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COMPARING ACI, BS AND EUROPE CODE USING LATERAL FORCES ON SHEAR WALLS EFFECT

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Master's Thesis

Supervisor

Prof. Dr. Tuncer ÇELİK

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I hereby declare that all information/data presented in this graduation project has been obtained in full accordance with academic rules and ethical conduct. I also declare all unoriginal materials and conclusions have been cited in the text and all references mentioned in the reference list have been cited in the text, and vice versa as required by the above mentioned rules and conduct.

Alfat AL-WAWI

Signature

DEDICATION

First and foremost, I dedicate this work to God Almighty my creator; He has been the source of my strength through my life. I would like to thank our family, especially my parents, my dearly sisters, for their encouragement, patience, and assistance over the years. We are forever indebted to our parents, who have always kept me in their prayers.

ABSTRACT

COMPARING ACI, BS AND EUROPE CODE USING LATERAL FORCES ON SHEAR WALLS EFFECT

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Comparing and contrasting analyses of concrete structures proposed for development in Baghdad and Cairo Seismic maps' impact on the examined structural system is taken into account in this study. The information offered included principles, illustrations of seismic maps and analytical techniques, and case study findings. The presence of a shear wall and where it is placed could alter the mass centre and stiffness of the structure. Asymmetrical models are very unlikely. This is a result of the interaction between gravity and the centre of mass of the object. The conditions are fulfilled when the shear wall moves the structure's centre of mass and centre of stiffness closer together. Because lateral stresses are absorbed, shear walls make buildings stronger and can sustain less movement. Installing shear walls to stop shear is a standard procedure in the construction sector (2). The building's performance has improved, and the displacement along the X and Y axes has decreased, according to analysis of its reaction spectrum. Finite element software, such as ETABS, is an incredibly useful tool for modelling large-scale infrastructures since performing actual research or simulations to analyse the behaviour of a system under a variety of situations can be prohibitively expensive and challenging to manage. Finite element software can be an important resource in situations like this (e.g. high rise buildings). The Egyptian code produces the highest base shear values, followed by the earthquake code published in (EUROCODE 8-2004), and finally the Iraqi code for 2019, with a structural system of shear walls producing the lowest results in terms of displacements, drifts, and base shear values, according to the current Cairo seismic map. Five: The Iraqi code results in the lowest values for base shear, drift, and displacement when the seismic coefficient is applied to the present Cairo map. The height of the building ranged linearly from 5 to 21 stories misplaced, and modal outlines were created utilizing dynamic and static analysis.

Keywords: American Concrete Institute, British Standard, Building Code, ETABS, Euro Code, Iraqi code, Shear Wall.

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ABBREVIATIONS

ACI	:	American Concrete Institute
BS	:	British Standards
RC	:	reinforced concrete
FRP	:	fibre reinforced polymer
TS	:	Turkish standards
UAC	:	Unified Arabic Code
CSA	:	Canadian code
ASCE	:	American Society of Civil Engineers
ATC	:	Applied Technology Council
TVLEM	:	Three Vertical Line Element Model
EC	:	European Code

1. INTRODUCTION

Shear walls made of reinforced concrete (RC) are one of the most popular types of lateral force resisting components used in building construction. Shear wall failure mechanisms may be classified into two categories: flexural failure and shear failure in the presence of seismic activity [1]. Different variables, such as the shear span ratio and the axial compression ratio, have an impact on the behaviour of each mode. Walls that have been properly planned are usually anticipated to fail due to ductile flexural failure. As a result of these low aspect ratios, shear walls in buildings with wide voids on the bottom level are particularly vulnerable to brittle shear failure during earthquakes [2], [3]. It is the shear behaviour of low-rise shear walls that has an impact on the collapse mode of the buildings as well as their seismic performance [4], [5]. In reality, RC shear walls that have been built using a well-established technique and that have a consistent energy dissipation capability are often seen. However, issues such as seismic rehabilitation and corrosion resistance continue to be a source of contention. Recent years have seen a fast increase in the use of fibre reinforced polymer (FRP) in civil engineering [6] – [10] applications. Because of its low weight, high strength, linear elasticity, and good resistance to corrosion and fatigue, fiberglass reinforced plastic (FRP) has emerged as a viable alternative to steel reinforcement [11], [12], and [13]. As Abidi and Madhuri point out, the usage of shear walls is an excellent method to increase the degree of ductility and provide more stable behaviour. Shear walls also seem to be a new approach to decrease the amount of soft story in seismic response [14]. A 56-story reinforced concrete tall skyscraper was the subject of Esmaili's research on a structural RC shear wall system. Shear walls for both gravity and bracing systems are undesirable to him, both theoretically and economically. He also believes that not only are main walls expected to carry seismic loads, but that they will also hold a substantial proportion of gravity loads [15] as well. Design of a concrete shear wall building for earthquake-induced torsion has been worked out by Humar and his colleague Yavari [16]. A building code (also known as building control or building regulations) is a collection of laws that specifies the requirements for built items such as buildings and non-building infrastructure, such as bridges and roads. Buildings must comply with the code in order to receive planning approval, which is often granted by a local municipality. Generally speaking, the primary aim of building codes is to safeguard the public's health, safety, and general welfare as they pertain to the construction of buildings and structures, as well as their occupancy. When a building code is officially adopted by the proper governmental or private authority in a specific jurisdiction, it becomes legislation in that jurisdiction. In most cases, building codes are intended to be applied by architects, engineers, interior designers, contractors, and regulators, but they can also be used for a variety of other purposes by safety inspectors, environmental scientists, real estate developers, subcontractors, manufacturers of building products and materials, insurance companies, facility managers, and tenants, among other people. Whenever codes are accepted into legislation, they govern the design and building of structures.

1.1. RESEARCH QUESTIONS

In the presence of a large number of building codes, the designer needs a clearer comparison between them. Even though the effects of lateral pressures on shear walls are an essential element in determining the building structure, a comparison between various codes in terms of the effects of lateral forces on shear walls has become necessary.

In order to do this, the following questions must be answered:

a. What are the consequences of lateral pressures on shear walls?

b. What is the American code of conduct?

c. What is the European Union's code of conduct?

d. What is the British code?

e. How do we go about designing a structure while adhering to these codes?

Our goals will be achieved via the answers to these and other issues that are addressed in this thesis.

1.2. AIMS AND OBJECTIVES OF THE RESEARCH

For a 21-story structure, this study seeks to enhance the comparison between various building codes in terms of diverse circumstances, particularly lateral pressures on shear walls and the consequences of these forces.

a. A comprehensive comparison study of the provisions of both codes is needed in order to validate their validity and uncover inconsistencies between the two laws, which is the goal of this research.

b. To investigate the provisions of various building codes' safety ideas, design assumptions, cross-sectional moment capacity, ductility, minimum and maximum reinforcement ratios, and load safety factors.

c. To create a comprehensive construction plan for a 21-story skyscraper.

1.3. METHODOLOGY

Starting with a review of reinforced concrete structure building design code documents, this study determined the effects of lateral forces on shear walls on the design of a 21-story building, in order to answer the research questions by reviewing the existing system and implementing the following measures:

a. A thorough study of the literature, including books, papers, web sites, and e-journals, was conducted in order to develop and assess competence models.

b. Investigate the impact of lateral pressures on shear walls caused by differences in the codes of reinforced concrete structures:

i. Examine the functions and applications of the methodology and procedures.

ii. Design and construct the structural framework for a 21-story skyscraper.

c. Using the findings from theory, the literature review, and practice, develop a general framework that can be tested by discussing it with practitioners, and then implement the framework.

d. Draw up a set of suggestions and conclusions.

1.4. THE CONSTRAINTS AND CHALLENGES

There are many restrictions on the building industry, which will be addressed in more detail later. According to the findings of the researcher's study, these limitations are as follows:

a. The behaviour of RC members under stress necessitates the acceptance of many simplifying assumptions, all of which must be tested experimentally.

b. Given the increasing usage of high- and ultra-high-strength concrete, as well as high-strength and high-ductility steel, the description of behaviour, especially in the support zones, is always applicable today.

c. If certain simplifications are made, the determination of the maximum values of significant tensile stresses that occur in the support zone may be restricted to the determination of shear stresses in the neutral axis. As a result, the bearing capacity is most often referred to as the shear capacity of the material.

1.5. SUMMARY OF CHAPTERS

This dissertation consists of six main chapters as follows:

d. Chapter One: Introduction. This chapter provided a summary of the major goals of the research, as well as a description of issues, as well as the study's aims and objectives.

e. Chapter Two: Background and Literature Review; It the purpose of this chapter to provide an overview of shear reinforcement provisions in codes of practice for reinforced concrete beams and to draw attention to the knowledge, tasks, and techniques that are required to comprehend the fundamental philosophy and principles of building design code documents for reinforced concrete structures.

f. Chapter three: Similar Work. This chapter shows the similar project and research in the same domain with comparison.

g. Chapter Four: Structural Analysis.

h. Chapter Five: Verification and Experimental Work

i. Chapter Six: Results and Discussions.

j. Chapter Seven: Conclusions and Suggestions for Further Studies

1.6. CONCLUSION

In this research:

a. The design of 21-story building construction will be addressed as a subject, with an emphasis on understanding how various codes may influence the lateral pressures on shear walls effect.

b. Cases in other designs that are similar to this one are compared.

c. The outcomes of the building design are evaluated and researched with the assistance of a literature study once the structure is completed.

2. BACKGROUND AND LITERATURE REVIEW

The phrase "structural design" is used to refer to the process of deciding on the amount and kind of materials to use, as well as the best layout for supporting loads without compromising safety or functionality. It is the branch of engineering that deals with fixed objects like bridges and buildings [17].

The design of reinforced concrete slabs, beams, columns, and foundations is often done within the confines of regulations that outline specific requirements for material selection, structural analysis, member proportioning, and other considerations. Sometimes the codes used are referred to as design codes. These documents are legally binding and define the barest of requirements for building safe structures; they are drafted by people with deep engineering skill.

Turkish standards (TS 500), the Unified Arabic Code (UAC), the Canadian code (CSA-A23.3-94), and British standards are only a few of the numerous regional and national structure design codes utilized across the world (BS 8110). Some nations and areas also make use of American codes (ACI 318). While some countries and regions have developed their own national or international codes like Europe's Euro code and the United States' ACI 318, others (often developing nations) do not make use of the application of specific design standards. Oftentimes, foreign national codes are referenced by structural engineers in these countries.

To design structures in the lack of a national design code, structural engineers in Nigeria turn to international design codes such as the BS8110, Euro code 2, ACI 318 and a slew of other international design codes. They believe these codes to be helpful in adhering to the legal requirements in the country. Designers and project owners, on the other hand, usually examine the specifications in these codes in order to find points of similarity and difference. However, while the primary goal of these design codes is to give instructions for the construction of safe and cost-effective structures based on a set of principles, methods, and assumptions, these codes may differ in their approach. Research has proven that some codes are more cost-effective than others.

In engineering, the goal is to create buildings that are both safe and high-quality, while also doing it at the lowest feasible cost. When it comes to engineering, safety and economy go hand-in-hand; therefore, a structure that's both safe and cost-effective is considered a good engineering construction. These distinctions can be better understood and interpreted through comparative investigations. As a result, the structural engineer will be better able to select the most cost-effective code for the design of the proposed construction. On the other hand, Despite the shear walls' poor detailing or construction with low strength materials, reinforced concrete buildings with a large amount of reinforced concrete shear walls have shown satisfactory performance in severe earthquakes such as those that occurred in Nicaragua in 1972, Chile in 1960, Armenia in 1988, and Venezuela in 1967 [17]. According to Badaux and Peter (2000), shear wall structures have a high degree of stiffness, lateral resistance, and interstory distortions, and they are quite stable. After the Chile earthquake, Fintel [4], Shear walls were shown to be efficient in preventing structural and non-structural damage to buildings even when cracking was present [18]. Thus, it is crucial to adopt appropriate shear wall area to floor area ratios in order to boost the seismic resistance of reinforced concrete buildings. In this chapter, the shear walls characteristics and kinds will be discussed. Then, a deep research about codes will be represented.

2.1. SHEAR WALLS' GENERAL CHARACTERISTICS, DESCRIPTION AND CLASSIFICATION

To withstand external vertical and horizontal loads, civil engineers utilize two basic types of structural systems: concrete frame and concrete frame-wall systems. However, ATC 40 [19] specifies that in concrete frame systems, the horizontal and vertical loads are borne by the frames; however, in concrete frame-wall systems, the shear walls are responsible for lateral resistance as well as certain local vertical loads. System of big weakly reinforced walls; ductile wall system; inverted pendulum; and torsional flexible systems are all included in the six structural categories of the Eurocode 8 for reinforced concrete structures. According to Turkish Earthquake Code [20], Concrete frame systems, concrete wall systems (with or without apertures), and concrete frame-wall hybrids are the three major categories of structural systems. Academics, researchers, and engineers all agree that in earthquake-prone areas, shear walls should be used in combination with a concrete frame-wall system. Gulkan and Utkutu [21] report that despite significant earthquakes, these buildings did not collapse and that the majority of them satisfied immediate occupancy conditions. Concrete frame-wall structures have shown superior seismic performance and resilience than concrete frame systems in experiments and analyses [21]. During the Caracas Earthquake in Venezuela, Fintel [4] found that structures with shear walls performed better. The building's seismic performance, which is the building's performance under earthquake loading, is determined by the building's strength, stiffness, and deformation capacity. The rigidity of the constructions is increased by the use of reinforced concrete shear walls, which reduces the observed distortion and drift values. Asymmetrical shear walls are also significant, since they ensure that inelastic deformations are evenly distributed during seismic activity.

Shear walls are vertical load-carrying features having a minimum length to thickness ratio of 7 and a minimum thickness of 0.2 meters, as prescribed in the Turkish Earthquake Code [20]. According to Gulkan and Utkutu [21], the ratio of wall height to wall length, which is 10:1, would not permit the use of a member with a thickness of 0.2 meters and a length of 1.4 meters in a five-story construction or a 14-meter-tall building. Some codes also categorize shear walls by their aspect ratios, which are described by the ratio of the wall's height to its length (hw/lw). According to ASCE 41 (2007), shear walls are classified as either squat (aspect ratio of 1.5 or less), short (aspect ratio of 3.0 or more), or intermediate (aspect ratio of 1.5 to 3.0) depending on their relative width to height. Despite the fact that ATC 40 (1996) requires both thin and thick shear walls to have an aspect ratio of 4 or greater, the code implies that squat shear walls have a height-to-length ratio of 2 or less. According to Aejaz and Wight [22], squat walls have an aspect ratio of 0.5 or less, while long walls have an aspect ratio of 2.0 or higher; the behaviour of narrow and squat shear walls is controlled by shear and flexure, while the behaviour of intermediate shear walls is affected by the combination of shear and flexure. The terms "slim walls" and "squat walls" are commonly used to describe shear walls that are more likely to fail due to shear or flexure loading. Thin and short walls are seen in Figure 2.1.

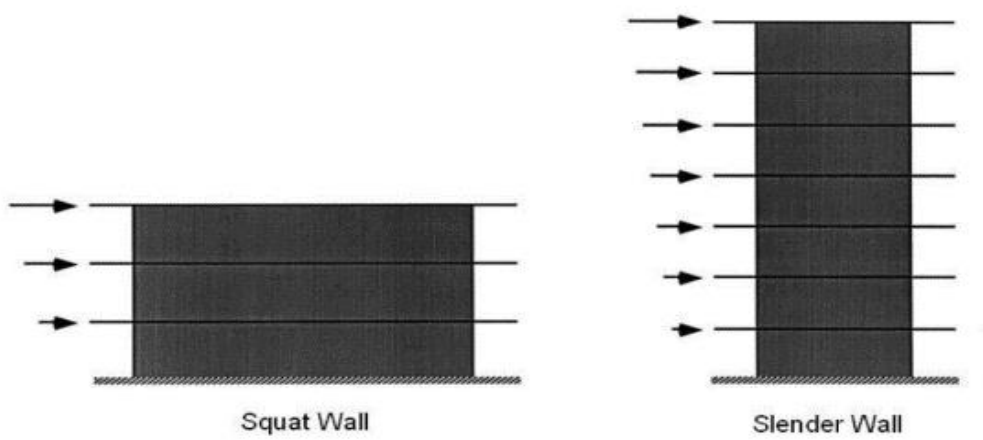


Figure 1.1 Types of Structural Walls

Because shear failure of walls is undesirable due to its brittle nature, Modifying squat shear walls such that they fail in flexure rather than shear is a major area of study. Since squat shear walls are structurally similar to deep beams in terms of geometry, it is possible to extrapolate findings from experiments conducted on deep beams to better comprehend the behaviour of squat shear walls. Squat shear walls can have their ductility needs lowered and an inelastic flexural response achieved with careful planning and specification of web reinforcement [22]. Thin shear walls, on the other hand, give way under flexure and can be employed in buildings of moderate to great height. The plastic hinges generated by the yielding of the flexural reinforcement at the base of the specimens was found to be sufficient for the inelastic deformation capacity of structures with thin shear walls, as discovered by Oesterle et al. [23]. Strategic placement of horizontal and vertical reinforcement in thin shear walls can increase the stiffness and lateral resistance of reinforced concrete shear walls and, by extension, reinforced concrete structures; nevertheless, Illiya and Bertero [23] also recognize the need for diagonal reinforcement. Illiya and Bertero [24] discovered that adding diagonal reinforcement to shear wall specimens improved seismic behaviour.

Coupled walls are another sort of structural wall (Figure 2.2). Coupled wall systems contain large apertures owing to architectural or technological requirements, and they are made up of shear walls and coupling beams that link the shear walls. To enhance the seismic performance of these structural systems, it is necessary to increase the energy dissipation capacity of the coupling beams, which absorbs and dissipates energy. Coupling beams with diagonal reinforcement give more structural strength, stiffness, and energy dissipation capacity than those with conventional reinforcement, as recommended by ASCE 41 (2007), and are thus essential for enhancing seismic behaviour. Pierced wall systems, like linked wall systems, have microscopic openings in the structural walls; however, unlike in connected wall systems, the apertures in pierced wall systems do not compromise the walls' seismic performance. Multiple structural wall systems are shown in Figure 2.2.

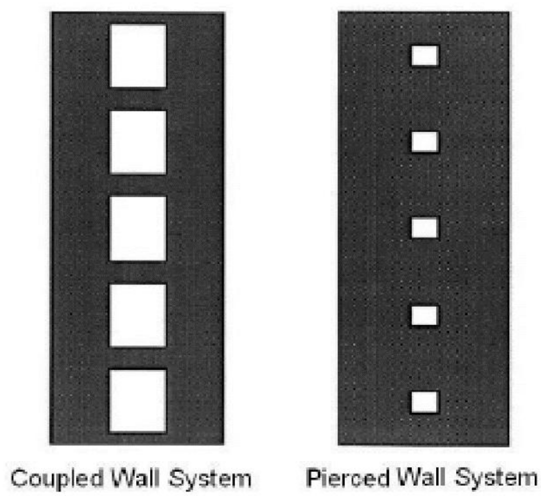


Figure 1.2 Types of Structural Wall Systems

In addition to the cross-sectional shape of the walls, reinforced concrete shear walls are classed as structural walls with barbell or channel, flanged, rectangular, T or L-shaped, or other shapes, among others. Barbell-shaped shear walls, in contrast to the web of structural walls, have huge, stiff boundary components and thin webs that are subjected to extreme shear stresses. However, in contrast to the web of the structural walls, appropriately detailed and constrained boundary components may resist greater shear pressures and axial loads, delaying inelastic bar buckling and maintaining shear strength. When stress and deformation are quite severe, the walls of the structure may show signs of web crushing [23]. In addition, Oesterle et al. [23] shown that well-designed boundary components may improve concrete's strain capacity, raise the material's shear capacity and wall stiffness, and prevent the inelastic buckling of vertical reinforcement.

Despite the wealth of research on the seismic behaviour of rectangular structural walls, little is known about the behaviour of nonrectangular reinforced concrete shear walls such channel, T, and L-shaped walls, which are also often observed in construction. These sorts of walls are typically seen in the vicinity of corridors or elevator shafts for architectural reasons. For T-shaped shear walls, the mode of failure is a combination of shear and flexure, with concrete crushing occurring at the bottom of the web and longitudinal web reinforcement reaching the limit of its deformation [24]. However, unlike their rectangular and L-shaped counterparts, T-shaped structural walls have a far lower weight bearing capability. In contrast, T-shaped shear walls are flexible, resilient, and able to dissipate energy. Load bearing capability of such walls may be improved by increasing the longitudinal reinforcement of the web edge [24]. Most importantly, pay attention to the diagonal orientation [25] of any channel- or U-shaped structural walls. When constructing channel-shaped structural walls, it is important to take into account the diagonal orientation of the walls in order to achieve appropriate analytical findings [24]. Finally, L-shaped shear walls outperform rectangular shear walls in terms of seismic performance, and as a result, they are frequently utilized in the corners of structures [24], [25].

According to the American Concrete Institute's 40th Annual Technical Conference, reinforced concrete wall-frame systems are prone to design problems such as vertical discontinuity, weak stories, shear cracking, diagonal tension/compression, and more (1996). For maximum stability, shear walls should be constructed up from the ground up (see Figure 2.3). Whenever columns or shear walls are removed to provide room for parking and businesses, a weak story is developed, which is a fairly prevalent flaw in Turkish construction. The stiffness and strength of these sorts of buildings differ dramatically from one level to the next (ATC 40 (1996)).

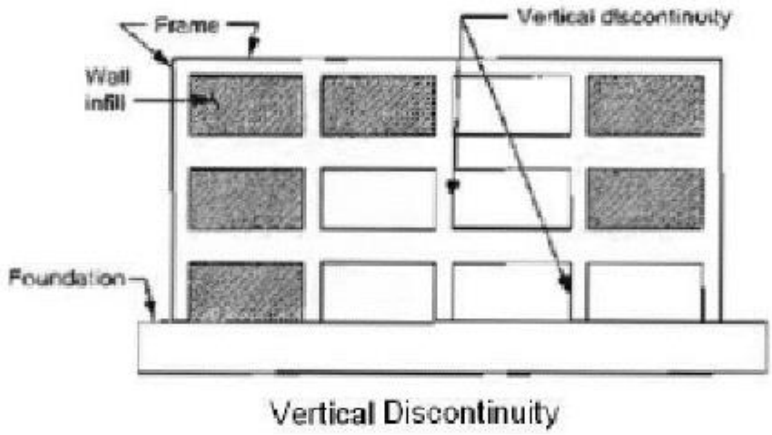


Figure 1.3 Typical Vertical Discontinuity (ATC 40 (1996))

2.2. MODELLING OF SHEAR WALLS

Reinforced concrete shear walls give excellent rigidity and seismic resilience to the building while also limiting the amount of interstory drifting that occurs. Heavily employed in the construction of new structures as well as the rehabilitation of existing structures in seismically active areas, shear walls are becoming increasingly popular in these areas. It is critical to precisely simulate the nonlinear behaviour of the structural walls in order to avoid errors in the design. Cross-sectional dimensions, aspect ratio, axial-flexure interaction (axial-flexure interaction = axial-flexure ratio), bond properties (bonding ratio), reinforcement detailing of the boundary elements, influence of connecting members (rigid-body rotation), and flexural capacity based on the shear capacity of the shear wall are all factors that need to be taken into account when modelling structural walls, as stated by the authors of Galal and Sokkary [26]. Although one may easily predict the flexural response of shear walls, accurately describing the combined flexural and shear response of the structural walls in a finite element analysis is more challenging. Analytical models of reinforced concrete shear walls often fall into one of two categories: either microscopic models or models that are scaled out to a macro level. Macroscopic models of shear walls incorporate the elements of the wall, such as concrete, reinforcement, and the relationship between concrete and reinforcement, to account for the entire reaction of the wall inferred from test data and observations. However, microscopic models account for the local behaviour of the structural walls in great detail since they are based on solid mechanics. Meso models, which fall somewhere in between macroscopic and microscopic in scale, has certain features of both but also have their own unique qualities [27].

The equivalent beam element model, the vertical line element model with a variable number of springs, the truss element model, the braced frame analogy, and the braced broad column analogy are the most often used macroscopic models in the industry. The microscopic approaches may be divided into two categories: the finite element method and the fiber technique. As a result of the simplicity, efficiency, and applicability of macroscale models, they are frequently used in statistical analysis.

2.2.1. Macroscopic Models of Shear Walls

Some of the most popular macroscopic models in use today are the comparable beam element model, often known as the broad column analogy. In this model, a line element is defined as the shear wall along its censorial axis; it shares the shear wall's moment of inertia and cross-sectional area. This line element is connected to the neighbouring components by infinitely stiff beams, which are positioned at each floor level of the building. Figure 2.4.a is a simple representation of the broad column analogy model. Half of a shear wall's length is the length of a stiff beam. For this model [28], it is assumed that flat floor surfaces would maintain their flatness after the application of lateral loads. Figure 2.4.b displays the use of the equivalent beam element model, which is straightforward and has minimal degrees of freedom, to calculate the seismic response of shear walls. However, the strain distribution of the wall is incorrect since the model does not account for the shifting of the neutral axis owing to flexural cracking and yielding of the wall reinforcement [27].

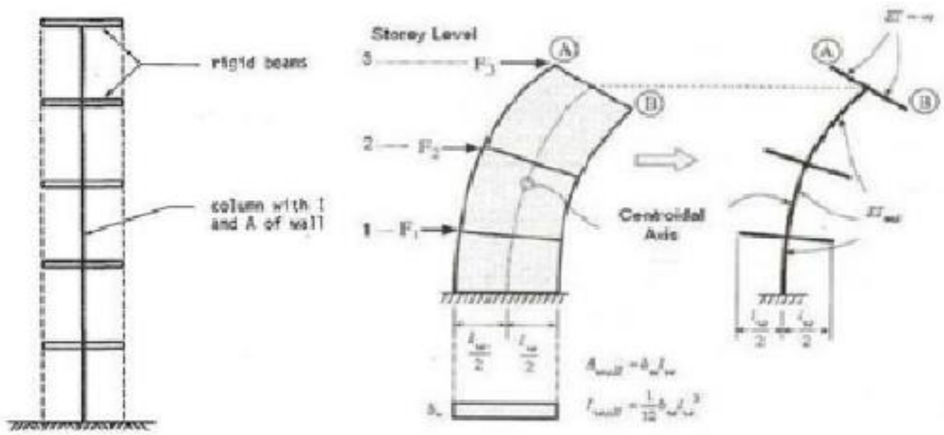


Figure 1.4 a) Wide Column Analogy, b) Equivalent Beam Element Model

As shown in Figure 2.5, the truss element model is comparable to the shear wall element model in that it also includes two vertical boundary truss elements at either end of the shear wall, a horizontal stiff element that represents shear reinforcement, and at least one diagonal truss element. For the model described by Galal and Sokkary [26], the boundary columns are vertical, the horizontal rigid beams carry the tension, and the diagonal truss components carry the compression under lateral stress; these columns resist the acting moment. Wall shear response to lateral stresses is studied by modelling the shear wall as a statically determinate truss. This model is inadequate for predicting the whole seismic response of the supporting walls. Comparative Model of Braced Wide Columns: Although the braced broad column analogy model also has rigid beams at the floor levels and a column element at the centroidal axis of the structural wall, it differs in that it also includes diagonal braces with hinged ends attached to the beam elements. The stiffness of the columns and shear wall is estimated in accordance with the guidelines of Smith and Girgis [29], which is essential for creating an accurate model of the structural wall. Using the bending moment, shear force, and axial force applied to the column, as well as the axial force applied to the diagonal braces, the model calculates the axial force, shear force, and moment capacity of the structural wall.

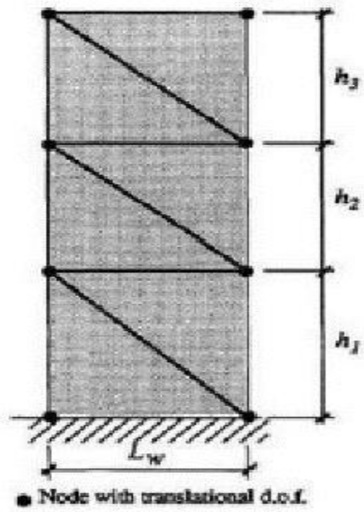


Figure 1.5 Truss Element Model

The braced broad column comparison is seen in its streamlined form in Figure 2.6.a. Smith and Girgis [29] also came up with the braced frame comparison, which may be shown in Figure 2.6b as another macroscale model. This model employs a number of structural components, including two column elements at either end of the shear wall, stiff beam elements at each floor level, and hinged diagonal braces. Stresses in the shear walls are determined by applying the same method to computing the bending, shear, and axial stiffness of the shear walls as was used in the braced broad column example. In terms of planar and nonplanar shear walls, it has been demonstrated that both the braced frame and braced wide column analogies are valid.

