering the horizontal performance-based seismic design criteria, the structural displacements of the reinforced concrete building are as shown below in **Error! Reference source not found.** and **Error! Reference source not found.** in comparison to the Design spectral and site-specific acceleration.

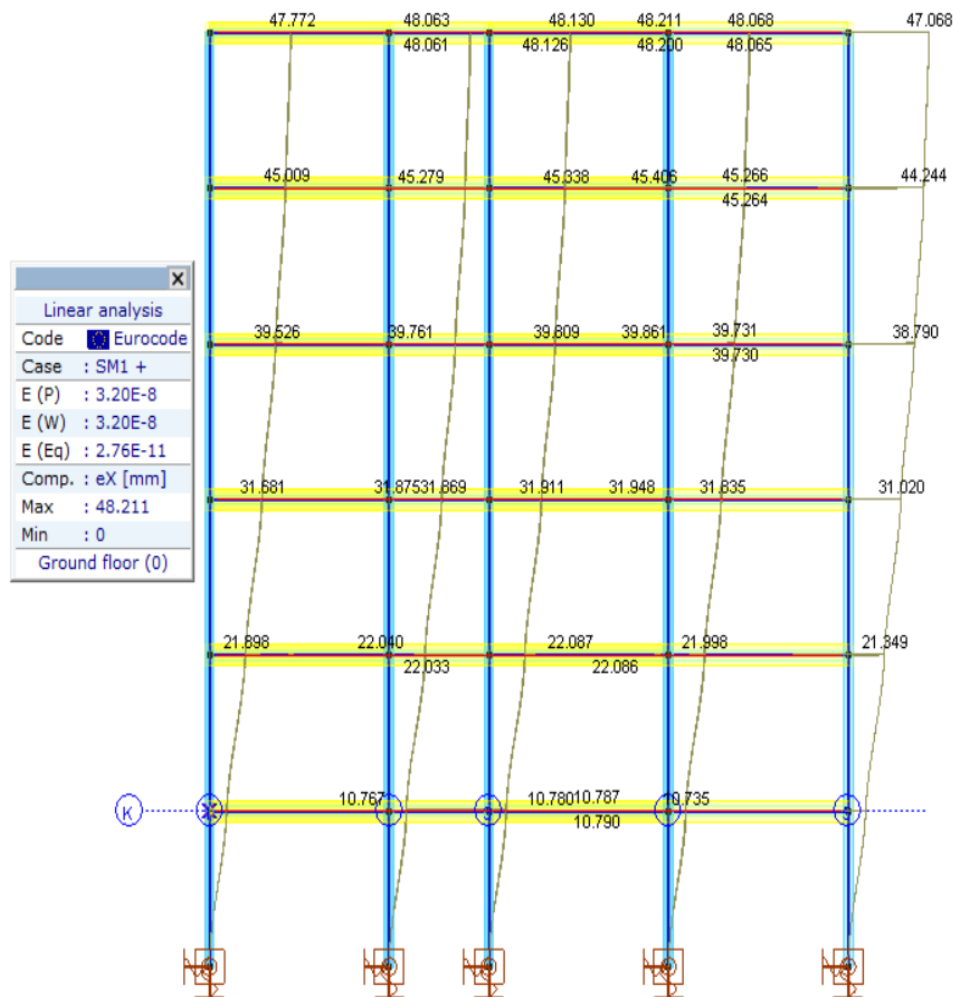


Figure 20 - Horizontal structural displacement performance based on Eurocode 8 Design Spectrum.

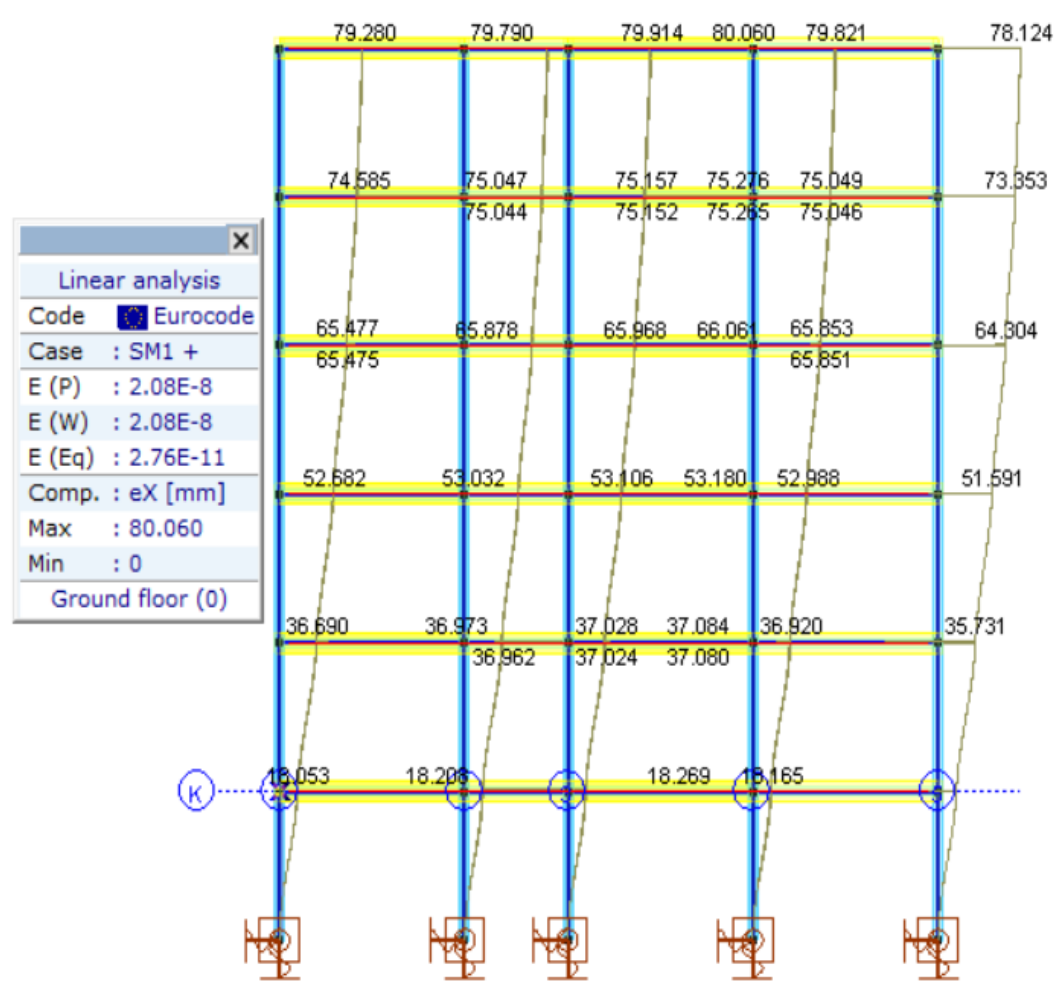


Figure 21 - Horizontal structural displacement performance based on Site-specific Spectrum.

In the overall structural displacements of the reinforced concrete structure, the maximum displacement when the design spectrum was applied is 48.13 mm and 79.28 mm for the site-specific spectrum respectively, showing a clear difference of 31.15 mm. This difference results in the variation between the design spectrum and site-specific spectrum based on the soil properties as shown in Figure 22.

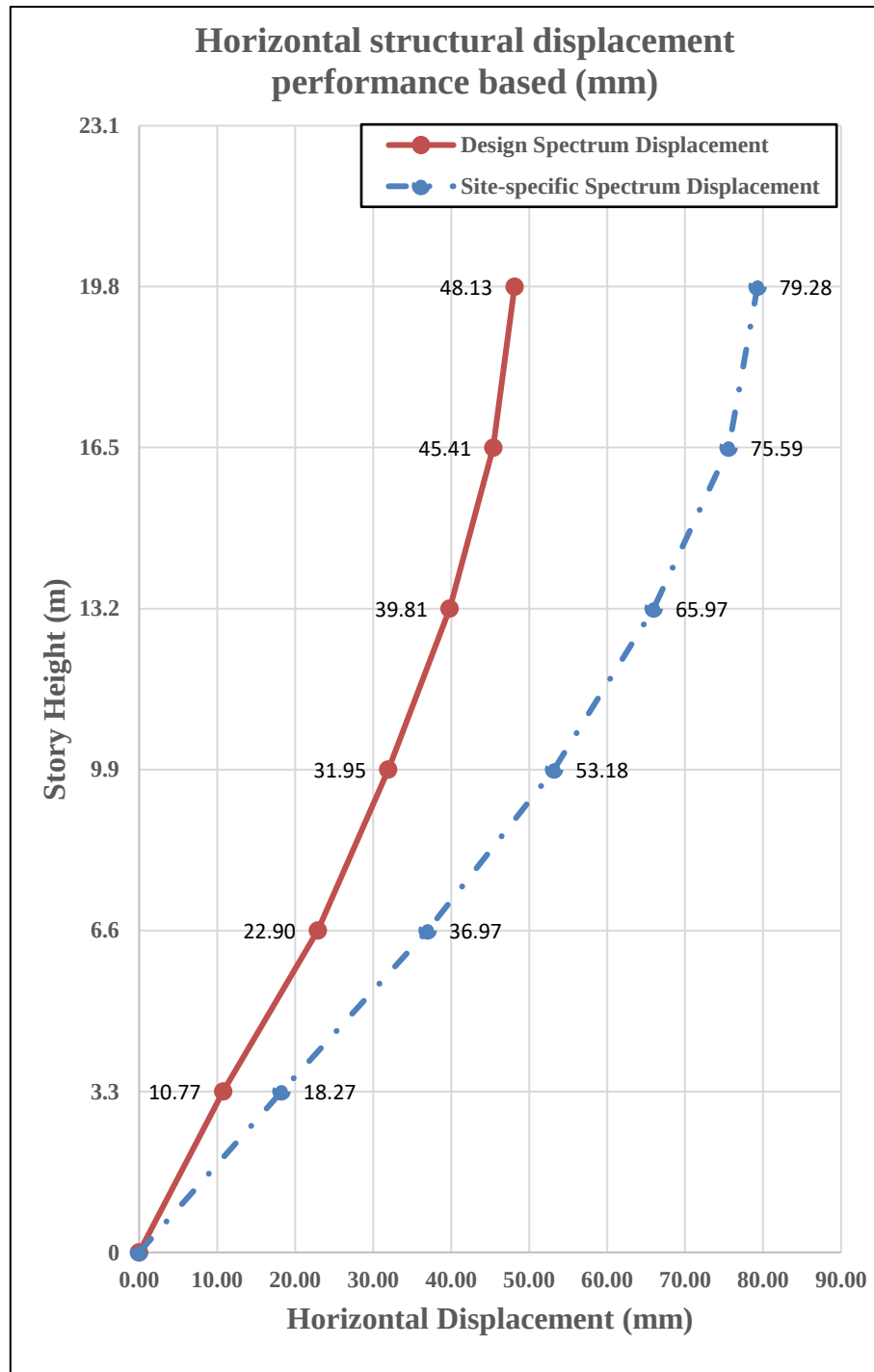


Figure 22 - Horizontal structural displacement performance based on Design and Site-specific Spectrum.

