

# **Investigation of Seismic Structural Response of Reinforced Concrete Buildings with Fixed Support and Foundation in Relation to Soil Properties and Life Cycle Cost Analysis**

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## **Abstract**

Recent earthquakes such as the 2023 Türkiye-Syria, Morocco, Afghanistan, 2015 Gorkha Nepal, 2009 Indonesia earthquake have shown the vulnerability of existing building stock. Throughout Europe, many existing buildings were constructed considering low or moderate standards, or without considering them. Therefore, this study investigates the seismic performance of reinforced concrete (RC) buildings in Hungary, exemplified as a six-story RC dormitory building, focusing on structures connected by fixed support, which present unique challenges under earthquake loads (various seismic scenarios – considering different peak ground acceleration values ranging from 0.1g to 0.6g representing seismicity of Europe), soil characteristics (soil type C: dense sand, gravel, and stiff clay), and site seismicity scenarios representing seismicity of Europe. The research aims to assess the potential effects of exceeding anticipated site seismic intensities, potentially leading to safer communities and infrastructure in the face of impending earthquakes. AutoCAD is used for architectural drawing and Robot Structural Analysis Professional software is used for structural analysis and design throughout soil-structure interactions and site seismicity considerations. Moreover, this study investigates the environmental implications and long-term viability of RC buildings, a representative of future building inventory in Europe, by conducting a life cycle assessment considering the environmental impact resulting in different amounts of material usage to design structures for complying with design standards (Eurocode 8). The methodology employed in this investigation aligns with the core principles of practical design encompassing economic and environmental sustainability.

**Keywords:** Seismic risk, reinforced concrete, fixed support, soil-structure interaction, sustainability, Hungary, earthquake design standards.

## **1 Introduction**

From ancient cities brought to ruin to modern tremors such as the Kathmandu (Nepal, 2015), Bam (Iran, 2003), Tangshan (China, 1976), Yogyakarta (Indonesia, 2006), Baghdad (Iraq, 2017), Ancona and Fano (Irrpinia, Italy 1930), causing widespread disruptions, earthquakes have played a significant role in shaping the physical and cultural landscape of the

world. Europe, often viewed as a continent of historical charm and stability, holds a hidden reality, it is actively shaped by earthquakes, recent years in Italy have seen a surge in data collection on earthquake damage to buildings[1]. This data, gathered from post-earthquake surveys, is fueling research efforts in the field of seismic risk assessment for existing structures. By analyzing past damage patterns, researchers can gain valuable insights into building vulnerabilities and develop targeted strategies for mitigating earthquake risk [2]. On April 6, 2009, a powerful earthquake struck L'Aquila, the capital of the Abruzzo region in central Italy, the powerful earthquake struck the Abruzzo Apennines in central Italy. A 6.1 magnitude tremor rocked the mountainous region, leaving devastation in its wake. L'Aquila, the main town in the area, and neighboring villages bore the brunt of the disaster, with around 300 people losing their lives [3]. The psychological trauma and emotional toll on survivors were immense, and the city faced a long road to recovery. Meanwhile, in February 2023, a devastating earthquake struck southeastern Türkiye (figure 1) and northern Syria, claiming over 50,000 lives and causing widespread damage [4]. As the deadliest earthquake in Turkey since 1939, it prompted a global response from disaster experts, including the Earthquake Engineering Field Investigation Team (EEFIT), the Geotechnical Extreme Events Reconnaissance Association (GEER), and the Earthquake Engineering Research Institute (EERI). Within a week, a team of 30 engineers and researchers from various countries deployed to assess the damage, analyze the impacts, and recommend strategies for reducing future vulnerabilities [5] (Y. D. Aktas).



*Figure 1. Damaged building after the 6 February 2023 Türkiye-Syria Earthquake (source: Magdalene College – University of Cambridge [6])*

In Hungary, the 1763 Komárom earthquake, the largest recorded in the Pannonian Basin in the last millennium, left a wealth of historical documentation for analysis. Building damage reports, aftershock observations, liquefaction reports, and economic impact surveys from Komárom, Győr, and surrounding villages provide valuable datasets for assessing the impact of the quake. While detailed damage records are limited to these two cities, a broader spatial distribution of shaking intensity can be inferred from other sources. In particular, the spatial patterns of secondary environmental effects, aftershocks, and village damage show interesting correlations, suggesting an epicentral location west of Komárom, north of the Danube River [7]. This earthquake is a reminder of the region's seismic vulnerability and the importance of preparedness. Studies of the 1763 Komárom earthquake contribute to the understanding of the historical earthquake activity in the Pannonian Basin and to the hazard assessment of future events.

After a major earthquake, assessing the damage to buildings is critical to managing the immediate crisis and recovery efforts. This is where post-earthquake surveys come in. These

surveys serve two main purposes (1) Determine building functionality: Assessing whether a building is still safe for use helps guide early response efforts such as search and rescue, evacuation, and possible demolition of unstable structures. (2) Assess damage for financial assistance: Assessing the extent of damage provides critical information for calculating the maximum financial assistance the government can provide for repairs or rebuilding [8]. The reality of the situation is that many European buildings are at risk from earthquakes as a consequence of their age and outdated design [9]. These buildings were built with outdated standards since many existing buildings were constructed without considering modern seismic design standards, making them vulnerable to the seismic loads imposed by these earthquakes (Valentina et al., 2018). Residential buildings dominate the global construction landscape, accounting for a staggering 90% of all existing structures. However, this dominance comes at a potential cost. These residential buildings are also disproportionately vulnerable to earthquakes, accounting for about half of the total losses from seismic events [10]. This vulnerability underscores the critical need to prioritize seismic design and construction practices in the residential sector. By implementing such practices, we can significantly improve the safety and resilience of communities in earthquake-prone regions. Over time, as buildings are used for years or decades, they experience wear and tear. Even if these structures were originally constructed to meet certain standards for energy efficiency and seismic safety, the effects of aging can gradually compromise these attributes. For example, insulation materials may deteriorate, resulting in reduced energy efficiency, and structural components may weaken, reducing the building's ability to withstand seismic forces. As a result, despite their original design intent, older buildings may become less energy efficient and more susceptible to earthquake damage as they age (Menna et al., 2022). Moreover, the built structures are reaching their limit as some buildings have simply reached the end of their design life, further increasing their vulnerability. These factors combine to create a significant seismic risk for many European structures, highlighting the need for evaluation and potential upgrades [11].

With the ever-present threat of earthquakes, it is imperative to design and construct structures that have the strength to withstand these devastating natural phenomena. The unpredictable nature of these seismic events underscores the critical need to exceed current seismic design standards. While Eurocode regulations provide a valuable tool for earthquake preparedness, the inherent uncertainty in earthquake intensity requires structures that can withstand even greater shaking than that anticipated by the established codes. By incorporating additional safety margins and implementing advanced seismic technologies, we can protect communities and infrastructure from the potentially catastrophic consequences of unanticipated seismic activity. Through this proactive approach, we implement mitigation strategies that enhance the structural integrity and seismic resilience of our built environment, enabling it to effectively withstand the unpredictable and potentially destructive forces of natural phenomena. During earthquakes, the geologic composition of the soil beneath a structure has a significant effect on the amplification or attenuation of ground motion. Several key soil characteristics affect how earthquakes affect a particular area, including

1. Soil type: Different soil types (e.g., sand, clay, rock) have different characteristics that affect their response to seismic waves. Soft, saturated soils such as loose sand tend to

amplify ground motion, while stiffer soils such as rock offer more resistance and dampen the intensity of shaking.

2. **Density:** Denser soils, with less pore space between particles, generally transmit seismic waves more efficiently, resulting in stronger ground motions. Loosely packed soils with more voids can absorb some of the energy from the waves, reducing their intensity.
3. **Water content:** Saturated soils, where water fills the pore spaces, are more susceptible to liquefaction, a phenomenon in which the soil loses its strength and behaves like a liquid. Liquefaction can cause severe damage to foundations and buried structures.
4. **Topography:** The topography of the land can also affect the effects of an earthquake. Hillsides and slopes with loose soils are more prone to landslides triggered by ground shaking.

The characteristics of the soil then have a direct influence on the behavior and propagation of earthquake-induced ground motions, including

1. **Amplification:** Soft, saturated soils can amplify seismic waves by resonance, similar to how sound waves are amplified at a particular tuning fork frequency. This can significantly increase the intensity of ground motion experienced at the surface.
2. **Attenuation:** Denser, stiffer soils tend to attenuate seismic waves, i.e., the wave energy is absorbed or scattered, reducing the intensity of ground motion reaching the surface.
3. **Site Response:** The combined effect of soil properties such as density, type, and topography on a given site is known as site response. This response can significantly affect the amount of ground shaking experienced at that location during an earthquake.

Seismicity, the frequency and intensity of earthquakes in a region, presents both challenges and opportunities for sustainable development. Earthquakes can destroy infrastructure, including buildings, transportation networks, and power grids. This disrupts daily life, hinders economic activity, and hinders access to essential services. Rebuilding infrastructure requires significant resources, potentially diverting funds from other sustainability initiatives, since repair activities could exceed 40% of the initial cost [12].

Driven by the constant threat of unpredictable and potentially devastating earthquakes, this study deals with the seismic performance of reinforced concrete buildings in Hungary. Specifically, we will study and investigate the structural response under different scenarios, including buildings connected by a fixed support, as this configuration presents a unique challenge in terms of force distribution and stress accumulation during seismic activity. This study will analyze how the interconnection affects the overall stability and potential failure modes compared to isolated buildings. This research will use similar soil characteristics by focusing on a specific soil type that allows for a more controlled and targeted investigation of the influence of soil conditions on seismic response. By controlling this variable, we can isolate the effect of building configuration and design on seismic performance. Furthermore, this study considers multiple Peak Ground Acceleration (PGA) values which will have earthquakes vary in intensity, and exceed predicted magnitudes considering that is not unpredictable/ uncommon. This research will investigate how structures respond to different levels of PGA beyond what is typically expected by the Eurocode standards. It will evaluate the safety margins built into current design approaches. By using these different scenarios and rigorous analysis, this study aims to gain a comprehensive understanding of how reinforced concrete buildings in Hungary will perform during earthquakes. This knowledge is critical for Identifying potential

vulnerabilities and weaknesses in existing structures. The study aims to inform future design and construction practices for improved earthquake resistance, develop effective retrofit strategies for existing buildings to improve their safety and provide more accurate and reliable seismic risk assessments for Hungarian communities. Ultimately, this study contributes to a future where the unpredictability of earthquakes does not translate into unacceptable risks to life and infrastructure. By building on the knowledge gained, we can create safer and more resilient communities in earthquake-prone regions.

In this study, we will leverage the capabilities of two software programs to design and analyze a six-story dormitory building. For the architectural drawings, we turn to AutoCAD (Autodesk, Inc.), a widely used software renowned for its precision and flexibility in creating detailed plans, elevations, and sections. This will allow us to meticulously capture the desired features and functionality of the dormitory, ensuring a clear and comprehensive representation of the building's design intent. Moving beyond the aesthetics, we will employ Robot Structural Analysis software (Autodesk, Inc.) to delve into the structural integrity of the building. This robust program will empower us to perform rigorous analyses across five distinct scenarios, each encompassing six individual cases. By systematically varying key parameters, we will gain a nuanced understanding of the building's behavior under different loading conditions. The first scenario will examine the built structure with fixed support, subjecting it to increasing PGAs ranging from 0.1g to 0.6g. This simulates the response of the building to earthquake ground motions, providing valuable insights into its seismic performance. The subsequent scenarios will incorporate the complexities of a support and soil type C. We will meticulously model the building's interaction with the ground, considering four distinct soil conditions: gravel, sand, clay, and silt. Each scenario will also explore six different cases within the given soil type based on the range of PGAs ranging from 0.1g to 0.6g, allowing for a comprehensive evaluation of the building's response to various soil-structure interactions. The purpose of this study is to analyze and compare the environmental footprint resulting from the various combinations discussed above. By examining these combinations, we aim to identify the most sustainable options and assess their potential impact on the environment. Sustainability will be a central focus of the investigation as we strive to understand how each combination affects the environmental footprint and identify opportunities for more environmentally friendly practices. Through this meticulous analysis framework, we aim to gain a comprehensive understanding of the structural behavior of the built structure under diverse loading conditions and soil interactions. This knowledge will be instrumental in optimizing the design for safety, efficiency, and resilience, ultimately contributing to the creation of a safe and **sustainable built structure**.

## 2 Soil Properties

Eurocode 8 (EC8), a European building code for earthquake resistance, establishes a system for classifying ground conditions based on their seismic behavior. It defines five standard ground types (A to E) that account for the influence of local soil properties on earthquake effects. Additionally, the code recognizes two special ground types (S1 and S2) that might require unique considerations in seismic design [13]. According to EC8, the following table illustrates each category of ground types identification.