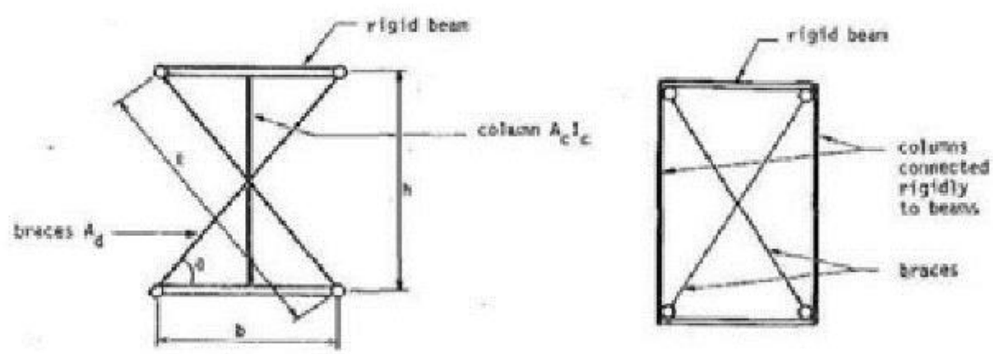


Figure 1.6 a) Braced Wide Column Analogy, b) Braced Frame Analogy

In order to determine the maximum roof displacement and maximum inter-story drift ratio under earthquake conditions in multi-story structures, the continuum technique is used as a macroscopic approach. Miranda et al. [30] discovered that this model used both the flexural cantilever beam and the shear cantilever beam with non-uniform lateral stiffness distribution throughout the height of the structural wall. For the sake of simplicity, the connecting links are shown as axially stiff beams in this plan view of the structural system (Figure 2.7). That's why under lateral stresses the horizontal deflections at each floor are identical. It is shown that differences in lateral stiffness do not significantly alter the ratio of spectral displacement to maximum roof displacement in a multi-story building, but do have a small effect on the ratio of maximum inter-story drift to roof drift [30].

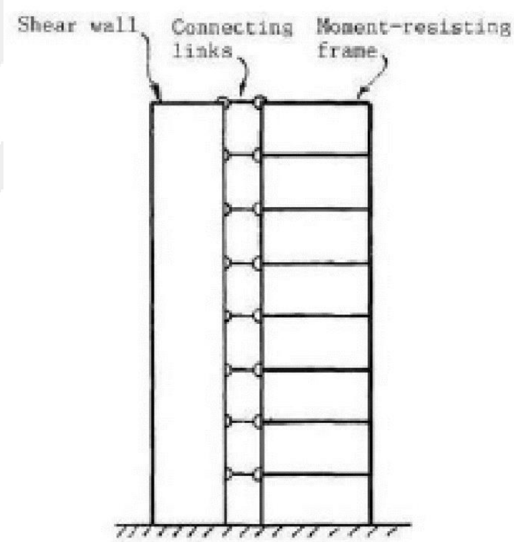


Figure 1.7 Continuum Model for a Multi-story Building

Horizontal rigid beams at each floor, like in the equivalent beam element model and the braced wide column model, two vertical truss elements at each end of the shear wall with the axial stiffness of boundary columns, and one central vertical line element representing the shear wall web are the components of the three vertical line element model (TVLEM) proposed by Kabeyasawa et al. [31]. Three vertical line components are used in this model, and five springs are put on each of them, as illustrated in Figure 2.8. The axial stiffness of the boundary elements is represented by nonlinear axial springs that are employed for each vertical truss element at the wall's ends and represent the axial stiffness of the boundary elements. Spring elements are positioned near the base of the primary vertical element, including horizontal, vertical, and rotating springs. Shear capacity is represented by the horizontal spring, while flexural capacity is represented by the rotating spring at the base of the central vertical element and the axial springs of vertical truss elements at the ends of the shear wall. The deformation and strength of a shear wall under bending are determined by a three-vertical line element model, which includes two exterior vertical truss elements and one core vertical line element, as shown in the figure. In the event of lateral loads, one of the outer vertical trusses bears tension while the other bears compression. Using the extension of the boundary column, which conveys tension, the bending deformation of the shear wall may be calculated and shown. According to Kabesayawa et al. [32], a three-vertical line element model may be utilized to calculate both the overall behaviour of the structural system and the member behaviour of a reinforced concrete shear wall.

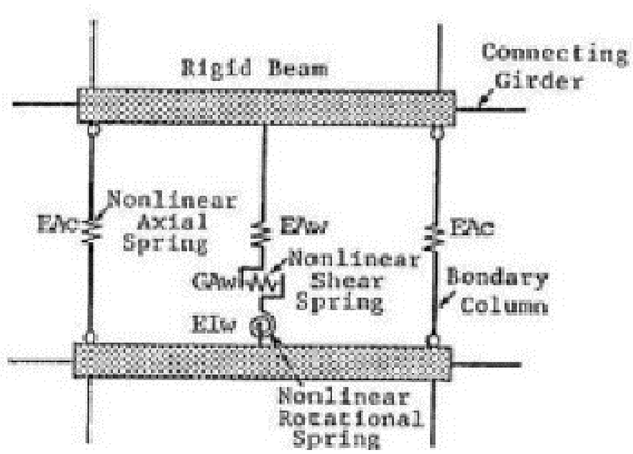


Figure 1.8 Three Vertical Line Element Model

Similar to the model produced by Kabesayawa et al. [32], but without the centre line element's spinning spring at its base, Linde [28] created a three-vertical line element model. Figure 2.9.a shows that this design utilizes a total of four axial springs. The shear wall's flexural behaviour is characterized in this model by the interaction between the two outside vertical springs and the central vertical spring, while the shear wall's shear behaviour is determined by the horizontal spring at the base of the centreline element. The nonlinear performance of a shear wall may be reliably estimated using Linde's [28] three-vertical line element model. The axial element in series model (AESM), also created by [32], is a three-vertical line element model called after the series connection of axial springs (Figure 2.9.b). A one-component element reflects the link between reinforcement and concrete, whereas a two-component element indicates the axial stiffness of the border components of the wall when there is no bond between reinforcement and concrete [32]. Predictions of the structural wall's flexural response can be made with this model, but the shear behaviour can't be studied because the model uses only axial elements in series (AESM).

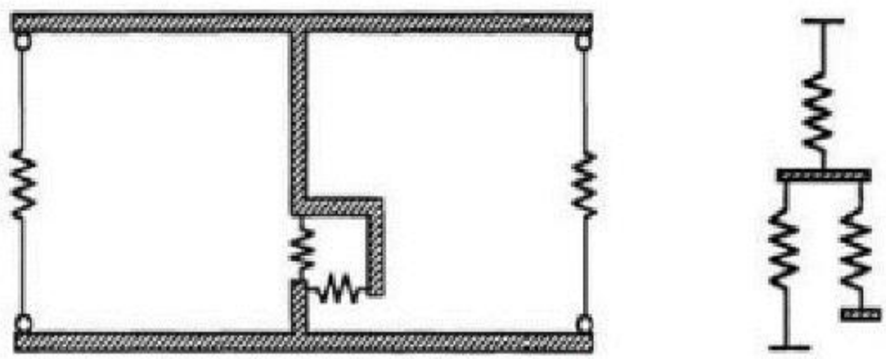


Figure 1.9 a) Three Vertical Line Element Model, b) Axial Element in Series Model

The multiple vertical line element model (MVLEM) proposed by Vulcano et al. [33] removes the rotational and vertical springs at the base of the central line element and replaces them with several vertical springs at many vertical trusses (Figure 2.10). The inelastic shear behaviour of the wall is modelled using a single horizontal axial spring positioned at the centre line element, in contrast to the three vertical line elements used in the models produced by Kabeyasawa et al. [32] and Linde [28]. The model also includes rigid beams at the floor levels to simulate the boundary elements of the wall. Shear wall activity is provided by additional axial vertical springs that combine axial and flexure behaviour. The multiple vertical line element method is more realistic and accurate than the other three vertical line element methods when studying the gradual yielding of the vertical reinforcement of the shear wall; however, this model is relatively more complicated than the other two methods due to the presence of multiple vertical springs.

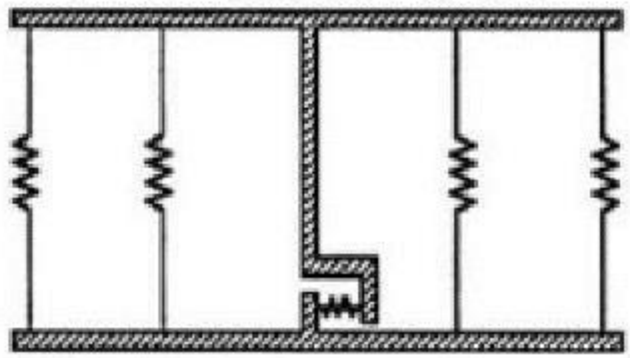


Figure 1.10 Multiple Vertical Line Element Method

Shear walls' global response to lateral force may be predicted with the help of a panel element model developed by Chen et al. [32]. This model has infinitely stiff beams at each floor level that are connected by axial springs and two outside vertical truss components with axial springs at either end of the shear wall (Figure 2.11.a). Shear wall web panels, also known as web panels, are used in the models as isoperimetric, incompatible rectangular elements (Figures 2.11.b and 2.11.c). In shear walls that fail due to flexure, isoperimetric elements in panel element models overestimate the shear deformation of the structural wall, while incompatible components in panel element models more precisely anticipate the shear and flexural deformations. It was observed by [31] that there was a strong correlation between the analytical and experimental findings of both elements.

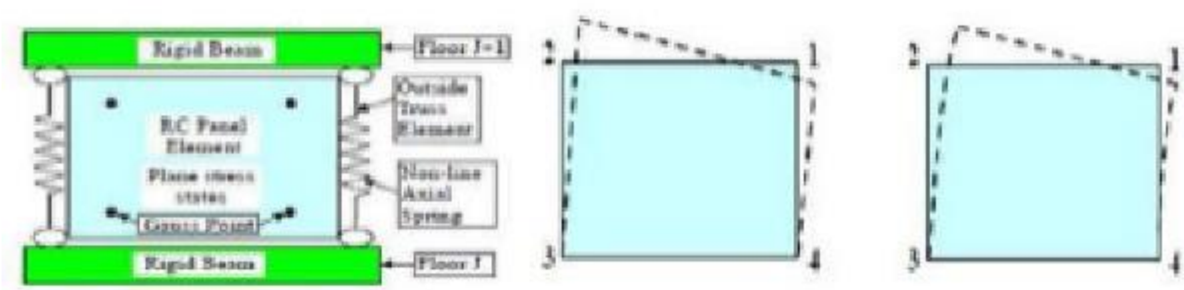


Figure 1.11 a) Panel Element Model, b) isoperimetric Element, c) Incompatible Element

2.2.2. Microscopic Models of Shear Walls

Microscopic approaches are also employed to simulate reinforced concrete structural walls, and the behaviour of each individual material, such as the reinforcement and concrete, as well as their interaction with one another is investigated. When a comprehensive study of local response of shear walls is required, a microscopic method is conceivable; however, It takes a long time to complete and can only predict how particular elements of a structure would react to a tremor. The finite element model (FEM) and the fibre model are two examples of microscopic models, with their respective representations shown in figures 2.12.a and 2.12.b.

The shear wall's global and local behaviour may be calculated using the finite element technique (FEM), which is commonly used to estimate the seismic behaviour of structural walls using a limited number of microscopic elements. The results also show that the shear wall's displacement response to seismic loads, as well as its yield point, yield strength, initial stiffness, and yield strength, can be approximated using the finite element method (FEM). To achieve the nonlinear behaviour of the shear wall, the fibre model, like the finite element technique, divides the member into several small elements. By utilizing the fibre model, one may make approximations regarding the moment curvature relationship of the structural wall at each load increment, the axial load - bending moment connection, and the flexibility distribution throughout the length of the shear wall.

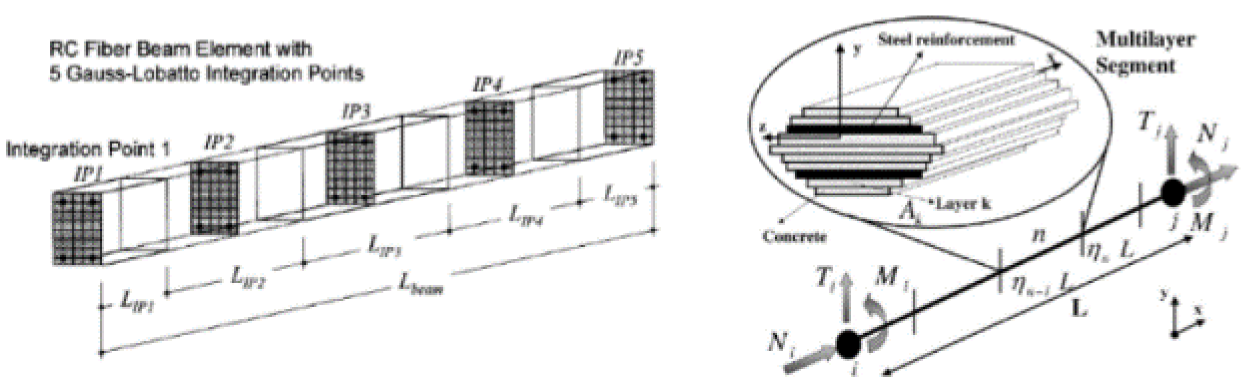


Figure 1.12 Microscopic Methods

2.3. SHEAR WALL RATIO OF THE STRUCTURES

According to the EERI report, even shear walls made of low-quality materials and poorly detailed can provide significant seismic capability to a building. Strengthening the nonlinear behaviour of reinforced concrete buildings, shear walls can withstand the lateral stresses generated by an earthquake. Therefore, adequate seismic resistance necessitates the adoption of a sufficient number of shear walls in the structural system. Shear wall ratios and column ratios are used to identify high-risk buildings in the evaluation of reinforced concrete low-rise monolithic structures provided by Hassan and Sozen [34]. This method was applied to 46 structures in order to provide an evaluation technique for them, and it simply requires the structural dimensions, shear wall ratio, and column ratio. Figure 2.13's X- and Y-axes stand for the column index (CI) and the wall index (WI), respectively. A comparison of the column index (CI) and the wall index (WI). The wall index (WI) is defined as the ratio of the total cross sectional area of reinforced concrete shear walls at the base of a building and a percentage of masonry walls at the base of a structure in the loading direction to the total floor area at the base of a building.

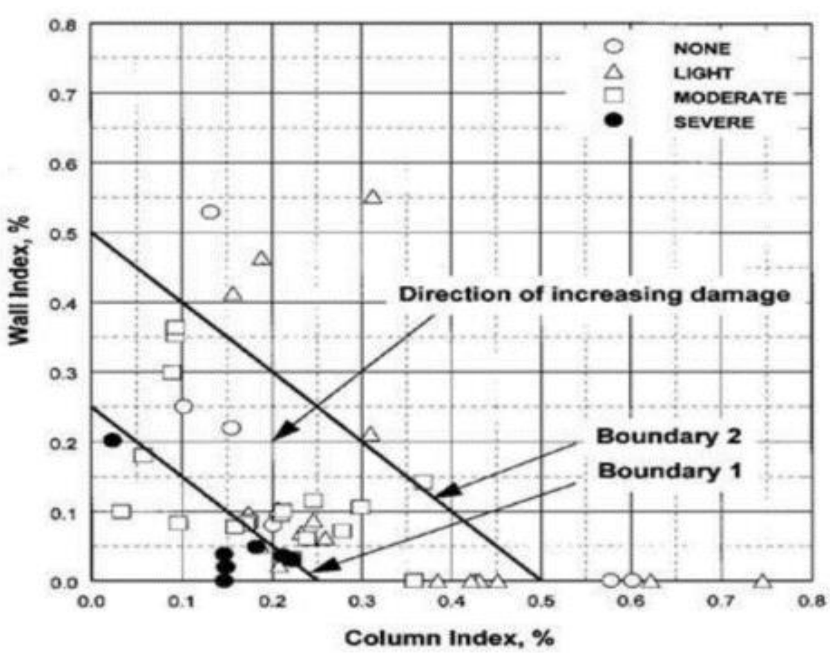


Figure 1.13 Evaluation Method Proposed by [33]

2.4. CORRELATION BETWEEN SHEAR WALL RATIO AND DRIFT

A structural element's deformation capacity as well as its seismic performance is two significant factors that contribute to seismic resistance and the prevention of excessive structural damage in reinforced concrete structures. Reinforced concrete structural walls play an important function in delivering high stiffness and deformation capacity to structures when they are subjected to earthquake loads. A building's shear wall ratio is also necessary in order to assess the degree of predicted drifts, such as roof and inter-story drifts, which may then be utilized in the seismic evaluation to estimate the level of damage in a structural system. However, there has only been a few numbers of research investigations conducted on the relationship between shear wall ratios and drifts in the literature.

2.5. AMERICAN CONCRETE INSTITUTE (ACI)

"ACI Building Code" is the official name of the American Concrete Institute's ACI 318-83 "Building Code Requirements for Reinforced Concrete" (ACI). To utilize the ACI Building Code as a contract document for construction is not permitted. The ACI Building Code is based on the premise that building codes are designed to provide the minimal standards necessary to ensure public safety. Reinforced concrete buildings must be constructed in accordance with the code's guidelines. The ACI Building Code is included into the building codes of a wide number of governmental entities and agencies, making it a legal obligation in their territories. Several standard requirements have been established by the ACI as well. ACI 301-84, "Specifications for Structural Concrete for Buildings," is perhaps the most well-known of them. These requirements are not legally enforceable, but they can be referenced in project specifications by an architect or engineer. As a result, the ACI requirements are now binding for that project. In addition to the ACI guidelines, the project's architect or engineer adds additional requirements based on the project's needs. ACI 301-84 includes the vast majority of the essential specifications for the construction of reinforced concrete structures in general. As published in the official report of ACI [35], To be utilized in a legally recognized building code, "Building Code Requirements for Structural Concrete (ACI 318-95)" must be distinct in form and content from publications that contain precise specifications, suggested practices or design assistance. The code is meant to encompass all sorts of structures, large and small. For a typical building, stricter requirements than the regulation may be desirable. However, code and discussion cannot substitute for technical expertise. A building code merely specifies the minimal criteria for public health and safety. This is the ACI Building Code. The owner or structural designer of any project may seek higher quality materials and construction than the regulation requires to safeguard the public. But lesser standards are not allowed. The commentary refers to additional publications that offer recommendations for implementing the code's criteria. They are not meant to be part of the code. The code is not legally binding until it is accepted by government agencies responsible for building design and construction. Even if the code isn't accepted, it can serve as a guide to good behaviour. The code establishes basic criteria for design and construction approval by a legally appointed building authority or his agents. A disagreement between the owner, engineer, architect, contractor or their representatives is not resolved by the code or commentary. Thus, in normal construction, the code cannot describe each party's contract responsibilities. Because the contractor is rarely in a position to assume responsibility for design specifics or construction requirements that require extensive understanding of the design, general references to ACI 318 should be avoided. Generally, the drawings, specifications, and contract papers should include all code-related requirements. Specific code parts in the task specs might help. Construction contract documents, such "Specifications for Structural Concrete for Buildings" (ACI 301).

2.5.1. Strength Design Method

The strength reduction factor is zero; while Rn is the nominal resistance in ACI 318M-11 reinforced concrete design. It is called strength design because the given strength must exceed the needed strength to handle the calculated loads. Member strength is calculated by checking the ultimate limit states against serviceability limit states.

2.5.2. Load Combinations

Section 9.2.1 of the 2011 ACI Code gives load factors and load combinations to be utilized with the strength-reduction factors in Sections 9.3.1-9.3.5.

Table 1.1 Load Combinations

Load cases	Load
D	U = 1.4D
D+L+Lr or S or R	U = 1.2D + 1.6L + 0.5(Lr or Sor R)
D+ Lr or S or R +L or W	U = 1.2D + 1.6(Lr or S or R) + (1.0L or O.SW)
D+L+W+ Lr or S or R	U = 1.2D + 1.0W + 1.0L + 0.5(Lr or S or R)
D+L+E+S	U = 1.2D + 1.0E + 1.0L + 0.2S
D+W	U= o.9D + 1.ow
D+E	U = 0.9D + 1.0E

2.6. BRITISH STANDARD (BS)

BS 8110-97 structural use of concrete is based on Limit-States Design principle.

2.6.1. Limit-States Design

Limit state design is considered as a mix of elastic (keeping stresses in the structure within the material's elastic range) and plastic (load factor) design. BS 8110 blends these two approaches well. The major goal of the limit state design process is to ensure that the structure fulfils its purpose throughout the design life. Excessive bending, cracking, and deflection can render a structure unsuitable. They are called limit states. The Ultimate limit state can cause partial or total failure of a building, whereas the Serviceability limit state impacts the structure's appearance. Overall stability is estimated by calculating the load that will induce collapse, while serviceability is checked under typical operating loads. It entails identifying major limit states (i.e., all conceivable mechanisms of failure), determining the acceptable levels of safety against each limit state. Design codes that describe load combinations, factoring and structural design for important limit conditions.

2.6.2. Partial Factors of Safety for Materials

To account for errors in material strengths, imperfections in the design equations, differences in concrete section size and reinforcement placement, the importance of members in structural approximations, etc. partially safe materials (BS8110) (Ym).

$$\text{Design strength} = \frac{\text{Characteristic strength}}{\text{Material partial factor of safety (Ym)}} \tag{2.1}$$

Table 1.2 Material Partial Factors of Safety (Ym) At the Ultimate Limit State

Limit state	concrete	steel
flexure	1.5	1. 15
Shear	1.25	1. 15
Bond	1.4	

2.6.3. Partial Factors of Safety for Loads

BS8110-1997 also imposes a partial factor of safety for loads to account for mistakes and inaccuracies related to design assumptions, calculation errors, unexpected load increases, and construction inaccuracies.

$$\text{Design load (U)} = \text{characteristic load} * \text{partial load factor of safety (Yr)} \tag{2.2}$$

2.6.4. Percentage of Longitudinal Reinforcement

BS 8110-97 specifies the minimum and maximum longitudinal reinforcement as a percentage of the column's gross area Ag. The lower limit is to account for possible analytical mistakes and to lessen the effect of column creep and shrinkage. High reinforcement ratios are not only uneconomical, but also pose practical issues in concrete placement due to the reinforcements' density. This increases the likelihood of honeycomb in the concrete, reducing the column's load-bearing capability.

2.7. EUROPEAN CODE (EC)

The European Union commissioned CEN - European Committee for Standardization to create a set of standards known as the Euro codes with universal principles for structural design inside the European Union. Euro code 8 is a European seismic code developed by the European Commission that is used for the design and construction of buildings in seismic zones. Its primary purpose is to keep people safe while also minimizing structural damage in the event of a disaster such as an earthquake. This code takes into consideration the capacity design requirements based on the ultimate limit states and serviceability constraints of structural systems. Flexural failure and shear failure are two distinct types of failure for RC walls, according to the European Commission. As demonstrated by the projected kind of failure, the behaviour factor to be used in the analytical method indicates the system's expected ductility, which may be determined by the predicted type of failure. This section contains design statements that are based on various modes of failure. Shear wall design approaches are discussed in detail in detail in the following parts of this document.

2.7.1. Standard method of shear design

According to the ratio of minimum design strength at shear failure mode (diagonal compression, diagonal tension, or shear sliding) and design flexural failure of the critical wall region scaled by global factor, it is generally possible to distinguish two failure modes. The global factor is used to compensate for the selection of a partial safety factor of steel and to cover partial hardening effects in the critical wall region.

In order to guarantee that the predicted mode of failure is achieved, a realistic -value must be established at the initial design of the wall reinforcement. A higher limit than one should be assumed for thin walls constructed for high or medium ductility (DC H or M) and for flexural failures, with the premise that the top limit should be larger than one. If squat walls (height-to-length ratio less than 0.75) are erected for one of the aforementioned ductility classes, it is reasonable to assume that they will fail in shear. Mixed wall failure is considered to occur in the event of walls with a height-to-length ratio between 0.75 and 2.0, depending on the wall height (flexural and shear failure). When attempting to forecast the development of a flexural or shear failure mode in a wall, the layout of the wall's reinforcement in the "critical" area is a significant component to take into consideration. This region, which is generally comparable to the height of the shear wall or one-sixth of the height of the structure, covers a section of the lower portion of the shear wall and is roughly equivalent to the height of the wall. When estimating the shear capacity of a wall, it is required to take into consideration the shear resistance of the concrete and reinforcing steel for a variety of different forms of failure. For concrete's diagonal compression and tension failure to occur, the following inequality must be satisfied:

a. For diagonal compression failure of the web:

$$V_{Sd} \leq V_{Rd\,2} \tag{1.1}$$

For

$$V_{rd\,2} = 0.4\left(0.7 - \frac{f_{ck}}{200}\right) f_{cd} \cdot \; b_{w0} \cdot z \tag{1.2}$$

b. For diagonal tension failure of the web:

$$V_{Sd} \leq V_{Rd\,2} \tag{1.3}$$

For

$$V_{Rd\,3} = V_{cd} + V_{wd} \tag{1.4}$$

c. For $\alpha_s \leq 1.3$, the following expression should be satisfied:

$$V \leq \left[\rho_h \left(\alpha_s - 0.3\right) f_{yd,h} + \rho_h \left(1.3 - \alpha_s\right) f_{yd,h}\right] b_{w0} z + V_{cd} \tag{1.5}$$

2.7.2. Criteria for Designing Lightweight Reinforced Concrete Shear Walls

When designing earthquake-resistant lightweight shear walls, it is essential to ensure that the structure is capable of withstanding both horizontal and vertical loads while releasing sufficient energy to do so effectively. The maximum practicable shear and moment values are frequently required by designers when assessing the distribution of seismic forces in a structural design. Given the various load combinations, the designer should be able to anticipate whether flexural or shear failure would occur in key wall sections when deriving shear and flexural capacities in key wall sections (Figure 2.14). Ideally, the shear capacity of a ductile system should be greater than the strength that can be obtained from its available moment capacity, such that the structural walls of the system have more strength. Accordingly, it is probable that in the wall design, shear load capacity will be required in two times the amount that flexural load capacity would be required. The various "ductility classes" utilised in earthquake resistant concrete construction are taken into consideration in the various design requirements in EC 8. This is especially true for RC thin and squat walls. An example of a slim wall is one with a height to length ratio more than 2, whereas an example of a squat wall is one that has a height to length ratio less than or equal to 2. The two most typical types of failure, flexural and shear are seen in the figure 2.1.

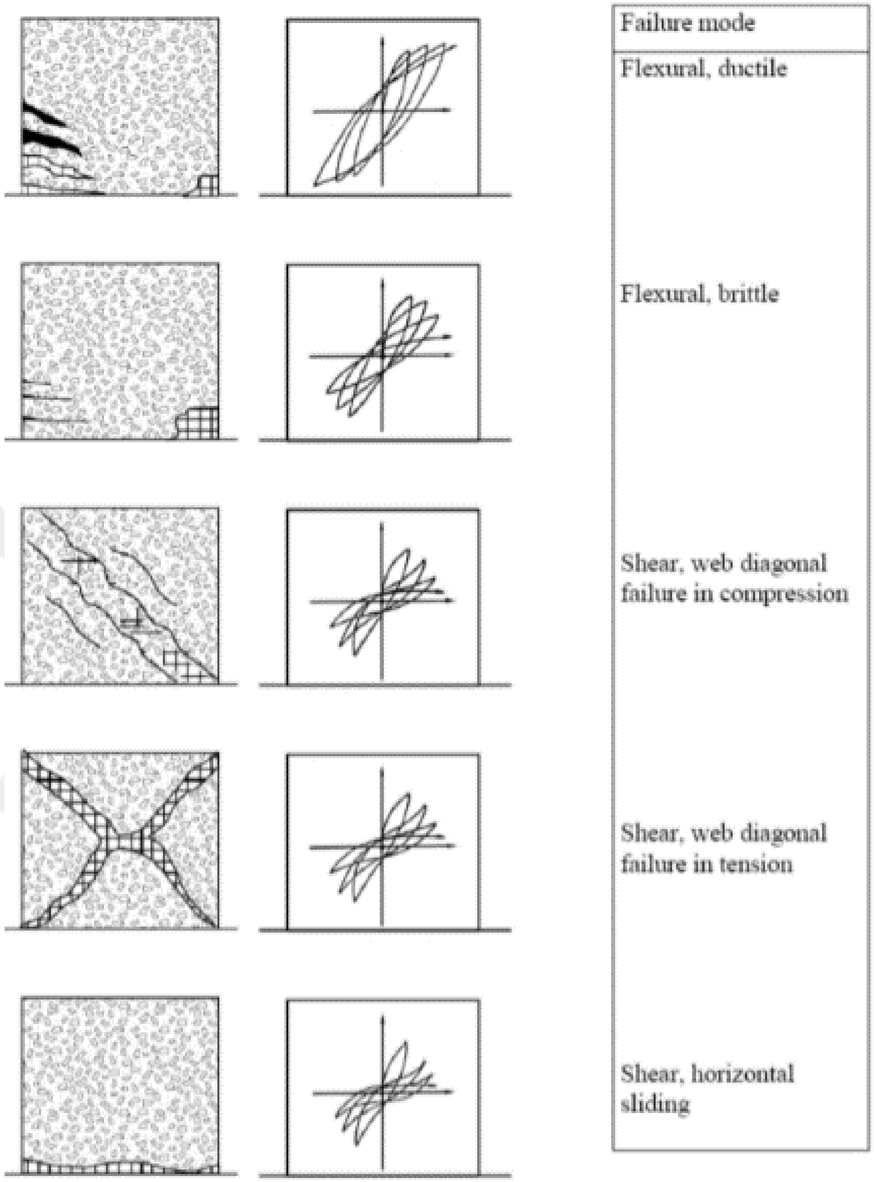


Figure 1.14 The RC wall's failure modes

2.7.3. Design of specimens according to EC 8

The LW-1, LW-2, LW-3, and LW-4 lightweight reinforced concrete shear wall specimens have been created using the procedures that have been previously developed. To begin with, a wall panel with an edge element of 250mm square has been selected for the preliminary design, which is 100mm thick.