For the Mansory building, dynamic analysis was employed to compute displacements and stresses arising from specified dynamic loading encompassing Design Spectral and Site-specific Response Accelerations based on the soil data (the same loading used for the RC building) and the 3D modeled shape drawing as shown in **Error! Reference source not found.** below.

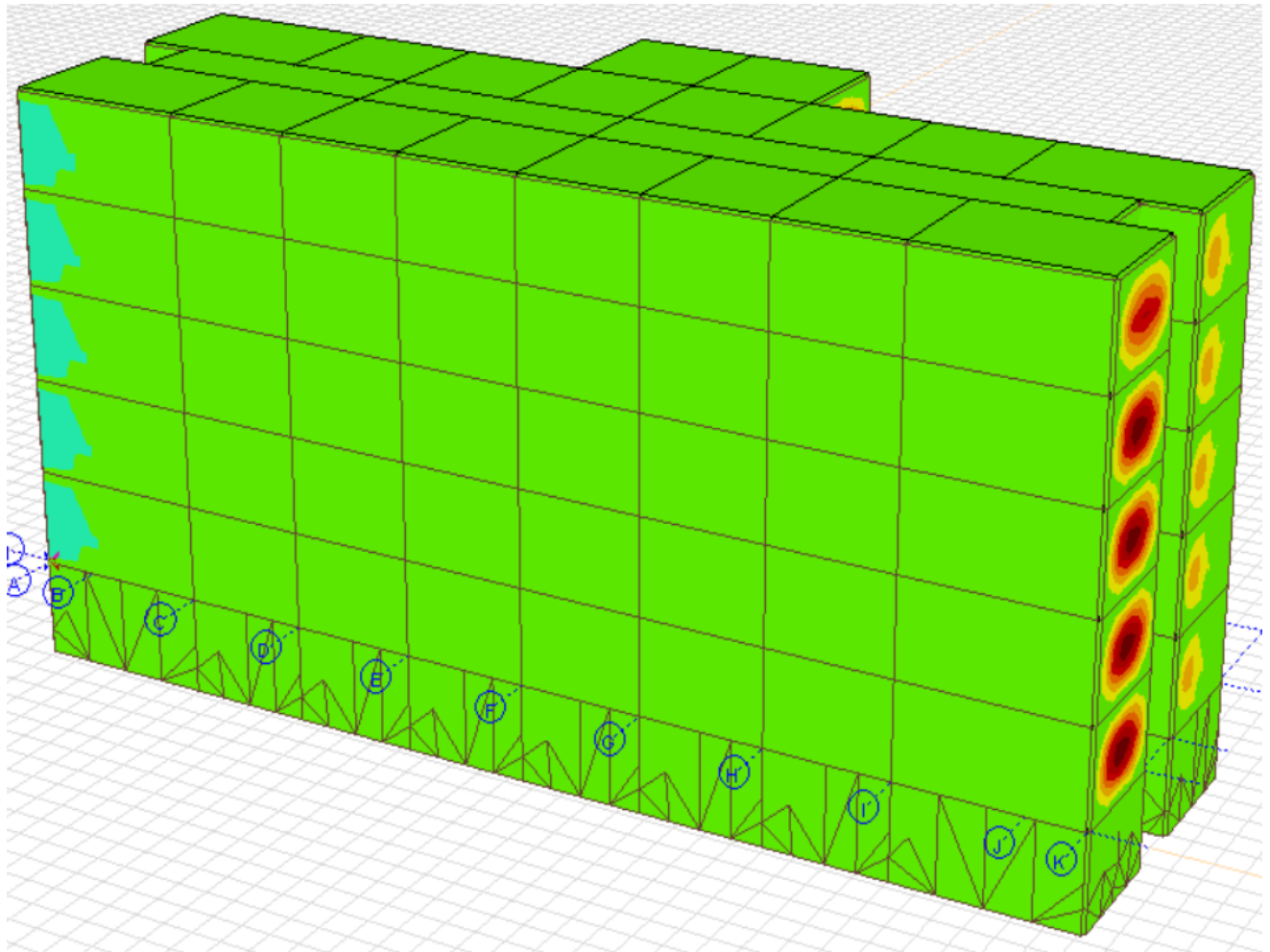


Figure 23 - 3-D physical model showing stresses arising from dynamic loadings of the mansory building.

The horizontal performance-based seismic design criteria were considered, and the resultant structural displacements of the reinforced concrete building are shown below in **Error! Reference source not found.** and **Error! Reference source not found.** in comparison to the Design spectral and site-specific acceleration. This was done by allowing movement and rotation in the weakest direction of the y-axis to evaluate the ultimate strength and displacement capacity of the structure and to verify that the failure mechanism is appropriate.



Figure 24 - Horizontal Masonry building displacement performance based on Eurocode 8 Design Spectrum.

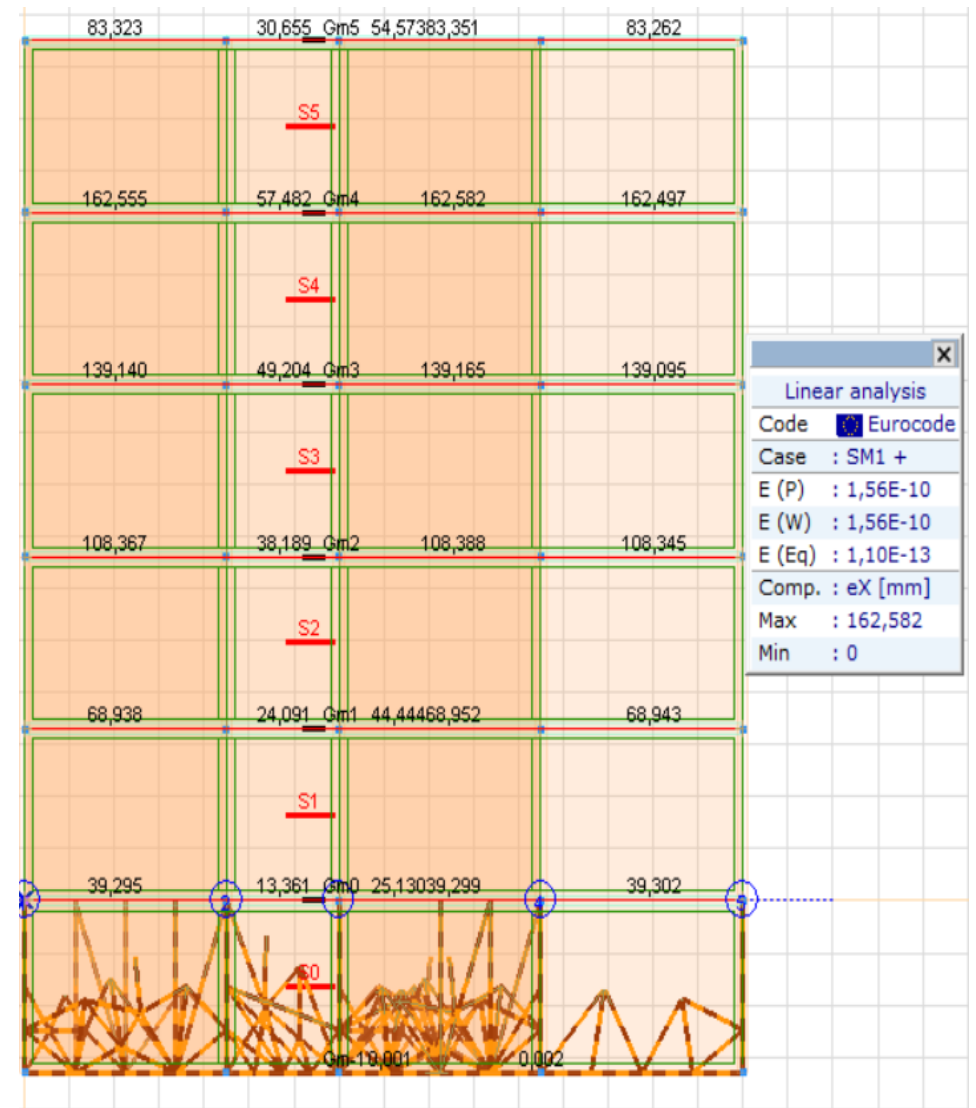


Figure 25 - Horizontal Masonry building displacement performance based on Site-specific Spectrum.

In the overall masonry building displacements, the maximum displacement when the design spectrum was applied is 91.409 mm and 162.582 mm for the site-specific spectrum respectively, showing a clear difference of 71.173 mm more than reinforced building by 38.762 mm. This difference results in the variation between the design spectrum and site-specific spectrum based on the soil properties as shown in Figure 26.