Table 1. EuroCode 8 categorization of ground type and its soils

| Ground type | Description of stratigraphic profile                                                                                                                                                         | Parameters            |                           |             |
|-------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------|---------------------------|-------------|
|             |                                                                                                                                                                                              | $v_{s,30}$ (m/s)      | $N_{SPT}$<br>(blows/30cm) | $c_u$ (kPa) |
| A           | Rock or other rock-like geological formation, 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 metres in thickness, characterised by a gradual increase of mechanical properties with depth.              | 360 – 800             | > 50                      | > 250       |
| C           | Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.                                                               | 180 – 360             | 15 - 50                   | 70 - 250    |
| D           | Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.                                                   | < 180                 | < 15                      | < 70        |
| 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, 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<br>(indicative) | –                         | 10 - 20     |
| $S_2$       | Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or $S_1$                                                                            |                       |                           |             |

In simple terms, EC8 categorizes soils based on their shear wave velocity ( $V_s$ ), a critical parameter for understanding how earthquake shaking propagates through the ground. The general explanation of each soil type with additional considerations includes:

1. Type A soil (rock or very dense granular materials with  $V_s \geq 800$  m/s), represents the most stable foundation condition. Rock provides the highest  $V_s$ , which means minimal ground vibration amplification and excellent load bearing capacity.
2. Soil Type B (Dense to very dense granular materials or stiff cohesive soils with  $360 \text{ m/s} \leq V_s < 800 \text{ m/s}$ ), considered to be very good foundation materials. Dense granular soils such as gravel or very stiff clays have good drainage and high bearing capacity. However, the specific characteristics within this category can vary. For example, a very dense layer of sand may behave differently from stiff clay in terms of earthquake response.
3. Soil Type C (Medium dense granular materials or cohesive soils with  $180 \text{ m/s} \leq V_s < 360 \text{ m/s}$ ) have moderate stiffness and drainage characteristics. Moderately dense sands or cohesive soils with some plasticity fall into this type. While these soils may provide adequate support for structures, they may experience some settlement under load and require more detailed investigation for seismic design than Types A and B.
4. Soil Type D (Loose granular materials or soft cohesive soils with  $V_s < 180 \text{ m/s}$ ) are most susceptible to amplification of earthquake shaking due to their low stiffness. Loose sands or

soft clays with high water content fall into this category. Foundations on these soils require careful design considerations, possibly including soil improvement techniques or deeper foundations, to mitigate settlement and ensure stability during earthquakes.

5. Soil Type E (Very soft soils and sensitive clays), represents the most challenging soil conditions for seismic engineering. Very soft clays or organic soils with extremely low  $V_s$  and high-water content can experience significant amplification and liquefaction (loss of strength) during earthquakes. Special foundation design approaches and advanced geotechnical analysis are often required for safe construction in these soils.
6. Site Condition S1 (Rock sites with very shallow overburden), represents sites where the underlying rock is near the surface with minimal overburden. Rock sites experience minimal amplification of earthquake shaking due to the high stiffness of the underlying bedrock.
7. Site Condition S2 (other site conditions not classified as S1), includes a wider range of soil profiles and potential for ground motion amplification. Sites with deeper soil layers, varying soil types within the profile, or topographic features such as slopes may fall into S2.

Understanding the properties of the underlying soil and the broader site conditions is paramount in earthquake engineering for several reasons, as outlined in EC8. These reasons include the seismic design parameters, as the soil type (A to E) and site classification (S1 or S2) play a critical role in the selection of appropriate design response spectrum for a given site [13], [14]. These response spectra, defined in EC 8, represent the expected characteristics of ground motion (frequency and acceleration) during different earthquake scenarios. This information is then used for structural analysis and design to ensure that the building can withstand the expected earthquake forces. Subsequently, the critical role of soil classification in seismic design extends to foundation design, where soil mechanical properties strongly influence the selection of appropriate foundation types (shallow vs. deep) and their design parameters (Kramer, 1996) [15]. Stiffer soils (Types A and B) with higher bearing capacity allow for shallow foundations. Conversely, softer soils (Types C, D, and E) with lower capacities may require deeper foundations or soil improvement techniques such as soil compaction or grouting to ensure stability during earthquakes. Finally, consideration of soil properties and site conditions is essential for structural design. This is because the interaction between a building and the soil on which it rests significantly affects its seismic performance [15]. By understanding how the soil behaves during an earthquake, engineers can design structures that act as efficient channels, safely transferring seismic forces through the foundation and minimizing damage to the upper parts of the building.

### **3 Seismicity**

Earthquake Hazard Maps for Europe play a critical role in assessing seismic risk and guiding mitigation efforts. These maps show the expected level of ground shaking PGA at different locations due to potential earthquakes, both local and distant. The PGA values, expressed as a percentage of the Earth's gravitational acceleration ( $g$ ), are derived from sophisticated models such as the recently updated European Seismic Hazard Model 2020 (ESHM20) [16]. ESHM20 incorporates a comprehensive analysis of historical earthquake data, geological information and active fault systems across Europe. Through complex calculations, the model estimates the probability of earthquakes occurring in different regions and their expected magnitudes. Based on these predictions, the model then simulates the intensity of

ground shaking at various points across the continent. Figure 1 represents the European seismic hazard map in 2020 by the "European Facilities for Earthquake Hazard and Risk (EFEHR)" [16]. The PGA values on the map range from 0 to 0.6 g.

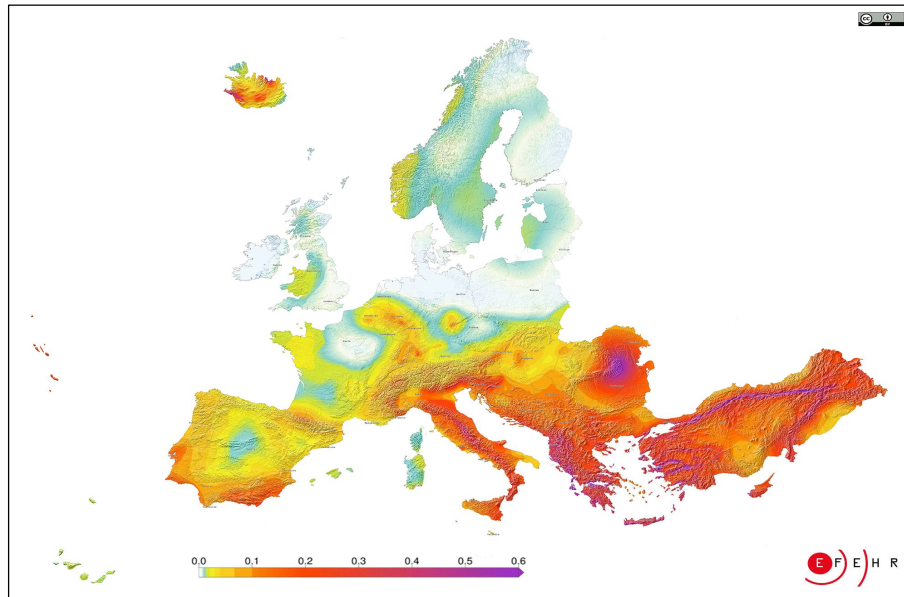


Figure 2. Earthquake Hazard map of Europe 2020 (source: European Facilities for Earthquake Hazard and Risk (EFEHR) [16])

This Earthquake hazard map (figure 2) uses a vivid color scheme to communicate the intensity of shaking expected in different areas. The colors of white ranging to green shades indicate low hazard, with minimal shaking expected, yellows and oranges represent moderate hazard, indicating a greater potential for shaking, and red to purple indicates high hazard, with the potential for significant shaking. These color-coded zones correlate with building design codes. Building codes in many regions require that earthquake-resistant structures, such as residential and commercial buildings, be designed to withstand a certain level of ground shaking. This level of shaking typically corresponds to a 10% probability of occurrence in a 50-year period [16], also known as the Design Basis Earthquake (DBE), as adopted by EC8. This reflects the design consideration in the seismic design standards of a standard residential building and translates to a return period of 475 years, meaning that the shaking intensity should only be exceeded, on average, once every 475 years [16]. The resulting earthquake hazard maps show the expected PGA values for different return periods (e.g., 50 years, 475 years). These maps provide valuable insight into areas likely to experience greater or lesser ground shaking during future earthquakes.

PGA is a fundamental parameter used to quantify the intensity of earthquake ground motion. PGA is typically measured in units of g (acceleration due to gravity) [17]. Higher PGA values correspond to more violent ground motions, with a greater potential for damage to buildings and infrastructure. PGA plays a critical role in earthquake engineering through two primary applications, including structural design and damage assessment. In structural design, engineers use PGA values to evaluate the seismic forces acting on structures. This information is critical for designing buildings and infrastructure that can withstand earthquake shaking without collapsing. Building codes in different regions often specify design PGA values based

on historical seismic activity and risk assessment. In terms of damage assessment, PGA can be used to estimate the potential level of damage that an earthquake could cause to existing structures [18]. This information is critical for post-earthquake response and recovery efforts by prioritizing damaged buildings for inspection and repair [18].

EC 8 includes importance factors ( $\gamma_i$ ) to account for the varying seismic vulnerability of buildings. These factors are assigned based on the societal importance of a building, the potential loss of life in an earthquake, and the economic consequences. Higher factors are assigned to buildings with greater societal importance (hospitals, fire stations:  $\gamma_i = 1.4$ ) compared to buildings with less importance (warehouses:  $\gamma_i = 0.8$ ). This prioritizes life safety and critical infrastructure by classifying buildings into four categories: buildings of lesser importance (warehouses,  $\gamma_i = 0.8$ ), ordinary buildings (baseline,  $\gamma_i = 1.0$ ), buildings important for seismic resistance (schools,  $\gamma_i = 1.2$ ), and buildings important for disaster response (hospitals,  $\gamma_i = 1.4$ ). This system ensures that buildings are designed to withstand earthquakes based on their societal importance [13].

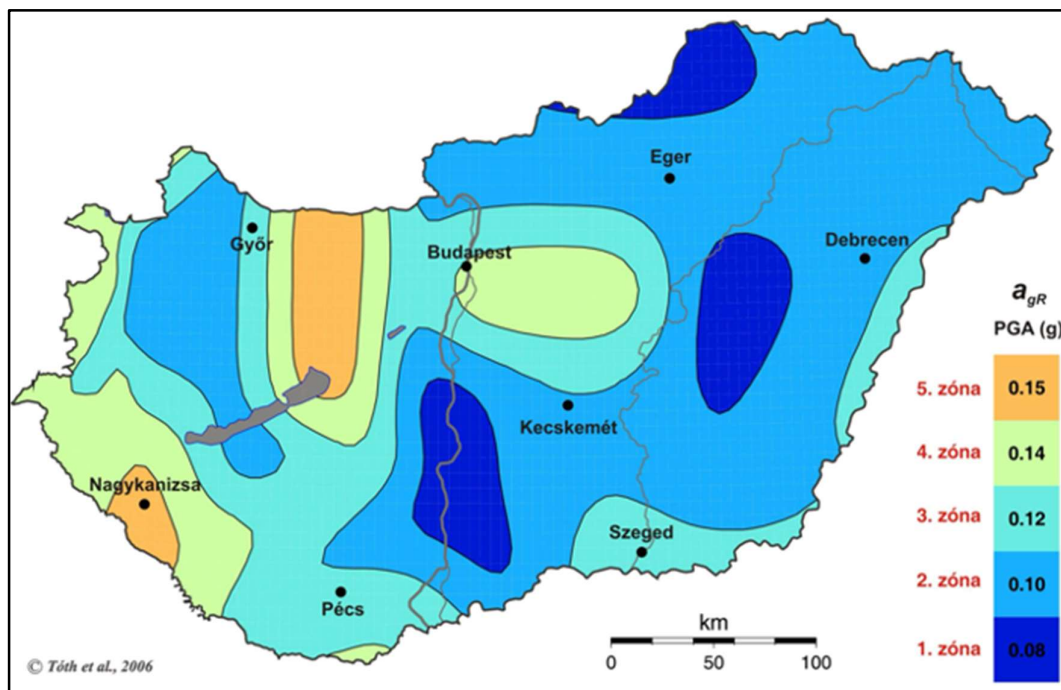


Figure 3. Earthquake Hazard map of Hungary (source: *Seismic Risk and Disaster Management – Kegyes Brassai* [6])

A closer look at Hungary's earthquake hazard map (figure 3) reveals five distinct zones (indicated by color) with PGA values ranging from 0.08g (blue) to 0.15g (orange). Notably, the map highlights a zone of heightened seismic activity just west of Győr, running along a north-south axis. Historical data confirms this pattern, indicating that past major earthquakes in this area significantly impacted buildings in both Komárom and Győr [19]

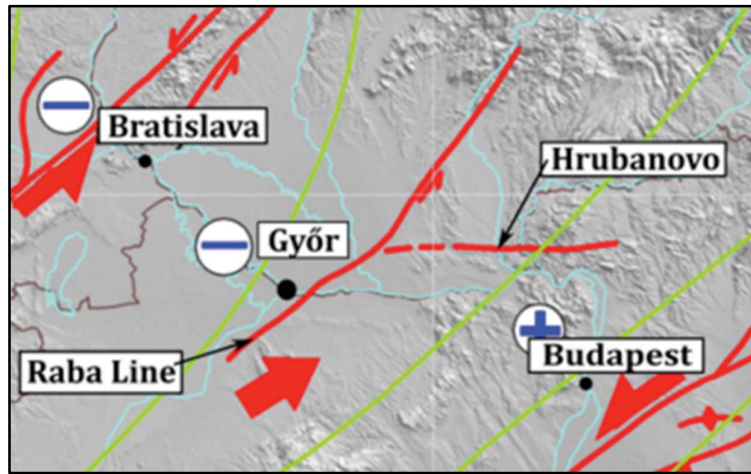


Figure 4. Earthquake Fault Line map of Hungary (source: : Seismic Risk and Disaster Management – Kegyes Brassai [6])

Beyond the broad seismic zone, specific faults further elevate the earthquake hazard for Győr. One such fault (figure 4), the Rába Line, runs beneath the Rába River and intersects with the Diósjenő-Őgyalla (or Hrubanovo) fault near Győr, located under the Danube River. Historical earthquake activity centered around Győr is primarily attributed to these faults [19].