LW-1	V_{Sd} = 440 kN	ρ_h= 0.78 %
	f_c ′= 30 MPa	ρ_v = 0.58 %
	fy = 400 N/mm^2	
LW-2	V_{Sd} = 433.2 kN	ρ = 0.57 %
	f_c ′= 30 MPa	
	fy = 400 N/mm^2	
LW-3	VSd = 558.8 kN	ρ = 0.75 %
	f_c ′= 30 MPa	
	fy = 400 N/mm^2	
LW-4	VSd = 530 kN	ρh= 0.87 %
	f_c ′= 30 MPa	ρ_v = 0.65 %
	fy = 400 N/mm^2	

2.8. CONCLUSION

The American Concrete Institute and the European Concrete Institute have significant differences in the design of reinforced concrete shear walls for shear. According to the American Concrete Institute (ACI), a concrete contribution is required to recognize the increased shear strength of walls with low shear aspect ratios. Higher concrete contributions are not permitted in the case of low thin walls, according to the European Code (EC). Shear span ratio is used to distinguish between the design equation for shear borne by the webbing reinforcements and another design equation for shear. There is only one requirement in the ACI Code: the ratio of vertical to horizontal reinforcements must not be more than 1. However, whereas EC8 explicitly acknowledges the possibility of sliding shear failure of squat walls, the ACI Code takes a more restrictive approach by setting an upper limit at the nominal shear stress level. Regarding wall reinforcement in accordance with both standards, the design according to the European Commission is more expensive than the design according to American Concrete Institute (ACI). While the ACI design is far more liberal than the EC design, the EC design is significantly more conservative. According to the findings of the study, both the ACI and BS codes successfully predicted the flexural capacity of single-reinforced cross-sections to within 4 percent of the true value. Herbarically, there are considerable variances in shear strengths between the two codes. For very slightly transversely reinforced sections, there is a 10 percent to 30 percent variation between the ACI code shear strength calculations and the BS code equations, depending on the application. When a result, as the longitudinal steel reinforcement ratio increases, the axial compressive strength predicted by the ACI code for concentrically loaded cross-sections falls between (10% and 25%) below the BS code projections. Dead weight, living weights and wind all contribute to the production of larger factored loads, which can reach 20 percent in some cases. This is especially true when the wind-to-dead weight ratios are low.

3. SIMILAR WORK

3.1 INTRODUCTION

In this chapter, a discussion about some similar work will be enhanced in order to make a clear idea about the common methodology in the fields.

3.2 REINFORCED CONCRETE DESIGNS BASED ON THE ACI 318 AND BS 8110 CODES

Building structures made of reinforced concrete are permissible in the United Arab Emirates if they are planned in line with the ACI 318 or the British Standard BS 8110 regulations. Because the requirements of the two codes are distinct from one another, it is necessary to compare the structural requirements of the two codes. Specifically, the ACI 318 code and the BS 8110 code are compared in terms of flexural, shear, and axial compression limitations and the results of this comparison are presented in [36]. The two codes are also compared in terms of load factors and load combinations, which is another important issue. In order to reach this aim, cross-sections with varied geometries, material properties, and reinforcement ratios are investigated in line with the methodologies in the two codes. The two algorithms additionally look at factored load combinations for a wide range of live-to-dead load ratios as well as a variety of wind-to-dead load ratios. According to the findings, there are minimum, moderate, and considerable differences in the design capabilities of the ACI 318 and BS 8110 codes for flexure, axial compression, and shear, respectively, between the two codes' design capabilities. Between the two codes, there are also minor to major differences in the computed load combinations for dead load, live load, and wind that are used in the calculations.

3.2.1 Factored Load Combinations

When many sources of load are applied to a structural element at the same time, there is a very little possibility that severe loads may occur at the same time. Possibly the most extreme case would be a mix of heavy snow, strong winds, and a huge earthquake. Some, but not all, of the group's loads may occur at the same time, but this is not guaranteed. As a result, while designing structures; structural design laws take into consideration realistic load combinations. The load combinations were created in such a way that all of them had roughly equal likelihood of exceeding their respective limitations. When designing for ultimate strength, the loads are multiplied by the load factors specified by the various codes of practice. Because of this method, these loads will only be occasionally exceeded throughout the duration of the structure's operational life. It is the combination of the load factors and the strength resistance factors that provides the overall factor of safety against strength failure. As specified by ASCE7, the load combinations in the ACI 318 code are based on the (ASCE7 2010). In the next section, you will find a list of the ACI 318 code factored loads involving service dead loads, floor lives loads, roof lives loads, and wind loads (D, L, and R) (W).

$$1.4D \tag{3.1}$$

$$1.2D + 1.6L + 0.5Lr$$

$$1.2D + 1.6Lr + (L\ or\ 0.8W)$$

$$1.2D + 1.6W + 1.0L + 0.5Lr$$

$$0.9D + 1.6W$$

Listed below are the BS 8110 code combinations that apply to the loads that we're thinking about.

$$1.4\mathrm{D} + 1.6\mathrm{L}\ 1.4\mathrm{D} + 1.4\mathrm{W}\ 1.0\mathrm{D} + 1.4\mathrm{W}\ 1.2\mathrm{D} + 1.2\mathrm{L} + 1.2\mathrm{W} \tag{3.1}$$

In addition, it should be noted that the ACI 318 code distinguishes between live loads on the floor (L) and roof (Lr), but the BS 8110 code does not make this difference and applies the same load factor to both types of loads.

3.2.2 Results

In order to calculate the factored (design) capacity of 400 distinct cases with varying cross-sections and material characteristics, the ACI 318 and BS 8110 rules were applied. The factored design capacity is determined by utilizing the ACI 318 code and the BS 8110 code, after all occurrences have been taken into account. On a graph, we can see how much of each code's design capacity corresponds to each of three potential limit states, and how much of each code's design capacity relates to each of the three possible limit states.

In this study, the formula f'c = fcu/1.2 was utilized since the ACI code equations use the concrete cylinder strength (f'c), whereas the BS code equations use the concrete cube strength (fcu) (BSI1881 1983). In the investigation of rectangular, singly reinforced cross sections, a wide range of width-to-depth ratios between 0.5 and 2.0 is taken into consideration. Specifically, we are interested in yield strengths of 250 MPa and 460 MPa for two distinct kinds of reinforcing steel. In the case of cubic concrete specimens, the limits of strength are between 20 and 50 MPa, but in the case of cylinder concrete specimens, the limits of strength are between 17 and 42 MPa. The research use both the ACI 318 code and the BS 8110 code, which are represented by the letters MACI and MBS, to estimate the factored flexural capacity of a certain cross-section. In 180 design samples analysed, the design capacity ratio, MACI/MBS, is shown to be highly correlated with the tension steel reinforcement index, fy/c'c. When the index is high, the cross section is more brittle; when the index is low, the cross section is more ductile. In this experiment, the stress reinforcement index ranges between 0.0125 and 0.25 percent. Figure 3-1 illustrates that the ACI 318 and BS 8110 codes provide results that are equivalent. According to Figure 3-1, when the cross-sectional moment arm is more than 0.95 (or fy/f'c is less than 0.56), which is the effective reinforcement depth from the extreme compressive fibers, the design capacity ratio MACI/MBS is lowered somewhat. Sections with z/d 0.95 (or 'c > 0.056), on the other hand, exhibit the opposite behaviour. Most of the time, the MACI/MBS ratio falls between (0.96 and 1.03) in all scenarios. This demonstrates that the flexural capacity of sections with insufficient reinforcement may be reliably predicted by both codes. As shown in Figure 3-2, the shear strength calculations employed in the ACI 318 code predict a lower capacity than the equations used in the BS 8110 code, which is a good thing. In the typical range of application, the ACI code generates shear strength values that are 10 percent to 30 percent lower than those produced by the BS code. Following the findings given in Fig. 3, the ratio of projected axial compressive strength predicted by the ACI 318 code to that predicted by the BS 8110 code decreases as the ratio of gfy/c increases. In the case of very little reinforced sections, the axial compression strength calculations in the ACI 318 code result in a capacity that is approximately 10% lower than that of the similar equations in the BS 8110 code as seen in Fig. 3. When gfy/f'c = 0.4, the compressive capacity is evaluated at the same level as before. For heavily reinforced cross-sections, the ACI 318 code predicts a design capacity that is up to 25 percent lower than the BS 8110 code, according to the manufacturer. As demonstrated in Figure 3-3, the ACI 318 code forecasts a lower ratio of projected axial compressive strength when compared to the BS 8110 code, which is consistent with the findings of the tests. According to Fig. 3, when there are relatively few reinforced sections, the axial compression strength calculations in the ACI 318 code produce a capacity that is approximately 10% lower than the analogous equations in the BS 8110 code. When Gfy/F'c = 0.4, there are no significant changes in the compressive capacity assessment. For heavily reinforced cross-sections, the ACI 318 code predicts a design capacity that is up to 25 percent lower than the BS 8110 standard, according to the manufacturer.

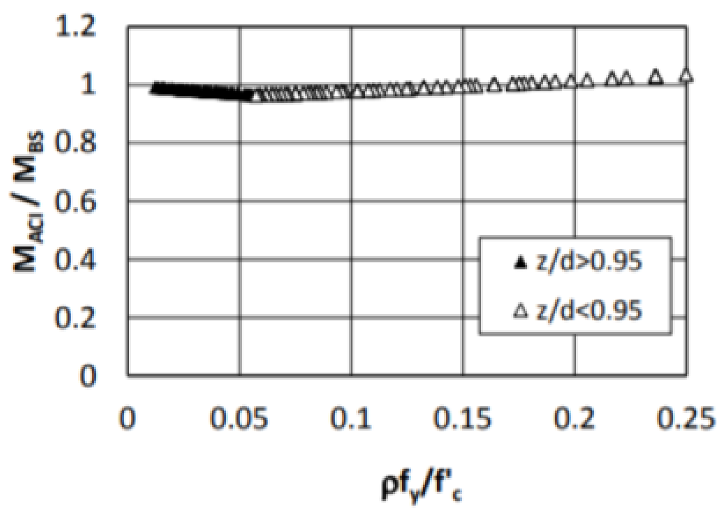


Figure 3.1: The ratio of flexural strength to the strength of the steel reinforcement

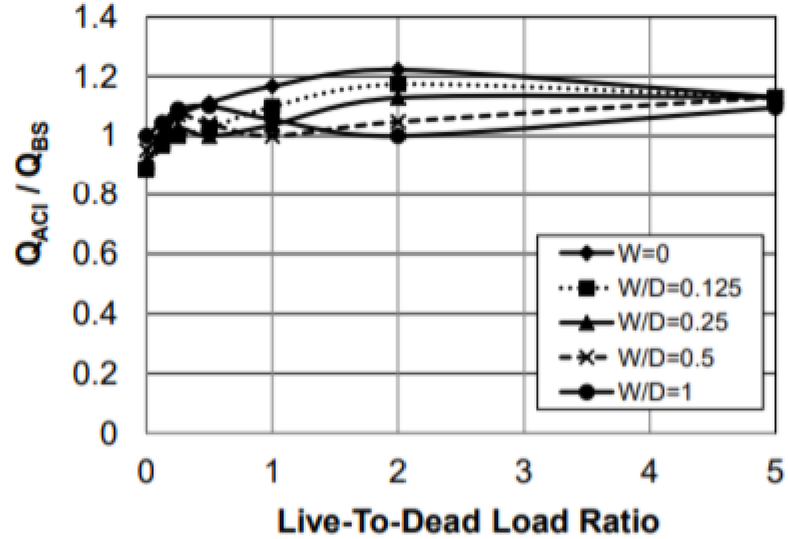


Figure 3.2: The ratio of shear strength to shear capacity

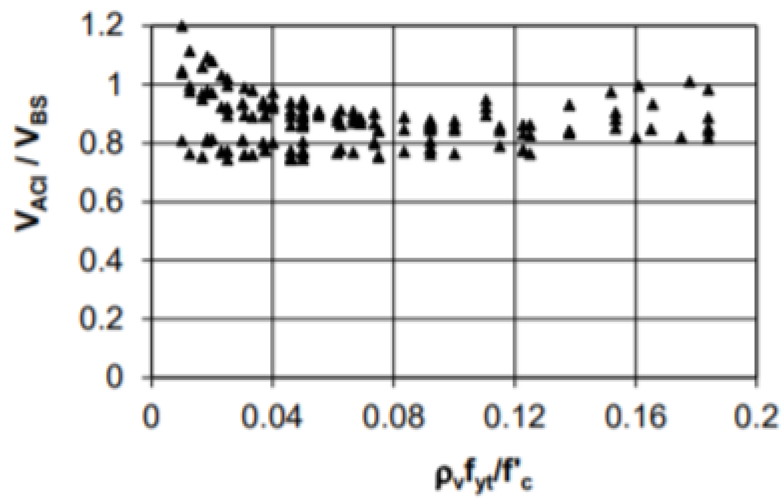


Figure 3.3: Gross steel reinforcement index in relation to axial compression capacity

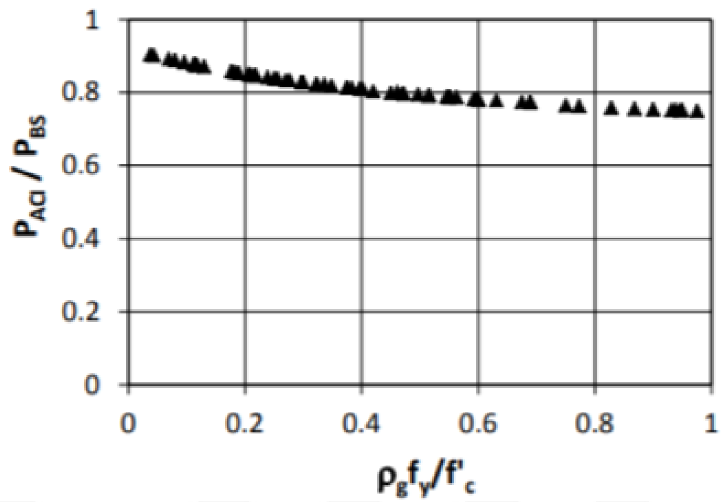


Figure 3.4: load ratio vs live-to-dead load ratio at different wind strengths

3.3 ACTIONS AND RESISTANCE IN VARIOUS ARCHITECTURAL DESIGN REGULATIONS

Mourad M. Bakhoum et al. [37] did a comparisons of the provisions for actions in flexural and compressive axial loading. Sections are compared in terms of their resistance (strength) to actions and loads when subjected to flexural and compressive axial loading. Some of the factors taken into consideration are variable occupancy and varied material strengths, to name a few of examples. When making this comparison, both concrete and steel constructions are taken into consideration, as well. Resistance and action from various codes, as well as the challenges and ramifications of doing so, are discussed.

3.3.1 Actions and resistances

The outcomes of various acts and resistances are compared and contrasted for a number of different circumstances. These sorts of structural components include reinforced concrete beams, reinforced concrete columns, steel beams, steel columns, and composite beams, to name a few examples. The first stage consists of a comparison of the actions and loads described in different design code versions. Among the variables considered in the study were those that are listed as follows: Residents' occupancy in a structure can vary depending on whether it is used for residential, office, or retail purposes, as can bending and axial forces caused by different types of occupants in a structure. Following that, the resistance of axially loaded short columns and beams to various structural components is investigated. The following are the characteristics of the materials that will be used during the research: yield strength of structural steel is measured as 360-500 N/mm2, yield strength of concrete cylinder is measured as f_y = 240 N/mm2, and yield strength of concrete cylinder is measured as f_{ck} = 25-40 N/mm2.

3.3.2 Actions in the Considered Codes

As you look at the figure 3-5, you can see how many people are living in different parts of the building at different times. Variable actions can be seen to have different values when you compare different programs. For balconies and corridors in homes, as well as stair loads in stores, there are big differences in live load intensity. In some cases, the design live load intensity went up by 60%.

When you reach the ultimate limit state, you combine the values of variable actions (L.L.) and permanent actions (D.L.) and each is multiplied by the load factor that applies. This is shown in the figure.

There are a few things that are thought about when looking at the figure 3-6:

It is used in the same place by both DL and L. As examples of D.L intensities, 3 kN/m2 and 7 kN/m2 could be that can be used in many different types of buildings. These are both examples of D.L. intensities that can be used in a lot of different types of buildings. EC2's ultimate load is used to figure out and figure out the final limit state values. Final loads for the EC2 are shown in kN/m2, which is how much weight it can hold.

Use	Code	Floors (kN/m²)	Corridors (kN/m²)	Stairs (kN/m²)	Balconies (kN/m²)
Residential	ASCE 7-10 [1]	1.92	4.79	4.79	2.88
	ACI 318-14 [2]	1.90	4.80	4.80	4.80
	EC2 [5]	2.00	2.00	3.00	4.00
	ECP 201-2011 [8]	2.00	2.00*	3.00	3.00
Offices	ASCE 7-10 [1]	2.40	3.83	4.79	3.60
	ACI 318-14 [2]	2.40	4.80	4.80	4.80
	EC2 [5]	3.00	3.00	3.00	3.00
	ECP 201-2011 [8]	2.50	2.50*	4.00	4.00
Shops	ASCE 7-10 [1]	6.00**	6.00**	4.79	–
	ACI 318-14 [2]	6.00**	6.00**	4.80	–
	EC2 [5]	5.00	5.00	5.00	–
	ECP 201-2011 [8]	5.00***	5.00***	5.00***	–

* This value is assumed to be same as that of floors.
** This value is assumed for light manufacturing.
*** The variable action intensity for warehouses and stores is given by $\geqslant 10$ kN/m² (according to the stored materials).

Figure 3.5 Action intensity values for different building uses under various codes

Use	Dead load (kN/m²)	ACI318-14/EC 2	ECP203-2007/EC2	ECP205-2007/EC2	EC2 ultimate value (kN/m²)
Residential (Floors)	L.L. Ratio	0.95	1.00	1.00	**2.00**
	3.00	0.94	1.05	0.96	7.05
	4.00	0.93	1.05	0.95	8.40
	7.00	0.92	1.04	0.93	12.45
Residential (Stairs)	L.L. Ratio	1.60	1.00	1.00	**4.00**
	3.00	1.32	1.05	0.98	9.00
	4.00	1.26	1.05	0.97	10.40
	7.00	1.15	1.05	0.95	14.60
Residential (Balconies)	L.L. Ratio	1.20	0.75	0.75	**4.00**
	3.00	1.12	0.90	0.84	9.00
	4.00	1.09	0.91	0.84	10.40
	7.00	1.04	0.94	0.85	14.60
Offices (Floors)	L.L. Ratio	0.80	0.83	0.83	**3.00**
	3.00	0.87	0.96	0.89	8.20
	4.00	0.87	0.97	0.89	9.60
	7.00	0.88	0.99	0.89	13.80

Case	Loads considered	Dead loads (D)		Live loads (L)		Wind loads (W)
		Adverse	Beneficial	Adverse	Beneficial	
ACI 318-14 [2]						
1	D, L	1.2	0.9	1.6	0.0	–
2	D, L, W	1.2	–	1.6	–	0.5
		1.2	–	1.0	–	1.0
3	D, W	–	0.9	–	–	1.0

For simplicity, L refers to floor live load and the roof live load case is neglected

Case	Loads considered	Permanent loads (G_k)		Variable imposed loads (Q_k)		Wind loads (W_k)
		Adverse	Beneficial	Adverse	Beneficial	
EC2 [5]						
1	G_k, Q_k	1.35	1.00	1.50	0.00	–
2	G_k, Q_k, W_k	1.35	1.00	1.35	0.00	1.35
3	G_k, W_k	1.35	1.00	–	–	1.50

Simplified combination rules with only one variable action are considered.

Case	Loads Considered	Dead Loads (D)		Live Loads (L)		Wind Loads (W)
		Adverse	Beneficial	Adverse	Beneficial	
ECP 203-2007 [9]						
1	D, L*	1.4	0.9	1.6	0.0	–
2	D, L, W	0.8×1.4	0.8×1.4	0.8×1.6	0.8×1.6	0.8×1.6
3	D, W	1.4	0.9	–	–	1.3

Case	Loads Considered	Dead Loads (D)		Live Loads (L)		Wind Loads (W)
		Adverse	Beneficial	Adverse	Beneficial	
ECP 205-2007 [10]						
1	D, L	1.2	0.9	1.6	0.0	–
2	D, L, W	1.2	–	1.6	–	0.8
		1.2	–	0.5	–	1.3
3	D, W	–	0.9	–	–	1.3

For simplicity, L refers to floor live load and the roof live load case is neglected

Notes: Values written in bold font represent the Variable Action Intensity according to EC2 [5] and as indicated in Table 1.
* For cases when live loads does not exceed 0.75 of the dead loads, the ultimate load (U) is calculated as follows: $U = 1.5$ (D + L).

Figure 3.6 Final loads and partial safety factors for a variety of building uses

3.3.3 Results

As seen in the figure 3-7, the percentage of reinforcement necessary for a single-reinforced concrete beam is calculated. Throughout the section, the design codes that were utilized to compute the ultimate moment of resistance are depicted on the horizontal axis, while the required reinforcement ratio is depicted on the vertical axis. When using the same design code to compute the straining actions and ultimate resistance, the first bar chart illustrates how the reinforcement ratio evolves as a function of time. The remaining bars depict the mixed designs that were created in response to the loads shown in the figure. As stated by the percentage figures above the bar chart, the reinforcement ratio varies depending on the mixed code case being considered. Positive numbers suggest a scenario that is uneconomical in comparison to the design code under consideration, whilst negative values indicate a potentially harmful condition. While ACI 318-14 provides harmful results when the European loading standards are utilized in conjunction with it, when the Egyptian loading criteria are applied, conservative results are discovered. The European design moment of resistance as well as the American and Egyptian loading standards offers cautious numbers of 3.8 percent and 5.3 percent, respectively, for the design moment of resistance. It is unsafe to build structures based on Egyptian standards for resistance mixed with loading criteria that do not correspond to Egyptian requirements for resistance. As indicated in the figure 3-8, the section modulus necessary for limited compact steel sections is determined by comparing the three codes that were examined. The concrete beams are compared in the same way as the steel beam is compared to the concrete beam. In this instance, it is evident that the differences between the various codes are small compared to one another. When Egyptian requirements are coupled with other loading codes, the results might be potentially hazardous to the environment. When combined with other standards, European requirements provide conservative results, but they are more flexible than American specifications.

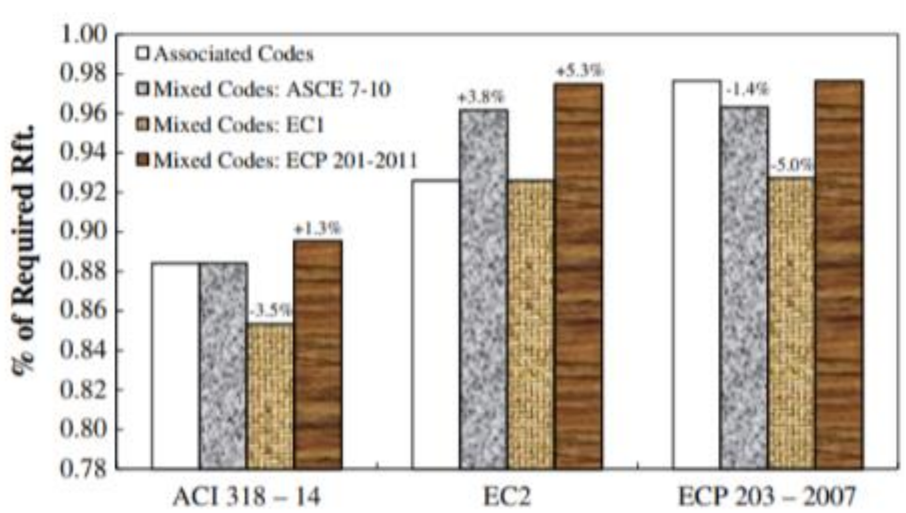


Figure 3.7 Calculations using related codes and mixed codes to compare the reinforcement ratio (percent) for a single reinforced beam segment in flexure

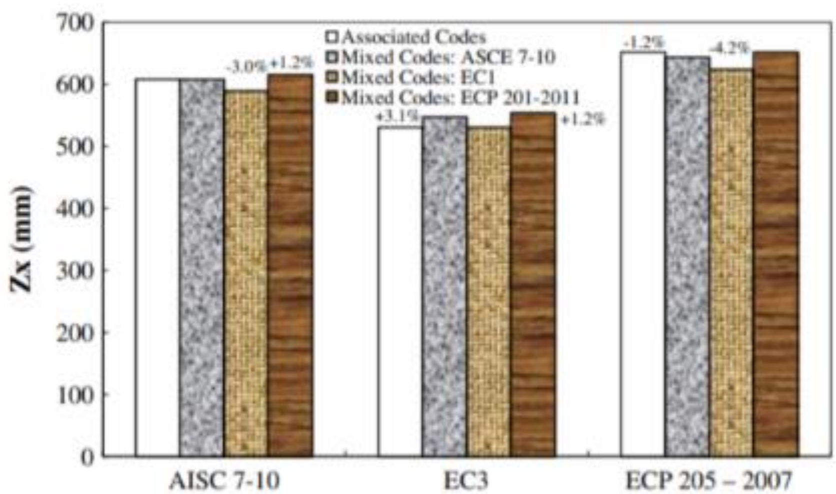


Figure 3.8 Comparative analysis of a steel beam's plastic modulus requirements for related and mixed code requirements

3.4 SHEAR WALLS MADE OF STRONG CONCRETE AND STEEL PLATES

The use of shear walls in high-rise constructions is a cost-effective method of lowering lateral stresses. It is shown in [38] that a composite shear wall system consisting of high-strength concrete walls with embedded steel plates may be evaluated experimentally for seismic performance. Dongqi Jiang and colleagues Construction and evaluation of five reinforced concrete shear walls (RCSW) and six reinforced concrete-steel plate shear walls (RCSP) were carried out under quasi-static reversed cyclic loads with varied aspect ratios (height/width, 1.5 and 2.7). (RCSPSW). In order to compare the damage development, failure mechanisms, and load-displacement responses of test specimens, experimental observations were employed. Through testing, it was discovered that the high-strength RCSPSW system had superior lateral load strength and acceptable deformation capabilities than the standard RCSPSW system. RCSW and RCSPSW have been shown to be reliant on the axial compressive load, and a maximum axial compression ratio (0.5) is recommended for use in engineering procedures when utilizing high strength RCSPSW (high strength RCSPSW). Design strength models were used to forecast the shear and flexure peak strengths of the RCSPSW system, and the models' applicability and reliability were proved by comparing the results of various tests.

It is proposed in this research that reinforced concrete-steel plate composite shear wall systems be used as lateral load-resisting components in high-rise building designs. To investigate the effect of critical factors on the seismic performance of RCSW and RCSPSW specimens, quasi-static cyclic lateral stress was applied to 11 RCSW and RCSPSW specimens. The results showed that the influence of critical parameters was significant. Important metrics such as lateral load capacity, ultimate displacement, the ductility factor, and the EVD coefficient were computed during the course of the test, and the progression of damage and failure modes was tracked throughout the course of the test. In addition, design models were used to forecast the shear and flexure strengths of the RCSPSW material. Using the information acquired during studies, the following conclusions can be drawn: The shear and flexure modes of failure prevail in specimens with an aspect ratio between 1.5 and 2.7, respectively. A total of four flexure-tension cracks and two flexure-shear fractures were found in the lower half of both the boundary components and the wall web of the two flexure specimens, whereas the shear specimens only contained inclination cracks. RCSPSW specimens, in contrast to RCSW specimens, exhibit more densely distributed cracks that are finer in texture. As axial compressive pressures increase, the compaction of compression forces lowers the amount of fractures and the width of cracks.

In the RCSW specimen hysteresis loops, there was a clear "pinching" effect. While the hysteresis curves appear to be plumper with higher peak lateral load capacities, a more severe post-peak strength and stiffness decline occurs with the embedment of the steel plate, as seen in Figure 1. RCSPSW systems outperform RCSW systems in terms of lateral load capacity, for the most part, according to the research. The ultimate drift value of 1.0 percent for the RCSPSW shear specimens demonstrates that the deformability is sufficient for design purposes in this application. It is found that when the axial compression ratio is less than 0.50, the final drift is larger than one percent and the ductility factor is roughly four. The ratio of axial compression to axial expansion of wall specimens has a significant impact on the lateral load performance of the specimens. The peak lateral load increases as the axial compression ratio increases, whereas the ultimate displacement reduces as the compression ratio grows. Greater loss of strength and stiffness was seen in specimens with an axial compression ratio higher than 0.50, according to the findings. Increased compression ratios of 0.58 and higher dramatically reduce ductility factors of 2.61, lowering the likelihood of drift to less than one percent in the long run. The brittleness of high-strength concrete can be related to the lower deformability of RCSPSW specimens when subjected to high axial compression ratios. Also probable is that the thin wall thicknesses of the test specimens have a moderate concrete confinement effect on the steel plate that has been put into the test specimen. Concrete spalling can cause the steel plate to buckle if the damage is serious enough. This can result in a substantial loss of strength and stiffness. Steel plate embedded in RCSW effectively improves the energy dissipation capacity, which is frequently lower in shear mode of failure than in flexure-controlled failures due to the flexure-controlled nature of the failure. When it comes to shear and flexure strength, the RCSPSW's conservative design values are used in the majority of the suggested designs. We may be able to obtain a cautious estimate if we use the correction factors ks = 0.9 and kf = 1.1. According to experimental data, a maximum axial compression ratio of 0.5 should be calculated for RCSPSW, and shear walls should be erected with caution when subjected to significant axial compressive loads. Tie or shear studs should be used in the design of high-strength reinforced concrete plate shear wall systems, as well as in the design of steel plates and concrete. A higher transverse reinforcement ratio should be employed in order to further increase the concrete confinement effect.