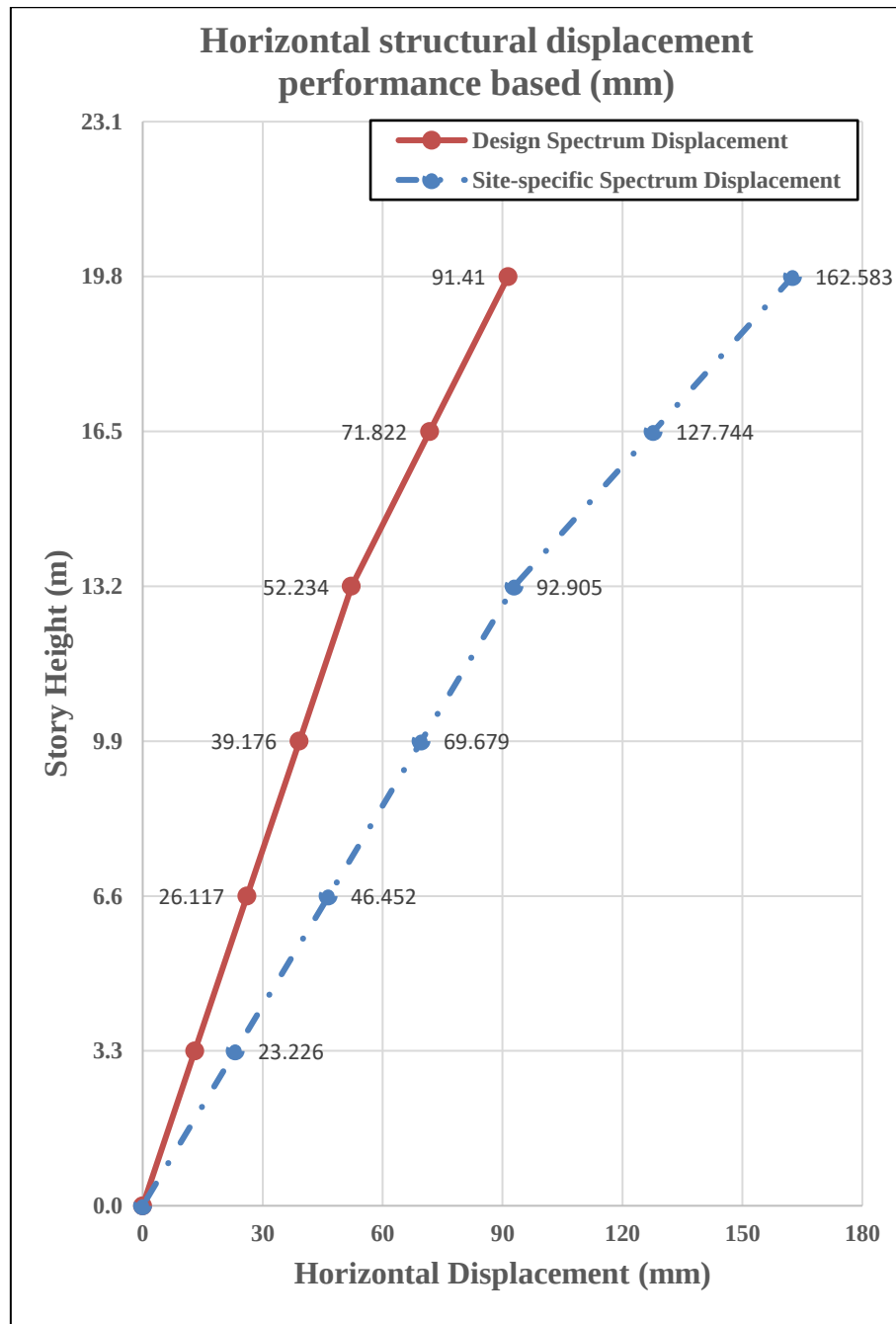


Figure 26 - Horizontal structural displacement performance based on Design and Site-specific Spectrum for the Masonry building.

To understand the collapse or deflection behavior of these structures under seismic or earthquake forces, the collapse mechanisms of the reinforced and masonry buildings are considered and presented in the graphs shown below.

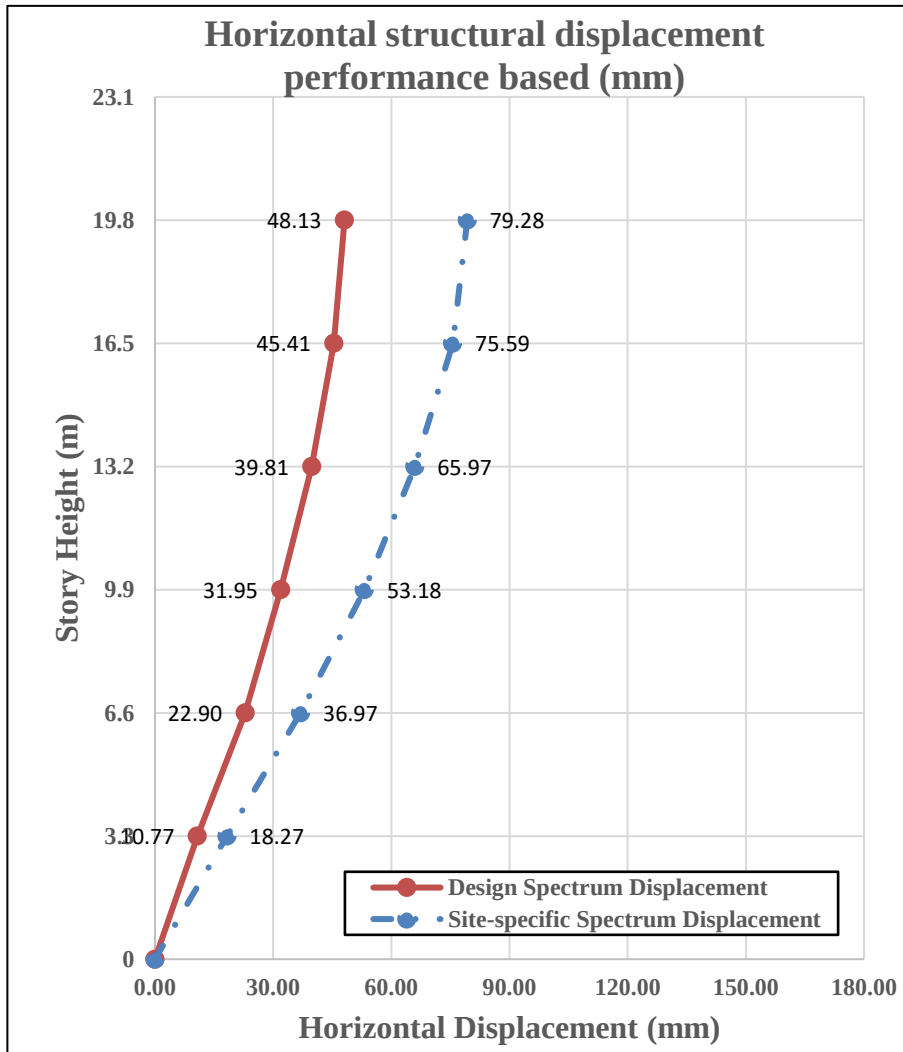


Figure 27 - The displacement curve sway mechanism shows a clear overturn from the middle points of the structure.

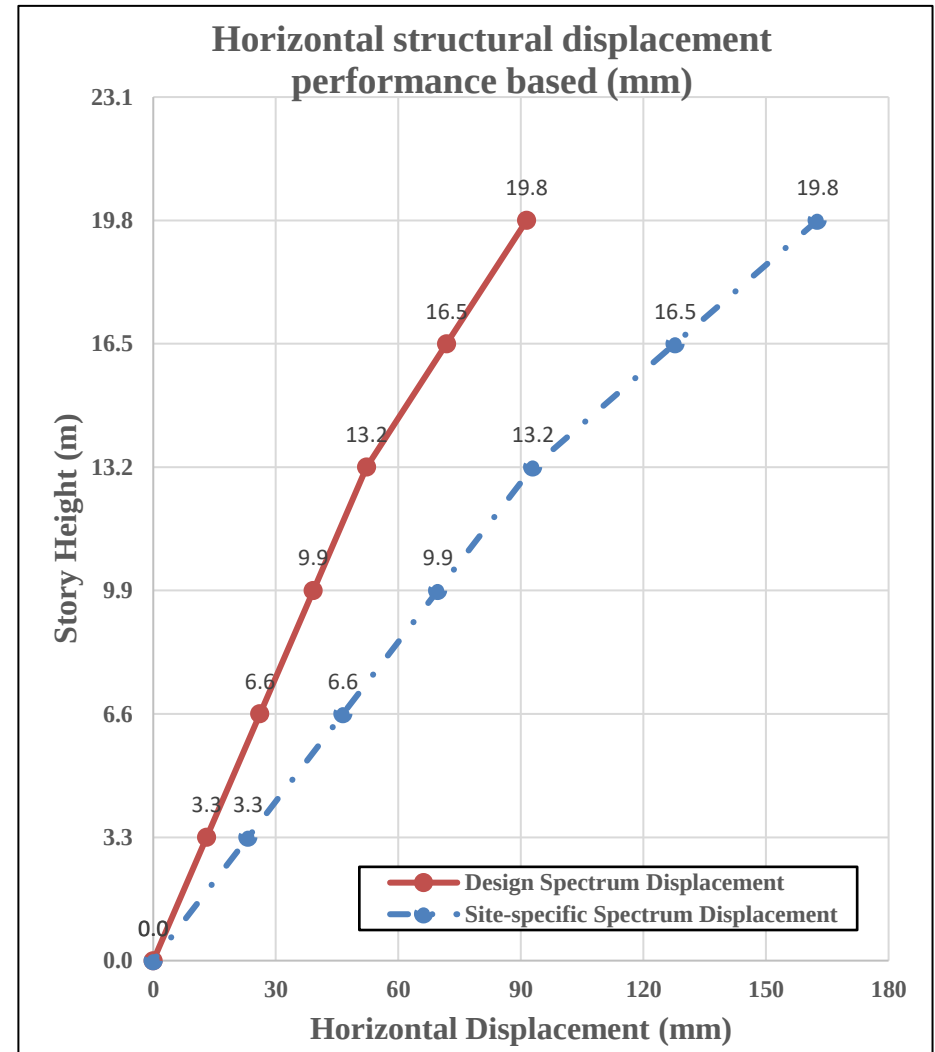


Figure 28 - The displacement curve shows areas of weakness are more general from the top as the building height progresses.

4.2.1 Vibration Analysis

The first and second-order vibration analyses were utilized to ascertain natural frequencies and mode shapes, which are frequently vital in solving dynamic problems and understanding the deformation mechanism and behavior of structural members. In seismic analysis, modal mass factors and moments of inertia are computed for the first dominant mode shapes in the x direction to establish the failure mode and these were correlated with Design and site-specific response, as depicted in Figure 29 below.