In the field of earthquake engineering, the Design Response Spectrum has become the preeminent method for the dynamic evaluation of structures subjected to seismic loads. In this study, response spectrum analysis was used to evaluate the seismic response of the building. The response spectra were generated using a range of PGA values from 0.1g to 0.6g. An importance factor ( $\gamma_I$ ) of 1.2 was used to reflect the increased seismic design requirements for the tall building due to its importance. In addition, 5% viscous damping, characteristic of reinforced concrete (RC) structures, was included. Finally, the analysis considered a soil type C, consistent with the case study site in Hungary.

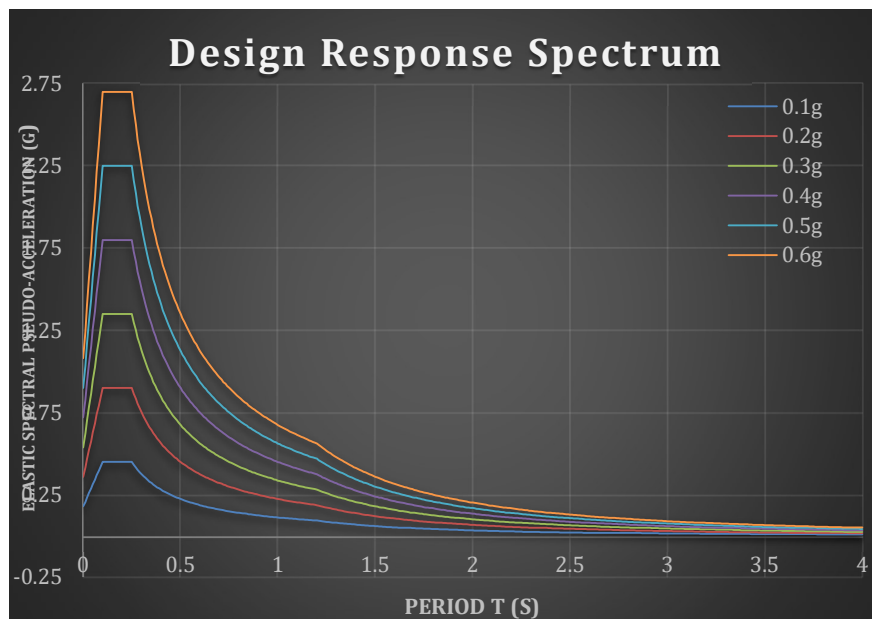


Figure 5. 0.1g to 0.6g design response spectrum (source: edited by authors)

As shown in Figure 5, the design response spectra represent the variation of the elastic spectral pseudo acceleration ( $S_a$ ) with respect to the structural vibration period ( $T$ ). Six different spectra are shown, each corresponding to a PGA value ranging from 0.1g to 0.6g in 0.1g increments. The spectra show a characteristic peak at a period of 0.1s. There is also a clear trend of increasing  $S_a$  with increasing PGA. The  $S_a$  values at the peak period progressively increase from 0.45g for the 0.1g PGA spectrum to 2.7g for the 0.6g PGA spectrum, demonstrating a potential increase of 0.45g for each 0.1g increment in PGA.

## 4 Structural Modelling and Analysis

In this research, for our case study, we design a building model created using AutoCAD for the architectural drawings. To illustrate the concept, since our research investigates the seismic performance of RC buildings in Hungary, the case study uses a six-story RC dormitory building model. The design is inspired by a similar dormitory building at the researcher's university, Széchenyi István University of Győr. The architectural design specifies a total building height of 21 meters with a rectangular footprint measuring 14.5 meters by 43 meters. A figure below (figure 6) showcases the CAD model of this example building.

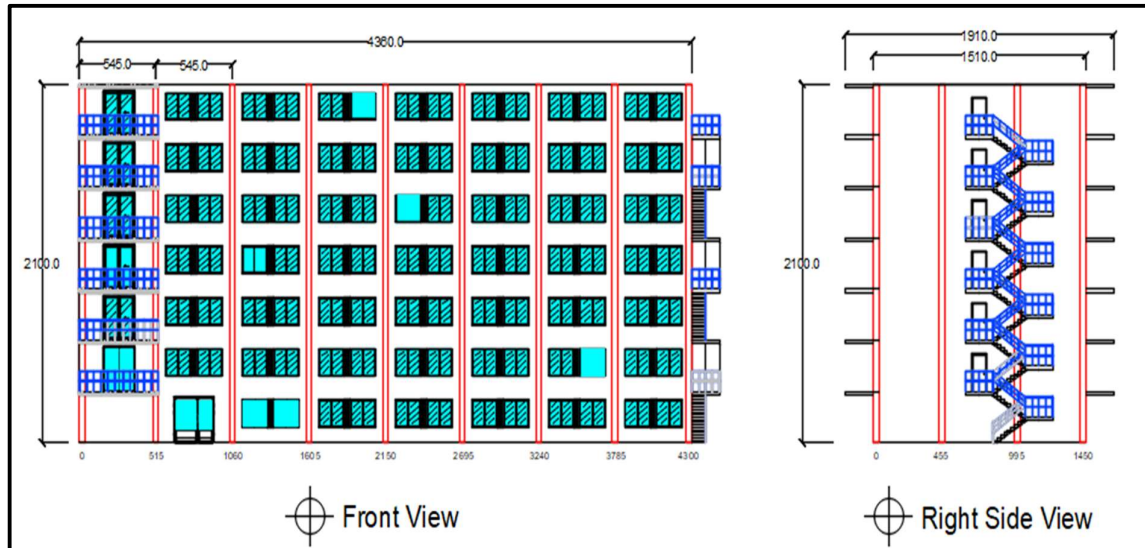


Figure 6. Front and Right Side view of RC dormitory structure (source: edited by authors)

While the building depicted in the figure avoids vertical irregularities in its design, it does have an irregularity in its plan. This is due to the significant difference in the dimensions of its short side compared to its long side. As a result, the building may be more susceptible to seismic loads acting on its short side, potentially affecting its overall structural performance.

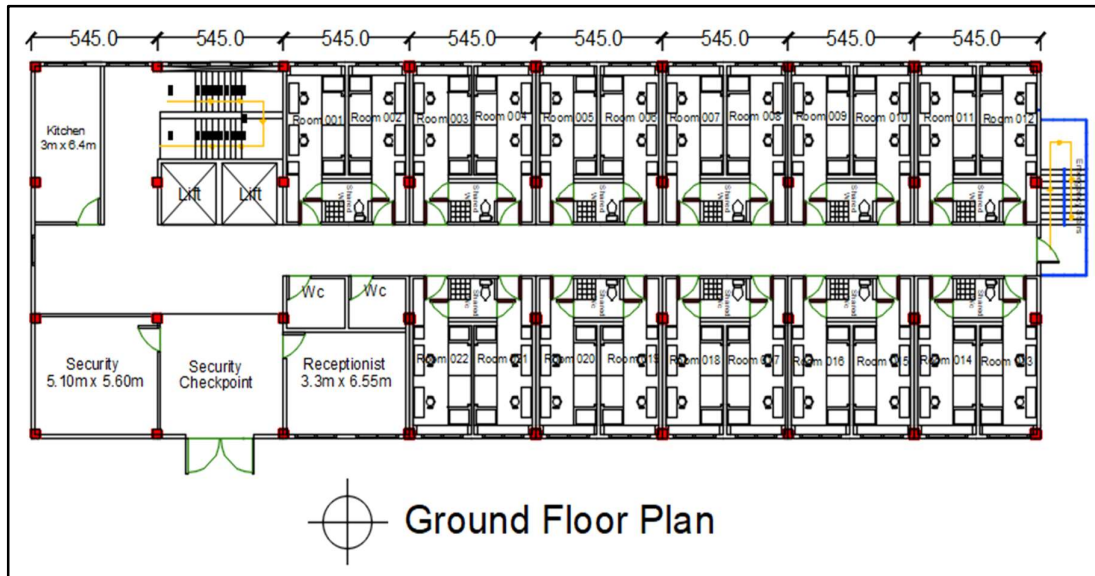


Figure 7. Ground Floor plan view of RC dormitory structure (source: edited by authors)

Reflecting its purpose as a dormitory, the building's floor plan (figure 7) incorporates features typical of such structures. Starting with the ground floor, it provides 22 shared rooms for residents. In addition, the ground floor provides essential common areas, including a main entrance with a security checkpoint, a reception desk, a security office, a kitchen, two common restrooms, two elevators, stairs, and an emergency exit with its own staircase. The layout for the first through sixth floors is the same as the ground floor, with 22 rooms each (Figure 8). However, these upper floors include functional spaces beyond the basic living quarters. These spaces consist of a common study room, a storage room, two balconies located on opposite sides of the building, and a dedicated laundry facility.

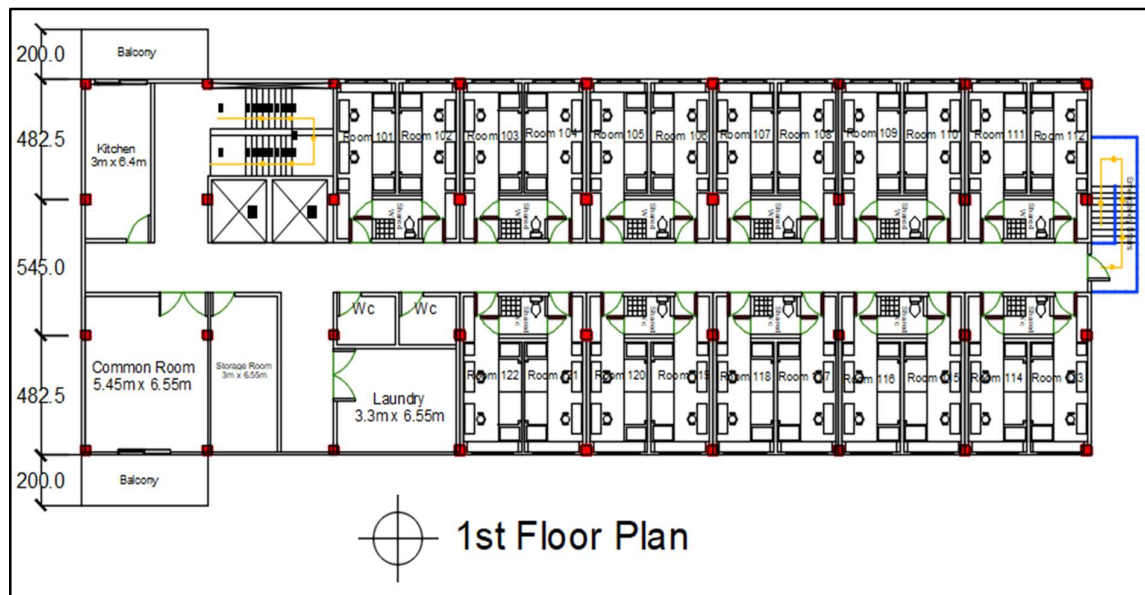


Figure 8. 1st Floor plan view of RC dormitory structure (source: edited by authors)

In total, the dormitory has 132 rooms with a capacity of 264 students. Shared facilities throughout the building include 14 restrooms, 7 kitchens, 6 common study rooms, 6 storage rooms and 6 laundry rooms. Accessibility is provided by two elevators and staircases, with a dedicated emergency exit with its own staircase. The ground floor houses the main entrance, reception desk, security checkpoint, and security office for overall building control. In addition, the building has 12 balconies strategically located on opposite sides of the building. Since the building emphasizes a regular design, both vertically and in its overall layout, the building design has favorable characteristics for potential modular expansion strategies. This prioritization of a simplified orthogonal layout facilitates the theoretical possibility of future implementation where multiple units could be stacked diagonally to create additional student housing.