4. STRUCTURAL ANALYSIS

4.1. INTRODUCTION

Earthquakes have the potential to cause the largest amount of damage since urbanization and population expansion have led to a rise in the demand for tall buildings [42 - 43]. Modelling the structural systems analysis of reinforced concrete multi-story buildings is a very challenging endeavour. They are often dealt with as finite beam systems, either in two or three dimensions, when this occurs. Because seismic forces are random and unpredictable, the instruments used by engineers need to be fine-tuned before they can be used to study structures that are being affected by them. In order to investigate the actual behaviour of a building while being aware that earthquake damage might be anticipated but must be mitigated, a load model for earthquakes is required. In the past several years, there has been a marked increase in the significance of doing a structural analysis for prior earthquakes of varied intensities as well as screening for a multitude of criteria at each level.

4.2. STATE OF THE PROBLEM

Analysis of the building's structure according to American code (ACI 318-14) and the Iraqi seismic code (2019), followed by a discussion of the findings [45].

In accordance with the British Code (BS 8110-97) and the Egyptian seismic code in which the European (EUROCODE 8-2004) is needed, doing an analysis of the structure while also providing an explanation of the findings.

4.3. USING COMPUTER PROGRAMS FOR ANALYSIS

Although computer programs are capable of doing analyses, the results will ultimately depend not only on the model that was established but also on the input data that was employed and generated by an engineer. In addition, the kind of analysis performed as well as the arrangement of the data is essential components in achieving precise outcomes. Because of the complexity of the analytical computation, the process of calculating the load on large structures or 3D (three-dimensional) constructions typically takes a long time and is prone to inaccuracy. Only by employing the method of manual computation will one be able to arrive at a figure that is either more accurate or more suitable. However, the iteration method or geometric stiffness that is almost exactly the same as what is used in hand calculations is the centre of every piece of software. For instance, the following are some of the ways in which manual calculations and computer-based analyses are different from one another.

4.3.1. Hand Computations Limitation

There are various restrictions imposed by hand computations:

a. Use for less significant problems and structures.
b. Even problems of a moderate scale might be challenging to resolve.
c. At this point in time, it is almost impossible to do analysis in three dimensions.
d. The likelihood of making a mistake increases proportionally with the size of a structure.
e. It takes a significant amount of time to analyse the data.

4.3.2. Advantages of the computer's innovation

a. Analysis of the structure employing matrices and their methods.
b. The continued development of numerical analysis as a field.
c. The finite element method.
d. There are now more programming languages available than ever before.
e. The organization of data input and output is made simpler by the use of drawings.

4.4. ANALYSIS WITH FINITE ELEMENTS FINITE ELEMENT ANALYSIS (FEM)

The finite element approach may be used to numerically solve issues involving structural analysis, hydrodynamics, and other branches of physics. When utilizing the finite element approach, it is important to build and solve huge algebraic systems, and computers supply the tools that are required for this task. The finite element method (FEM) is utilized for the purpose of doing analysis on all significant engineering designs as well as almost all scientific research. These days, using commercial finite element programs is the primary way to put this technology into effect (ANSYS, ABAQUS, REVIT, ETABS, NASTRAN, SAP, etc.).

These applications are executed on mainframes, workstations, and personal computers, all of which are utilized in the process of resolving exceedingly challenging issues. With the finite element method (FEM), a large, complex physical issue is subdivided into a number of more manageable, smaller problems that, if certain boundary conditions are fulfilled, may be solved. The results of the smaller problems are then combined using mathematical methods that are based on matrix methods to get the results of the larger problem, which are close to the exact solution that is dependent on the differential equations. This is done in order to get the results of the larger problem. The finite element technique is a software solution that is used in structural engineering. This approach separates a structural element into a certain number of elements depending on the level of accuracy that is required for the particular type of element that is being researched. In construction applications that are referred to as 3D analysis, such as ETABS, it is possible to "divide" or "Mesh" the evaluated structural sections in order to obtain the best suitable form for division.

4.5. THE FUNDAMENTALS OF SHEAR FORCE AND HOW TO USE THEM

The "base shear force," which is an imaginary force that is employed in structural calculations, is a representation of the internal forces that can be exerted on structures as a result of earthquakes and the movement of the earth. These forces are responsible for causing damage to the structures. To calculate the total design basal shear forces (V), either identical static procedures [39] or dynamic methods are used. This is due to the fact that the structure's dynamic reaction is equivalent to a static response that fluctuates slowly over time (from the point of view of the dynamics of structures). This and other static techniques take into consideration dynamic reaction since earthquake force is measured at its greatest strength and with an escalation in safety. However, the fact is that each method has limits and restrictions because each method has its own set of limitations and requirements. In spite of the fact that dynamic analysis methods, which will be covered in greater depth in the following paragraphs, are more complex yet may be used in any facility, it is necessary to calibrate (compare) them with static methods that are equal in nature. This is due to the fact that a given approach may be suitable for one facility but not for another, but other more sophisticated ways may be used in any facility. Structures are built to withstand earthquakes by having the highest feasible force introduced into them [48]. However, this does not mean that the force should be increased, as it is initially set to be equal to the base shear forces, and increasing it will cause damage to the structure if it is increased further. Shear walls and frames, for instance, are able to withstand horizontal loads because their mass is distributed in such a way that the effective mass is able to respond to lateral pressures to the greatest extent that is practically possible. According to Newton's second law, the force in question is proportional to the structural system's compliance in resisting lateral forces. This is because the force in question is equal to mass time the acceleration of gravity. As a consequence, seismic forces are dissipated in proportion to the structural system's compliance in resisting lateral forces.

The earthquake can appear to be made up of merely four horizontal and two vertical waves, but in reality, it is made up of six 3D waves in space (3D waves). Because it is hard to pinpoint the direction from where the earthquake originated, horizontal forces, denoted by the symbol "Eh," are exerted on the building from all four of its sides. Static analysis requires that the vertical component, denoted by "Ev," be applied to the structure twice, once for up and once for down, in accordance with the code. The actual number that is assigned to "Ev" is a percentage of the total dead load. When doing dynamic analysis, the design earthquake for the relevant seismic zone is used to estimate the real value of each vehicle. This value is then applied to the design structures. One of these two compounds can wind up being the determining element in the construction of the structure depending on the structural system and how it reacts to changes in the earth's surface. For instance, the design of an engineering facility like as a bridge is going to be quite different from the design of an engineering facility such as a structure that is going to be quite tall more specifically, in accordance with the information presented in the second chapter.

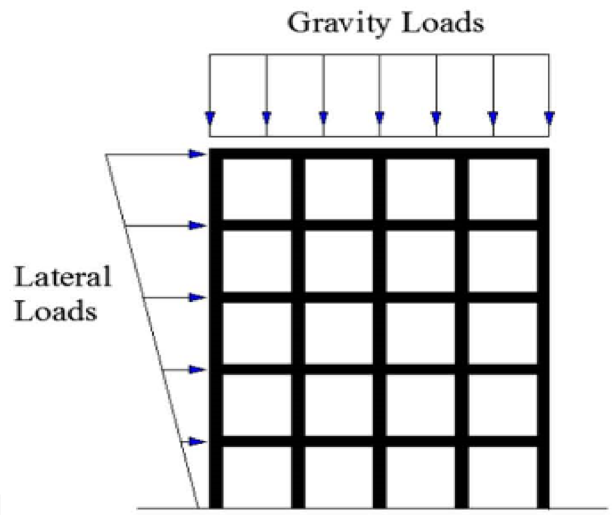


Figure 3.9 Displays a Frame That Can Withstand Momentum

4.6. THE REASON FOR DOING THIS RESEARCH ON THE FACILITY

In the course of this study, each international code will be investigated in order to find out how it varies from other codes in terms of the analysis it provides and the impact it has on the building and the components that make it up structurally, in the capital, Baghdad, Iraq and Cairo, Egypt.

4.7. AN OVERVIEW OF THE STRUCTURE

This structure is made of reinforced concrete and has 21 stories. The floor space is 32.6 by 33.6 meters. The thickness of the slabs is 0.2 meters, while the thickness of the stairs is 0.15 meters. There are no moveable supports in the structures. The dimensions of the beams are 0.3 by 0.6 meters. The height of the bottom floor is 3.4 meters, while each of the other storeys is 3.2 meters in height. In the structural analysis of the building that was being looked at, columns and shear walls were utilized. The building had a column system with a diameter of (0.4*0.8) meters, a core wall structure with a thickness of 0.3 meters, and shear walls with dimensions of (0.3*100) meters, (0.3*130) meters, (0.3*150) meters, and (0.3*190) meters, as well as a flat slab system. The modeling and analysis of the structure are both done with the REVIT and ETABS programs. The building was designed in accordance with the Iraq seismic code requirements for buildings (2019) and Cairo's seismic regulation, which is why it's situated in such close proximity to (Euro code 8-2004). The entirety of the construction may be seen in Figure 4.2. An area that is 1057 square meters in size will be explored in the capital of Baghdad for 5 additional stories, with the height of the top floor being (3.2 m) and the height of the bottom floor being (3.4 m) from the natural ground level. Figure 4.2 depicts these measurements. On some of the roofs, live loads at a rate of 0.3 tons per square meter were positioned, while on the remaining roofs, dead loads at a rate of 0.4665 tons per square meter were positioned.

Table 3.1 Data base and Modelling

Data base and Modelling	
Beam size	30x60 m
Column size	40x80 m
Elevators and stairs Core Shear Wall thickness	30cm
All long Stories Structure Shear Wall thickness	30cm
Slab	20 cm, 15 cm
Duplicate Story height = 3.2 m	3.2 m
Ground story height = 3.4 m	3.4 m

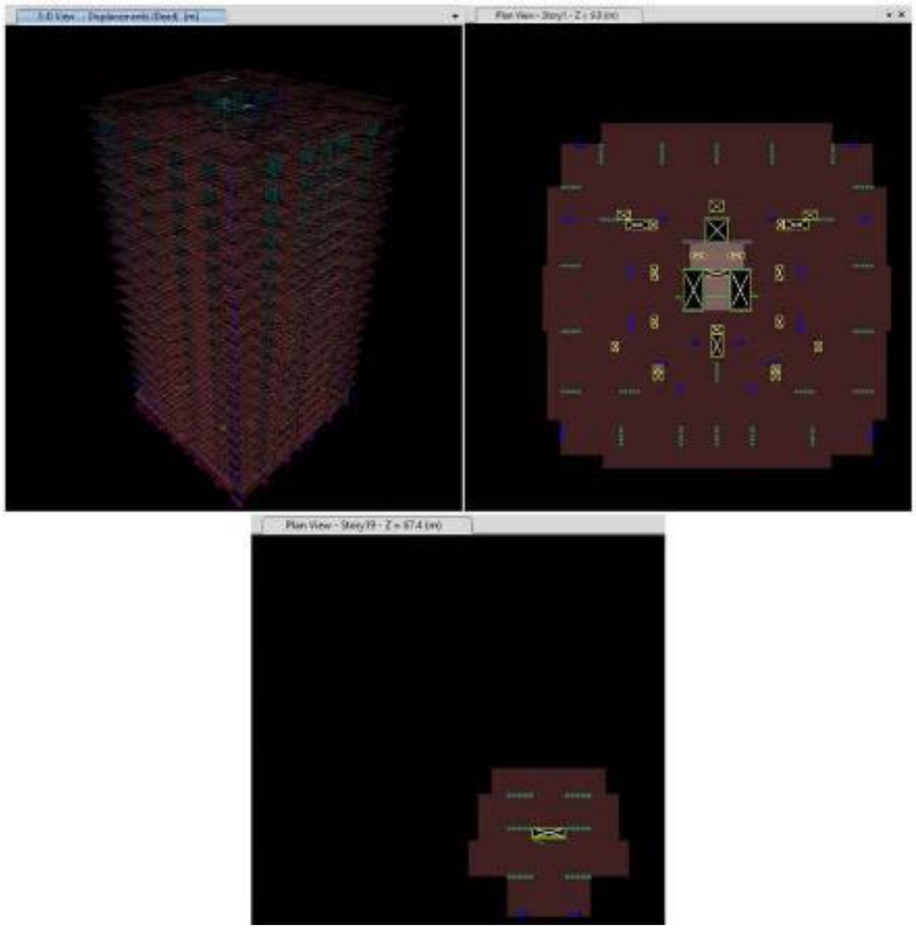


Figure 3.10 Projection of the Building under investigation in a horizontal plane

Table 3.2 Building's Material Characteristics

Parameter	**Symbol**	**unit**	**Confinement steel**	**Longitudinal steel**	**Concrete**
Modulus of Elasticity	E	Mpa	20037.5	204	261.2
Weight per unit Volume	/	Ton/m3	/	/	25
Specified concrete compressive strength	fc	Mpa	240	360	35
Minimum Yield Stress	Fy	Mpa	350	520	/
Minimum Tensile Stress	Fu	Mpa	464.03	464.03	/
Expected Yield Stress	Fye	Mpa	696.04	696.04	/
Poisson	V		0.3	0.3	0.2

4.8. THE FACTOR OF SEISMIC ZONE

Because it is the most important of all the seismic coefficients, the value of the coefficient z is what determines the design requirements for the building's structural system as well as the type of structural system that should be used for the building depending on the seismic zone. All of the other seismic coefficients are also connected to the coefficient z in some way (z). On the basis of previous records as well as geological and seismic information, it has been determined through calculations that the effective ground acceleration of the design will be greater than 10 percent after a period of 50 years. This level of acceleration corresponds to an earthquake movement that is greater than the seismic hazard level in the study area [41]. This consideration is taken into account by the factor z. In the course of our investigation, we are concentrating on the disparities that have been discovered in labs between the currently applicable codes and the studies that have been conducted by geologists from a variety of countries, including Baghdad in table 4.2. Below is a map showing spectral response acceleration in Iraq (ISC2019) [49].

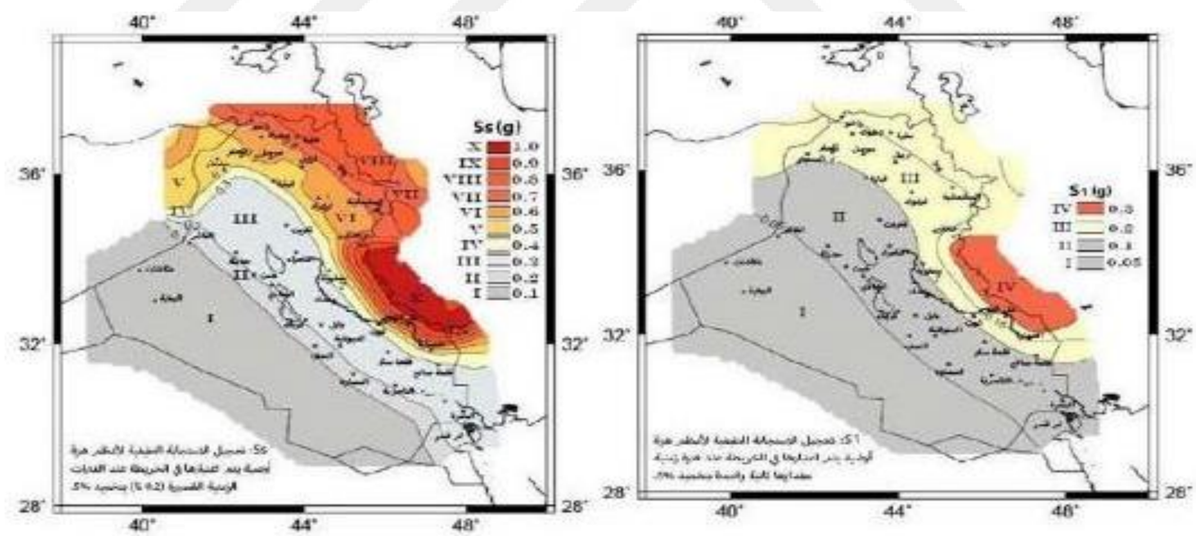


Figure 3.11 Iraq's spectral response acceleration is depicted on a map (ISC2019)

Table 3.3 Code details

Code	Seismic coefficients	
	S_S	S_1
Iraqi seismic code 2019	0.3	0.1

4.9. IMPORTANT ASPECT (I)

When developing buildings that are vulnerable to earthquakes in accordance with the philosophy of seismic structural design, one of the most important considerations is taking this into account. The earthquake has a severe influence on the structure that is being evaluated if it does not possess this attribute. This quality is determined by the kind of building work that was done on the structure in accordance with the specifications (Euro code 8-2004).

Because structures are systematically categorized according to their worth, essential establishments like hospitals, centres for civil defence, and other essential facilities are able to continue operating normally even after an earthquake. A structure that has a greater value of the design base shear is more likely to resist earthquakes than a structure that has a lower value of the design base shear; this is why the main purpose of such a structure is to protect the individuals who are located inside of it.

Because the facility was considered to be a standard, run-of-the-mill construction, the examination did not have any effect on the structural components of the building. Because of this, it is imperative that a residential or administrative building (I = 1) be constructed.

4.10. PROFILE OF SOIL

Investigations of the soil are necessary to determine how much of a seismic force a particular structure is able to withstand, as well as how much of a seismic force it can manage, according to the qualities of the soil in the region (subsidence, endurance, etc.). These qualities are determined by conducting soil tests. In order to correctly investigate and design the foundations of the facilities, it is also required to have a comprehensive grasp of engineering geology, as well as the attributes and needs of the soil. The scale of these investigations should be appropriate for the magnitude of the project. However, it is important to keep in mind that the cost of the field study can account for anywhere between two and five percent of the total cost of the project. On the other hand, the maintenance costs that can result from an incorrect study of the soil are typically much lower than this cost, and as a result, trying to save money on soil investigations will result in a higher cost when consolidating. Soil tables have already been constructed in the Baghdad governorate and the Cairo governorate in preparation for the construction of vertical housing projects with a height of more than 30 meters.

Table 3.4 Profile of the Soil's Efficiency (Iraq, Baghdad)

oil class	City	Vs (m/sec)	N (Kn/m^2)	Cu (Kn/m^2)
C, D	Baghdad city centre	180 to 760	15 to > 50	50 to 100
D, E	Outskirts of Baghdad	<180 to 370	> 15	< 50
F	Other	/	/	/

Table 3.5 Profile of the Soil's Efficiency (Egypt, Cairo)

oil class	Soil Type	Vs (m/sec)	N (Kn/m^2)	Cu (Kn/m^2)
B, C	Sediments composed of (sand – gravel) dense or clay, having a cohesion resistance (Cu) shown in the table.	180 to 800	15 to > 50	70 to 250
C, D	Non cohesive soil composed of (sand – sand) clay or muddy, having a cohesion resistance (Cu) shown in the table.	<180 to 360	> 15	< 70
E	The soil sector consists of a surface layer fluvial sedimentation.	/	/	/

4.11. STRUCTURE PERIOD T

When the building only has one degree of freedom, the dynamic equilibrium equation (general equation of motion) demonstrates that the structure period (T) is inversely proportional to stiffness and directly proportional to mass. This demonstrates that the location of the building's implementation does not have an effect on the structure period (T).

$$T = 2\pi\sqrt{M/K} \tag{4.1}$$

However, in dynamic analysis, all modes of the building are considered using sophisticated mathematical methods. Because of this, in static analysis, the building's structure periods in both the x-direction (x) and the y-direction (y) are taken into account. In dynamic analysis, all modes of the building are taken into consideration. The Raleigh methodology is a method for determining the role (T) value of a building. This may be done manually or more precisely using the ETABS program. We used this method for the purpose of our study.

$$T = 2\pi\surd\ (\sum wi\ \delta i2)\ \div ni = 1(g\ \sum Fi\ \delta ini = 1) \tag{4.2}$$

According to EUROCODE 8-2004 and ASCE 7-10, there is an estimated method for determining the building's actual structural period (T).

$$T = Ct *\ (hn)\ 3/4 \tag{4.3}$$

Table 3.6 Values of Approximate Period Parameters Ct (EUROCODE 8-2004)

	RC MRF	Steel MRF	EBF eccentrically braced frames	RC/ Masonry shear wall	Other
EUROCODE 8-2004	T =Ct h $^{3/4}$				
	Ct =0.075	Ct =0.085	Ct =0.075	Ct =0.05 or, Ct =0.075 / √AC	Ct =0.05

The value of Ct = (0.05) for the studied building is taken from the above table as follows:

T= 0.05*(67.4) ^3/4 = 1.176 sec for 21 floors.

$$Ta = Ct *\ (hn)x \tag{4.4}$$

Structure Type	Ct	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028(0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

Figure 3.12 Values of Approximate Period Parameters Ct and x (ASCE7-10)

4.12. SEISMIC COEFFICIENT CA AND SEISMIC COEFFICIENT CV AND DESIGN RESPONSE SPECTRUM

The seismic coefficients at the site of construction indicate the acceleration at (T = 1 second) and are defined as the greatest effective acceleration in the soil and the velocity for the propagation of seismic waves in the layers of the earth's crust. These seismic coefficients are measured in g's. These factors, which are illustrative of the basics of the location and the area, help to enhance the potential for ground vibration generated by the earthquake that was triggered at the site that is the subject of this research (S & Z). According to tables (16 - R) and (16 - Q) of code ISC2019 code, the Ca value will be 0.12, and the Cv value will be 0.18. The seismic acceleration of the shorter portion of the response spectrum (which is determined by acceleration) is accounted for by Ca, whereas the seismic velocity of the longer part of the response spectrum is accounted for by Cv (governed by velocity).

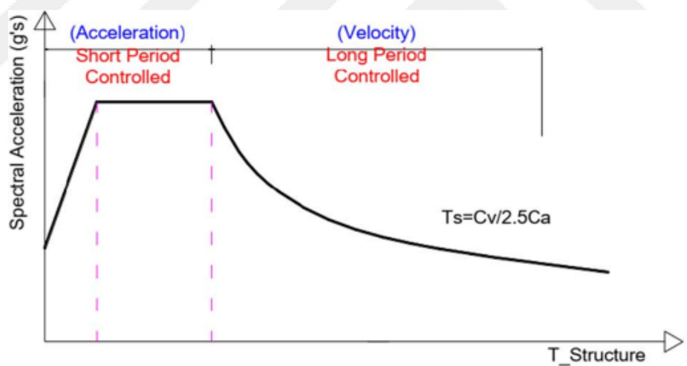


Figure 3.13 Shows Design Response Spectrum

4.13. STRUCTURAL SYSTEM (R)

From the point of view of seismic structural design, it would not be cost-effective for the structure to be constructed to take all of the horizontal pressures that resulted from the earthquake while it was still within the building's confines (elastic phase). Instead, in order to keep the structure contained inside the elasticity field, it is intended to have a resistance that is lower than the crisis. As a result of this significant reduction in resistance, the structure is now functioning in an area of non-linearity, also known as plasticity (plastic phase). Common examples of locations where multiple construction elements enter the plasticity stage and lead to the formation of flexible joints include places of stress concentration, points of structural weakness, and locations adjacent to nodes (joints). These are all examples of locations that can be found in a structure and these results in a significant amount of the earthquake's energy being absorbed or dissipated by the building itself during an earthquake.

R is a coefficient that represents the value of the building's excess resistance (Over Strength) and the total ductility of its structural elements. The concept behind R can be understood by consulting the table for R in the ISC 2019 code. In order to accomplish this, the response modulation factor (R) is used. See also: STRUCTURAL SYSTEMS, which may be found in TABLE 16-N.

The task of the structural designer is to bring a certain (R) so that the building can form a number of plastic joints (plastic hinges) to dissipate the seismic energy (suppress the movement of the building). This is necessary because a portion of the earthquake's energy is converted into kinetic energy within the building. Shear forces are simply mass times acceleration. That the structure could be able to continue performing its typical activities after an earthquake; such instances have occurred and still continue to occur in a variety of countries all over the world.

The IMRF (Moment - Resisting Frame systems - Intermediate Reinforced Concrete Moment Frames), which was used as the stimulus for the study, may be found listed in table 12.2-1 of ASCE7-10.

R equals 5 in accordance with the data shown in Table 2 of ASCE7-10 (TB, TC, TD, and S).

4.14. THE LOAD INCLUDED IN THE SEISMIC CALCULATION (W)

Effective load within the vibrating height (h_n) is the seismic load or so-called effective mass in the structure that is influenced by the movement of the earth. This load is defined by the code, and it is the seismic load (earthquake). According to the code, it is equal to the sum of the dead load, self-weight, borrowed roof weight, and any fixed component weight that is present during the earthquake. Additionally, the percentages applied from other loads that are presumably present during the earthquake (design logic in the code), such as self-weight and must also be taken into account.

4.15. EXTENDED THREE-DIMENSIONAL ANALYSIS BUILDING SYSTEM (ETABS) [50]

Since it was first made available to the public in 1984, it has grown to become one of the most well-known, oldest, and powerful programs in the world that is used to do three-dimensional structural analysis of a range of different systems. It was created at the University of Berkeley in California, which is located in the United States of America and is often regarded as the top engineering school in the entire world. In the year 1984 A.D., Computer & Structures Inc. was given the task of providing structural designers all over the world with a copy of the software and distributing it to them. The method of finite elements is used in the construction of the analytical model that ETABS uses (FEM). It incorporates nonlinear static analysis in addition to linear static analysis and both types of dynamic analysis. It is utilized in the process of rehabilitating structures so that they are able to withstand horizontal stresses (such as winds and earthquakes) with the assistance of linear static analysis. We conducted our study using this particular piece of software since it is universally acknowledged to be the industry standard for excellence. The results of the software are quite close to those obtained by manual calculations, but at the same time a significant reduction in processing time is achieved.

4.16. MODELLING STEPS FOR DIFFERENT FACILITIES IN ETABS

There are three key axes that are utilized in the organization of the majority of worldwide building projects.

1. Modeling
2. Structural Analysis
3. Design or Checking

The version of the software used in this study is ETABS V18.1.0.

4.17. SETUP OF THE SOFTWARE

During the course of our investigation, we measured distances using the meter, forces using Newton's, and temperatures using degrees Celsius. This was the initial stage in doing modeling, and it took careful attention to verify that the inputs and outputs were appropriate and reasonable before commencing the project. This stage came before beginning the project itself (Figure 4.6).

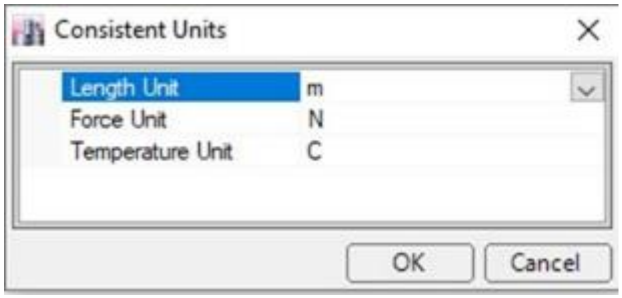


Figure 3.14 Project Units

In view of the earlier discussion and study of the elements impacting earthquake structural design, the following are some important and needed inputs to the program that we would want to clarify:

a. Modulus of Elasticity (E): The answer to this question is dependent on the particulars of the materials used since the value of (E), which is determined for standard concrete from, is directly related to the hardness of the structure (ACI 318-14).

$$Ec = 4730\sqrt{f'c} \;:\; f'c\ MPa\ , normal\ concrete = 25\ Mpa\ , Design\ concrete = 35\ Mpa \tag{4.4}$$

$$Eco = 4700\sqrt{f'} \;:\; f'c\ MPa\ , normal\ concrete = 25\ Mpa\ , Design\ concrete = 35\ Mpa \tag{4.5}$$

b. Mass per Unit Volume w/g**:** It is essential that the mass's value in the volume unit be accurately found; ETAPS does so to an accuracy of 14 digits after the comma. Since the mass is the starting point for the dynamic analysis of buildings, it is critical that its value in the volume unit be determined correctly.

c. Poisson's Ratio (υ): This ratio is comparable to 0.2 when used to concrete, but it is equivalent to 0.3 when applied to steel.

d. Shear Modulus (G): This is decided entirely on its own by the computer program in line with the law that is presented here:

$$G = E2(1 - \upsilon) \tag{4.6}$$

e. Specified Con Comp Strength f'c: Because the compressive strength of cylinders and required in the program, as the cubic resistance of concrete, is greater than the cylinder used in the American code by approximately 10% to 15%, depending on the dimensions of the samples, it is important to pay special attention to this detail during the design phase. This is because the compressive strength of cylinders and required in the program, as the cubic resistance of concrete.

f. Bending Rein f Yield Stress: The yield stress or flow stress of longitudinal steel reinforcement that is resistant to bending forces ranging from GRAD 40 to 60 must be met for conventional concrete, which means that it must not be pre-stressed in seismic structural design. The requirements for conventional concrete can be found in ACI - Se 21.1.5.2 [35]. (240 to 350 MPa).