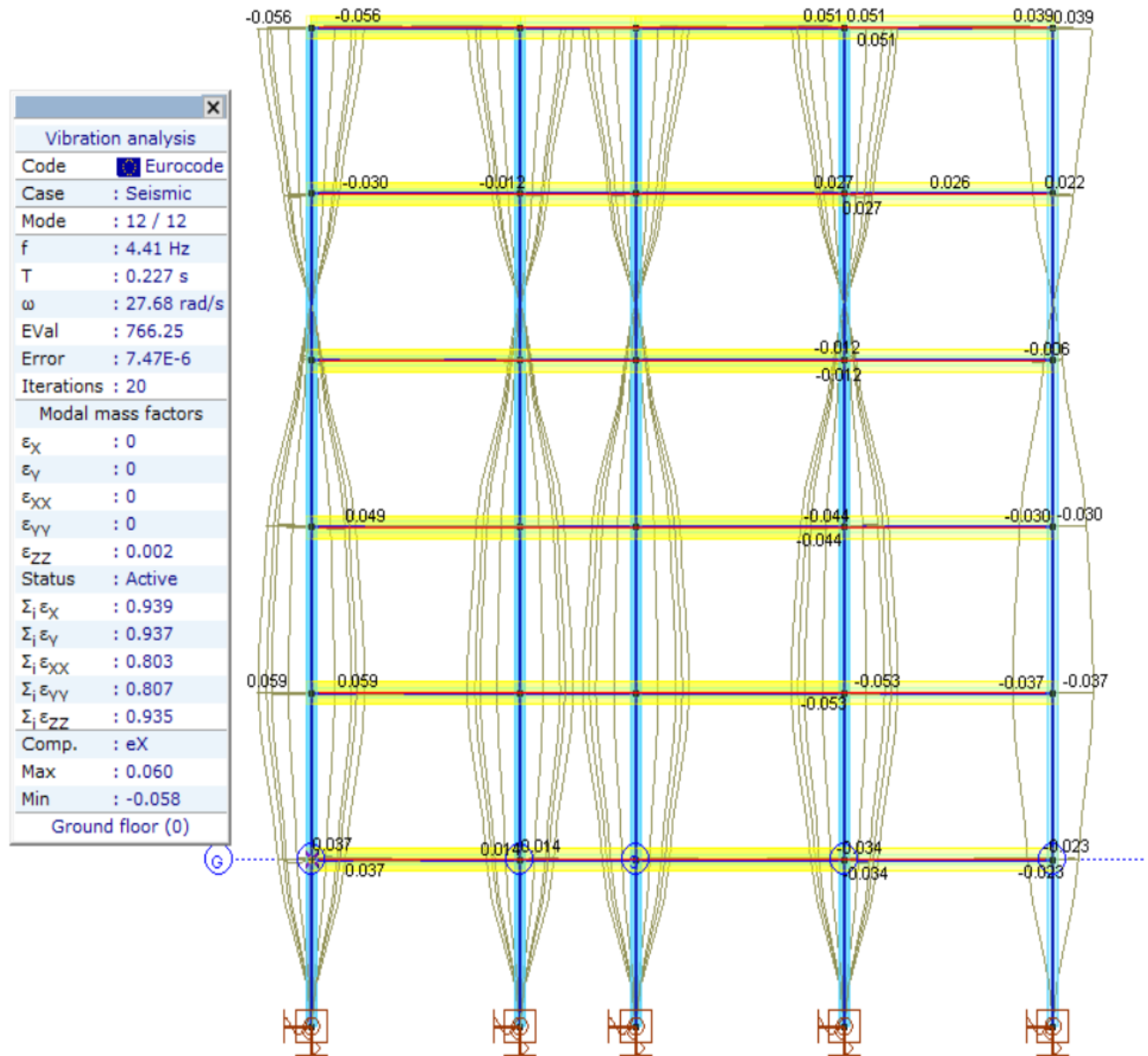


Figure 29 - For vibration the Maximum acceleration spectral design for Types 1 & 2, mode shape deformation for the Design Spectral acceleration showing negligible damage on the RC and Mansory structural design members (mode shape 10) correlating with the vibrator.

Equally vibration analysis was done for the Mansory building considering the highest modal mass factor and the difference in deflection of the members is considered negligible at 0.087mm.

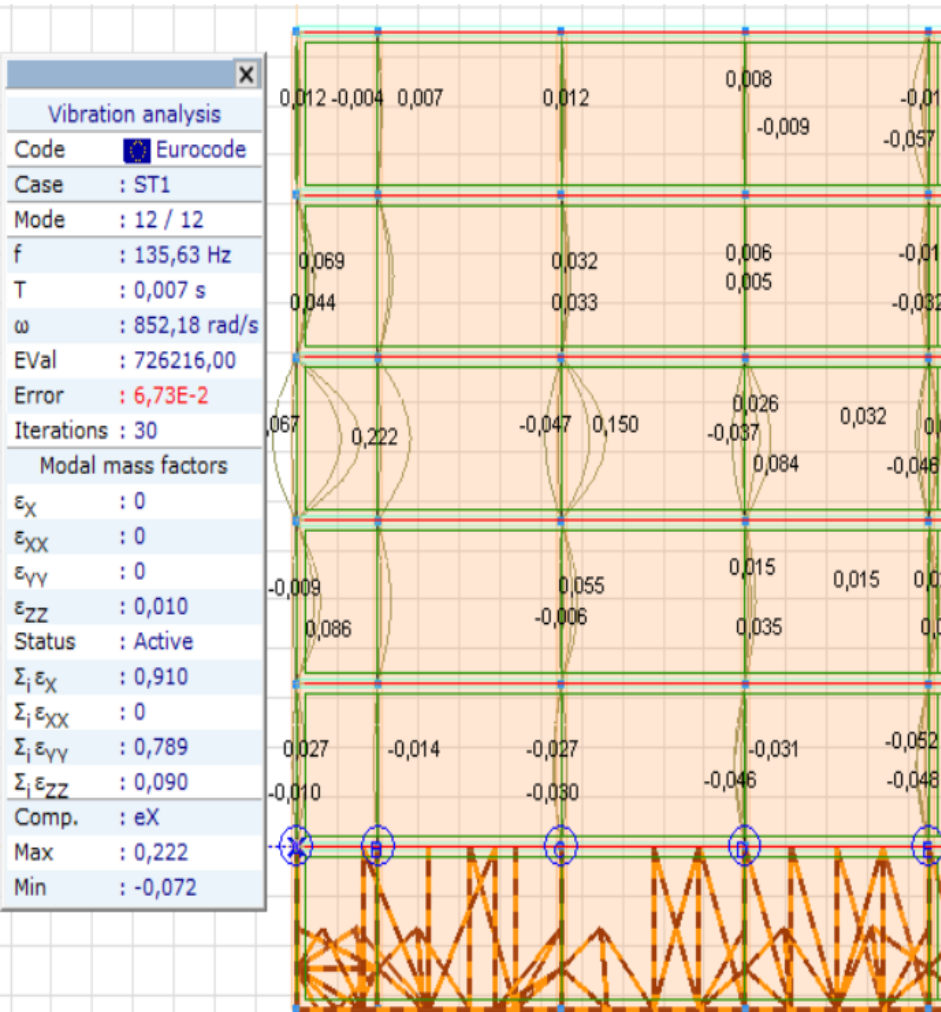


Figure 30 - Deflection of the Mansory building at Design spectra.

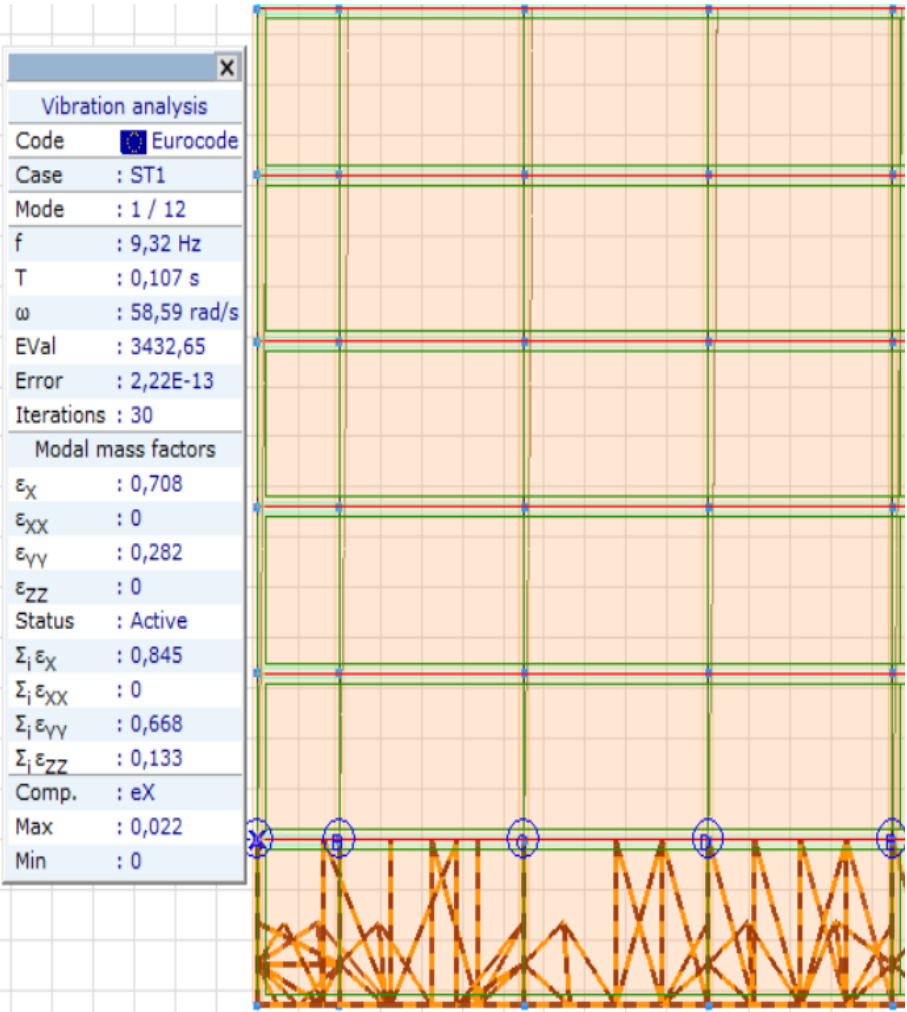


Figure 31 - Deflection of the Mansory building at Site-specific spectra.

Frequency Mode Shapes Factors for Design & Site-specific Spectrum Structure				
Mode Shape	f [Hz]	T [S]	ω [rad/s]	Modal Mass [%]
1	1,12	0,892	7,04	0
2	1,15	0,866	7,26	82,0
3	1,16	0,862	7,29	0,27
4	2,24	0,447	14,05	0
5	2,83	0,354	17,77	0
6	3,23	0,31	20,27	0
7	3,35	0,299	21,02	0
8	3,43	0,292	21,52	0.3
9	3,44	0,291	21,61	0.89
10	3,92	0,255	24,66	0
11	4,27	0,234	26,83	0
12	4,4	0,227	27,68	0

Frequency Mode Shapes Factors for Design & Site-specific Spectrum Structure				
Mode Shape	f [Hz]	T [S]	ω [rad/s]	Modal Mass [%]
1	7,55	0,132	47,42	71,6
2	22,28	0,045	139,98	0.74
3	35,79	0,028	224,87	0.22
4	46,79	0,021	293,97	0.06
5	53,29	0,019	334,84	0.01
6	122,12	0,008	767,29	0.91
7	133,89	0,007	841,28	0
8	134,52	0,007	845,2	0
9	134,55	0,007	845,43	0
10	134,81	0,007	847,01	0
11	135,5	0,007	851,39	0
12	135,63	0,007	852,18	0

Table 4 - Reinforced Concrete Structure's Time-Frequency mode shape factor displacement for the Design and site-specific Spectrum highlighting the dominant times and frequencies in correlation to the Eurocode 8 spectrum based on the highest modal factor and Mansory building's Time-Frequency mode shape factor displacement for the Site-specific Spectrum highlighting the dominant times and frequencies in correlation to the site-specific spectrum based on soil's properties and the highest modal factor.