Table 2. Performance level of built structures

| Performance Level       | Abv | EMS-98 [27] damage states | % inter-story drift ratio |
|-------------------------|-----|---------------------------|---------------------------|
| Operational Performance | OP  | No / Slight               | 0.5                       |
| Immediate Occupancy     | IO  | Slight / Moderate         | 1                         |
| Damage Control          | DC  | Moderate / Heavy          | 1.5                       |
| Life Safety             | LS  | Heavy / Very Heavy        | 2                         |
| Collapse Prevention     | CP  | Destruction               | 2.5                       |

This data table (Table 2) summarizes the findings from a scientific study on building performance during various stress levels. It categorizes building functionality based on five levels: Operational Performance (OP), Immediate Occupancy (IO), Damage Control (DC), Life Safety (LS), and Collapse Prevention (CP). Each level corresponds to a specific inter-story drift ratio, an engineering measure of how much floors move relative to each other. The table indicates the level of damage a structure can withstand at each drift ratio. For example, a building with an inter-story drift ratio of 1 (calculated based on deflection) might be classified as Immediate Occupancy (IO), meaning it could sustain minor to moderate damage. In contrast, a building designated as Life Safety (LS) with a drift ratio of 2 could experience significant or severe damage due to deflection. This table serves as a valuable resource for emergency responders and engineers who design safer structures for urban environments.

## 5 Environmental Impact Assessment

In contemporary society, the notion of sustainability (figure 9) has gained significant importance. It denotes the capacity to fulfill current requirements while safeguarding the ability of forthcoming generations to fulfill their own necessities [20]. It encompasses environmental, social, and economic considerations. Environmental sustainability focuses on using resources and energy in ways that minimize negative impacts on the planet. This includes issues such as mitigating climate change, reducing resource depletion, and controlling pollution [21]. Social sustainability considers the human element, focusing on fair labor practices, social equity, and community well-being [22]. In addition, economic sustainability ensures the financial viability of systems and practices so that they can be maintained over the long term (Asheim, 1994) [23]. These three pillars of sustainability are interrelated. For example, economic practices that are

highly polluting may not be sustainable in the long term. Similarly, social inequalities can hinder economic development.



Figure 9. Sustainability Venn-diagram (source: ConceptDraw – Sustainable Development [24])

One way to measure sustainability is through Life Cycle Assessment (LCA). LCA is a powerful tool for assessing the environmental impact of a product, process or service over its entire life cycle [25]. This includes all stages from raw material extraction and manufacturing to use, maintenance and end-of-life disposal [26]. Through LCA, we are able to identify areas for improvement and encourage the development of more sustainable products and systems.

This "full lifecycle" approach recognizes that each stage of a product's life is interrelated and can contribute to its overall environmental footprint. It's important to understand that LCA goes beyond simply adding up the impacts of each stage. It uses specialized tools (e.g., LCA Software tools "SimaPro", "OpenLCA", "OneClick LCA") and methodologies (e.g., Life Cycle Inventory (LCI), Life Cycle Impact Assessment (LCIA), Life Cycle Interpretation) to quantify and compare different environmental impacts from cradle to grave. These include not only obvious impacts such as energy consumption and emissions, but also hidden impacts such as resource depletion, water pollution and land use change. By providing a comprehensive and quantitative assessment of environmental impacts, LCA helps researchers, companies, and policy makers to (1) Optimize product design and process selection: Identify and minimize environmental hotspots throughout the product life cycle, (2) Improve environmental transparency: Communicate the true environmental impact of products and processes to stakeholders, and (3) Support informed decision making: Guide sustainable product development, resource management, and waste reduction strategies [27].

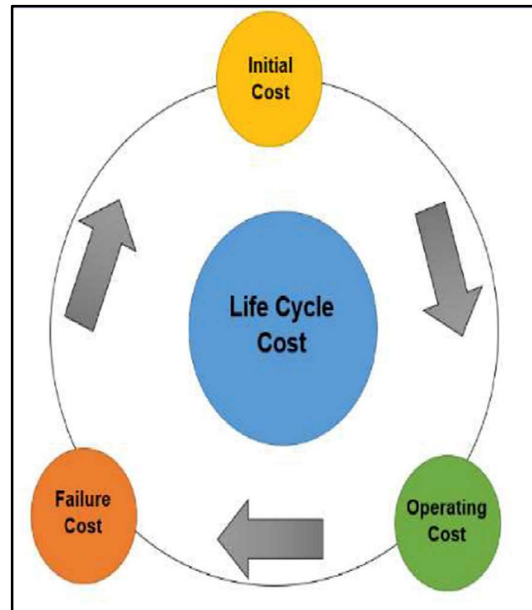


Figure 10. Life Cycle Cost framework categories (source: RailwayAge – Life Cycle Design [28])

LCC (figure 10) provides a framework for evaluating the total financial burden associated with a product or system over its lifetime. It includes three primary cost categories:

1. Initial Cost (IC): This cost reflects the up-front investment required to acquire the product. It includes the cost of raw materials, manufacturing processes, transportation to the intended location, installation, and any engineering or design fees associated with its development.
2. Operating Costs (OC): These are the ongoing costs incurred during the operational phase of the product to maintain its functionality. This category includes the energy consumption required for operation, the cost of routine maintenance to ensure proper functioning, the cost of consumables required for ongoing use, and the associated costs of waste disposal throughout the life cycle of the product.
3. Failure Cost (FC): This category represents the financial impact of product failures or breakdowns. These costs can manifest themselves in a variety of ways, including repairs necessitated by failures, replacement of defective parts, downtime resulting in lost productivity due to equipment failure, warranty claims that must be satisfied, and any associated disposal costs of the failed components.

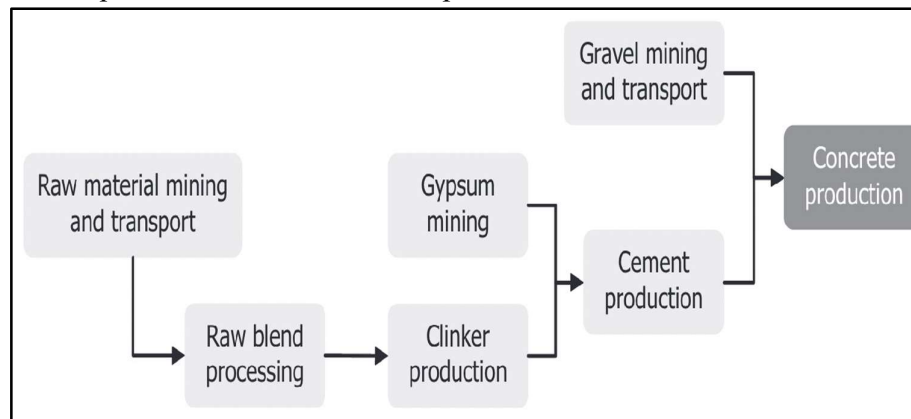


Figure 11. Concrete production flow diagram

In construction projects, LCCA is a key tool for assessing the sustainability of materials throughout their entire life cycle. This analysis is particularly important for concrete, a common building material. Shown in figure 11, the process flow diagram depicts the movement of materials throughout the concrete manufacturing process. It begins with the extraction and transportation of raw materials such as gravel from quarries and gypsum from mines. These processed materials are then mixed to produce clinker, a critical component of cement. This initial stage is directly linked to Life Cycle Cost Analysis (LCCA) through the cost of acquiring raw materials, as shown in the diagram. In essence, the cost of acquiring these essential ingredients for concrete contributes to the overall LCCA of the material. Following the raw material extraction stage, the diagram shows the processing of these materials. The mined gravel and gypsum will likely be crushed and screened to meet the specific particle size requirements for concrete production. However, before concrete is produced, cement must be manufactured. This process involves two key steps: first, raw material blending, where the processed gravel and gypsum are precisely proportioned and mixed to create a base for cement production. Second, clinker production, where this raw mixture is heated intensely in a kiln to form clinker, the core component of Portland cement. It's important to note that this heating process contributes to the LCCA through the energy consumption costs associated with crushing, grinding and heating materials during production. The final stage, shown in the diagram, focuses on the concrete production itself. Clinker, the key component from the previous stage, is cooled and ground into a fine powder. This powder is then combined with other ingredients such as limestone and fly ash to make Portland cement, the glue that holds concrete together. Finally, at the concrete batch plant, the cement is mixed with aggregates (sand and gravel), water, and optional admixtures to achieve the desired concrete properties for construction. The mixed concrete is then loaded into trucks for delivery to the construction site. It's important to note that the entire concrete production process can have environmental impacts beyond those shown in the diagram. Activities such as mining and quarrying can have lasting impacts, and a comprehensive LCCA can include these environmental costs through methods such as environmental impact assessments.

The foundation, the critical link between a building and the ground it stands on, has a significant impact on its structural performance. The stiffness of a building, its ability to resist deformation under load, is directly influenced by the stiffness of its foundation soil (Bowles, 1996) [29]. Different soil types have different mechanical properties, such as bearing capacity, shear strength, and settlement characteristics. For rock, the foundation provides the stiffest support, with high bearing capacity and minimal settlement. This results in a stiffer building structure that requires less material for reinforcement against lateral loads. For coarse-grained soils (sand and gravel), the soils provide good drainage and moderate bearing capacity. However, they may experience some settlement under load, requiring foundation designs that account for potential movement. Buildings on such foundations may have some flexibility, but may still be quite rigid, depending on the specific soil characteristics. For fine-grained soils (silt and clay), the soils can have lower bearing capacity and higher susceptibility to settlement, especially under dynamic loading such as earthquakes. Clay soils can also experience volume changes due to moisture fluctuations, which can affect foundation stability. Buildings on fine-grained soils may require stiffer foundation designs and possibly more material to compensate for the lower soil stiffness [29].

The choice of foundation type and the amount of material used is directly related to the stiffness of the underlying soil [30]. For stiffer soils, rock and some coarse-grained soils allow for shallow foundations and potentially less material to be used in foundation construction and structural reinforcement. This is because the stiffer soil provides inherent support, reducing the need for extensive reinforcing elements within the building itself. However, as with less stiff soils, fine-grained soils often require deeper foundations, piles, or other techniques to ensure stability and mitigate settlement [30]. In addition, buildings on such soils may require more robust structural elements within the superstructure to compensate for the reduced foundation stiffness, potentially increasing material usage. Therefore, a thorough geotechnical investigation is critical to understanding the characteristics of the foundation soil (ASCE 7). The geotechnical investigation helps to determine the appropriate foundation type, depth, and required building stiffness to withstand the anticipated loads. The type of soil underlying a building has a significant impact on its stiffness and material requirements. Stiffer soils, such as rock, allow for more efficient designs that use less material. Conversely, less rigid soils such as clay may require deeper foundations, extensive foundation treatments, and potentially more material in the superstructure to achieve the desired building stiffness. A thorough understanding of soil properties through geotechnical investigation is essential for safe and cost-effective building design.

## 6 Results

A comprehensive structural analysis was performed considering several design scenarios with varying member dimensions. These scenarios were evaluated for seismic performance, with deflection due to earthquake activity as a key factor. In addition, the analysis considered the sustainability implications of material usage. Before presenting the final analysis results, the pre-analysis data used as input for the software application is presented. The architectural design features a fixed-support structural system for a building with a total height of 21 meters and a rectangular footprint of 14.5 meters by 43 meters. The building comprises seven stories, each with a height of 3 meters. The structural system utilizes 252 columns, each 3 meters tall, to provide vertical support. The horizontal framing system consists of 413 beams with an approximate length of 5 meters and an additional 28 beams with a length of 2 meters specifically designed as cantilevers for balconies. The column spacing is approximately 5 meters, and the floor slabs have a thickness of 15 centimeters. A separate table will detail the various loads acting on the structure.

Table 3. Pre-data of loads acting upon structure

|                   | Load on Floor 1 – 6<br>(kPa) | Load on Balcony<br>(kPa)    | Load on Roof<br>(kPa)       |
|-------------------|------------------------------|-----------------------------|-----------------------------|
| Self-weight (DL)  | Automatically<br>calculated  | Automatically<br>calculated | Automatically<br>calculated |
| Floor Finish (DL) | -1.2                         | -                           | -                           |
| Live Load (LL)    | -1.92                        | -0.35                       | -0.96                       |

Prior to the seismic analysis, a modal analysis was performed that considered the effects of both dead load (DL) and live load (LL) through a load-to-mass conversion. The modal

analysis used a load combination of  $1.0 \times DL$  and  $0.3 \times LL$ . Following the modal analysis, the seismic properties were determined by incorporating parameters that influence seismicity. These parameters included: ground response acceleration (ag) in meters per second squared ( $m/s^2$ ), soil type, response spectrum, spectrum type, analysis direction, and behavior factor.