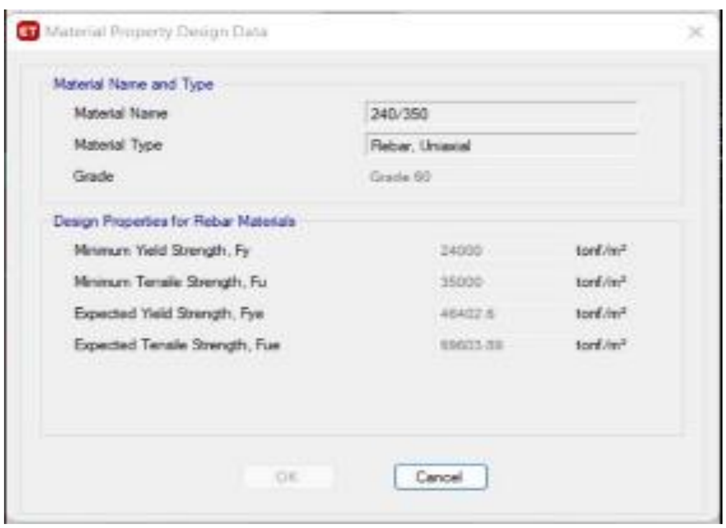


Figure 3.15 Bending Reinforcement Yield Stress

g. Shear Reinforcement, Yield Stress: The shear stress at which a tangential steel reinforcement that is resistant to shear fails. Because of the ease with which it may be applied, the secondary steel reinforcement, also known as stirrups, that is typically used to enclose the longitudinal reinforcement is typically of the low-strength wrought variety (fy = 360 to 520 MPa).

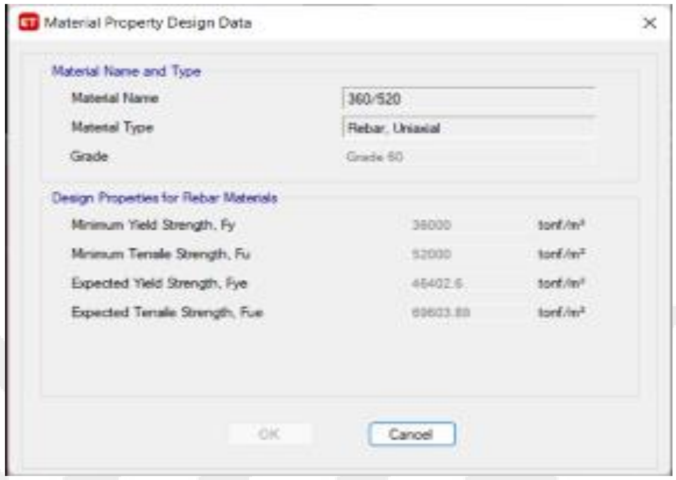


Figure 3.16 Shear Reinforcement and Yield Stress

h. Sections Define: Before identifying the components that will make up the final, required, and structurally successful shape, it is common practice to first carry out a preliminary or partial design.

i. Frame Section Define: The program receives clips as input in order to validate their structural integrity, and in accordance with the rules of the code, the program specifies the sections in the manner depicted in figure 4.9 below, regardless of whether the source is being manually created or validated. In the course of our investigation, we decided to use a beam with dimensions of (3060) cm and a column with dimensions of (4080) cm; the shear walls of both were constructed to have a thickness of (30cm).

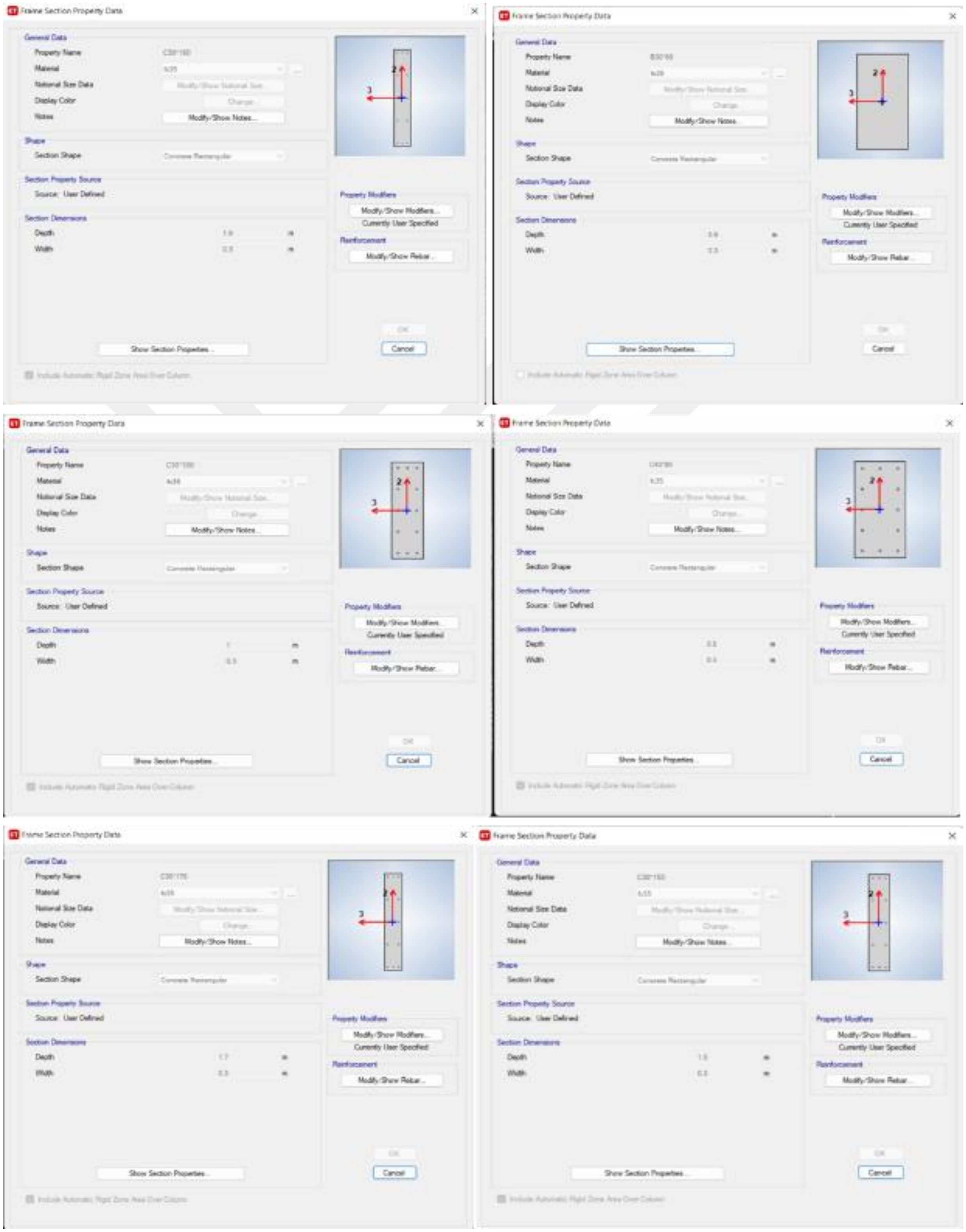


Figure 3.17 Frame Section Define

j. Shell /Area section define: Within the program, you have the ability to define the slabs, walls, and slabs, in addition to the required thicknesses for each. Utilizing the program, one is able to calculate not only the mass of the slab but also its resistance to axial loads, shear, and bending thickness. In order to calculate the bending moment resistance of the slab, the thickness (20 cm) and (15cm) of the slab were entered into the computer, as shown in Figure 4.10.

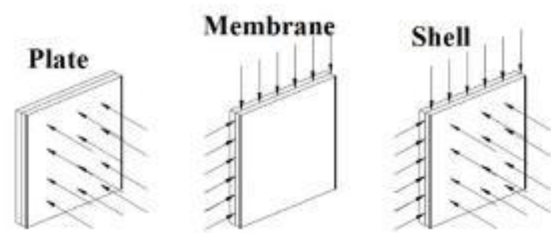


Figure 3.18 Shell / Area Section

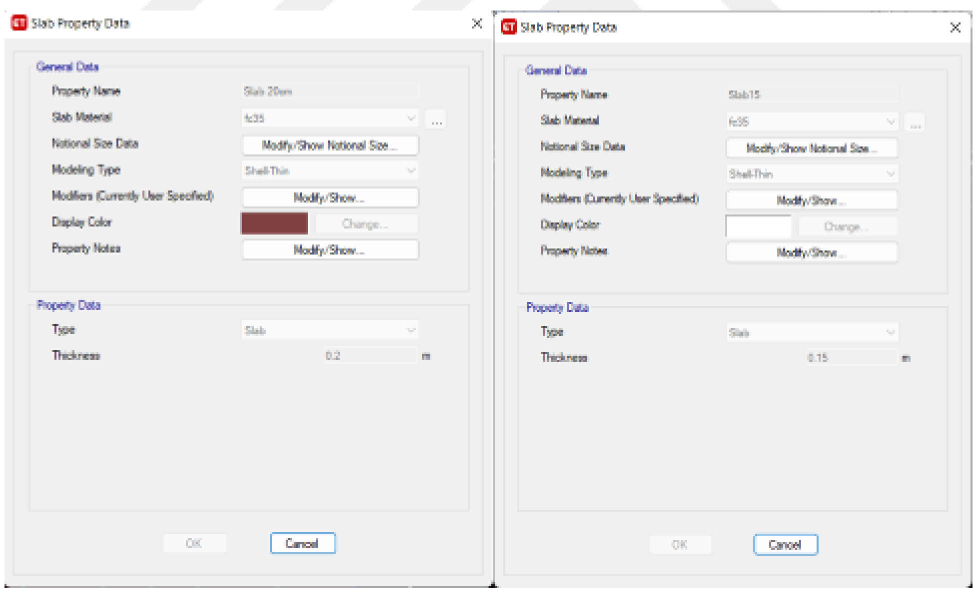


Figure 3.19 Shell / Area Section

As was briefly covered above, constructing the analytical model in such a way that it conforms to the code's requirements and makes use of the finite element method (FEM) is what has to be drawn or developed. It is crucial to have accurate modeling in order to prevent mistakes from becoming compounded during the later stages of analysis, design, and inquiry. This involves making use of the proper instructions or interconnection tools to guarantee that the structural pieces are connected in the appropriate manner. Using the ETBAS program's set of simple and advanced modeling capabilities, the designer is able to model any sort of element in any shape. During earthquake studies of buildings, staircases should not be modeled for a variety of reasons connected to dynamic assumptions that are based on the dynamics of structures.

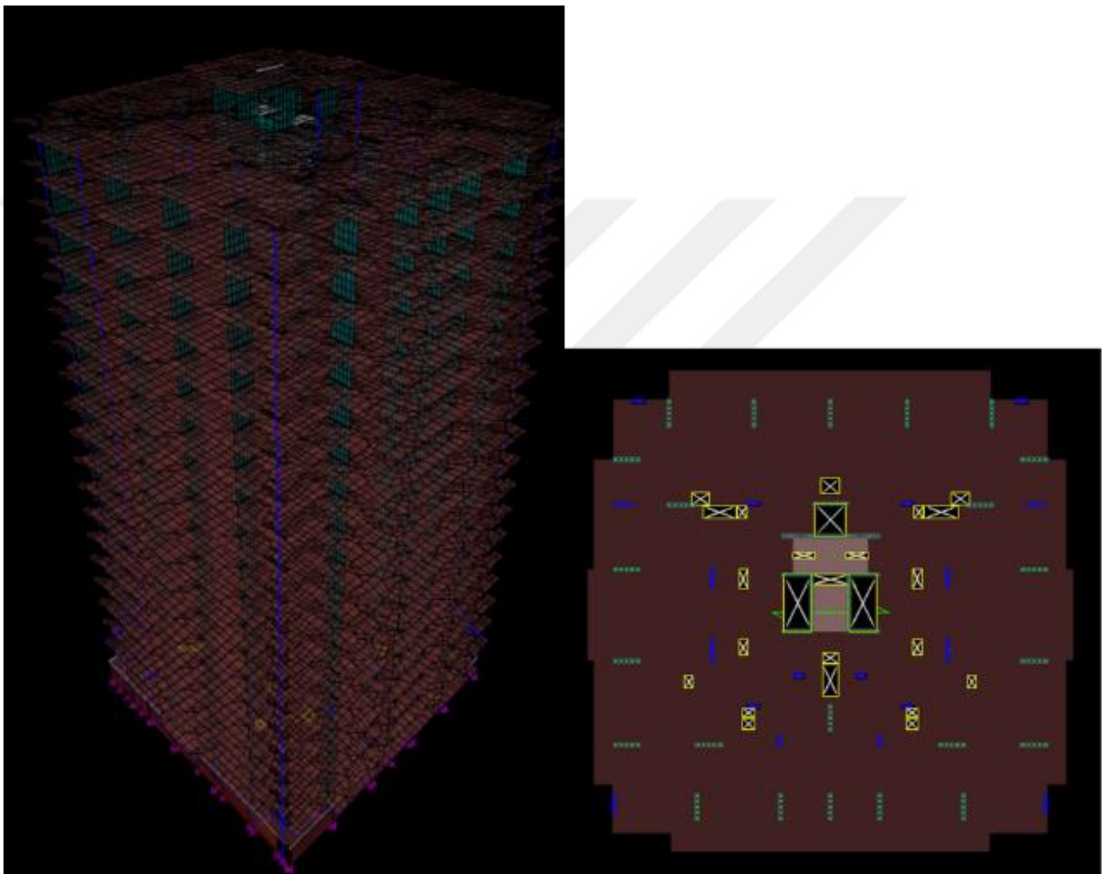


Figure 3.20 the Studied Building

k. Mesh of Elements (ETABS) [50]**:** The finite element method recommends breaking down large structural elements into smaller ones in order to get more precise results for the deformations of linear and areal elements as the size of the stiffness matrix grows with the increase in smoothing mesh. This is done in order to account for the growing complexity of the problem. The application gives users a variety of options to choose from in order to carry out this task, each of which comes with its own set of benefits and drawbacks. When reading the deformations in (slabs) or studying the dynamics in order to elicit an accurate response from the structure, more consideration is given to the kind and number of divisions in the stress locations of structural components, particularly for irregular forms. This is the case especially for irregular forms. If the number of element divisions grows significantly, the structural designer may be required to examine the problematic component in a model that is distinct from the structure itself. This is because increasing the number of element divisions would make it take significantly longer to evaluate the structure as a whole. Cracking causes a reduction in the effective depth of the structural section, which is determined by using the moment of inertia I. This depth reduces as a result of the cracking. Therefore, when cracking reduces the torque stiffness of the section (EI), values of vertical (deflections) and horizontal (displacements) transitions rise, and the moment of inertia decreases, bringing the design closer to the actual reality of the structural behavior of the structural sections. This is because the moment of inertia is directly proportional to the magnitude of the vertical and horizontal transitions.

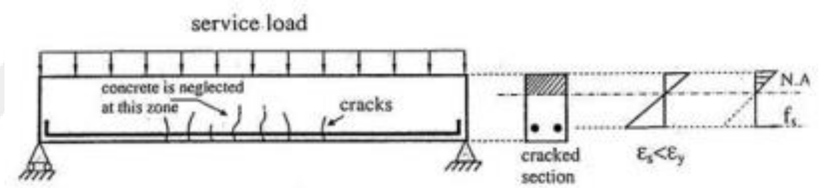


Figure 3.21 Clarification of cracks on the value of the height of the section entered in the calculation (I)

The code (ACI- 11-R8.7.1) [35] specifies that the element hardness (EI & GJ) be calculated in such a way that it represents the degree to which the elements fracture and the inelastic action that occurs to the elements before they achieve yielding. This must be done in order for the element hardness to accurately represent the degree to which the elements fracture. What this indicates is that the linear analysis of the structure has to be carried out using the code that has been supplied. According to the following formula, we were able to determine the hardness reduction coefficients for the structural components for both BS 8110-97 se 10.10.4.1 and ACI-11 se 10.10.4.1, Compressive tension on elements:

Columns = 0.70 Ig

Walls - uncracked = 0.70 Ig

Cracked = 0.35 Ig

Elements subjected to major flexural members

Beam = 0.35 Ig

Flat plate and flat slab = 0.25 Ig

l. Customization of slabs as rigid membranes (Rigid Diaphragms): The transmission and distribution of lateral forces, such as shear forces and torsion moments, from the structure to the vertical sections that resist them can be altered by the use of diaphragms, which are structural systems that can be horizontal or inclined. The study of buildings those are able to survive horizontal vibrations like earthquakes absolutely need this information. In other words, the slab is both flat and solid, and it features retractions along the x and y axes as well as a single Torsion rotation in the z axis. By computing the structure's various patterns of deformation, one may acquire information on the dynamic properties of the structure's behavior. The overturning moment of a structure happens when it rotates around two axes (x, y), which has an effect on the design of the foundations and causes the structure to be raised from one side (Uplift), which may cause the structure to collapse over if it is not properly supported. Torsion about the axis (z) has a significant impact on the design condition of the elements of the structure that are resistant to lateral forces. In the course of our investigation, we defined (diaphragms) due to the fact that this is an essential step in locating the structure's centre of mass and centre of stiffness, and consequently in locating the appropriate eccentricity to apply to the structure in order to take into account the effects of torsion on the structure in accordance with the requirements of the code (ASCE7-10).

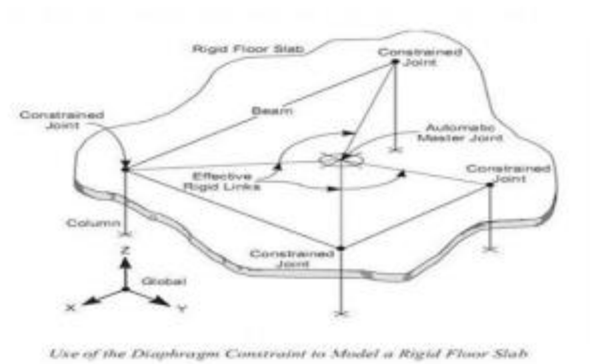


Figure 3.22 diaphragm constraint

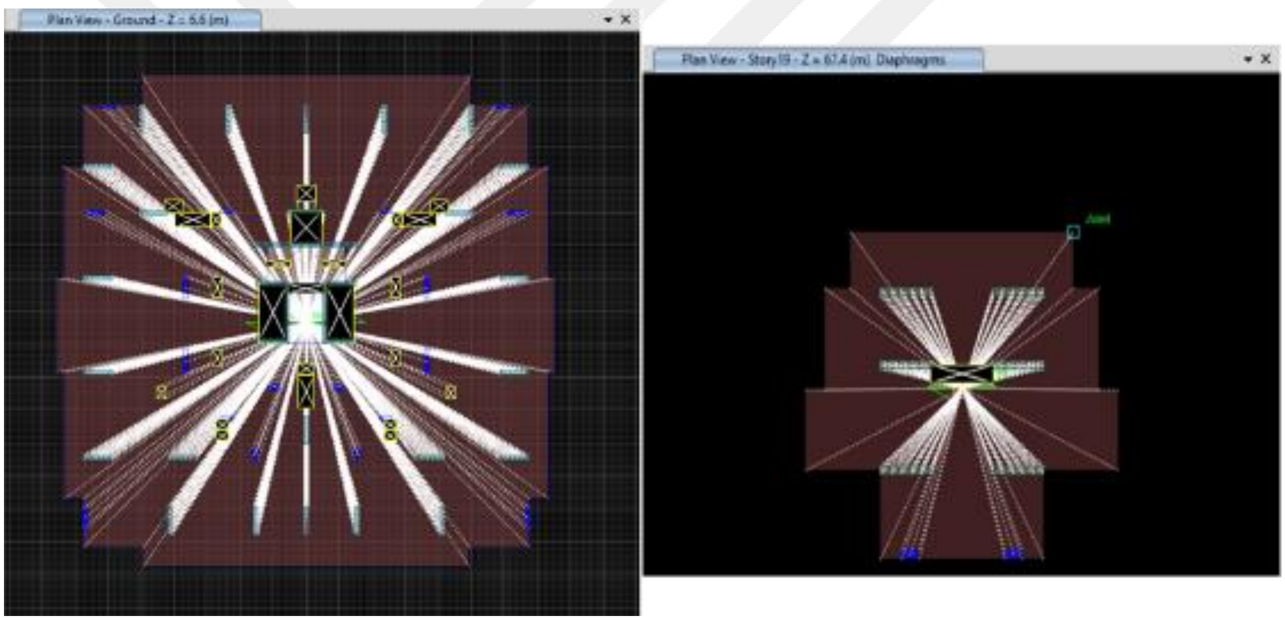


Figure 3.23 Diaphragms

m. Define Load (Static Loads Cases): The second analogous static technique involves entering seismic factors into the ETABS software, one at a time and for each direction of the structure. The goal of this method is to select the most risky design scenario while keeping in mind all of the concepts that have been presented in this chapter. Access to a wide variety of global earthquake calculation codes is provided by the program, which is both comprehensive and helpful for the purposes of our investigation. Additionally, we have the ability to compare these codes, which is made easier by the fact that we are aware of the extent to which each local code is both covered by and influenced by the worldwide codes. In addition to the ACI 14-18 code

(ASCE7-10), the Iraqi code 2019 (BS 8110-97), and the code, the Iraqi government has also adopted the code (Euro code 2004-8). When assessing whether or not a given place is situated inside a seismic zone, the location of the area in question, the kind of soil present there, and the relative weight of these elements are all taken into consideration. The AISC 7-10 decentralization value, the height of the structure that is seismically vulnerable, and the value of the excess resistance coefficient (R) are all shown in the figure below. The AISC 7-10 decentralization value is derived by subtracting the dimension of the building that is perpendicular to the direction of the horizontal forces applied to the structure (initially by 0.05), and the figure also shows the value of the excess resistance coefficient (R).

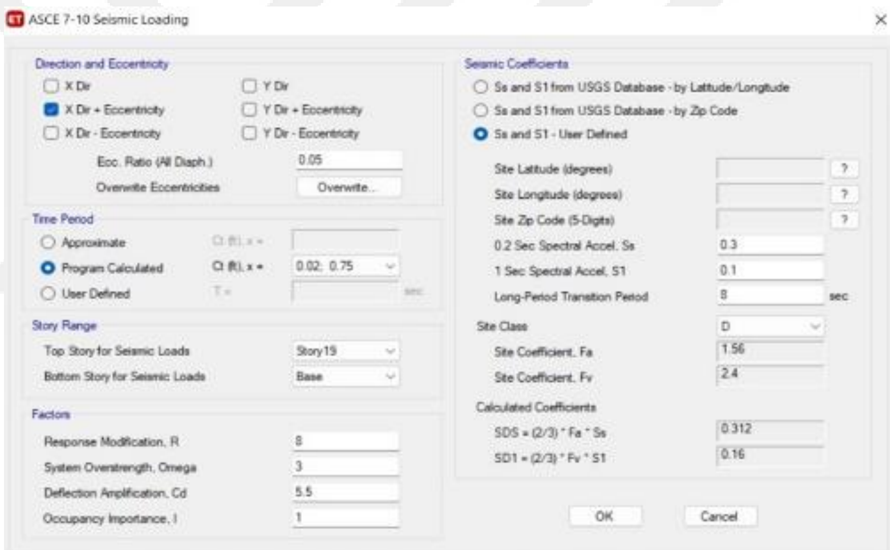


Figure 3.24 Specify where on the structure to enter the coefficients that determine how seismic forces are to be calculated

n. Deformation Mode and software seismic study hypotheses: Because it enables approximation to discoveries that are extremely close to the precise answer via static or dynamic solution, the finite element technique is the cornerstone of earthquake structural research. This is the case because it can be applied to both static and dynamic problems. The (diaphragm) level or the narrative level is responsible for putting the pieces together. When the results of the dynamic equilibrium equations are separated, which is equal to the number of aggregated masses at the levels of the diaphragm in the analytical model, and when the calculated deformation patterns of the computational-analytical model, which represents the physical model, are used for use in the conduct of the dynamic analysis, these patterns provide the basis for the calculation of the dynamic properties of structures. If the number of modes is equal to the number of floors, then the response spectrum analysis may be used to characterize the deformation patterns of a structure with three stories, given that the number of floors is also equal to the number of modes. It is very important to make a distinction between the (Analytical Model) and the (Mathematical Model), where the former is a set of differential equations that describe the behavior of a structure, and the latter is a representative model of the construction sentence that is similar to the real sentence but whose mathematical analysis is easier. Both models are used to describe the behavior of a structure, but the distinction between them is very important.

In our study, the deformation patterns were dealt with for high accuracy and accurate results. It was assumed that each floor has three degrees of freedom for each floor.

$$mode = N * DOF \tag{4.7}$$

$$N = number\ of\ story$$

$$DOF = degree\ of\ freedom = 3$$

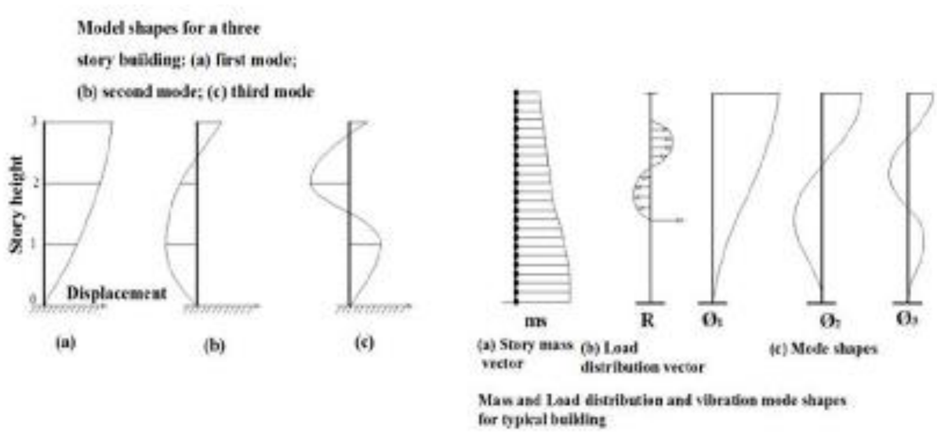


Figure 3.25 The First Three deformation modes of a hypothetical Three-Story Structure

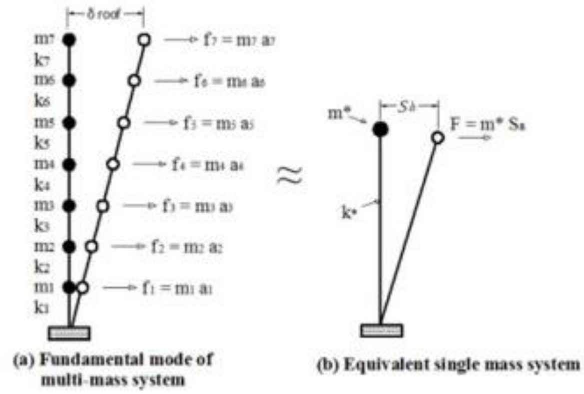


Figure 3.26 Analytical representation of the patterns of a multi-story (Multi-Degree of Freedom) Structure, approximating it to a single-floor or single-degree of freedom model

o. Mass Source: In accordance with Newton's second law (Fi=m*a), a force of inertia acting on the structure is equal to the product of the seismic weight (W) that was included in the calculation of the design base shear force (V) divided by the gravitational acceleration (m=W/g). In other words, the seismic weight multiplied by the gravitational acceleration results in the value of the seismic force. This mass is the origin of the horizontal force that is brought about by the influence of the earthquake (V). Because it is common knowledge that the design bases shear strength (V) is equivalent as a percentage of the seismic weight (W), these methods are referred to as equivalents. This is due to the fact that they produce the same results regardless of the source of the blocks that are used in the analysis. This is particularly true with regard to the dynamic analysis. A joint has to be able to perform three axis transitions (Tx, Ty, Tz) and three axis rotations (Rx, Ry, Ryz) in order to be able to move in any direction (Rx, Ry, Rz). As a programmatic analytical model, in accordance with the code (ASCE7-10 Se 12.7.1), typically adopting supports of the restricted type in all directions (Fixed), but when designing the foundations, a soil spring (subgrade of soil) is added in accordance with the permissible soil bearing capacity, and the nodes that represent the foundations of the structure are assigned supports that are related to the type of soil in the site. This ensures that the foundations of the structure are adequately supported. Assessing the soil's interconnection with the structure in an accurate manner is a challenging endeavor that calls for a comprehensive dynamic analysis that is based on the pressure and takes into account the results of geotechnical experiments and tests performed on the site. Our investigation also encompassed the preparation of the site for the project (Fixed).