The objective of modal response spectrum analysis is to find the highest seismic impact in each of the N modes caused by the seismic force in directions X, Y, or Z as specified in Eurocode 8 for modal mass.

4.3 Capacity Design and Strengthening of Weak Members

Based on EN 1998-1:2004 (E) design method for every element in the structural system should be chosen and carefully designed and detailed for energy dissipation under severe deformations and providing other structural elements with sufficient strength so that energy dissipation can be maintained to avoid damage and collapse by seismic actions.

To unravel the effectiveness of the structural designs for the design and site-specific response in Axis-VM, member elements are compared to conventional capacity design in terms of failure and the ability to withstand seismic forces applied. The results presented herein can be seen as a first demonstration of the potential advantages of the design spectrum to site-specific when applied. This will also help in practical applications when designing to withstand seismic forces by extensively verifying both analytical and experimental designs.

For the reinforced structure, all columns at site-specific response have failed as compared to the columns at the design spectra failing to satisfy the condition to resist the applied force and moment as shown in the equation and figures below.

$$\eta_N = \frac{N_{Ed}}{N_{Rd}} < 1 \quad (4.4.1)$$

Where:

N_{Ed} is the design load in the Column and N_{Rd} is the resistant load in the column respectively.

In the case of design spectra, $\eta_N = 0.1729$ which is less than 1 and has satisfied the condition according to EC2, EN: 3.1.6, 1992 and $\eta_N = 1.464$ greater than 1.

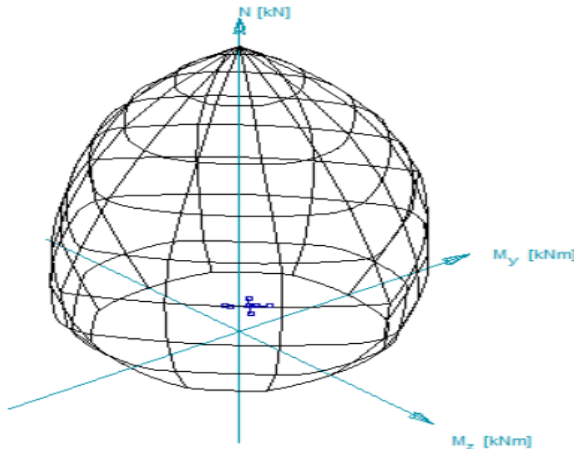


Figure 33 - Column carrying capacity for the Design Spectra showing no deformation or failure.

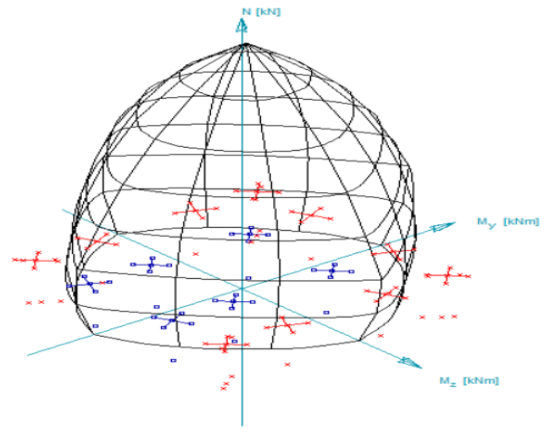


Figure 32 - Column carrying capacity for the Local site Spectra showing deformations or failure.

Hence all columns in the reinforced building will be redesigned by selecting new dimensions. This will increase the resistance and the stiffness of the structure thereby reducing deflection caused by seismic forces.

Similarly, the Mansory building was also checked in terms of capacity and its ability to bear the loading from the structure at designed and site-specific spectra. The findings are presented below.

- All walls have proven to be incapable of carrying the loads from the building at both the design and site-specific response.
- Due to the high deflection mechanism arising from the weak direction, all joints turned to sway positively causing the walls to be in Tension instead of compression resulting in total failure.

- c) As shown in the horizontal performance displacement curve, the more stories the higher the displacement causing more stresses in the building at higher heights.

4.3.1 Proposed Remedy Method for the New Design

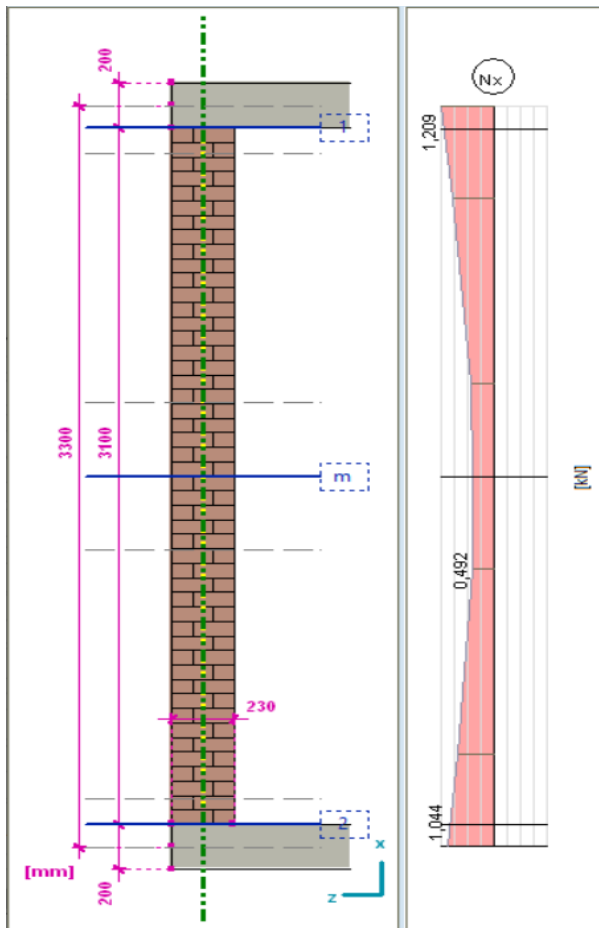


Figure 34 - Horizontal displacement of solid brick members resulting in tension instead of compression.

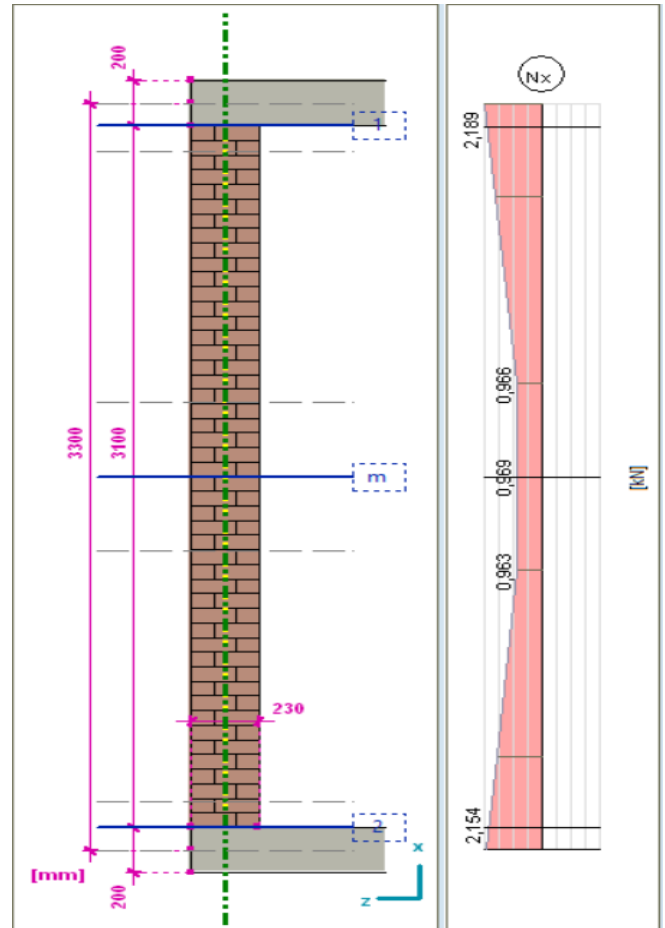


Figure 35 -, Horizontal displacement of solid brick members resulting in tension instead of compression.

The following redesign technologies will be applied to the failed members, in the case of columns for the reinforced building, re-selection of the columns are the most ideal and have proven in the remodeled building that columns of dimensions 230 x 450 mm can withstand and resist shock forces while shear walls will be applied at major corners of the bricks (Mansory) building to act as a resistance to the seismic forces.