For each case, the ground response (ag) was calculated by multiplying the PGA value by the building importance factor (IF) of 1.2. This resulted in a range of ag values from 0.12g to 0.72g, corresponding to PGA values of 0.1g to 0.6g. Finally, the ag values were converted to units of  $m/s^2$ , yielding values approximately equal to 1.2  $m/s^2$ , 2.4  $m/s^2$ , 3.6  $m/s^2$ , 4.7  $m/s^2$ , 5.9  $m/s^2$ , and 7.0  $m/s^2$ . The following parameters were used for the seismic analysis input:

- Soil type : C (dense or medium density sand, gravel, or stiff clay)
- Spectrum type : Design Spectrum
- Analysis Direction : Horizontal
- Behavior Factor : 1.5 (constant value)

## 6.1 Structural Performance

### A. Structural Performance of using PGA 0.1g

A series of ten seismic analysis cases were performed, each considering a different structural configuration (column and beam dimensions) subjected to a PGA of 0.1g, corresponding to a ground response of 1.2  $m/s^2$ . The resulting deflections for each case are summarized in a dedicated table for further evaluation.

Table 4. Deflections imposed and performance level from 0.1g seismic analysis

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Deflections (cm) | Inter-story Drift Ratio (%) |
|----|------------------------|----------------------|------------------|-----------------------------|
| 1  | 45x45                  | 25x40                | 9.1              | 0.4333                      |
| 2  | 45x45                  | 25x45                | 8.6              | 0.4095                      |
| 3  | 45x45                  | 25x50                | 8.2              | 0.3904                      |
| 4  | 45x45                  | 25x55                | 7.9              | 0.3761                      |
| 5  | 45x45                  | 25x60                | 7.7              | 0.3666                      |
| 6  | 50x50                  | 30x40                | 8.3              | 0.3952                      |
| 7  | 50x50                  | 30x45                | 7.8              | 0.3714                      |
| 8  | 50x50                  | 30x50                | 7.4              | 0.3523                      |
| 9  | 50x50                  | 30x55                | 7.1              | 0.3380                      |
| 10 | 50x50                  | 30x60                | 6.9              | 0.3285                      |

An analysis of the table shows that the structural configuration with column dimensions of 50 cm x 50 cm and beam dimensions of 30 cm x 60 cm has the minimum deflection of 6.9 cm among the ten cases studied. This finding suggests that this configuration provides the optimal design solution in terms of seismic performance. Conversely, the configuration with column dimensions of 45 cm x 45 cm and beam dimensions of 25 cm x 40 cm results in a deflection of 9.1 cm. It is important to note that although this deflection value is higher, the Inter-story drift ratio in all cases remains below the critical threshold of 0.5%, indicating operational performance with negligible to minor damage expected under the specified seismic loading.

### B. Structural Performance of using PGA 0.2g

Thirteen variations of the building structure were analyzed for earthquake resistance. Each variation had different column and beam sizes. All structures were subjected to a simulated earthquake with a PGA of 0.2g, which corresponds to a ground motion of 2.4 m/s<sup>2</sup>. The amount of bending (deflection) experienced by each structure under this earthquake scenario is presented in a separate table for further study.

Table 5. Deflections imposed and performance level from 0.2g seismic analysis

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Deflections (cm) | Inter-story Drift Ratio (%) |
|----|------------------------|----------------------|------------------|-----------------------------|
| 1  | 45x45                  | 25x40                | 18.3             | 0.8714                      |
| 2  | 45x45                  | 25x45                | 17.2             | 0.8190                      |
| 3  | 45x45                  | 25x50                | 16.4             | 0.7809                      |
| 4  | 45x45                  | 25x55                | 15.8             | 0.7523                      |
| 5  | 45x45                  | 25x60                | 15.4             | 0.7333                      |
| 6  | 50x50                  | 30x40                | 16.7             | 0.7952                      |
| 7  | 50x50                  | 30x45                | 15.6             | 0.7428                      |
| 8  | 50x50                  | 30x50                | 14.8             | 0.7047                      |
| 9  | 50x50                  | 30x55                | 14.2             | 0.6761                      |
| 10 | 50x50                  | 30x60                | 13.7             | 0.6523                      |
| 11 | 55x55                  | 30x40                | 15.8             | 0.7523                      |
| 12 | 55x55                  | 30x60                | 12.5             | 0.5952                      |
| 13 | 60x60                  | 30x60                | 11.6             | 0.5523                      |

Analysis of the table shows that the structural configuration with column dimensions of 45 cm x 45 cm and beam dimensions of 25 cm x 40 cm has the largest deflection among the cases studied, reaching a value of 18.3 cm. Conversely, the configuration with column dimensions of 50 cm x 50 cm and beam dimensions of 30 cm x 60 cm results in a deflection of 13.7 cm. It is important to note that both configurations have high inter-story drift ratios (IDR) greater than 0.5%, placing them approaching the immediate occupancy performance level characterized by light to moderate seismic damage. In an attempt to achieve operational performance with an IDR below 0.5%, a configuration with column dimensions of 55 cm x 55 cm and beam dimensions of 30 cm x 60 cm was analyzed. However, this configuration did not meet the desired IDR threshold. A configuration with even larger column dimensions of 60 cm x 60 cm and beam dimensions of 30 cm x 60 cm was also investigated. While this configuration yielded an IDR of 0.5523, approaching the operational performance range, the significant increase in material usage due to the larger member sizes may present an economic and potentially unsustainable design issue.

### C. Structural Performance of using PGA 0.3g

A series of ten structural configurations were evaluated for seismic performance. Each configuration used different column and beam dimensions. All structures were subjected to a simulated earthquake with a PGA of 0.3g, corresponding to a ground motion of 3.6 m/s<sup>2</sup>. The resulting deflections experienced by each structure under this earthquake scenario are presented in a separate table for further analysis.

Table 6. Deflections imposed and performance level from 0.3g seismic analysis

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Deflections (cm) | Inter-story Drift Ratio (%) |
|----|------------------------|----------------------|------------------|-----------------------------|
| 1  | 45x45                  | 30x40                | 27.1             | 1.2904                      |
| 2  | 45x45                  | 30x45                | 25.6             | 1.2190                      |
| 3  | 45x45                  | 30x50                | 24.6             | 1.1714                      |
| 4  | 45x45                  | 30x55                | 23.8             | 1.1333                      |
| 5  | 45x45                  | 30x60                | 23.3             | 1.1095                      |
| 6  | 50x50                  | 35x40                | 24.7             | 1.1761                      |
| 7  | 50x50                  | 35x45                | 23.2             | 1.1047                      |
| 8  | 50x50                  | 35x50                | 22.1             | 1.0523                      |
| 9  | 50x50                  | 35x55                | 21.3             | 1.0142                      |
| 10 | 50x50                  | 35x60                | 20.7             | 0.9857                      |

Examination of the table shows that the structural configuration using 50 cm x 50 cm columns and 35 cm x 60 cm beams has the lowest deflection of 20.7 cm among the ten cases studied. This configuration also achieves the lowest IDR of 0.9857%, which exceeds the operational performance threshold ( $IDR < 0.5\%$ ) but falls within the immediate occupancy performance level ( $0.5\% < IDR < 1.0\%$ ). This performance level is characterized by light to moderate seismic damage. Conversely, the configuration with column dimensions of 45 cm x 45 cm and beam dimensions of 30 cm x 40 cm results in a deflection of 27.1 cm and an IDR of 1.2904%. Similar to the previous configuration, this IDR value falls between the immediate occupancy and damage control performance levels ( $1.0\% < IDR < 1.5\%$ ), indicating expected moderate to severe seismic damage.

#### D. Structural Performance of using PGA 0.4g

Twelve variations of structural design were studied to evaluate their seismic performance. Each design used different column and beam dimensions. All configurations were subjected to a simulated earthquake with a PGA of 0.4g, representing a ground motion of  $4.7 \text{ m/s}^2$ . The resulting deflections experienced by each structural configuration under this earthquake scenario are presented in a dedicated table for further analysis.

Table 7. Deflections imposed and performance level from 0.4g seismic analysis

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Deflections (cm) | Inter-story Drift Ratio (%) |
|----|------------------------|----------------------|------------------|-----------------------------|
| 1  | 45x45                  | 40x60                | 31               | 1.4761                      |
| 2  | 45x45                  | 40x65                | 30.6             | 1.4571                      |
| 3  | 45x45                  | 40x70                | 30.4             | 1.4476                      |
| 4  | 45x45                  | 40x75                | 30.3             | 1.4428                      |
| 5  | 45x45                  | 40x80                | 30.2             | 1.4428                      |
| 6  | 50x50                  | 50x50                | 29               | 1.3809                      |
| 7  | 50x50                  | 50x55                | 28.2             | 1.3428                      |
| 8  | 50x50                  | 50x60                | 27.7             | 1.3190                      |
| 9  | 50x50                  | 50x65                | 27.3             | 1.3000                      |
| 10 | 50x50                  | 50x70                | 27.1             | 1.2904                      |

|    |       |       |      |        |
|----|-------|-------|------|--------|
| 11 | 50x50 | 55x70 | 29   | 1.3809 |
| 12 | 50x50 | 60x70 | 28.2 | 1.3428 |

Analysis of the table shows that the structural configuration with column dimensions of 45 cm x 45 cm and beam dimensions of 40 cm x 60 cm has the highest deflection of 31 cm among the eighteen cases studied. This configuration also achieves the highest inter-story drift ratio (IDR) of 1.4761%. This IDR value exceeds the immediate occupancy performance level ( $IDR < 1.0\%$ ) and approaches the damage control performance level ( $1.0\% < IDR < 1.5\%$ ). The damage control performance level is typically associated with moderate to severe seismic damage. Conversely, both configurations using column dimensions of 50 cm x 50 cm and beam dimensions of either 50 cm x 70 cm or 55 cm x 75 cm resulted in a deflection of 27.1 cm and an IDR of 1.2904%. This deflection value is the lowest observed in this analysis. Similar to the first configuration discussed, this IDR value falls in the range between immediate occupancy and damage control performance levels, indicating expected moderate to severe seismic damage.

#### **E. Structural Performance of using PGA 0.5g and 0.6g**

Structural analyses were performed for PGA values of 0.5g and 0.6g, corresponding to ground responses of  $5.9 \text{ m/s}^2$  and  $7.0 \text{ m/s}^2$ , respectively. While deflection analysis was feasible for both cases, achieving 34.9cm and 37.3cm in deflection respectively, reinforcement design using previously explored member dimensions proved problematic. These configurations resulted in error messages indicating insufficient shear or shear-torsion capacity, and limitations in the calculation of required reinforcement areas according to code requirements (EN 1992-1-1 6.2.2 (1)). Through iterative analysis, alternative member dimensions suitable for reinforcement were identified. In both cases, 50 cm x 50 cm columns were used. However, the required beam dimensions differed depending on the PGA value. For the 0.5g case, a 60 cm x 80 cm beam could be reinforced, while the 0.6g case required a larger 60 cm x 100 cm beam. Given the significant increase in member sizes required for higher PGA values, it is recommended that alternative structural systems be investigated for such scenarios. These could include the incorporation of shear walls to supplement the primary structural frame and reduce the seismic effects on the primary members.

## **6.2 Material Usage for Design**

#### **A. Material usage of using PGA 0.1g**

Building on the previous analysis of the structural performance under a PGA value of 0.1g, this investigation focuses on a comparative evaluation of the deflections experienced by each structural configuration in relation to the total cubic meters of columns and beams used. A table summarizing the deflection values and total member cubic meters for the ten configurations is presented for further review.

Table 8. Total cubic meters of each structural configuration for 0.1g

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Deflections (cm) | Total Cubic Meters (m <sup>3</sup> ) |
|----|------------------------|----------------------|------------------|--------------------------------------|
| 1  | 45x45                  | 25x40                | 9.1              | 365.19                               |
| 2  | 45x45                  | 25x45                | 8.6              | 391.7025                             |
| 3  | 45x45                  | 25x50                | 8.2              | 418.215                              |
| 4  | 45x45                  | 25x55                | 7.9              | 444.7275                             |
| 5  | 45x45                  | 25x60                | 7.7              | 471.24                               |
| 6  | 50x50                  | 30x40                | 8.3              | 443.52                               |
| 7  | 50x50                  | 30x45                | 7.8              | 475.335                              |
| 8  | 50x50                  | 30x50                | 7.4              | 507.15                               |
| 9  | 50x50                  | 30x55                | 7.1              | 538.965                              |
| 10 | 50x50                  | 30x60                | 6.9              | 570.78                               |

The data presented in the table is further visualized by constructing a combination graph (figure 12). This graph correlates the deflection values experienced by each structural configuration with the total volume (in cubic meters) of the columns and beams used.