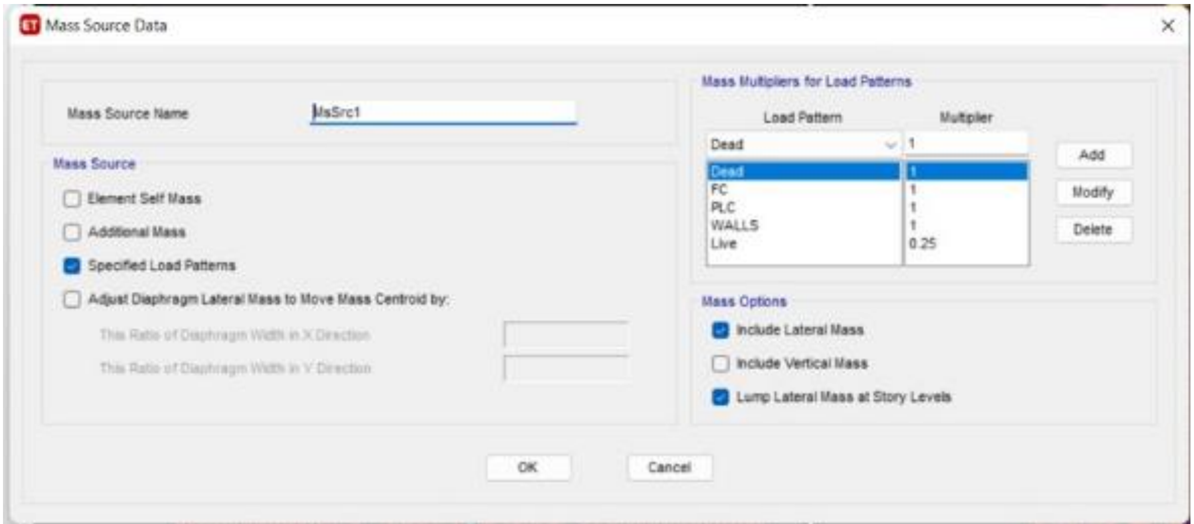


Figure 3.27 the entry of the Mass Source into the EATBS Program in our Study

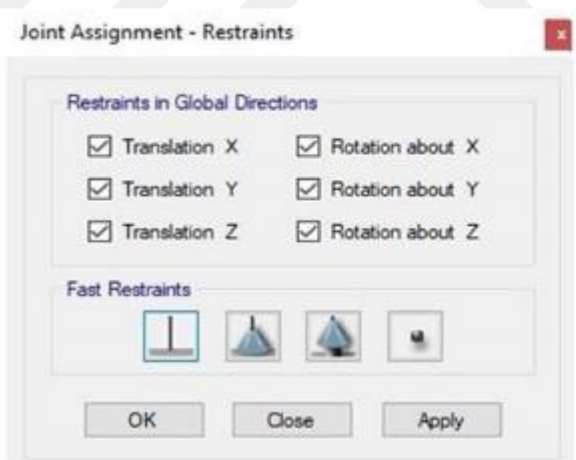


Figure 3.28 the entry of Supports Assignment into the EATBS program in our Study

p. Design Loads Combinations: When developing a bearing system, a designer must always take into consideration both types of loads. This is due to the fact that an earthquake cannot occur until the structure is subject to dead loads (DL), yet continuous loads may be present regardless of whether or not there are earthquakes. It is necessary to combine the effects of seismic loads with the effects of fixed loads and other forms of loading in accordance with certain logic in order to find the most dangerous design condition for the structural part that has been investigated. This will allow for the identification of the most dangerous design condition. The bulk of well-known international codes result in building designs that are ultimately relatively comparable to one another in terms of their final architectural consequences. Following an analysis of each and every conceivable Load Combination, we chose those with the highest potential for harm and based our research on their findings.

q. Dynamic Parameters: ASCE 7-10 stipulates that all allowed phases must contribute at least 90% of the overall mass as one of the key elements in the structural analysis (Sum of Mass). Because of this, the spectral response analysis and the fixed equivalent approach were both utilized in this investigation. As a result, it was found that the (Number of Mode) was far higher than the required minimum percentage and was extremely near to 99 percent.

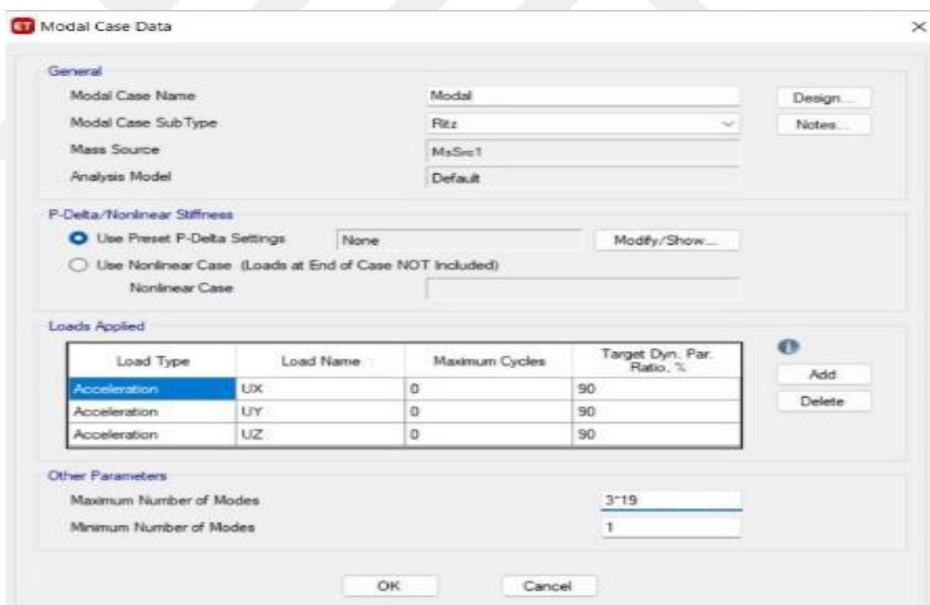


Figure 3.29 the entry of Dynamic Parameters into the EATBS program in our Study

r. Check Model: These helps to verify that the structural elements do not overlap, that the Mesh is accurate, that the loads have been assigned correctly and are within the allowable accuracy (Tolerance), and that the modeling has been completed in accordance with all of the requirements and international codes that have been entered. Any errors in the modeling are corrected until a notice arrives, as seen in Figure 4.23.

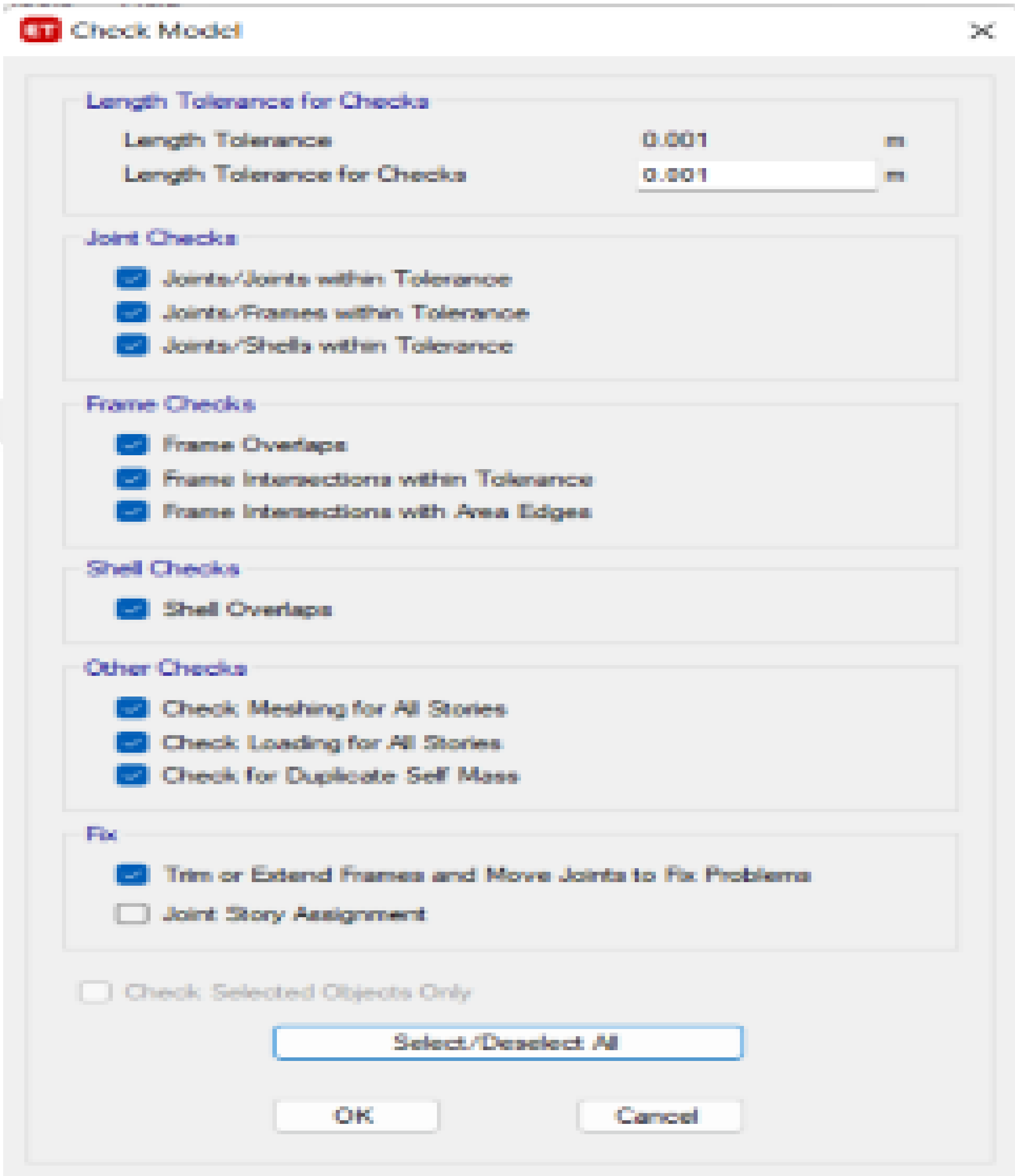


Figure 3.30 the check Model in the EATBS program in our Study

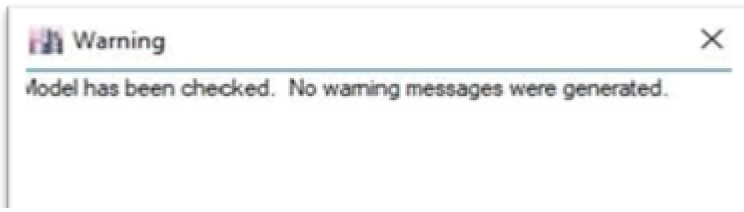


Figure 3.31 Shows the massage Check Model in the EATBS Program in our Study

4.18. RUN ANALYSIS

The program then runs the analysis, which can take several hours depending on the size of the project, and then the necessary results are extracted for the purpose of checking and reading for two different projects in Codes. This is done after finishing all of the necessary modeling, inserting, loading, checking, and treating of errors.

This methodology was utilized in order to give an accurate comparison between Baghdad, the capital of Iraq, and Cairo, the capital of Egypt, as well as to take into consideration the ongoing investment projects in both cities.

The fifth and final chapter will provide the findings of two projects that have been modeled, reviewed, and clarified throughout this chapter.

5. VERIFICATION AND EXPERIMENTAL WORK

5.1 GENERAL

This chapter is a crucial component of the whole topic, and no one can dispute the fact that computer programs are entirely dependent on correct data input and modelling.

In the second part of this chapter, an experimental construction was solved by hand, and the results of this were compared to the discoveries produced by the algorithm that was used in this chapter's conclusion, both manual and computer analysis are compared and contrasted.

5.2 ECCENTRICITY

Only if a diaphragm is given for each level is it possible to calculate and display the centres of mass and stiffness of the structure. The value (5.1) may be found in the table, and the fact that it has a negative sign suggests that the centre of stiffness is located closer to the studied axis than the centre of mass. This is due to the quality of horizontal distribution (shear walls and columns) in the horizontal plane, which creates the problem of actual decentralization between the mass centres and rigidity centres, and these two things need to be reconciled. Eccentricity is also noted to be small in comparison to the size of the structure. This is due to the quality of horizontal distribution (shear walls and columns) in the horizontal plane.

5.3 TORSION (RIGIDITY CENTRE, MASS CENTRE)

Because it creates an actual fundamental torsion, the difference between the rigidity centre and the mass centre is known as the actual eccentricity. Horizontal projections are the only ones that matter for calculating rigidity, which is assessed by the rigidity centre and the mass centre. Only horizontal projections require rigidity to be taken into consideration. It is possible that the shape of the structure's deformation will lead to incompatible or homogeneous transitions of nodes. This is because extroversion causes the additional torsion that was produced to be fundamentally different from the additional torsion that was induced by the eccentricity that was computed using the horizontal projection (V). If this turns out to be the situation, it will be absolutely necessary to install mass dampers in the places where the structure calls for them in order to achieve a state of equilibrium in the structure's deformation (or what is called damping).

5.4 TOTAL SEISMIC FORCE (V) AS IT TRAVELS VERTICALLY UP THE BUILDING

The shear force is dispersed in accordance with the following equations, which are derived from the dynamic assumption that is utilized in the MODF formula for the calculation of multi-story buildings in which the masses are aggregated at the levels of the floors (ASCE 7-10).

$$V = Ft + \sum_{i=1}^{n} Fi \tag{5.1}$$

$If\ T \leq 0.7\ sec,\ then\ Ft = 0$

$If\ T > 0.7 sec,\ then\ Ft = 0.07\ T\ V \leq 0.25\ V$

$$Fx = (V - Ft) * Wx * hx \sum Wi * hini = 1 \quad (5.2)$$

There are no shear forces along the Z axis since the seismic force is perpendicular to the axis, which is consistent with the equivalent static approach, and there is also no force along the y axis (Fy = 0) because there is no force along the y axis. (X).

The improvement brought about by the addition of brings the relationship between shear forces and height closer to being perfect (Figure 5.1). (Ft).

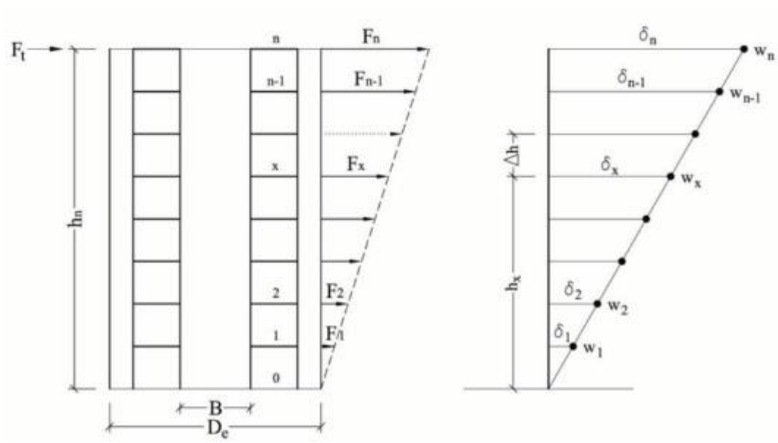


Figure 3.32 the distribution of the shear forces exerted by the base on the floors

5.5 RIGIDITY AND A LACK OF REGULARITY

According to one theoretical explanation, the trigonometric distribution of the base shear force and the height of the structure hint at the presence of a state of irregularity in the structure's stiffness (soft story). It is possible to infer the stiffness of the floor relative to the stiffness of the other floors through the program by the value of the displacements or deformations in general, which are entirely related to a linear way. This is especially true when the heights of the floors are not equal, as this is the situation in which the triangular or linearly distributed of the shear force (V) results from the assumption of a triangular shape for the first response pattern. As a consequence of this, it is possible to compare the values of the lateral transitions (displacements) or (drifts) that arise from the analyzed lateral pressures for the floors of the building. This comparison should take into account the difference in stiffness between the floors.

5.6 EXPERIMENTAL MODEL

An experimental case will be solved using both the manual technique and the software in order to check the results and remove any doubt that may have been there. Comparison of the results to one another will be used to assess the validity of the findings.

The seismic base shear (V) and lateral seismic forces (Fx) will be computed for each of the seven levels of the concrete structure using the figure that can be seen in Figure 5.2.

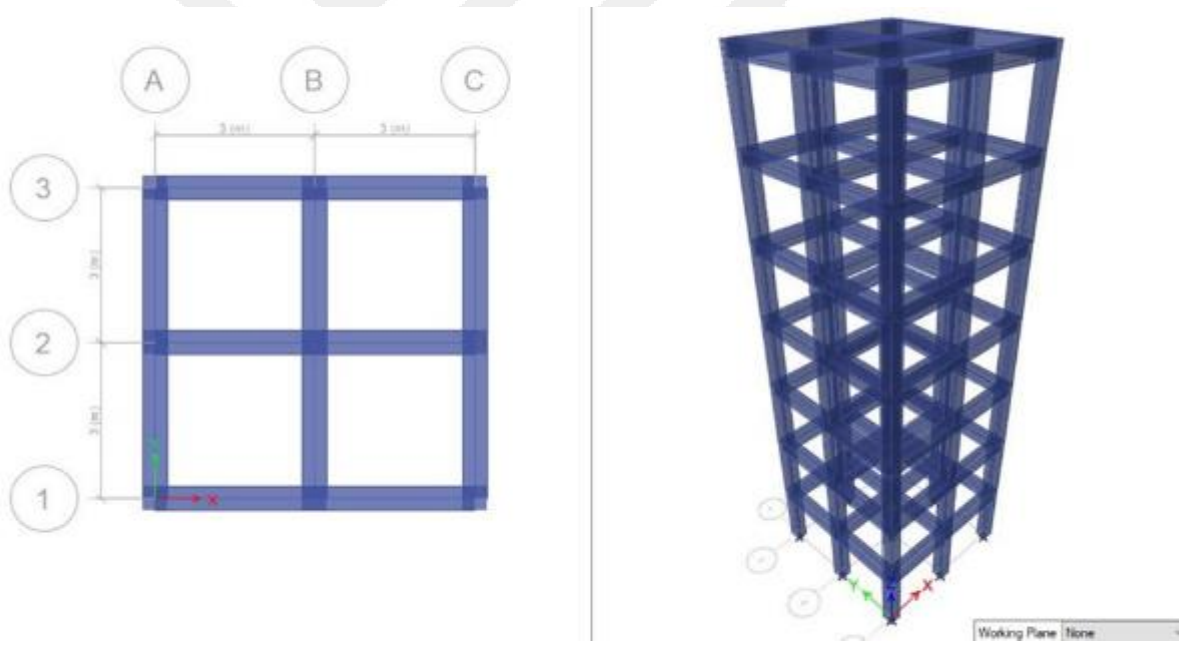


Figure 3.33 Experimental Model

5.6.1 Description of the Prototype Building with Square Cross-Section

In the calculation of the effective seismic weight, it was thought that the total dead load as well as the effective parts of the other loads should be included (W). At floor X is where you'll find a fraction of the building's total gravity, denoted by the letter W. The building in Baghdad, Iraq, was designated as an administrative structure since the floor heights were required to be a minimum of three meters and the soil on the site was classified as solid (Sd) [40]. The actual framework of the building is made up of beams, shear walls, and columns made of concrete.

The horizontal plane of the building is depicted here in the figure 5.2.

For the purpose of analyzing this building, either the (ISC2019) [46] or the (ASCE7-10) [47] code will be utilized while the seismic ground's spectral acceleration is Ss for (0.2) seconds = 0.3 and S1 for one second = 0.1 when measured over one second.

According to the information found in ISC2019 Table (1-7-1), category (D) shows that the soil is stable. This is in compliance with the Iraqi Earthquake Code for 2019, which mandates that this be done. After looking at the table (2-2/1A), we have determined that the magnitude of the position coefficient (Fa) at the greatest acceleration of the spectrum response to the earthquake and for short periods is equal to 1.35. This was determined after conducting our research using the table. Using the table (2-2/1b), we can determine that the position coefficient, Fv, is equal to 2.4 when the acceleration of the spectral response of the earthquake is at its highest. Using the equations for the item (2-2/3), we have the ability to alter the two values (Ss, S1) in order to acquire the value that we want (SMS, SM1).

$$S_{MS} = F_a * S_s = 1.35 * 0.3 = 0.405 \tag{5.3}$$

$$S_{M1} = F_v * S_1 = 2.4 * 0.1 = 0.24 \tag{5.4}$$

$$S_{DS} = 2/3 * S_{MS} = 2/3 * 0.405 = 0.27 \tag{5.5}$$

$$S_{D1} = 2/3 * S_{M1} == 2/3 * 0.24 = 0.16 \tag{5.6}$$

$$S_a = S_{DS}\,(0.4 + 0.6\,T/To)\; for\; T < To \tag{5.7}$$

$$S_a = S_{DS}\;\; for\; T >= To \tag{5.8}$$

$$S_a = S_{D1}/T \;\;\; for\; T >= T \tag{5.9}$$

$$To = 0.2\left(\frac{S_{D1}}{S_{DS}}\right) = 0.2(0.16/0.27) = 0.119\ sec \tag{5.10}$$

$$Ts = \frac{S_{D1}}{S_{DS}} = 0.16/0.27 = 0.594\ sec \tag{5.11}$$

Because it houses administrative operations, the building in question belongs to the second category of structures listed in Annex (A). As a result, a significant coefficient of 1 may be derived from the data shown in Table (3-2/1).

The aforementioned SDS and SD1 values may be extracted, and then we can use tables (4-2/1) and (4-2/a) to calculate the seismic design category from those values (D).

The value of R is determined to be 6.5 by referring to a table (3-2/1) and doing the calculations based on the design category of the earthquakes (D) RCS is an abbreviation for "Reinforced Concrete Structures System."

Our investigation will make use of the similar static force technique since compliance with the requirements of the Iraqi Code of Earthquakes mandates that we do so.

$$Ta = Ct * hnx\ From\ table\ (2/9-3), Ct = 0.044\ and\ X = 0.9 \tag{5.12}$$

$$Ta = 0.044 * 210.9 = 0.681\ sec$$

$$Cs = 0.044\ SDS\ I \quad < \quad Cs = SDSRI \quad < \quad Cs = SD1T * RI \tag{5.13}$$

$$0.012 \qquad < \qquad 0.0415 \qquad > \qquad 0.036$$

$$V = Cs\ W = 0.036 * 4855 = 174.78\ KN \tag{5.14}$$

$$Fx = Cvx\ V \tag{5.15}$$

$$\mathrm{Cvx} = \frac{\mathrm{Wx\ h}_x^k}{\sum_{i=1}^{n} \mathrm{Wi\ h}_i^k} \tag{5.16}$$

$K = 1.0905\ (from\ Ts = 0.681\ in\ between\ 0.5\ sec\ and\ 2.5\ sec)$

Table 3.7 Fx Value Calculations

Floor	Wx (KN)	hx (KN)	h_x^k	Wx h_x^k	Cvx	Fx (KN)
G	687	3	3.31	2274	0.03	5.28
F-1	682	6	7.06	4815	0.06	11.18
F-2	676	9	10.98	7422	0.10	17.23
F-3	671	12	15.03	10085	0.13	23.41
F-4	665	15	19.17	12748	0.17	29.60
F-5	661	18	23.38	15454	0.21	35.88
Roof	813	21	27.66	22488	0.30	52.21
				Σ75286		

Load Pattern	Type	User T (sec)	Top Story	Bottom Story	R	Ω	Cd	I	Ss	S1	TL	Site Class	Fa	Fv	Sds	Sd1	Period Used	Coeff. Used	Weight Used	Base Shear
Ex	Seismic	0.681	F- 7	Base	6.5	3	5.5	1	0.3	0.1	8	D	1.4	2.4	0.3	0.2	0.681	0.036	4811.4	188.346
Ex	Seismic	0.681	F- 7	Base	6.5	3	5.5	1	0.3	0.1	8	D	1.4	2.4	0.3	0.2	0.681	0.036	4811.4	188.346
Ex	Seismic	0.681	F- 7	Base	6.5	3	5.5	1	0.3	0.1	8	D	1.4	2.4	0.3	0.2	0.681	0.036	4811.4	188.346
Ey	Seismic	0.681	F- 7	Base	6.5	3	5.5	1	0.3	0.1	8	D	1.4	2.4	0.3	0.2	0.681	0.036	4811.4	188.346
Ey	Seismic	0.681	F- 7	Base	6.5	3	5.5	1	0.3	0.1	8	D	1.4	2.4	0.3	0.2	0.681	0.036	4811.4	188.346
Ey	Seismic	0.681	F- 7	Base	6.5	3	5.5	1	0.3	0.1	8	D	1.4	2.4	0.3	0.2	0.681	0.036	4811.4	188.346

Figure 3.34 Earthquake forces results for distribution of horizontal forces

5.6.2 ETABS Software analysis

In this study, ETBS is used. Model is shown in Figure 5.5.

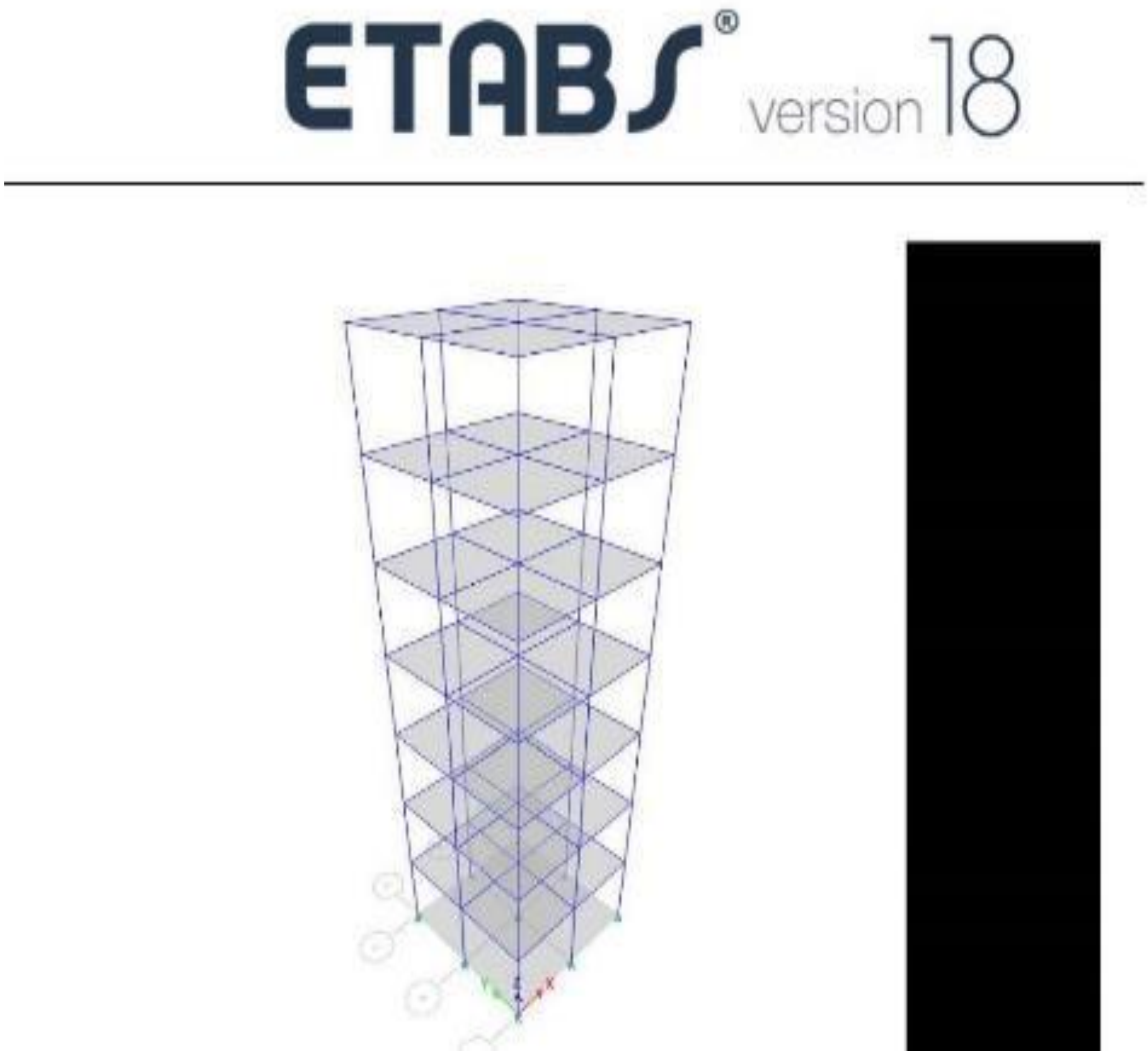


Figure 3.35 ETABS software

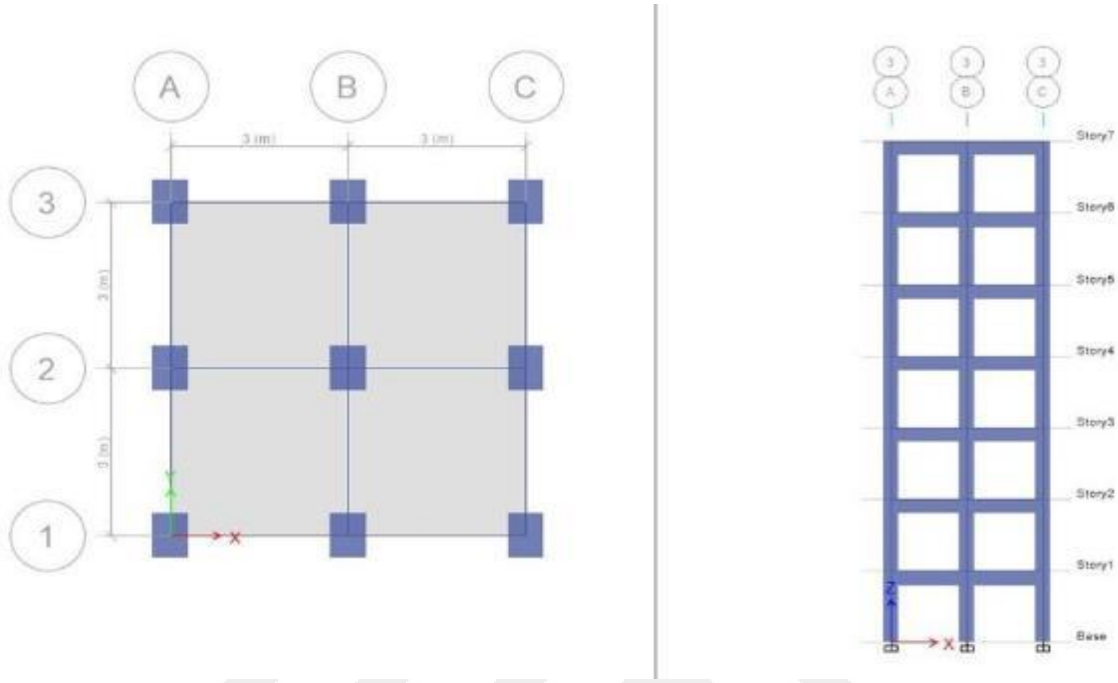


Figure 3.36 Model in software ETABS

The manual analysis came up with a number for the base shear forces that was comparable to 174.78 KN, which is in accordance with the seismic rules and the code of practice in Iraq for the year 2019 (ASCE7-10). The study performed by the ETABS software provided us with a result of 188.346 KN, which is equivalent to a matching ratio of 92.7%. This demonstrates the reliability and consistency of the findings, and it ensures that the program's outputs will receive full and unqualified acceptance (Figure 5.6-5.14).