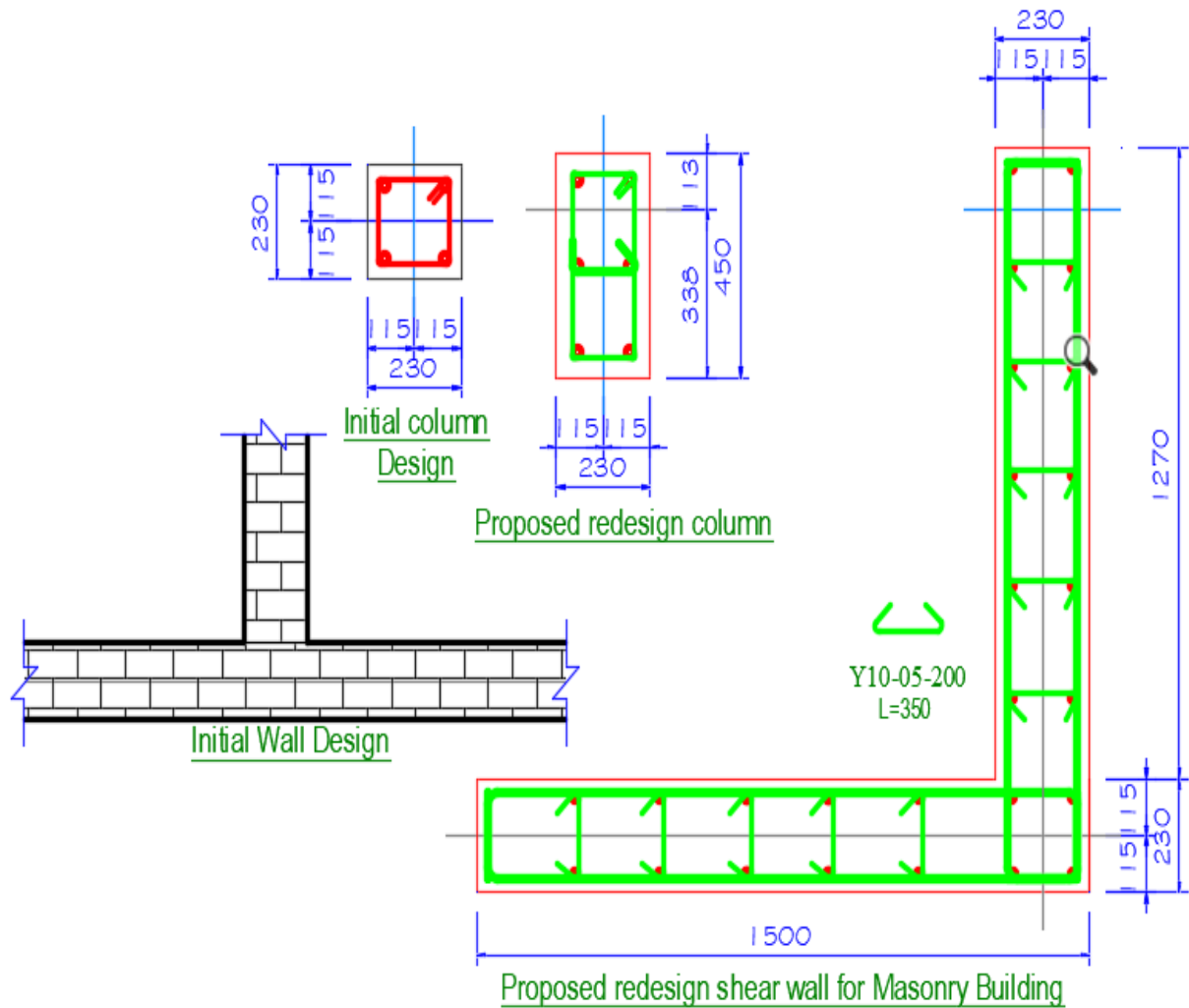


Figure 36 - Proposed cross-sections for the retrofitting of columns and shear wall.

To determine the most suitable, sustainable and to reduce the rate of environmental impact, the building members (columns and walls) that are to be strengthened are examined by performing LCA in addition to the structural analysis to decide the most suitable redesign technique and the best in terms of ecological cost and carbon footprint.

Below is the general arrangements of columns positions for the redesigned reinforced building to limit deformations and damage to the structure allowing it to withstand seismic forces.

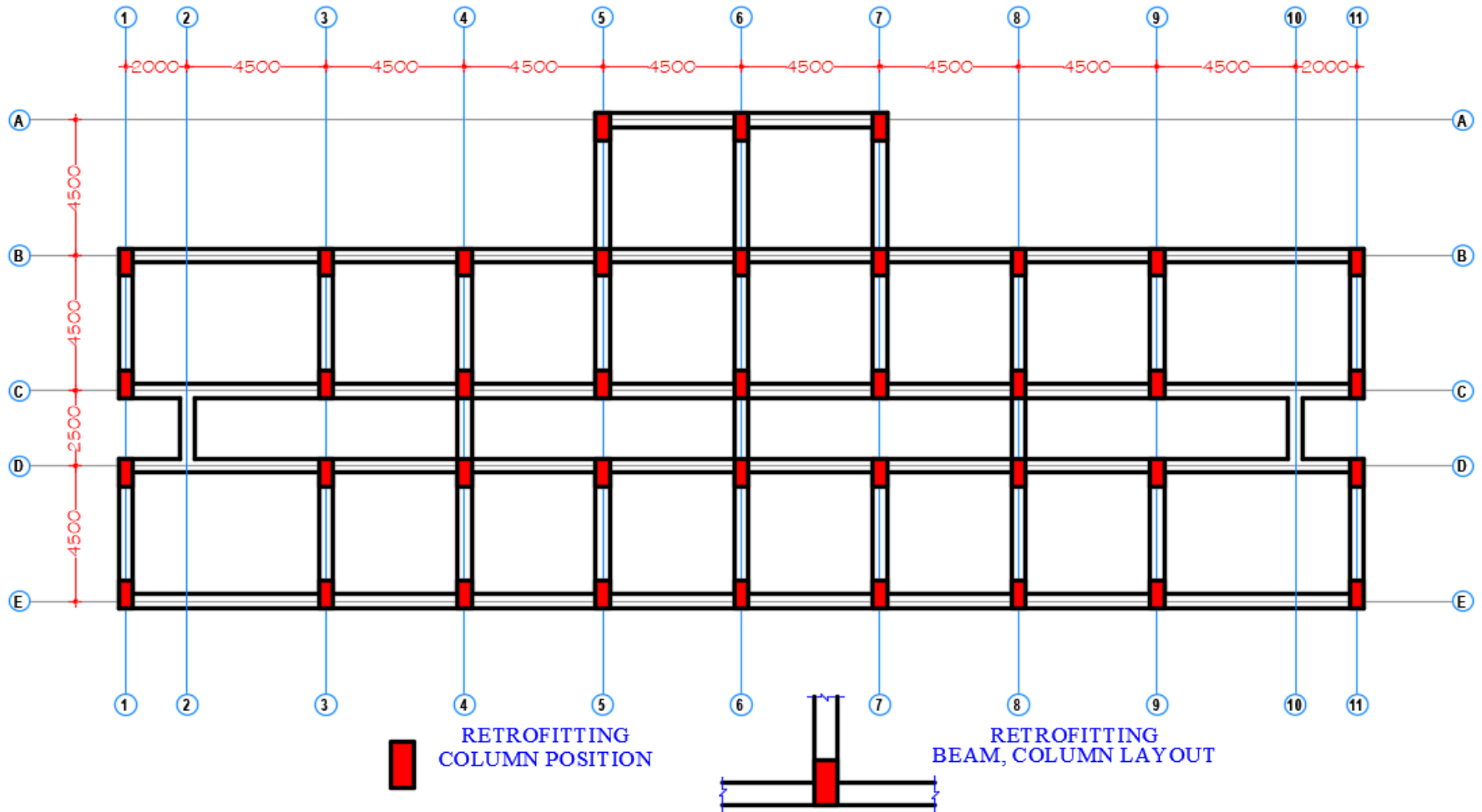


Figure 37 - Column layout for Redesign of the Reinforced Concrete Building.

In the case of masonry building, proposed shear walls positioning for the redesigned building as seen below in Figure 38 to limit deformations and damage to the structure allowing it to withstand seismic forces.

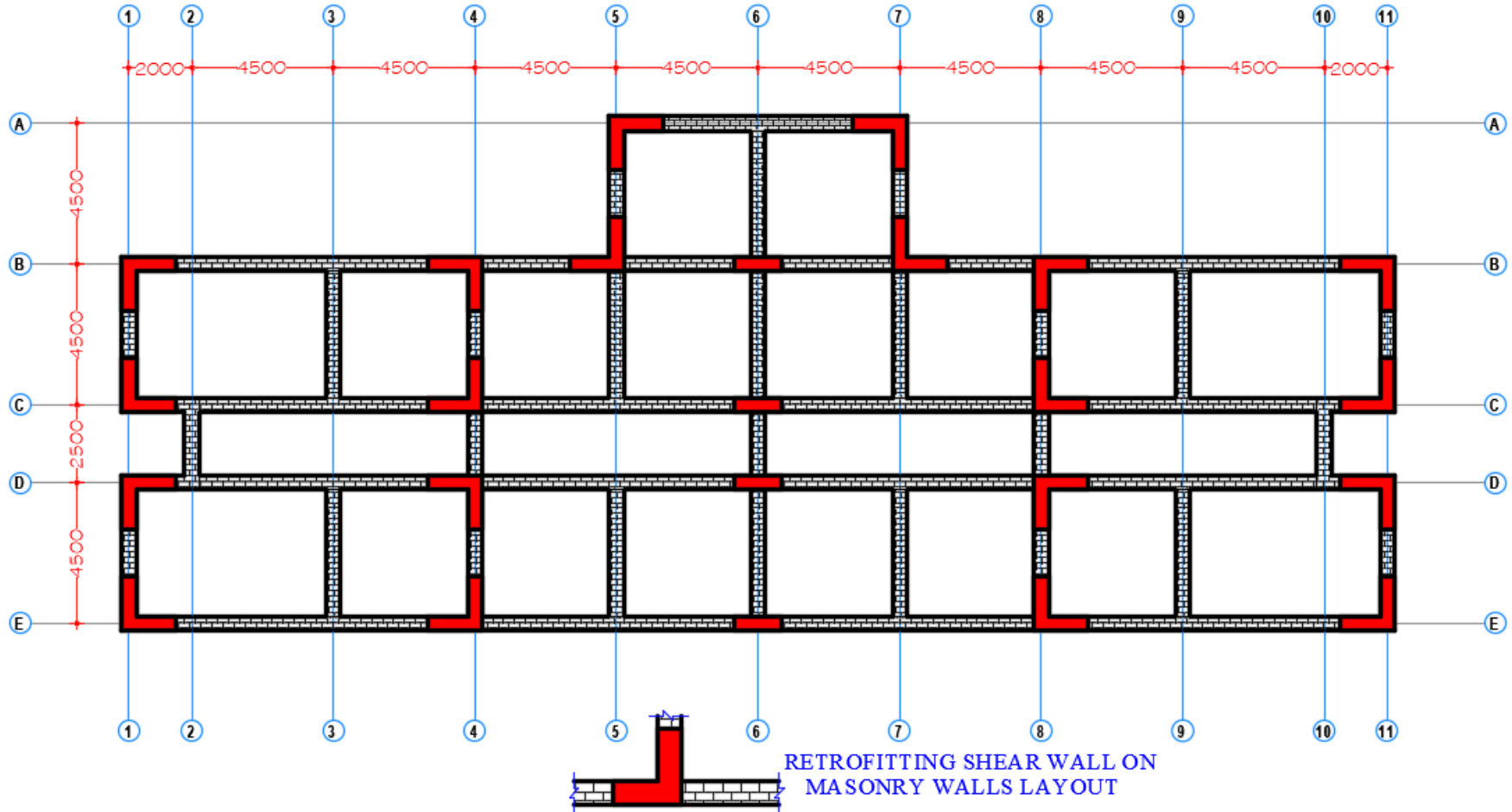


Figure 38 - Redesigned Shear wall layout for the Masonry building.