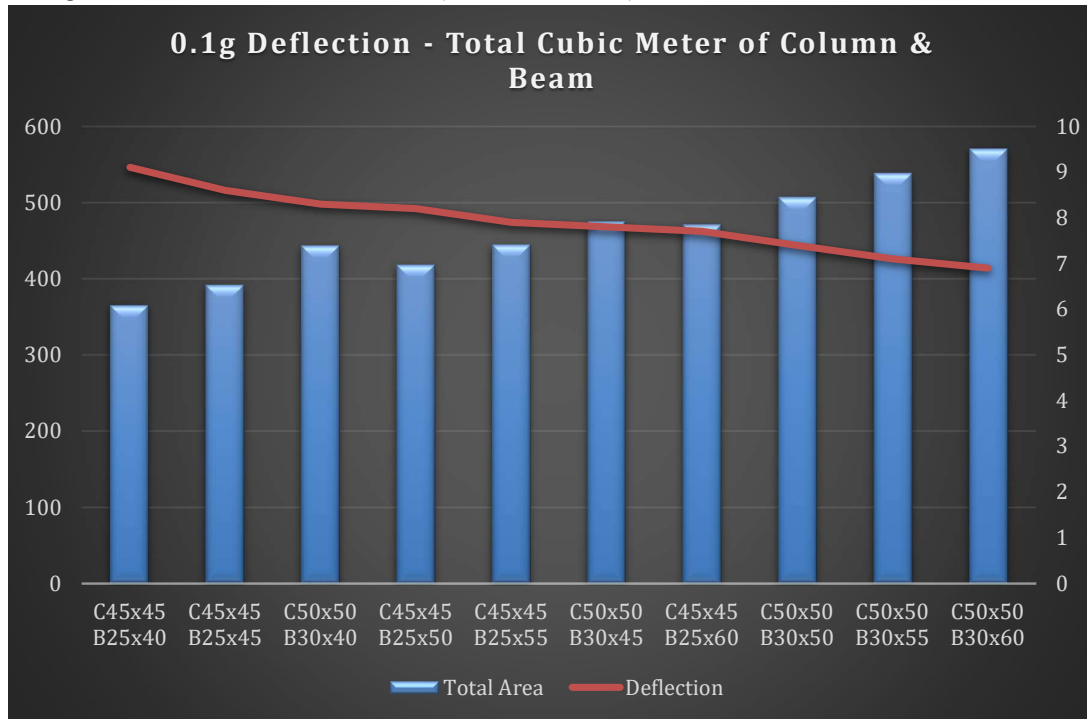


Figure 12. Relationship between deflection to total cubic meter of structure (PGA: 0.1g) (source: edited by author)

By analyzing the combination diagram, three specific structural configurations will be selected for a comparative evaluation of their material usage. These configurations will represent:

1. Minimum deflection (achieved with column dimensions of 50 cm x 50 cm and beam dimensions of 30 cm x 60 cm)
2. Maximum deflection (achieved with column dimensions of 45 cm x 45 cm and beam dimensions of 25 cm x 40 cm)

3. An intermediate deflection value (achieved with a configuration using column dimensions of 45 cm x 45 cm and beam dimensions of 25 cm x 60 cm)

The following table provides a detailed breakdown of material usage for the three selected structural configurations. The data includes the total weight of concrete and the total weight of reinforcing steel for each case.

Table 9. Total weight of concrete and steel for each structural configuration at 0.1g

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Total weight of Concrete (kg) | Total weight of Steel (kg) |
|----|------------------------|----------------------|-------------------------------|----------------------------|
| 1  | 50x50                  | 30x60                | 1,451,366                     | 73,162.3                   |
| 2  | 45x45                  | 25x40                | 926,605                       | 53,882.4                   |
| 3  | 45x45                  | 25x60                | 1,198,441                     | 65,350.45                  |

#### B. Material usage of using PGA 0.2g

Building on the previous analysis of the structural performance under a PGA value of 0.2g, this investigation focuses on a comparative evaluation of the deflections experienced by each structural configuration in relation to the total cubic meters of columns and beams used. A table summarizing the deflection values and total member cubic meters for the ten configurations is presented for further review.

Table 10. Total cubic meters of each structural configuration for 0.2g

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Deflections (cm) | Total Cubic Meters (m <sup>3</sup> ) |
|----|------------------------|----------------------|------------------|--------------------------------------|
| 1  | 45x45                  | 25x40                | 18.3             | 365.19                               |
| 2  | 45x45                  | 25x45                | 17.2             | 391.7025                             |
| 3  | 45x45                  | 25x50                | 16.4             | 418.215                              |
| 4  | 45x45                  | 25x55                | 15.8             | 444.7275                             |
| 5  | 45x45                  | 25x60                | 15.4             | 471.24                               |
| 6  | 50x50                  | 30x40                | 16.7             | 443.52                               |
| 7  | 50x50                  | 30x45                | 15.6             | 475.335                              |
| 8  | 50x50                  | 30x50                | 14.8             | 507.15                               |
| 9  | 50x50                  | 30x55                | 14.2             | 538.965                              |
| 10 | 50x50                  | 30x60                | 13.7             | 570.78                               |
| 11 | 55x55                  | 30x40                | 15.8             | 483.21                               |
| 12 | 55x55                  | 30x60                | 12.5             | 610.47                               |
| 13 | 60x60                  | 30x60                | 11.6             | 653.94                               |

The data presented in the table is further visualized by constructing a combination graph (figure 13). This graph correlates the deflection values experienced by each structural configuration with the total volume (in cubic meters) of the columns and beams used.

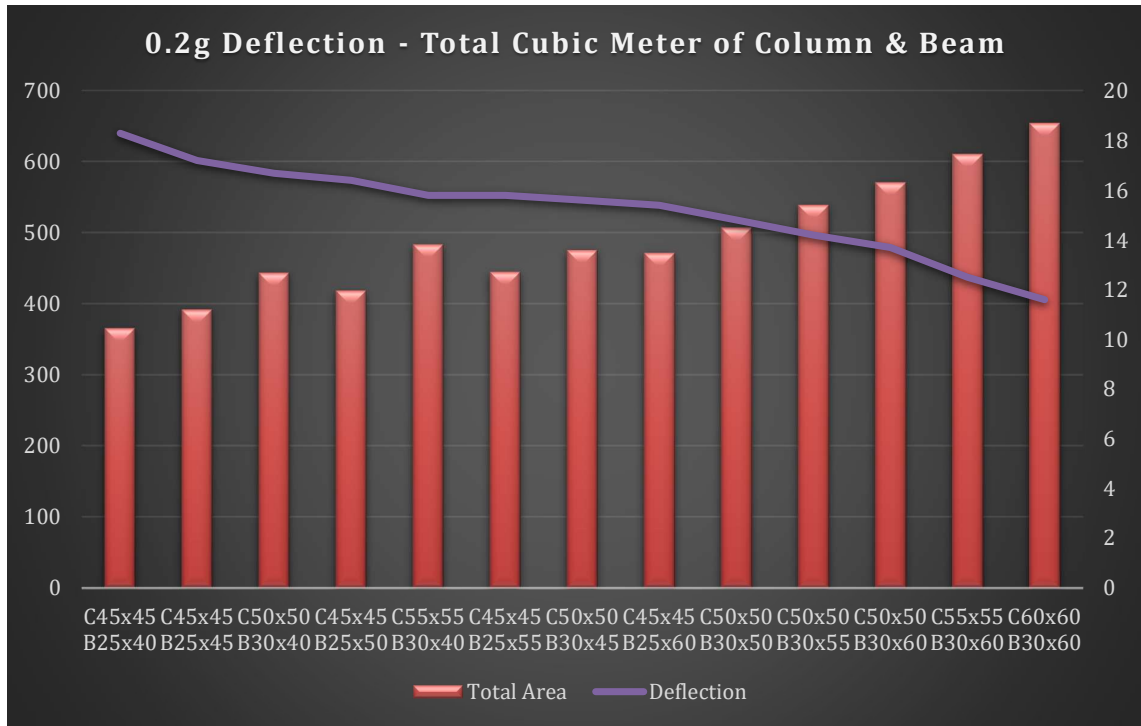


Figure 13. Relationship between deflection to total cubic meter of structure (PGA: 0.2g) (source: edited by author)

By analyzing the combination diagram, three specific structural configurations will be selected for a comparative evaluation of their material usage. These configurations will represent:

1. Minimum deflection (achieved with column dimensions of 60 cm x 60 cm and beam dimensions of 30 cm x 60 cm)
2. Maximum deflection (achieved with column dimensions of 45 cm x 45 cm and beam dimensions of 25 cm x 40 cm)
3. An intermediate deflection value (achieved with a configuration using column dimensions of 50 cm x 50 cm and beam dimensions of 30 cm x 50 cm)

The following table provides a detailed breakdown of material usage for the three selected structural configurations. The data includes the total weight of concrete and the total weight of reinforcing steel for each case.

Table 11. Total weight of concrete and steel for each structural configuration at 0.2g

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Total weight of Concrete (kg) | Total weight of Steel (kg) |
|----|------------------------|----------------------|-------------------------------|----------------------------|
| 1  | 60x60                  | 30x60                | 1,659,380                     | 165,241.3                  |
| 2  | 45x45                  | 25x40                | 926,605                       | 106,441.1                  |
| 3  | 50x50                  | 30x50                | 1,288,265                     | 138,771.2                  |

### C. Material usage of using PGA 0.3g

Building on the previous analysis of the structural performance under a PGA value of 0.3g, this investigation focuses on a comparative evaluation of the deflections experienced by each structural configuration in relation to the total cubic meters of columns

and beams used. A table summarizing the deflection values and total member cubic meters for the ten configurations is presented for further review.

Table 12. Total cubic meters of each structural configuration for 0.3g

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Deflections (cm) | Total Cubic Meters (m <sup>3</sup> ) |
|----|------------------------|----------------------|------------------|--------------------------------------|
| 1  | 45x45                  | 30x40                | 27.1             | 407.61                               |
| 2  | 45x45                  | 30x45                | 25.6             | 439.425                              |
| 3  | 45x45                  | 30x50                | 24.6             | 471.24                               |
| 4  | 45x45                  | 30x55                | 23.8             | 503.055                              |
| 5  | 45x45                  | 30x60                | 23.3             | 534.87                               |
| 6  | 50x50                  | 35x40                | 24.7             | 485.94                               |
| 7  | 50x50                  | 35x45                | 23.2             | 523.0575                             |
| 8  | 50x50                  | 35x50                | 22.1             | 560.175                              |
| 9  | 50x50                  | 35x55                | 21.3             | 597.2925                             |
| 10 | 50x50                  | 35x60                | 20.7             | 634.41                               |

The data presented in the table is further visualized by constructing a combination graph (figure 14). This graph correlates the deflection values experienced by each structural configuration with the total volume (in cubic meters) of the columns and beams used.

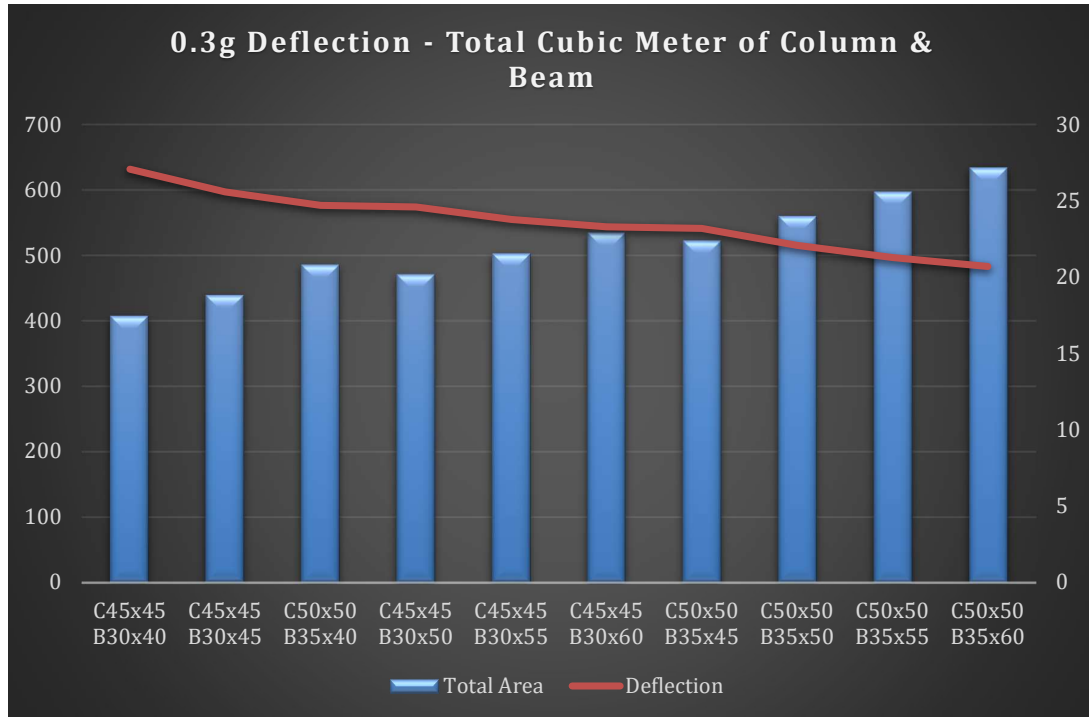


Figure 14. Relationship between deflection to total cubic meter of structure (PGA: 0.3g) (source: edited by author)

By analyzing the combination diagram, three specific structural configurations will be selected for a comparative evaluation of their material usage. These configurations will represent:

1. Minimum deflection (achieved with column dimensions of 50 cm x 50 cm and beam dimensions of 35 cm x 60 cm)

2. Maximum deflection (achieved with column dimensions of 45 cm x 45 cm and beam dimensions of 30 cm x 40 cm)
3. An intermediate deflection value (achieved with a configuration using column dimensions of 50 cm x 50 cm and beam dimensions of 35 cm x 45 cm)

The following table provides a detailed breakdown of material usage for the three selected structural configurations. The data includes the total weight of concrete and the total weight of reinforcing steel for each case.