Name	Type	Notes
Modal	Modal - Eigen	
Dead	Linear Static	
Live	Linear Static	
Ex	Linear Static	
Ey	Linear Static	
RSx	Response Spectrum	
RSy	Response Spectrum	

Figure 3.37 Load Cases

Output Case	Case Type	Step Type	Step Number	Step Label	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Dead	LinStatic				0	0	0	0	0	0
Live	LinStatic				0	0	0	0	0	0
Ex	LinStatic	Max			-188.3457	0	0	0	-2820.4114	565.037
Ex	LinStatic	Min			-188.3457	0	0	0	-2820.4114	565.037
Ey	LinStatic	Max			0	-188.3457	0	2820.4114	0	-565.037
Ey	LinStatic	Min			0	-188.3457	0	2820.4114	0	-565.037

Figure 3.38 output case

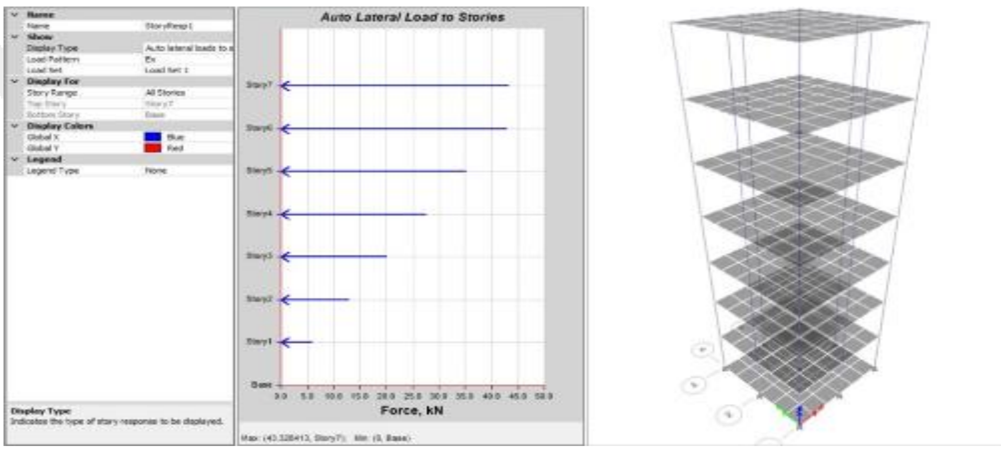


Figure 3.39 Lateral Load Stories

Loads 06/08/2022

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2] $C_S = \frac{S_{DS}}{(\frac{R}{I})}$

[ASCE 12.8.1.1, Eq. 12.8-3] $C_{S,max} = \frac{S_{D1}}{T(\frac{R}{I})}$

[ASCE 12.8.1.1, Eq. 12.8-5] $C_{S,min} = max\ (0.044\ S_{DS}\ I, 0.01\ ) = 0.023232$

[ASCE 12.8.1.1, Eq. 12.8-6] $C_{S,min} = 0.5\frac{S_1}{(\frac{R}{I})}$ for $S_1 = 0.6g$

$C_{S,min} \le C_s \le C_{S,max}$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
X	0.681	0.060243	3126.4203	188.3457
X + Ecc. Y	0.681	0.060243	3126.4203	188.3457
X - Ecc. Y	0.681	0.060243	3126.4203	188.3457

Figure 3.40 applied story load

Loads 06/08/2

ASCE 7-10 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Ex according to ASCE 7-10, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = User Specified

User Period		$T = 0.081$ sec
Long-Period Transition Period, T_L [ASCE 11.4.5]		$T_L = 8$ sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]		$R = 6.5$
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]		$\Omega_0 = 3$
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]		$C_d = 5.5$
Importance Factor, I [ASCE Table 11.5-1]		$I = 1$

Ss and S1 Source = User Specified

Mapped MCE Spectral Response Acceleration, S_S [ASCE 11.4.1]		$S_s = 0.6g$
Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.1]		$S_1 = 0.2g$

Site Class [ASCE Table 20.3-1] = D - Stiff Soil

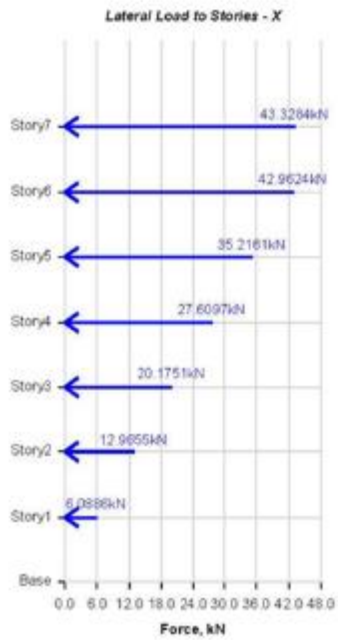
Site Coefficient, F_a [ASCE Table 11.4-1]		$F_a = 1.32$
Site Coefficient, F_v [ASCE Table 11.4-2]		$F_v = 2$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.3, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 0.792g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.3, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.4g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.4, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.528g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.4, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.266667g$

Figure 3.41 load calculation

Loads



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story7	21	43.3284	0
Story6	18	42.9624	0
Story5	15	35.2161	0
Story4	12	27.6097	0
Story3	9	20.1751	0
Story2	6	12.9655	0
Story1	3	6.0886	0
Base	0	0	0

Figure 3.42 Lateral Load Stories

Loads 06/08/2022

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-2] $C_S = \frac{S_{DS}}{(\frac{R}{I})}$

[ASCE 12.8.1.1, Eq. 12.8-3] $C_{S,max} = \frac{S_{D1}}{T(\frac{R}{I})}$

[ASCE 12.8.1.1, Eq. 12.8-5] $C_{S,min} = max\ (0.044\ S_{DS}\ I, 0.01\ ) = 0.023232$

[ASCE 12.8.1.1, Eq. 12.8-6] $C_{S,min} = 0.5 \frac{S_1}{(\frac{R}{I})}$ for $S_1 = 0.6g$

$$C_{S,min} \le C_s \le C_{S,max}$$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kN)	V (kN)
Y	0.681	0.060243	3126.4203	188.3457
Y + Ecc. X	0.681	0.060243	3126.4203	188.3457
Y - Ecc. X	0.681	0.060243	3126.4203	188.3457

Applied Story Forces

Figure 3.43 Applied story forces

ASCE 7-10 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Ey according to ASCE 7-10, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

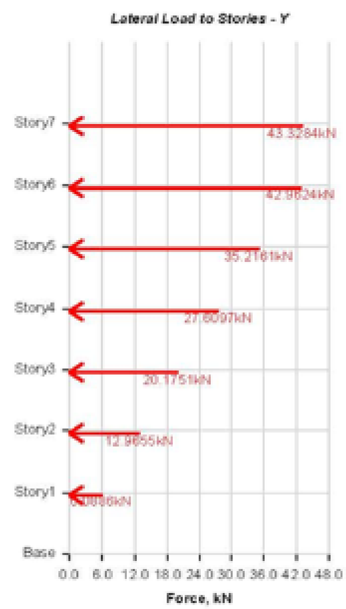
Structural Period

Period Calculation Method = User Specified

User Period		$T = 0.681\ sec$
Long-Period Transition Period, T_L [ASCE 11.4.5]		$T_L = 8\ sec$
Factors and Coefficients		
Response Modification Factor, R [ASCE Table 12.2-1]		$R = 6.5$
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]		$\Omega_0 = 3$
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]		$C_d = 5.5$
Importance Factor, I [ASCE Table 11.5-1]		$I = 1$
Ss and S1 Source = User Specified		
Mapped MCE Spectral Response Acceleration, S_s [ASCE 11.4.1]		$S_s = 0.6g$
Mapped MCE Spectral Response Acceleration, S_1 [ASCE 11.4.1]		$S_1 = 0.2g$
Site Class [ASCE Table 20.3-1] = D - Stiff Soil		
Site Coefficient, F_a [ASCE Table 11.4-1]		$F_a = 1.32$
Site Coefficient, F_v [ASCE Table 11.4-2]		$F_v = 2$
Seismic Response		
MCE Spectral Response Acceleration, S_{MS} [ASCE 11.4.3, Eq. 11.4-1]	$S_{MS} = F_a S_S$	$S_{MS} = 0.792g$
MCE Spectral Response Acceleration, S_{M1} [ASCE 11.4.3, Eq. 11.4-2]	$S_{M1} = F_v S_1$	$S_{M1} = 0.4g$
Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.4, Eq. 11.4-3]	$S_{DS} = \frac{2}{3} S_{MS}$	$S_{DS} = 0.528g$
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.4, Eq. 11.4-4]	$S_{D1} = \frac{2}{3} S_{M1}$	$S_{D1} = 0.266667g$

Figure 3.44 auto seismic load calculation

Loads



Story	Elevation	X-Dir	Y-Dir
	m	kN	kN
Story7	21	0	43.3284
Story6	18	0	42.9624
Story5	15	0	35.2161
Story4	12	0	27.6097
Story3	9	0	20.1751
Story2	6	0	12.9655
Story1	3	0	6.0886
Base	0	0	0

Figure 3.45 Lateral Load to Stories-Y

6. RESULTS AND DISCUSSIONGENERAL OVERVIEW

This chapter will introduce a number of different variables, such as the height of the building and the structural system, in order to understand the impact that these factors have on buildings, particularly the variation in the special code for earthquakes and the degree to which it is applied. In particular, this chapter will focus on understanding the impact that earthquakes have on buildings. In the preceding chapters, we will go over the variations in the outcomes that occur depending on the particular, with the intention of achieving the most accurate and best results that are humanly feasible. Not to add the disparities that exist between the legal systems of Egypt and Iraq (ISC 2019). (ESC), In this scenario, the software analyses the structure as many times as there are static and dynamic loading scenarios (Load Cases), and then collects data from each analysis, including forces, moments, and other internal activities for each part of the structure. In addition, there are load cases that include both static and dynamic loading. When the necessary conditions for structural analysis are met, such as those listed in (Conditions of Structural analysis), the (Principle of Superposition) states that "when a set of external forces are applied to the structure, their actions in the structure are equivalent to the actions of the forces forming them in the various elements of the structure."

Condition of one equilibrium: under the influence of externally imposed loads and reactions in the supports, the structure and all of its sections must be balanced or stable statically and dynamically in order to satisfy the requirements of this condition (based on the kind of applied loads and their connection to time).

Second, distortions Compatibility requires that the deformations of the source correspond to the limitations that have been put on it from without.

Thirdly, one is successful in acquiring the differential relations of the material. In order for the analysis to be static, it is necessary to achieve compliance with Hooke's law (=E), often known as conformity with the theory of elasticity. When there is a dynamic component to the study, that component of the study will always be the dynamic. In order to satisfy the requirements of the plasticity hypothesis, a condition of relative equilibrium needs to be accomplished.

6.2 FACTORS AFFECTING STRUCTURAL EARTHQUAKE

The constructional layout of a building is simply one of several factors that might influence how effectively a structure can resist an earthquake. Other factors include:

a. The way in which the structure resonates.

b. The insulating qualities that are possessed by the structure.

c. The third part of the structure is the building's foundation.

d. The importance of the structure's role in the whole.

e. The structure's adaptability to changing conditions.

f. The sixth distinguishing feature is the ability to deflect while bending laterally.

6.3 RESULTS AND DISCUSSION

Each zone included a column and shear wall structural component, and each structural system was investigated according to four different codes: (ACI 318-14, ISC 2019 (ASCE7-10), (BS 8110-79, and (ESC)) (EUROCODE 8-2004). The two buildings had exactly the same dimensions (67.4 meters in height, 21 floors), and the findings will be presented for two nations that are located in distinctly distinct seismic zones.

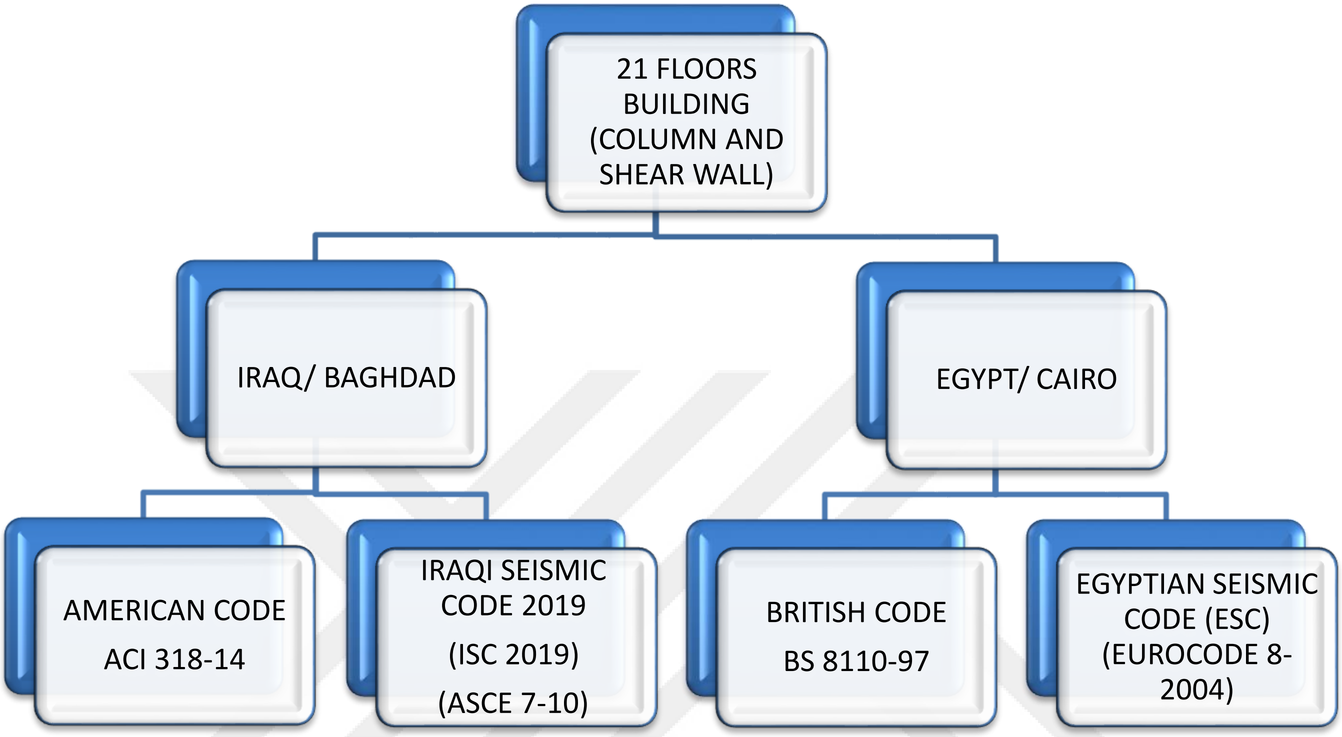


Figure 6.1 Structural Schedule shows a Model in ETABS Program for the two (67.4 m in height, 21 Floor-Buildings) based in (Iraq/ Baghdad) and (Egypt/ Cairo) according to ((ISC 2019-(ASCE 2017)) and ((ESC - (EUROCODE 8-2004)) respectively

6.3.1 Base Shear

The value of the base shear forces is one of the fundamentals of our comparison. This value is distributed later on the height of the floors, as is explained in the fourth and fifth chapters of each code, Code ((ISC 2019 (ASCE 7-10) and ((ESC (EUROCODE 8-2004)), which extract the base shear forces and then continue to the seismic analysis [44]. One of the fundamentals of our comparison is the value of the base shear forces (figure 6.2).

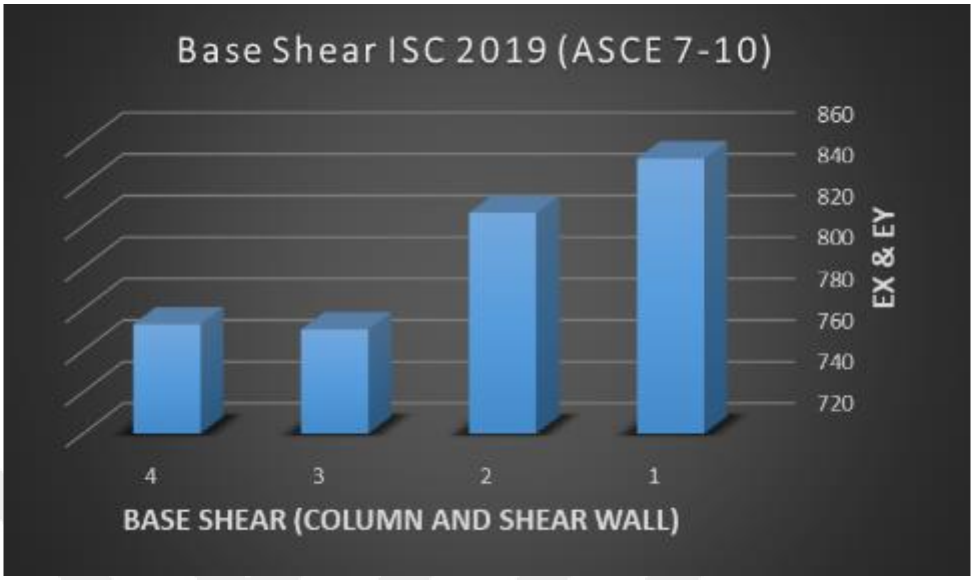


Figure 6.2 Shows Base Shear according to ((ISC 2019-(ASCE 7-10)) Code for the 21 Floor-Building in (Iraq/ Baghdad).

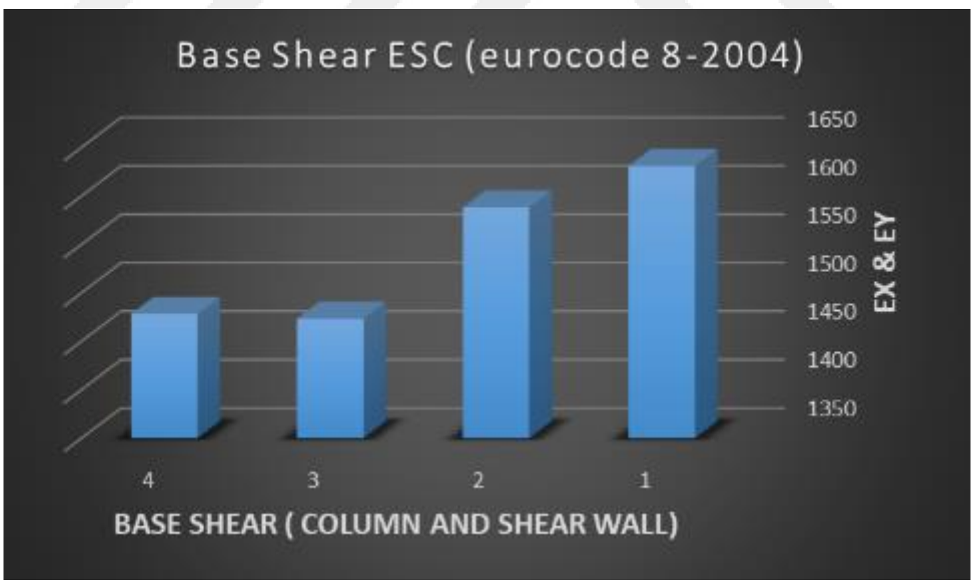
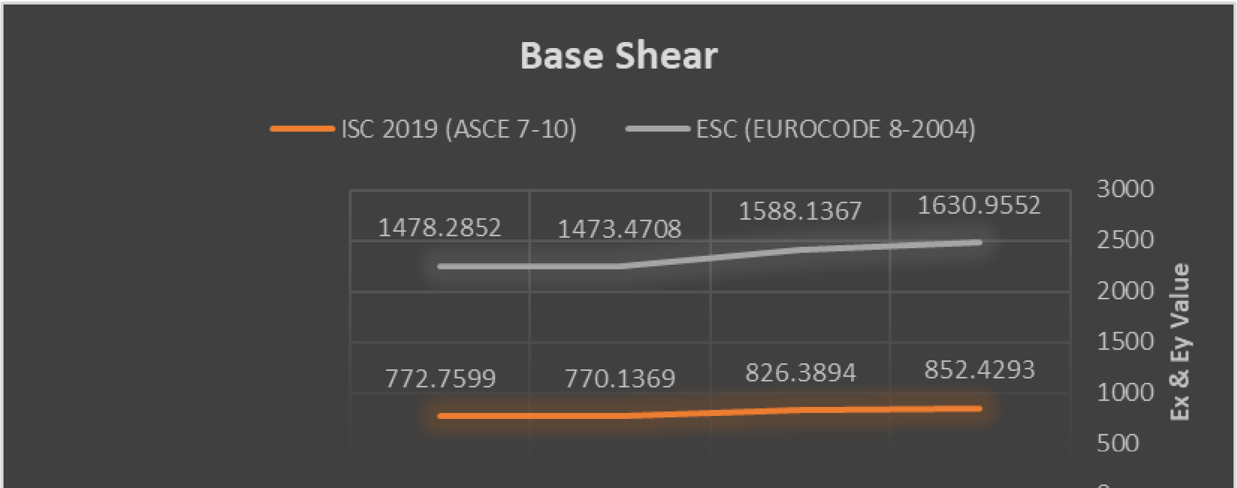


Figure 6.3 Shows Base Shear according to ((ESC - (EUROCODE 8-2004)) Code for the 21 Floor-Building in (Egypt/ Cairo).

For the same structure that was looked into, it was discovered that the shear forces in Baghdad for the same weight and loads in code ((ISC 2019 (ASCE 7-10) was = 852.429 Ton. This was determined via investigation. For the identical structure, the code that was utilized in

Egypt/Cairo that was equal was ((ESC (EUROCODE 8-2004)) =1930.955 Ton. As can be seen in figure 6.4 below, the 2019 revision of the Iraqi seismic code produced the best possible outcome, which was the lowest possible value.



	4	3	2	1
ESC (EUROCODE 8-2004)	1478.2852	1473.4708	1588.1367	1630.9552
ISC 2019 (ASCE 7-10)	772.7599	770.1369	826.3894	852.4293

Figure 6.4 Shows Base Shear according to ((ISC 2019-(ASCE 2017)) and ((ESC - (EUROCODE 8-2004)) Codes for the two (21 Floor-Buildings) in (Iraq/ Baghdad) and (Egypt/ Cairo) respectively.

6.3.2 Story Displacement

6.3.2.1 Maximum Story Displacement ACI 318-14 Code

Figures 6.5 a, b, c, and d from the ETABS Program indicate the maximum story displacement in compliance with the Iraqi Seismic Code ((ISC 2019-(ASCE 7-10)). These figures are used in conjunction with the Iraqi Seismic Code (ACI 318-14 Code).

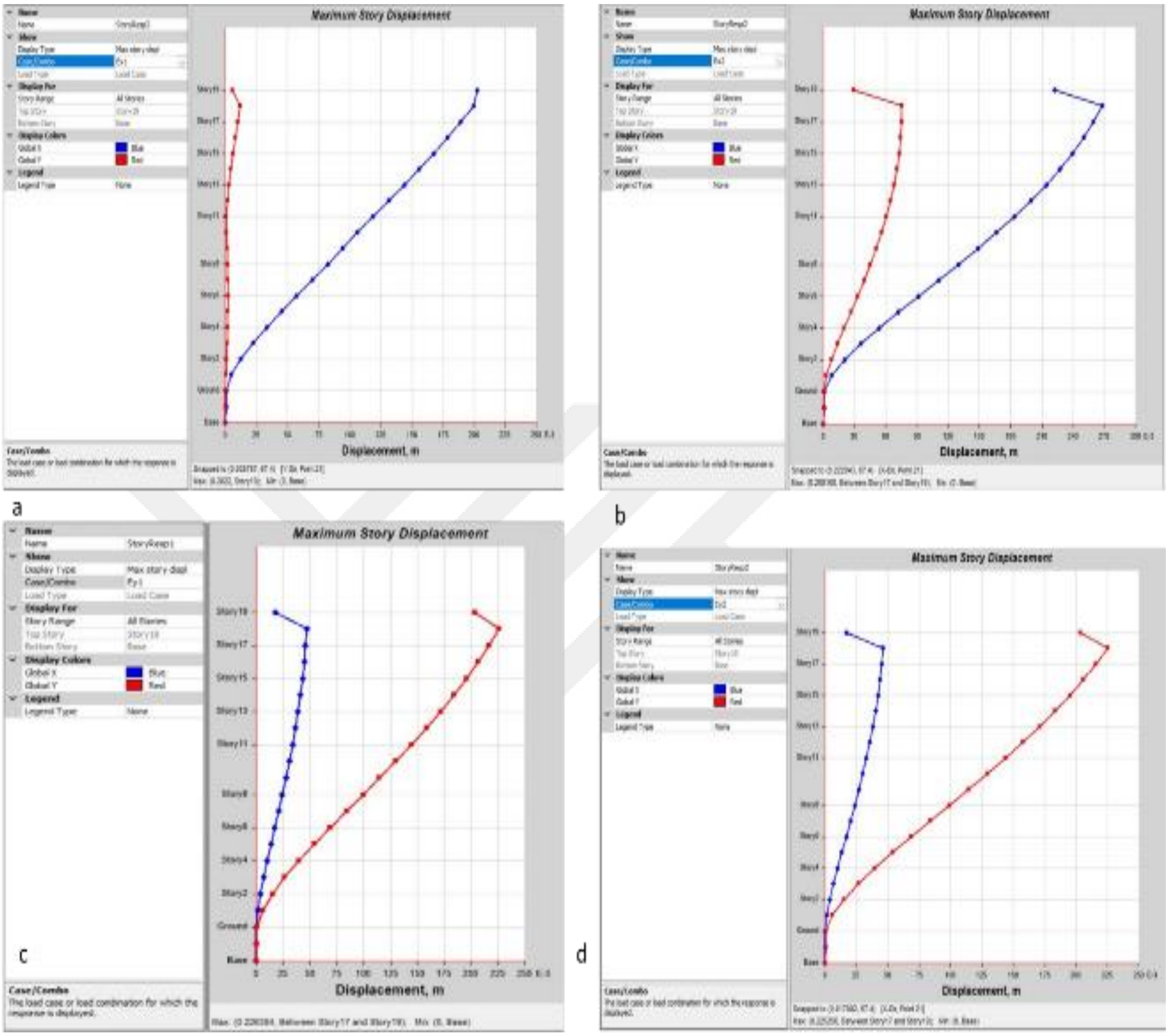


Figure 6.5 Maximum story displacement ACI 318-14 Code

6.3.2.2 Maximum Story Displacement BS 8110-97 Code

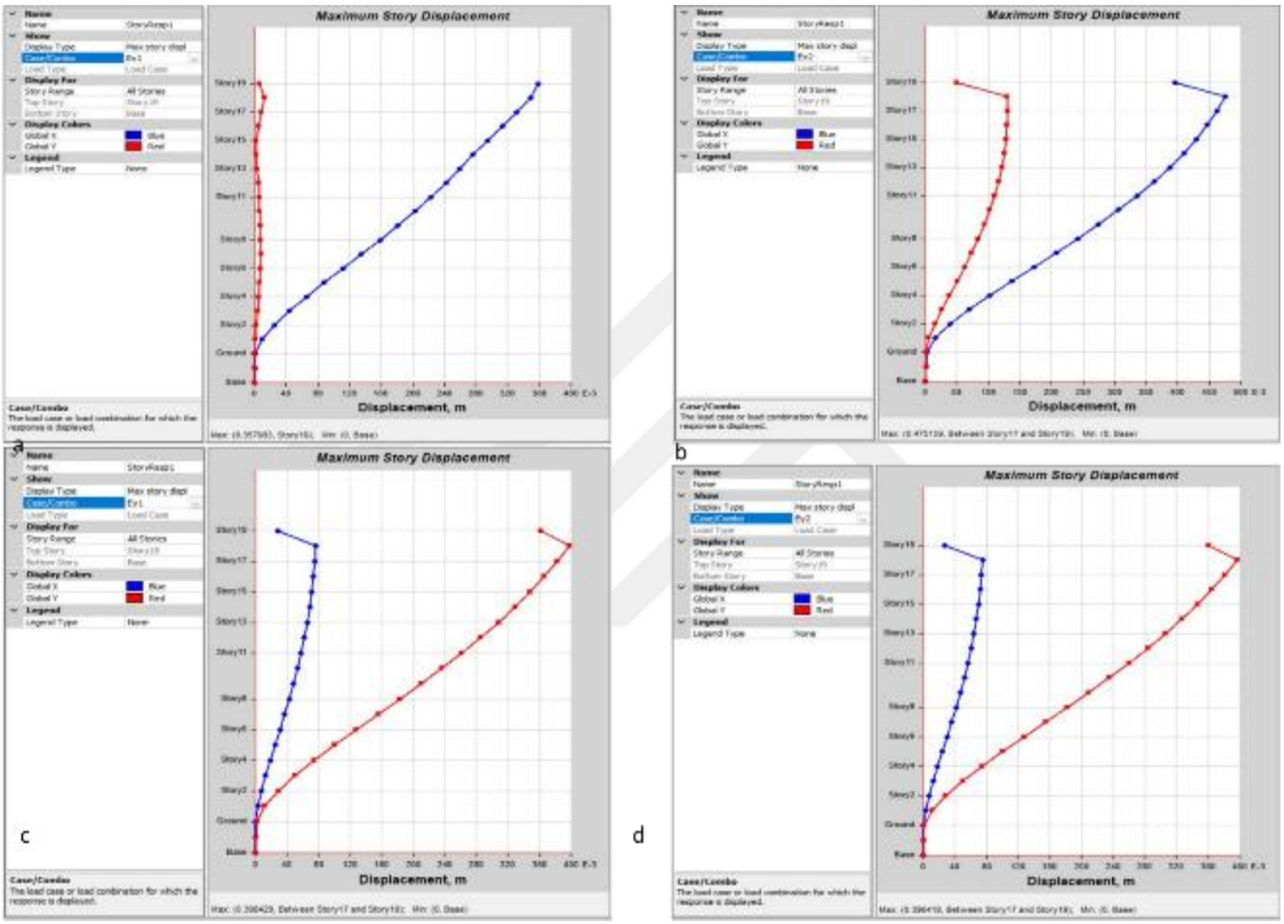


Figure 6.6 Maximum Story Displacement BS 8110-97 Code

Figure 6.7 is a graphical representation of the findings, which are detailed in further detail in the aforementioned scripts.