In comparison to the initial deformations in the reinforced and the masonry buildings, there has been a significant reduction in the deformation of the structure since the redesign elements were sufficient to withstand bending and twisting. Compression, or stretching without losing its original shape. It is worth knowing that a stiffer structure deforms less under seismic and eternal loading which is the case as seen below.

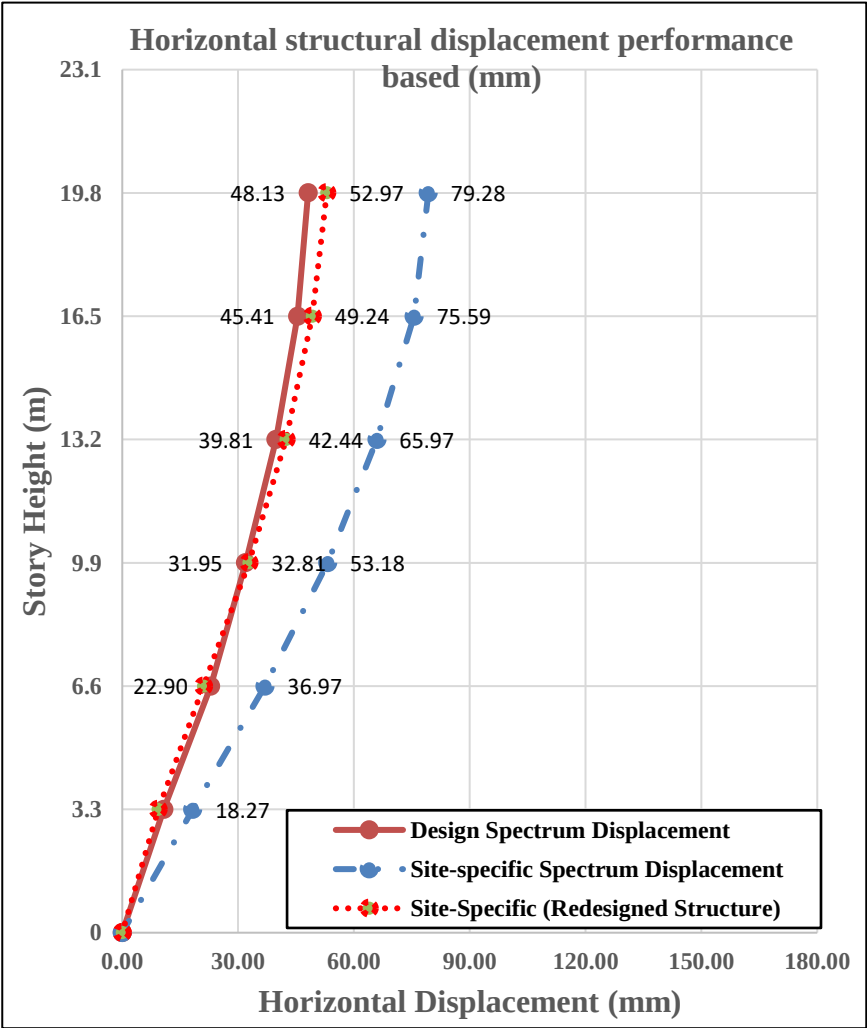


Figure 39 - Horizontal comparative deformation of the increased in stiffness of deformed members (RC Building)

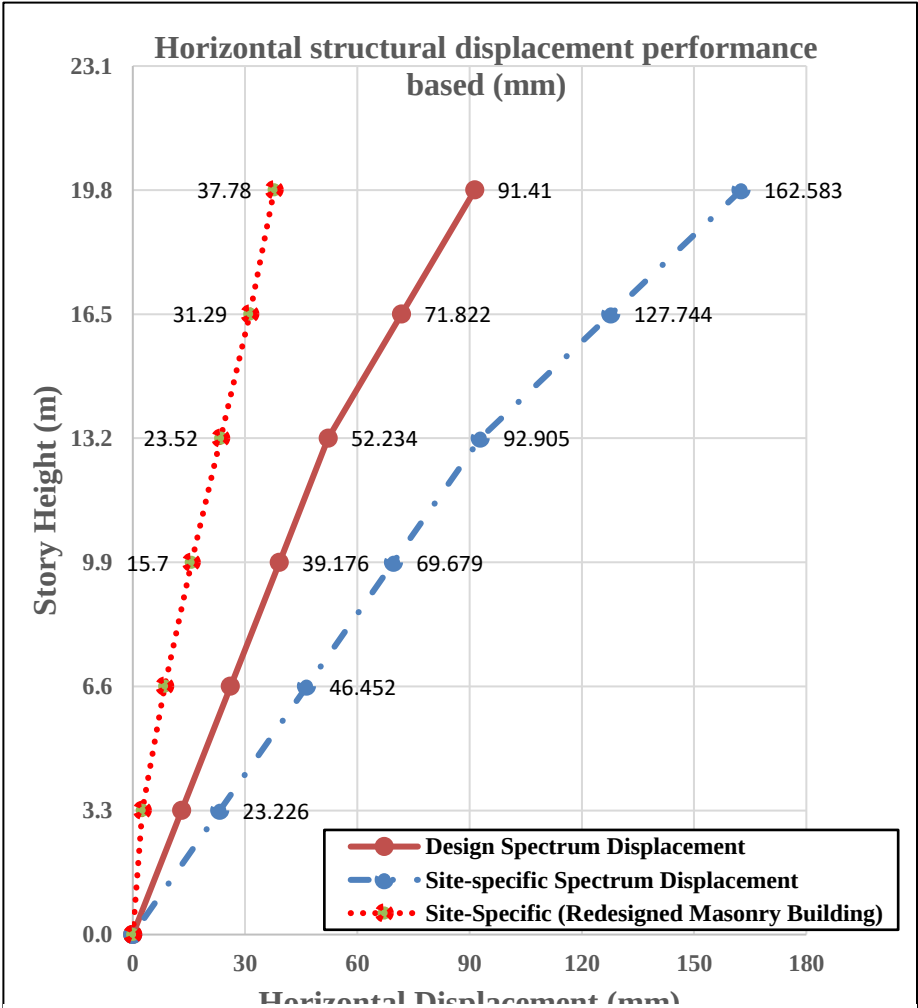


Figure 40 - Horizontal comparative deformation of the increased in stiffness of deformed members (Masonry Building)

4.4 Comparative Life Cycle Cost Assessments for the Masonry and Reinforced Buildings

For the LCA performance, key indicators, and major compositions for the elements such as the total volume of steel rebars in kilograms and the total concrete required based on key components are considered for effective determination of the eco-cost and carbon footprint in terms of resource depletion, eco-toxicity, and human health as shown below.

LCA performance for Reinforced Building		
Indicators (Parameters)	eco-costs (€)	Carbon footprint kg CO2 eq.
Steel Rebar (Reinforcement)	1535.82	6801.50
Cement	639.51	3385.64
Sand	4.51	17.30
Crushed Aggregates	33.05	120.17
Transportation	8.54	30.97
Electricity & Gas	79.64	480.75
Total	2301.07	10836.33

Table 5 - Life cycle assessment of retrofitting the reinforced building.

LCA performance for the Masonry Building		
Indicators (Parameters)	eco-costs (€)	Carbon footprint kg CO2 eq.
Steel Rebar (Reinforcement)	1286.41	5696.98
Cement	773.73	4096.22
Sand	5.46	20.94
Crushed Aggregates	39.98	145.39
Transportation	9.51	34.47
Electricity & Gas	72.73	439.06
Total	2187.83	10433.05

Table 6 - Life cycle assessment of retrofitting the Masonry building.

The evaluation of the life cycle assessment for the reinforced concrete and masonry buildings in terms of ecological cost and carbon footprint following EN 15804 standard shows that it will cost more to reduce the environmental pollution and materials depletion from the reinforced building than the masonry building at a cost difference of 114 euros and an equivalent carbon emission of 403 kilograms respectively.

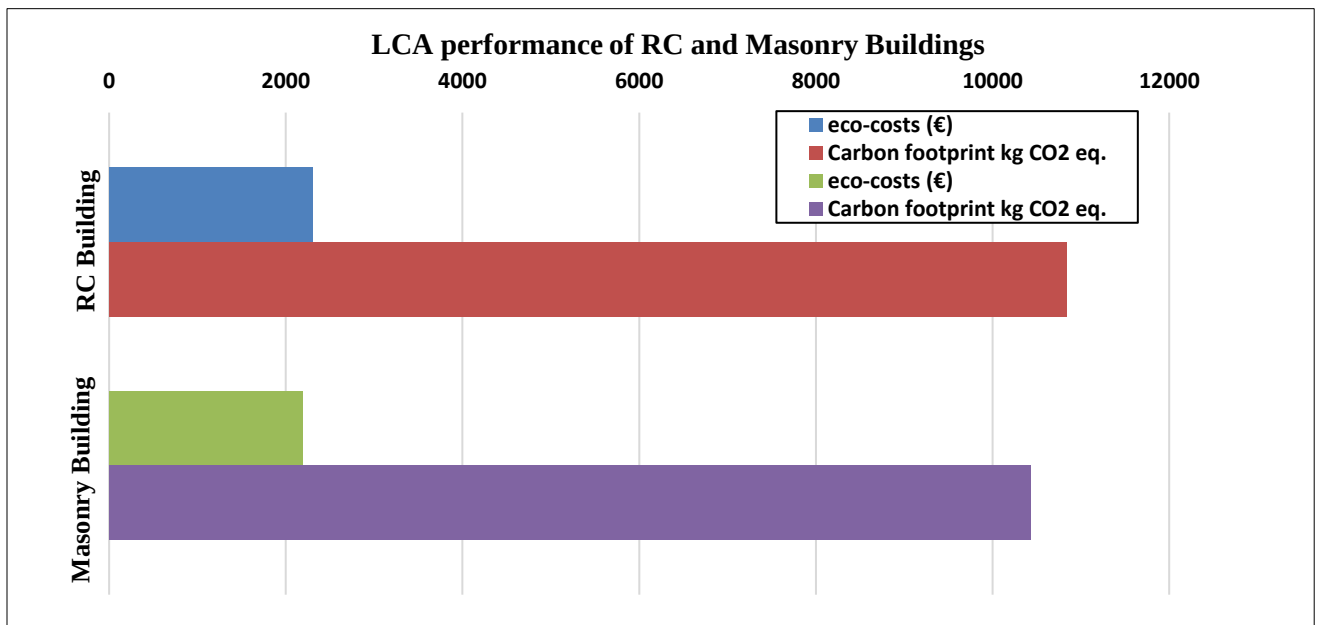


Figure 41 - Life cycle assessment performance difference of retrofitting the reinforced concrete building and the masonry building with their level of environmental pollution and materials depletion effects.