Table 13. Total weight of concrete and steel for each structural configuration at 0.3g

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Total weight of Concrete (kg) | Total weight of Steel (kg) |
|----|------------------------|----------------------|-------------------------------|----------------------------|
| 1  | 50x50                  | 35x60                | 1,614,408                     | 202,044                    |
| 2  | 45x45                  | 30x40                | 1,035,340                     | 149,471                    |
| 3  | 50x50                  | 35x45                | 1,329,040                     | 179,925                    |

#### D. Material usage of using PGA 0.4g

Building on the previous analysis of the structural performance under a PGA value of 0.4g, this investigation focuses on a comparative evaluation of the deflections experienced by each structural configuration in relation to the total cubic meters of columns and beams used. A table summarizing the deflection values and total member cubic meters for the ten configurations is presented for further review.

Table 14. Total cubic meters of each structural configuration for 0.4g

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Deflections (cm) | Total Cubic Meters (m <sup>3</sup> ) |
|----|------------------------|----------------------|------------------|--------------------------------------|
| 1  | 45x45                  | 40x60                | 31               | 662.13                               |
| 2  | 45x45                  | 40x65                | 30.6             | 704.55                               |
| 3  | 45x45                  | 40x70                | 30.4             | 746.97                               |
| 4  | 50x50                  | 50x50                | 29               | 719.25                               |
| 5  | 50x50                  | 50x55                | 28.2             | 772,275                              |
| 6  | 50x50                  | 50x60                | 27.7             | 825.3                                |
| 7  | 50x50                  | 50x65                | 27.3             | 878.325                              |
| 8  | 50x50                  | 50x70                | 27.1             | 931.35                               |
| 9  | 50x50                  | 55x70                | 29               | 1005.585                             |
| 10 | 50x50                  | 60x70                | 28.2             | 1079.82                              |

The data presented in the table is further visualized by constructing a combination graph (figure 15). This graph correlates the deflection values experienced by each structural configuration with the total volume (in cubic meters) of the columns and beams used.

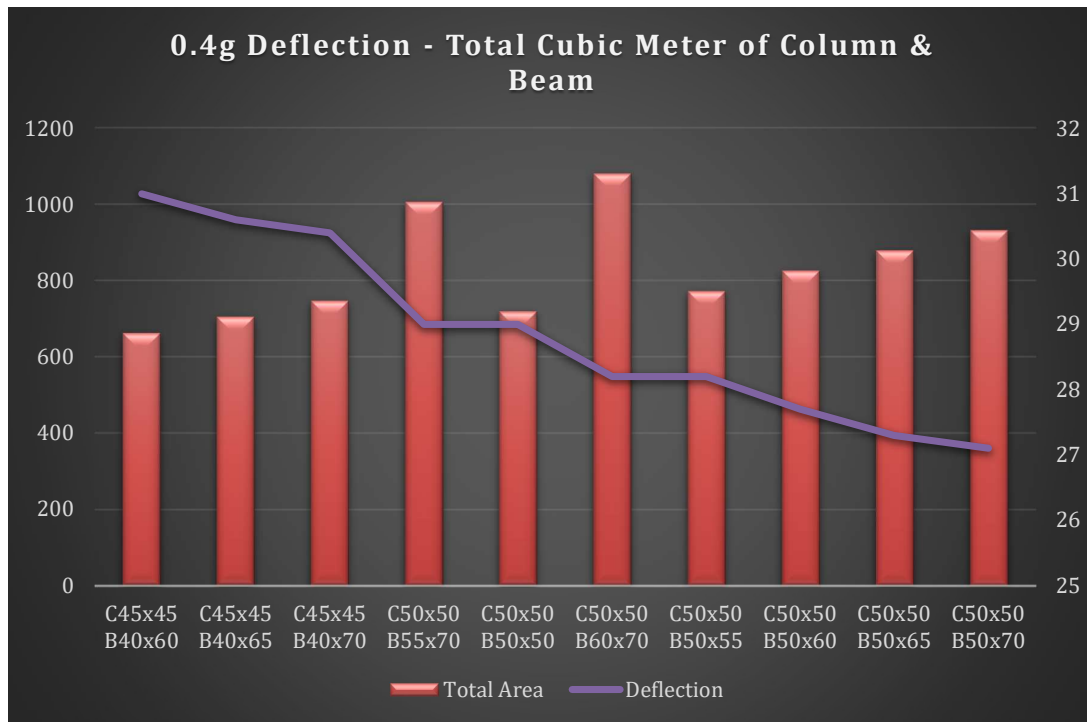


Figure 15. Relationship between deflection to total cubic meter of structure (PGA: 0.4g) (source: edited by author)

By analyzing the combination diagram, three specific structural configurations will be selected for a comparative evaluation of their material usage. These configurations will represent:

1. Minimum deflection (achieved with column dimensions of 50 cm x 50 cm and beam dimensions of 50 cm x 70 cm)
2. Maximum deflection (achieved with column dimensions of 45 cm x 45 cm and beam dimensions of 40 cm x 60 cm)
3. An intermediate deflection value (achieved with a configuration using column dimensions of 50 cm x 50 cm and beam dimensions of 50 cm x 50 cm)

The following table provides a detailed breakdown of material usage for the three selected structural configurations. The data includes the total weight of concrete and the total weight of reinforcing steel for each case.

Table 15. Total weight of concrete and steel for each structural configuration at 0.4g

| No | Column Dimensions (cm) | Beam Dimensions (cm) | Total weight of Concrete (kg) | Total weight of Steel (kg) |
|----|------------------------|----------------------|-------------------------------|----------------------------|
| 1  | 50x50                  | 50x70                | 2,375,608                     | 329,178                    |
| 2  | 45x45                  | 40x60                | 1,687,745                     | 269,777                    |
| 3  | 50x50                  | 50x50                | 1,831,936                     | 274,392                    |

#### E. Material usage of using PGA 0.5g and 0.6g

Due to the significant increase in member dimensions required for PGA values of 0.5g and 0.6g, material usage analysis for concrete and steel reinforcement was not performed for these scenarios. Computational limitations and time constraints also necessitated this decision.

### 6.3 Environmental Effect

This subsection focuses on a Life Cycle Cost (LCC) analysis of the materials used in the studied structural configurations. The IdematLightLCA software is used for this evaluation. The LCC analysis considers both concrete and steel materials used in the reinforced concrete structures. The material usage data for each configuration and the corresponding PGA value are obtained from the previous sections. The results of the LCC analysis for both concrete and steel are presented in a separate table for further review.

Table 16. Base Life Cycle Cost for Concrete and Steel

| Base Material              | Concrete (per kg) | Steel (per kg) |
|----------------------------|-------------------|----------------|
| Eco-costs (€)              | 0.06              | 0.22           |
| Carbon Footprint (kg)      | 0.31              | 0.93           |
| Closed Loop Recycling (kg) | 0                 | -0.29          |

Following the LCC analysis, an evaluation of the environmental impact associated with each structural configuration is conducted. This evaluation considers both the eco-costs and carbon footprint per kilogram of the materials used (concrete and steel). The total weight of concrete and steel for each case is multiplied by their respective eco-cost and carbon footprint values. A table summarizing these calculations for each case is presented below for further examination.

Table 17. Life Cycle Cost Analysis for Concrete and Steel used in each case for each PGA value

| PGA  | Column Dimensions (cm) | Beam Dimensions (cm) | Concrete      |                       | Steel         |                       |
|------|------------------------|----------------------|---------------|-----------------------|---------------|-----------------------|
|      |                        |                      | Eco-costs (€) | Carbon footprint (kg) | Eco-costs (€) | Carbon footprint (kg) |
| 0.1g | 50x50                  | 30x60                | 87,081.96     | 449,923.5             | 16,095.706    | 68,040.94             |
|      | 45x45                  | 25x40                | 55,596.3      | 287,247.6             | 11,854.128    | 50,110.63             |
|      | 45x45                  | 25x60                | 71,906.46     | 371,516.7             | 14,377.099    | 60,775.92             |
| 0.2g | 60x60                  | 30x60                | 99,562.8      | 514,407.8             | 36,353.086    | 153,674.4             |
|      | 45x45                  | 25x40                | 55,596.3      | 287,247.6             | 23,417.042    | 98,990.22             |
|      | 50x50                  | 30x50                | 77,295.9      | 399,362.2             | 30,529.664    | 129,057.2             |
| 0.3g | 50x50                  | 35x60                | 96,864.48     | 500,466.5             | 44,449.68     | 187,900.9             |
|      | 45x45                  | 30x40                | 62,120.4      | 320,955.4             | 32,883.62     | 139,008               |
|      | 50x50                  | 35x45                | 79,742.4      | 412,002.4             | 39,583.5      | 167,330.3             |
| 0.4g | 50x50                  | 50x70                | 142,536.48    | 736,438.5             | 72,419.16     | 306,135.5             |
|      | 45x45                  | 40x60                | 101,264.7     | 523,201               | 59,350.94     | 250,892.6             |
|      | 50x50                  | 50x50                | 109,916.16    | 567,900.2             | 60,366.24     | 255,184.6             |

Table 18. Combined Eco-costs and Carbon footprint in each case for each PGA value

| PGA  | Column Dimensions (cm) | Beam Dimensions (cm) | Combined Eco-Costs (€) | Deflection (cm) | Inter-story Drift Ratio (%) | Combined Carbon Footprint (kg) |
|------|------------------------|----------------------|------------------------|-----------------|-----------------------------|--------------------------------|
| 0.1g | 50x50                  | 30x60                | 103,177.666            | 9.1             | 0.4333                      | 517,964.4                      |
|      | 45x45                  | 25x40                | 67,450.428             | 6.9             | 0.3285                      | 337,358.2                      |
|      | 45x45                  | 25x60                | 86,283.559             | 7.7             | 0.3666                      | 432,292.6                      |
| 0.2g | 60x60                  | 30x60                | 135,915.886            | 11.6            | 0.5523                      | 668,082.2                      |
|      | 45x45                  | 25x40                | 79,013.342             | 18.3            | 365.19                      | 386,237.8                      |
|      | 50x50                  | 30x50                | 107,825.564            | 14.8            | 0.7047                      | 528,419.4                      |
| 0.3g | 50x50                  | 35x60                | 141,314.16             | 20.7            | 0.9857                      | 688,367.4                      |
|      | 45x45                  | 30x40                | 95,004.02              | 27.1            | 1.2904                      | 459,963.4                      |
|      | 50x50                  | 35x45                | 119,325.9              | 23.2            | 1.1047                      | 579,332.7                      |
| 0.4g | 50x50                  | 50x70                | 214,955.64             | 27.1            | 1.2904                      | 104,2574                       |
|      | 45x45                  | 40x60                | 160,615.64             | 31              | 1.4761                      | 774,093.6                      |
|      | 50x50                  | 50x50                | 170,282.4              | 29              | 1.3809                      | 823,084.7                      |

Table 19. Average deflection, cubic meter, eco-costs, and carbon footprint for each PGA value

| PGA  | Average Deflection | Average Cubic Meter | Average Eco-Costs (€) | Average Carbon Footprint (kg) |
|------|--------------------|---------------------|-----------------------|-------------------------------|
| 0.1g | 7.9                | 469.07              | 85,637.2177           | 429,205.1                     |
| 0.2g | 14.9               | 508.76              | 107,584.931           | 527,579.8                     |
| 0.3g | 23.6667            | 521.6925            | 118,548.027           | 575,887.8                     |
| 0.4g | 29.4               | 820.4               | 181,951.227           | 879,917.4                     |

## 7 Discussion

### 7.1 Structural Performance

The results from the previous chapter show a clear correlation between structural performance, PGA values, and the influence of member dimension variations. Adjustments to column and beam dimensions have a direct effect on the resulting inter-story drift ratio (IDR). The IDR, which is essentially the deflection experienced by the structure under load, serves as a key indicator of overall performance levels. The established performance levels range from operational performance (no/slight damage) to collapse prevention (destruction). The analysis

revealed that all of the structural configurations studied achieved performance levels in the operational or near damage control range. The latter category represents structures that may experience moderate to heavy damage under seismic loading.