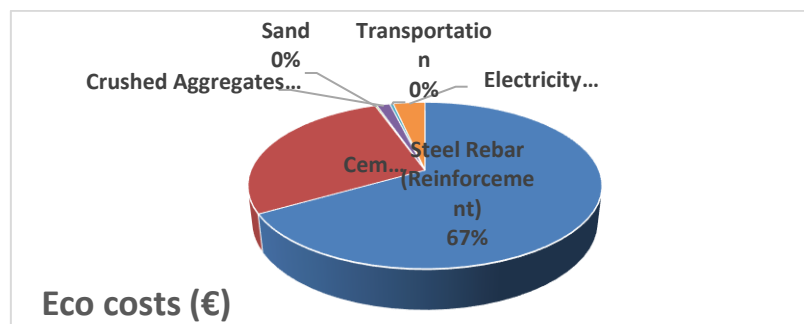


Figure 42 - Ecological cost of retrofitting the reinforced concrete building showing the percentage contribution of each indicator and parameter.

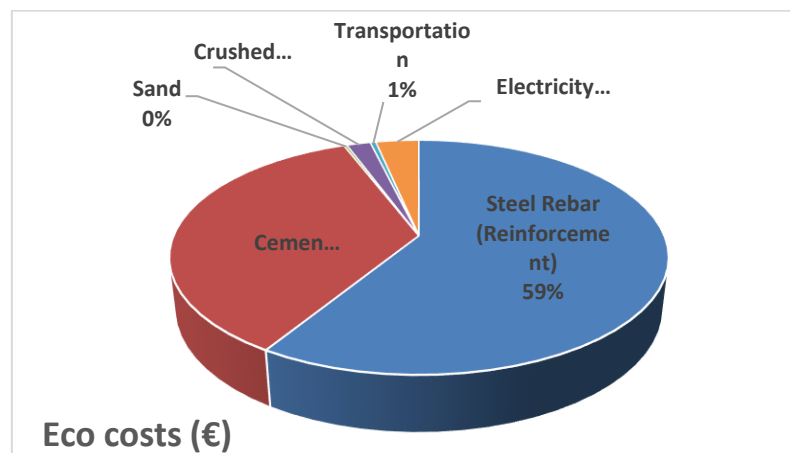


Figure 43 - Ecological cost of retrofitting the Masonry building showing the percentage contribution of each indicator and parameter.

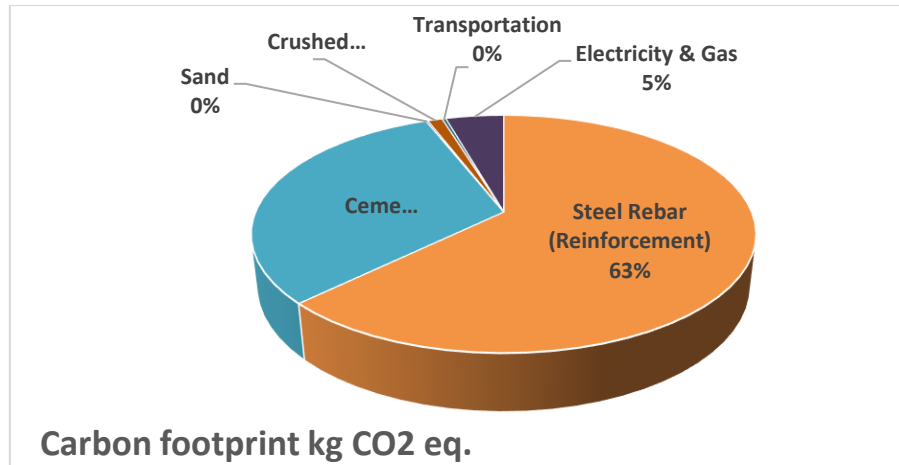


Figure 44 - Equivalent Carbon footprint (CO₂) of retrofitting the reinforced concrete building showing the percentage contribution of each indicator and parameter.

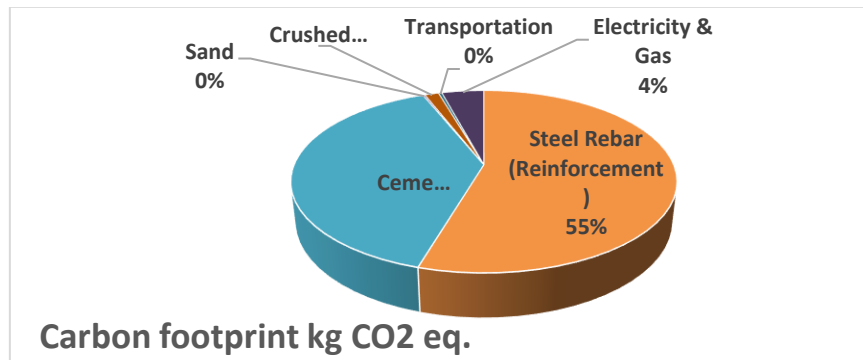


Figure 45 - Equivalent Carbon footprint (CO₂) of retrofitting the masonry building showing the percentage contribution of each indicator and parameter.

From the result above, the masonry building has higher deflection which will possibly incur more damage than the reinforced building. Most of the equivalent carbon footprint generated is majored more from the steel reinforcements followed by cement which is also the major constituent of concrete which of course has a larger percentage of environmental impact on the surrounding areas. In terms of ecological cost comparison, the difference in cost of stabilizing and preventing the effect is environmental pollution is negligible which implies that reinforced concrete building is better off in seismic-prone areas since they can withstand more forces and is less cost-effective in repairing damages more than the masonry building since it will still bounce back to repairs that has to do with reinforcing members with concrete and rebars to partially replaced the bricks walls.

CHAPTER FIVE

5 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This research sort to evaluate the site response of masonry and reinforced concrete buildings in consideration of soil properties and environmental impact assessments of retrofitting the failed members by modelling two buildings with different material properties using AxisVM, STRATA, OpenLCA and IdematLCA software respectively.

Hence, the site-specific acceleration response spectrum was evaluated using Strata software, providing a 1-dimensional response for ground motion data amplification with soil strata effects obtained from the site in accordance with Eurocodes 7 and 8, respectively. The local site-specific spectra were compared with the Eurocode 8 design spectra, and the difference in the two spectra was clearly noted.

The data from the amplification, considering the highest local spectral values, were fed into the AxisVM software. This finite element software is used for structural design, taking into account two different building types (unreinforced masonry and reinforced concrete structures) for a six-story building based on Eurocodes 8 and 2. These buildings were designed using the determined site-specific acceleration response spectrum from STRATA to study the effects of site and design responses for those buildings.

Results obtained show that the masonry building is in the most vulnerable condition compared to the reinforced building. To assess the overall performance of the buildings, the horizontal displacements of the reinforced concrete structure and the masonry building were compared. The maximum displacements at the designed spectrum were 48.13 mm and 79.28 mm for the reinforced concrete structure and 91.41 mm and 162.583 mm for the masonry building, respectively, for the site-specific spectrum. This reveals a clear overall difference in the horizontal deflection of 31.15 mm and 71.17 mm.

In the masonry building, walls were observed to have sheared into multiple zones, causing the entire top and bottom joints to be in tension instead of compression, exceeding the capacity and hence resulting in total failure. In the reinforced structure, the members were able to withstand the loadings and seismic forces at the design spectrum specified in Eurocode 8 under soil types C. Under consideration of the local spectra for the reinforced design, it was noted that all columns supporting the structure yielded a greater deflection, causing all the columns to exceed the designed capacity resulting in fail as well. A decision was reached based on the program specifications to strengthen the failed members, proposing redesigned methods of increasing the member stiffness.

For the reinforced building, 230 x 450 mm sizes were proposed and applied as against the initial columns' sizes of 230 x 230 mm which was deemed the most ideal, and it has been proven in the remodeled building that columns of dimensions 230 x 450 mm can withstand and resist seismic shock forces by significantly reducing the horizontal deflection to 52.97 mm against the initial deflection of 79.25 mm. In the case of the masonry building, shear walls were applied at major corners of the masonry building to act as resistance and braces to seismic forces which also significantly pulls down the horizontal deflections from 162.583 mm to 37.78 mm.

Environmental Impact Assessment (EIA) was conducted on the redesigned buildings (Masonry and RC) to ascertain their scrap value and impact on the supporting environment, using OPENLCA and IdematLCA software, professional Life Cycle Assessment (LCA) and Footprint software, was employed as a tool to estimate the effect of strengthening the failed members and calculate the effect of the environmental burden of a product on the basis of preventing that burden in terms of ecological costs.

Results of the ecological cost and environmental impact assessment show that rebars are consumed more in the reinforced building than in the masonry building, and concrete is consumed more for the masonry building in terms of retrofitting.

5.1 Conclusions

In this study, a 6-story RC and masonry buildings have been assessed by determining site soil properties effect on seismic load amplification and consequently the redesign and environmental impact and the followings were noted:

- The determined site-specific response spectrums have been seen higher than the Eurocode specification.
- In terms of the performance of buildings, the RC building has shown more resistance to seismic forces by yielding less deflection.
- When the site-specific response spectrum is considered for analysis of buildings the load-carrying elements (e.g., columns, walls) have exceeded.
- Environmental impact assessment of redesigned failed members for both building types has not shown a significant difference in terms of eco-cost and carbon footprint.

In conclusion, it is better to construct reinforced buildings in high seismic zones, as they have the greater advantage of withstanding the forces within the most peak response periods compared to masonry buildings.

5.2 Recommendations

Local site-specific design should also be considered in designing buildings in seismic-prone zones to ascertain the exact behavior of the structure under seismic loading. This is necessary because, in most cases, the design spectrum specified by Eurocode 8 may fall short of the actual reality on the ground.

Since in this study, only considered one building it is noteworthy to recommend the consideration of buildings with different numbers of stories in future investigations.

Adequate care should be taken when constructing masonry buildings in seismic zones going over two stories. The walls should always be filled with mesh and reinforcements to contribute to resisting tension forces, as the walls or bricks will be under a constant compression force.

Furthermore, the environmental impact should also be assessed to understand and demonstrate which methods and materials are more sustainable in terms of ecological costs, social costs, and environmental carbon footprint before full site mobilizations.

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