Analysis of structures subjected to a PGA value of 0.1g showed that all configurations using member dimensions ranging from 45x45 cm columns and 25x40 cm beams to 50x50 cm columns and 30x60 cm beams achieved operational performance. This level of performance is characterized by an IDR of less than 0.5%, indicating minimal to low seismic damage. Therefore, it can be concluded that these structures would likely experience no or minor damage under a 0.1g earthquake scenario. At a PGA value of 0.2g, the design configuration falls within the Operational Performance to Immediate Occupancy Performance Level range ( $0.5\% < \text{IDR} < 1.0\%$ ). This range indicates structures that may experience no to minor damage or minor to moderate damage (immediate occupancy) under seismic loading. In an attempt to optimize the design and achieve operational performance at 0.2g, modifications to the column and beam dimensions were explored. However, the analysis showed that these dimensional adjustments were not effective in changing the overall performance level of the structure for this specific seismic intensity.

The analysis of structures subjected to a PGA value of 0.3g showed that the initial configurations, using member dimensions ranging from 45 cm x 45 cm columns and 30 cm x 40 cm beams to 50 cm x 50 cm columns and 35 cm x 55 cm beams, fell within the damage control performance level ( $\text{IDR} > 1.0\%$ ). This level indicates structures that are expected to experience moderate to severe seismic damage. However, by increasing the member sizes to 50 cm x 50 cm columns and 35 cm x 60 cm beams, it is possible to improve the performance level to immediate occupancy ( $0.5\% < \text{IDR} < 1.0\%$ ). This indicates structures that may experience slight to moderate damage under seismic loading. For a PGA value of 0.4g, even with adjustments to column and beam dimensions, the performance level remained within the immediate occupancy to damage control range.

These results suggest that at higher seismic intensities, it becomes increasingly difficult to achieve significant performance improvements through changes in member size alone. In the case of 0.5g and 0.6g PGA values, this study highlights the potential limitations of relying solely on member size adjustments. In addition, since this study considered a moment frame reinforced concrete building, it is recommended to investigate the impact of incorporating shear walls into the structural model. Shear walls are structural elements that can significantly increase the lateral load resistance of a building, potentially leading to improved performance under seismic loading.

## 7.2 Material Usage

The relationship between material usage and deflection was investigated for PGA values ranging from 0.1g to 0.4g. For each PGA value, three structural configurations were selected from the analysis results. These configurations represented structures with

- Minimum deflection
- Maximum deflection
- An intermediate deflection value

The analysis of these configurations for a PGA value of 0.1g serves as an illustrative example. Configuration (a), with 50 cm x 50 cm columns and 30 cm x 60 cm beams, had the

minimum deflection (6.9 cm), but also had the highest total concrete and steel weight. Conversely, configuration (b), with 45 cm x 45 cm columns and 25 cm x 45 cm beams, experienced the maximum deflection (9.1 cm) but had the lowest material usage. Configuration (c), using 45 cm x 45 cm columns and 25 cm x 60 cm beams, represented an intermediate scenario in terms of both deflection (7.7 cm) and material usage.

Similar trends were observed for the 0.2g, 0.3g, and 0.4g PGA values. These results suggest a trade-off between material usage and deflection (and consequently seismic performance). While structures with lower deflections generally require more material, they can also achieve better performance levels under seismic loading and potentially suffer less damage. Finding a balance between these two factors is critical to designing sustainable and safe buildings. Sustainable design principles aim to minimize material consumption while ensuring structural integrity. Therefore, during the design process, it is essential to optimize the structural configuration to achieve the desired level of performance with minimal material use.

A detailed breakdown of material usage for the 0.1g PGA case shows variations in both concrete and steel quantities. The total weight of concrete ranged from 1,456,366 kg to 926,605 kg, while the total weight of steel ranged from 73,162.3 kg to 53,882.4 kg for the three structural configurations studied. From a purely technical point of view, configuration (c), which uses 45 cm x 45 cm columns and 25 cm x 60 cm beams, may appear to be a reasonable choice due to its intermediate position in terms of both deflection and material usage. However, it's important to consider the overall level of performance achieved by all three configurations. Since all three structures exhibited operational performance with little or no damage under seismic loading, an alternative approach can be considered. Specifically, configuration (a), using 45 cm x 45 cm columns and 25 cm x 40 cm beams, represents the most sustainable option due to its significantly lower material consumption. The deflection differences between configuration (a) and the other two configurations are relatively small, and all three configurations fall within the same performance level (operational). Therefore, prioritizing configuration (a) is well aligned with the principles of sustainable design, which seeks to minimize resource use while maintaining structural integrity.

Similar to the 0.1g case, a detailed analysis of the material usage for the 0.2g PGA scenario reveals variations in the quantities of concrete and steel. The total weight of concrete ranged from 1,659,380 kg to 926,605 kg, while the total weight of steel ranged from 165,241.3 kg to 106,441.1 kg for the three structural configurations. The selection of an optimal configuration in this case requires careful consideration of both performance level and resource consumption. All three configurations studied achieved an IDR between 0.5% and 1.0%, corresponding to a performance level ranging from operational (no/slight damage) to immediate occupancy (slight/moderate damage). Configuration (a) has the lowest deflection (11.6 cm), but also requires the most material. Conversely, configuration (b) minimizes material usage but has the highest deflection. High deflections can lead to higher maintenance costs in the long run. Therefore, configuration (c) may be a balanced option. It achieves a compromise between material usage and deflection while maintaining a level of performance within the operational to immediate occupancy range. This approach is consistent with sustainable design principles, where the goal is to find a balance between structural safety, material efficiency, and life cycle costs.

The material usage analysis for the 0.3g PGA scenario shows similar trends to the previous cases. While most configurations achieve a performance level within the immediate occupancy to damage control range ( $1.0\% < \text{IDR} < 1.5\%$ ), indicating structures susceptible to moderate to severe seismic damage, one configuration (labeled (c) for consistency) stands out. This exceptional configuration has a performance level that falls in the operational to immediate occupancy range ( $\text{IDR} < 1.0\%$ ), indicating minimal to light damage under seismic loading. Although it requires the highest material input (1,614,408 kg of concrete and 202,044 kg of steel), it provides the most desirable result in terms of damage prevention. Minimizing seismic damage not only improves occupant safety, but also reduces long-term maintenance costs. Therefore, in critical situations where a higher level of seismic protection is paramount, this configuration may be a justifiable choice despite its increased material consumption. Finally, for the 0.4g PGA scenario, all configurations exhibited performance levels within the immediate occupancy to damage control range. This suggests that achieving significant performance improvements through member size adjustments alone becomes increasingly difficult at this seismic intensity. Consequently, the selection of the most sustainable and safest option for the 0.4g case becomes more questionable. Configuration (a) (minimum deflection), configuration (b) (minimum material usage), or configuration (c) (intermediate deflection) could all be potential candidates. The choice may depend on project-specific priorities and constraints, such as the relative weight given to minimizing material usage versus achieving a slightly lower deflection.

The analysis revealed a significant increase in material usage as the PGA increased. For RC structures designed for a PGA of 0.1g and subsequently modified to meet the performance requirements of a 0.2g site, the quantities of concrete and steel increased by approximately 28% and 61%, respectively. This trend continued at higher seismic intensities. Moving from a 0.2g design to a 0.3g design required increases in concrete and steel of 20% and 31%, respectively. Similarly, moving from a 0.3g design to a 0.4g design resulted in an additional 32% and 38% increase in concrete and steel consumption, respectively. In essence, these results suggest that constructing an RC structure designed for a 0.1g PGA in a location with a 0.4g PGA would require approximately 60% more concrete and 83% more steel. This highlights the significant impact of seismic intensity on the material requirements for RC structures.

### **7.3 Environmental Effect**

Life cycle analysis considers not only the use of materials, but also the associated environmental impact through eco-costs and carbon footprint. The results show a clear correlation between the PGA value and the environmental impact of a structure. Higher PGA values correspond to areas of greater seismic risk. Building codes in such regions impose stricter safety requirements, often leading to increased material use in structural designs. The study found that both environmental costs and carbon footprints increase significantly as PGA increases. For example, reinforced concrete (RC) structures designed for a 0.1g PGA and then modified to meet the performance criteria for a 0.2g site showed an average increase in eco-cost and carbon footprint of 20% and 18%, respectively. This trend continued at higher seismic intensities. Moving from a 0.2g design to a 0.3g design resulted in a more modest increase of 9% and 8% for eco-cost and carbon footprint. However, moving from a 0.3g design to a 0.4g design resulted in a more substantial increase of 34% for both eco-cost and carbon footprint. In

essence, these results suggest that constructing an RC structure originally designed for a 0.1g PGA in a location with a 0.4g PGA would result in approximately a 52% increase in eco-cost and a 51% increase in carbon footprint. This highlights the significant impact of seismic intensity on the environmental footprint of RC structures. Finding the optimal balance between structural safety, material efficiency and environmental impact remains a critical challenge in earthquake engineering.

For the 0.1g PGA scenario, where the achieved performance level is operational (no/slight damage), minimal material use is preferred. This is in line with sustainability principles as it minimizes the environmental impact. However, for higher seismic intensities (0.2g, 0.3g, and 0.4g), the LCCA suggests that increasing member dimensions to improve structural performance may be justified. While achieving a balance between material use and sustainability remains a priority, prioritizing improved performance offers additional benefits. A structure with a higher level of performance is less likely to suffer significant damage during an earthquake. This not only protects human life, but also minimizes the environmental impact associated with earthquake-related waste generation. In addition, a high-performance building design can potentially eliminate the need for reconstruction, thereby reducing future material and energy consumption. This helps conserve virgin materials and energy resources.

## 8 Conclusion

This study investigates the seismic performance of reinforced concrete buildings in Europe in light of recent earthquakes. The authors focused on a six-story, fixed-support dormitory building and analyzed its response under different earthquake scenarios (0.1g to 0.6g peak ground acceleration) and soil conditions (dense sand, gravel, and stiff clay). The goal was to assess the impact of exceeding the expected seismic intensity and promote safer infrastructure. The study used AutoCAD and Robot Structural Analysis Professional software for both architectural drawings and structural analysis, taking into account soil-structure interaction and site-specific seismicity. In addition, the study examined the environmental impact of these buildings, including a life cycle assessment of the varying material usage required to meet design standards. This methodology emphasizes practical design that considers both economic and environmental factors. The key findings during our research are:

1. The study analyzed a reinforced concrete building model under various earthquake intensities (PGA) and found a strong relationship between member size, deflection (IDR), and overall performance. While member size adjustments can improve performance at lower PGA values (0.1g and 0.2g), achieving significant improvements becomes more difficult at higher intensities (0.3g and 0.4g). This suggests that alternative design strategies, such as the incorporation of shear walls, may be required for buildings in high seismic zones.
2. The analysis identified member dimension ranges that achieved operational performance (minimal damage) at lower PGA values (0.1g) and immediate occupancy performance (minor to moderate damage) at moderate PGA values (0.2g). However, at higher intensities (0.3g and 0.4g), performance remained in the damage control range (moderate to severe damage) even with member size adjustments. These results highlight the limitations of relying solely on member size optimization for high seismic regions.

3. The study examined the trade-off between material use and seismic performance (deflection) in RC buildings. The analysis showed that structures with lower deflections (achieved through higher material use) generally suffer less damage in earthquakes. This highlights the importance of finding a balance between these two factors for sustainable design, which aims to minimize material use while ensuring structural safety.
4. By analyzing different configurations for different PGA values (0.1g to 0.4g), the study showed that for lower seismic zones (0.1g and 0.2g), prioritizing configurations with minimal material use can be a sustainable approach, since all options achieved good performance (minimal damage). However, for higher seismic zones (0.3g), prioritizing configurations with slightly higher material usage may be necessary to achieve significantly better performance (minimal to low damage) and reduce long-term maintenance costs.
5. This study highlights the environmental impact of building in high seismic zones. Stricter building codes for these areas, which require more material use, result in higher environmental costs and carbon footprints. The analysis showed that for a representative reinforced concrete structure, a 0.1g PGA design built in a 0.4g PGA zone would result in a 52% increase in eco-cost and a 51% increase in carbon footprint.
6. The study suggests that Life Cycle Analysis (LCA) can be a valuable tool in balancing environmental impact with safety considerations. While lower material use is ideal for minimal seismic zones (0.1g PGA), prioritizing higher performance structures in zones with stronger earthquakes (0.2g, 0.3g, and 0.4g PGA) can be beneficial from an LCA perspective. These higher performance buildings are less likely to be damaged and require reconstruction, reducing waste and long-term environmental impacts.

## **9 Recommendations**

Given the exclusive focus on moment frame reinforced concrete (RC) buildings in this research, further research is recommended to explore the impact of incorporating shear walls into the structural model. Shear walls are vertical elements specifically designed to resist lateral loads, such as those generated by earthquakes. Their inclusion can significantly improve a building's ability to resist these horizontal forces. By analyzing the influence of shear walls on building performance under different seismic scenarios, valuable insights can be gained. These insights could include improved performance levels for a given material usage strategy, the potential to reduce member sizes while maintaining desired performance levels, or a combination of both. Ultimately, a comprehensive understanding of how shear walls interact with moment frames in RC buildings would contribute to the development of more efficient and seismically resistant structural systems.

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