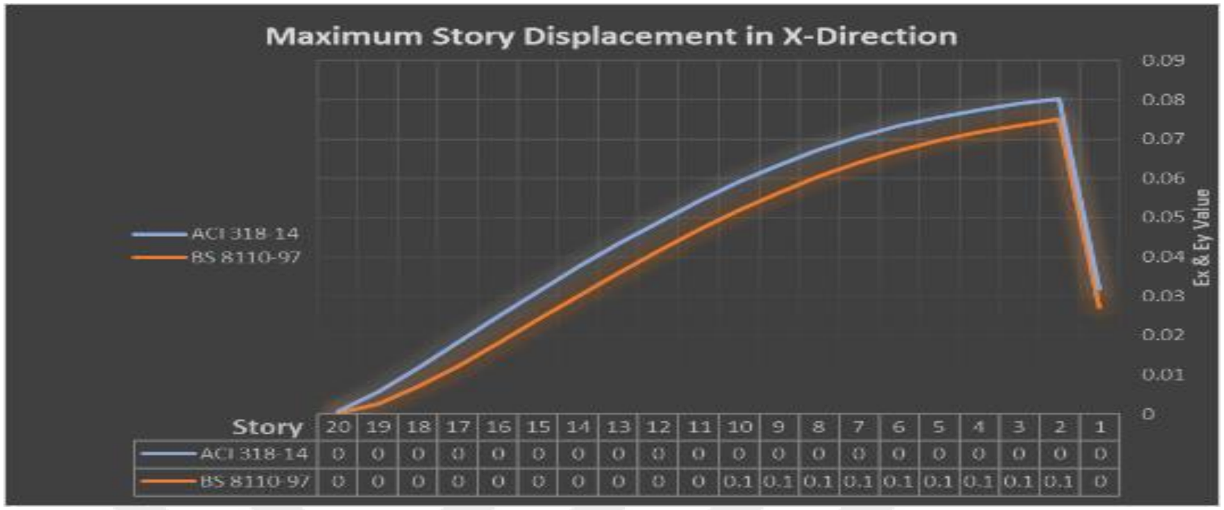


Figure 6.7 Shows Maximum Story Displacement according to (ACI 318-14) and (BS 8110-97) Codes for the two (21 Floor-Buildings) in (Iraq/ Baghdad) and (Egypt/ Cairo) respectively.

Based on a comparison with the particular code 0.002 of the building height (67.4 m), which is equal to 134.8 mm, we find that the Egyptian seismic code ESC (EUROCODE 8-2004) is satisfied, whereas the Iraqi seismic code 2019 ISC 2019-(ASCE 7-10) is not acceptable. This conclusion is reached after discovering that the Egyptian seismic code ESC (EUROCODE 8-2004) is met.

Table 6.1 Maximum Story Displacement according to ((ACI 318-14 (ISC 2019-(ASCE 7-10)) and ((BS 8110-97 (ESC (EUROCODE 8-2004)) Codes for the two (67.4 m in height -21 Floors) Buildings in (Iraq/ Baghdad) and (Egypt/ Cairo) respectively

Design Area	**Design Codes**	**Seismic codes**	**Elevation**	**Max. Displacement (mm) x**	**Max. Displacement (mm) y**	**Status**
Iraq/ Baghdad	ACI 318-14	ISC 2019-(ASCE 7-10)	Between (Story 19-61m & Story 21-67.4m)	268.1	226.4	Not Ok
Egypt/ Cairo	BS 8110-97	ESC (EUROCODE 8-2004)	Between (Story 19-61m & Story 21-67.4m)	396.43	396.43	Ok

6.3.3 Story Drift

Calculations of story drift are an essential part of the building design process. These calculations are used to show the placement of walls and windows in order to reduce the likelihood of fractures. The repercussions of neglecting story drifts are very severe, especially on outdoor surfaces when there are structural windows or brick walls present. In order to establish the plot drift for story I, we are in possession of.

$$DRi = \delta i - \delta(i-1)Hi \tag{6.1}$$

The DRĭ is story drift for story ĭ, δi displacement of story ĭ, δi–1 is the displacement of lower story h (i-1) and the hi is the story height.

6.3.3.1 Maximum Story Drift ACI 318-14 Code

According to the (ACI 318-14 Code) and the Iraqi Seismic Code ((ISC 2019-(ASCE 7-10)), the maximum story drift for the column and shear wall construction system is depicted in Figures 6.8a, 6.8b, 6.8c, and 6.8d from the ETABS program. These figures show the maximum story drift for the column and shear wall construction system.

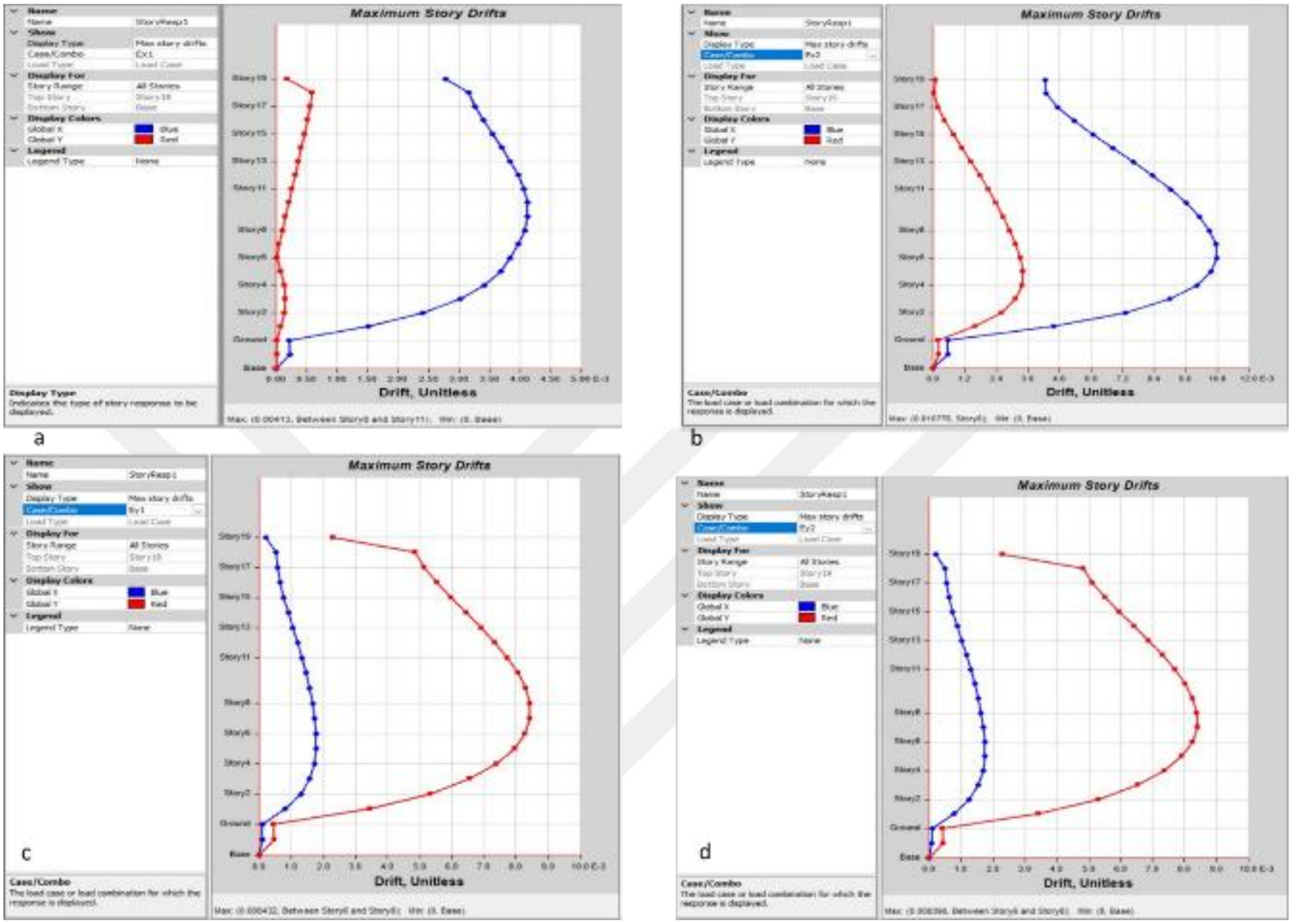


Figure 6.8 Maximum Story Drift ACI 318-14 Code

6.3.3.2 Maximum Story Drift BS 8110-97 Code

The maximum story drift for the column-and-shear-wall construction system is depicted in Figure 6.9a, b, c, and D from the ETABS software. This was done while utilizing the Egyptian seismic ((ESC (EUROCODE 8-2004)) data.

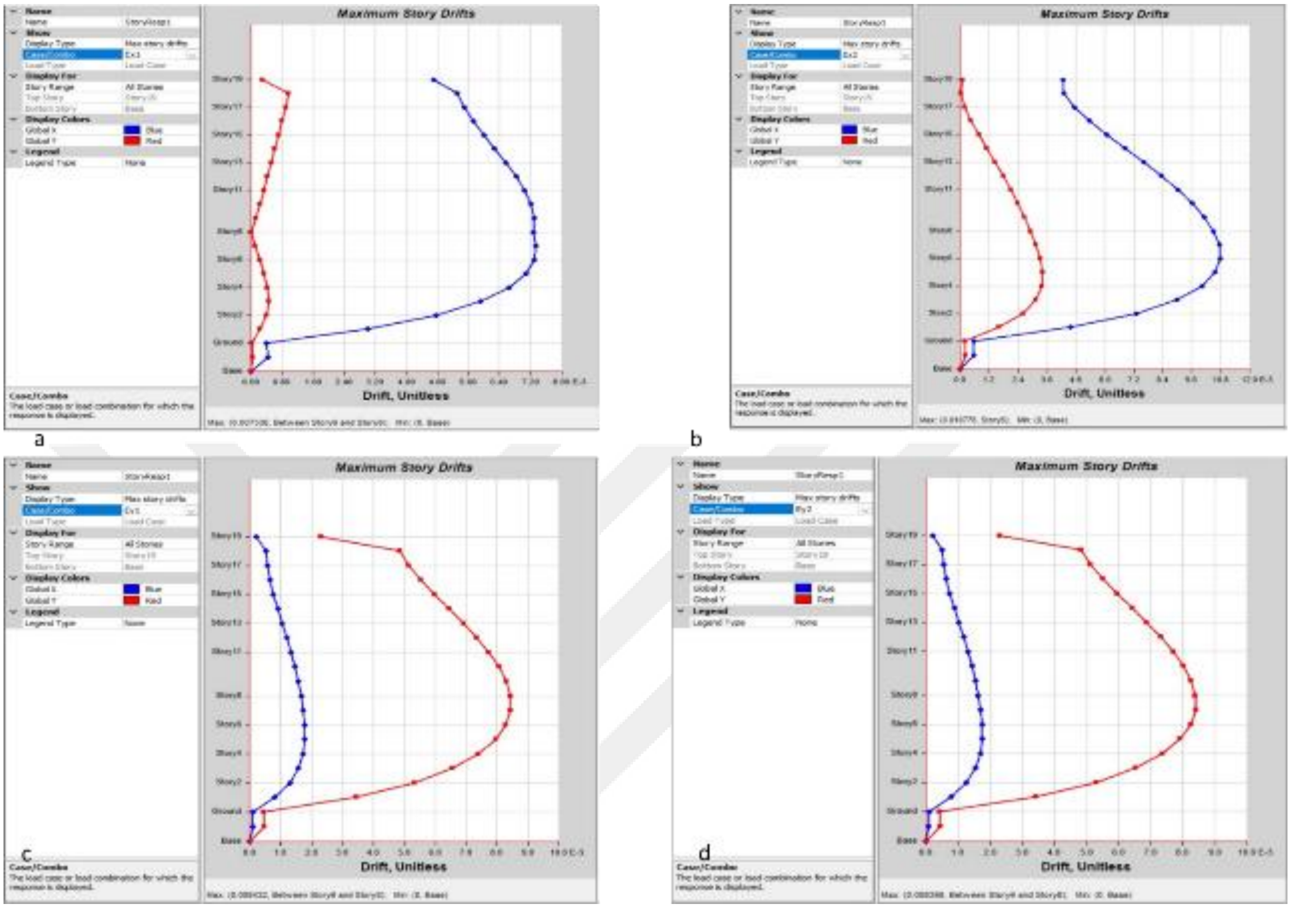


Figure 6.9 Maximum story drifts BS 8110-97 Code

When we keep the aforementioned codes in mind, we are now able to analyze the results shown in figure 6.10.

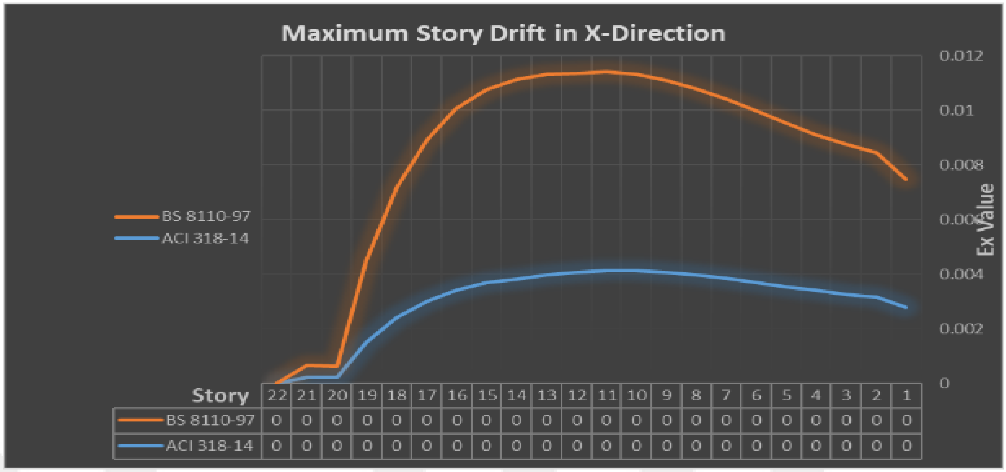


Figure 6.10 Shows Maximum Story Drift according to (ACI 318-14) and (BS 8110-97) Codes for the two (21 Floor-Buildings) in (Iraq/ Baghdad) and (Egypt/ Cairo) respectively.

When appropriate, the ISC 2019 and ESC Codes require that calculations be made to determine the horizontal displacements of the structure. In line with the provisions of this section, the Maximum Inelastic Response Displacement denoted by the letter M, of the structure as a result of the Design Basis Ground Motion needs to be computed for both the Allowable Stress Design and the Strength Design. The drifts associated with the seismic design.

$$\Delta M = 0.7\ R\ \Delta s \tag{6.2}$$

Table 6.2 Maximum Story Drift according to ((ACI 318-14 (ISC 2019-(ASCE 7-10)) and ((BS 8110-97 (ESC (EUROCODE 8-2004)) Codes for the two (67.4 m in height -21 Floors) Buildings in (Iraq/ Baghdad) and (Egypt/ Cairo) respectively.

Design Area	Design Codes	Seismic codes	Elevation	Max. Drift (unitless) x-dir.	Max. Drift (unitless) y-dir.
Iraq/ Baghdad	ACI 318-14	ISC 2019-(ASCE 7-10)	Between (Story 8-32.2m & Story 11-41.8m)	0.00413	0.00472
Egypt/ Cairo	BS 8110-97	ESC (EUROCODE 8-2004)	Between (Story 6-25.8m & Story 8-32.2m)	0.007336	0.00843

By applying the calculation that is presented in paragraph (15/9-3) of the International Building Code for 2019, (ISC 2019) "The maximum permissible amount of drift for each level of a building that has a basic period of less than 0.7 seconds shall be less than 0.025 times the height of the building. For structures with a basic period of at least 0.7 seconds, the computed narrative drift must be less than 0.020 times the story height." Since the floor height of the structure that is the subject of this study is, all of the structural systems that were employed in this research have a T that is less than 0.7 seconds (3.2 m).

ISC2019 Drift Limits: The following is a list of the ISC2019 drift limitations that were applied to the structure that was being evaluated, with categories according to the kind of works and the nature of construction.

$$\Delta a \leq 0.007 hsx \qquad (6.3)$$

$$hsx: the\ story\ height\ =\ 3200\ mm$$

$$\Delta a = 21.2mm$$

Table 6.3 Discussing drift results and comparing them with a Limits ISC 2019

Design Area	Seismic codes	Max. Drift (unitless)	Hsx (mm)	Δsi (mm)	R	ΔMi (mm)	Limit (mm)	Status
		Story$_{(i)}$					0.025*3200	
Iraq/ Baghdad	ISC 2019- (ASCE 7-10)	0.00413	3200	13.216	5	46.256	80	Ok
Egypt/ Cairo	ESC (EUROCODE 8-2004)	0.00734	3200	23.4752	5	82.163	80	Not Ok

7 CONCLUSIONS AND SUGGESTIONS FOR FURTHER STUDIES

7.1 GENERAL OVERVIEW

In this chapter, results are drawn based on static and dynamic seismic analyses of two models that were subjected to actual earthquake magnitude. These analyses took into account the effect of the design code as well as the differences between seismic maps. The following are some suggestions and recommendations for the conduct of more study.

7.2 CONCLUSIONS

This study compares and contrasts the analysis of concrete structures that are to be built or created in Baghdad and Cairo. The study takes into consideration the influence that seismic maps have when applied to the structural system. The principles of it, together with some sample seismic maps and analysis techniques, as well as the findings of a case study, were presented here. The following are likely the inferences you can make:

a. The mass centre and the stiffness of the structure can both be affected by the addition of a shear wall, as well as by the placement of that wall. It is quite unlikely that any asymmetric models exist. Both the gravitational force and the centre of mass contribute to this result. The conditions will be satisfied when both the structure's centre of mass and its centre of stiffness are brought into closer proximity to one another, as they are in a shear wall.

b. Shear walls are used to reinforce buildings and absorb lateral loads, which ultimately results in reduced displacement. Shear walls are utilized in construction (2). When response spectrum analysis is done to the building, the results show an improvement in performance as well as a reduction in displacement in the X and Y axes.

c. Finite element software, such as ETABS, is an especially useful tool for modeling large-scale infrastructures in situations in which actual studies or simulations to examine the behaviour of a system under a variety of circumstances are either too expensive or too complex to manage. This is one of the situations in which finite element software can be particularly useful (e.g. high rise buildings).

d. The seismic coefficient in the current Cairo map gives the Egyptian code the highest base shear values, followed by the earthquake code issued in (EUROCODE 8-2004), and finally the Iraqi code for the year 2019 with a structural system of shear walls giving the lowest results in terms of displacements, drifts, and base shear values.

e. The seismic coefficient in the current Cairo map gives the Iraqi code the lowest results in terms of displacements, drifts, and base shear values.

f. Modal contours are formed by dynamic and static analysis, and there was a linear link between the number of stories displaced and the building's height from 5 to 21 stories.

7.3 SUGGESTION FOR FUTURE STUDIES

Some ways in which the findings of the current study can be used to future research:

a. It is essential to investigate the effects of earthquakes on different types of buildings that are also utilized so that one may have a better understanding of how well these structures function under these loads.

b. Because performing a different analysis on a real building that was designed using a different design code leads to results that are different from the ones that were designed based on it, buildings need to be analyzed or developed on the same designed code, or at the very least, a unique amplification factor needs to be imposed for each building so that it can be dealt with.

c. Seismic research and building codes focus on one of the most secure tubular structures due to its resistance to impact loads and its high rigidity of structure in resisting lateral and gravitational stresses. This property makes it an excellent candidate for seismic engineering. Typically, it is made up of two tubes, with the larger tube located on the exterior of the building and the smaller tube located on the inside. However, if additional safety is required and if it can be demonstrated that the concept is successful, the design may include additional tubes within the tubes already present [51].

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APPENDIX A

SUMMARY ANALYSIS REPORT

A.1 FIGURES

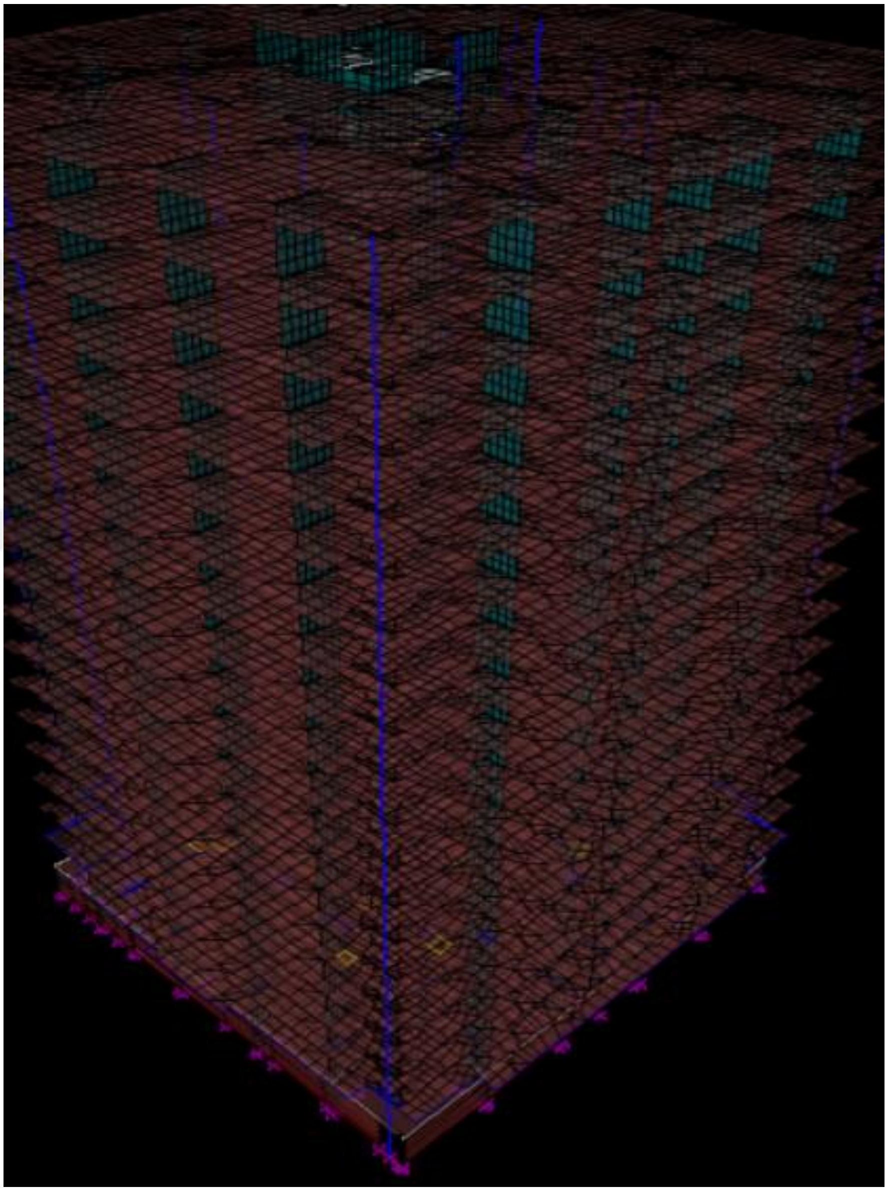


Figure A.1 Deformed Shape for the building caused by lateral loads

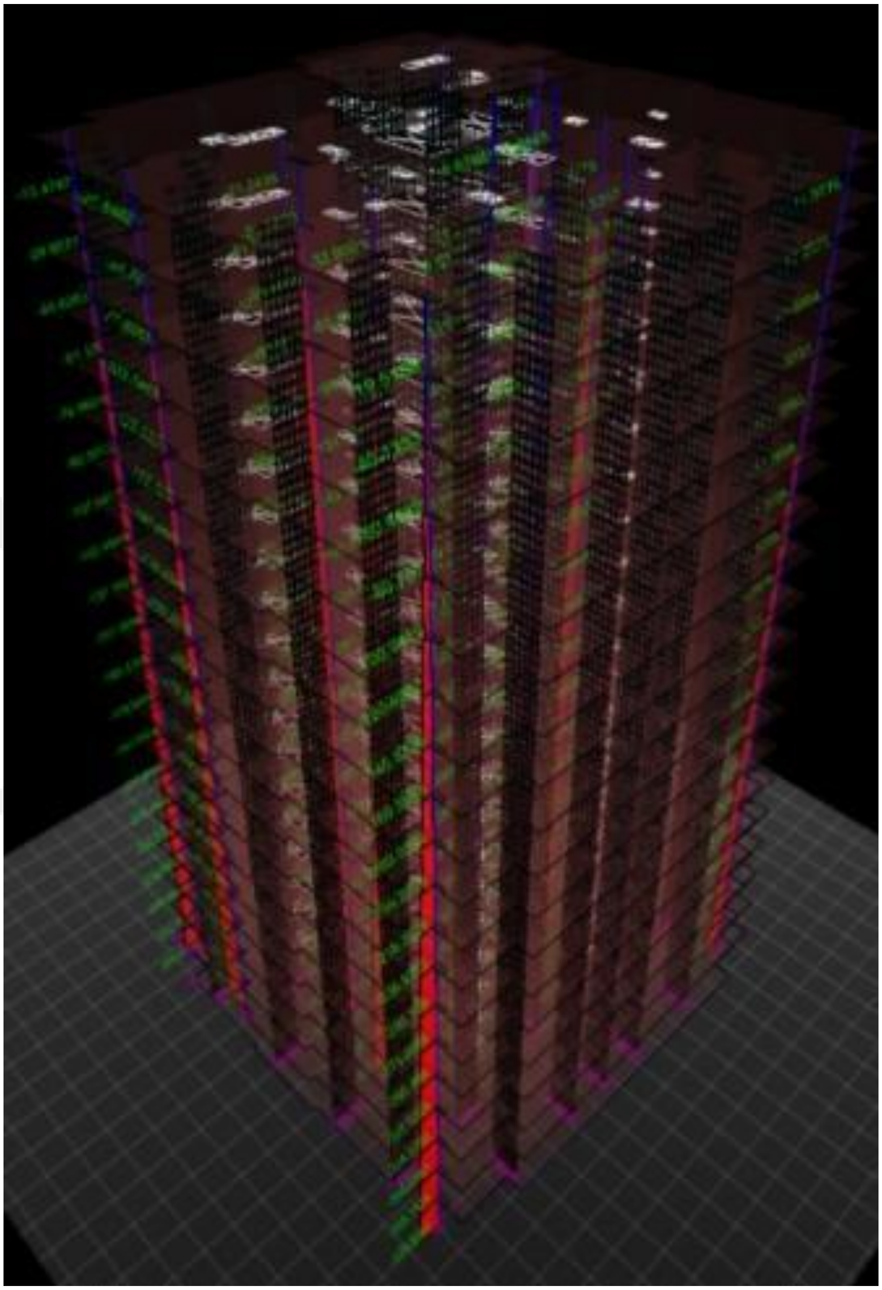


Figure A.2 3D Model Axial Forces in ETABS Program

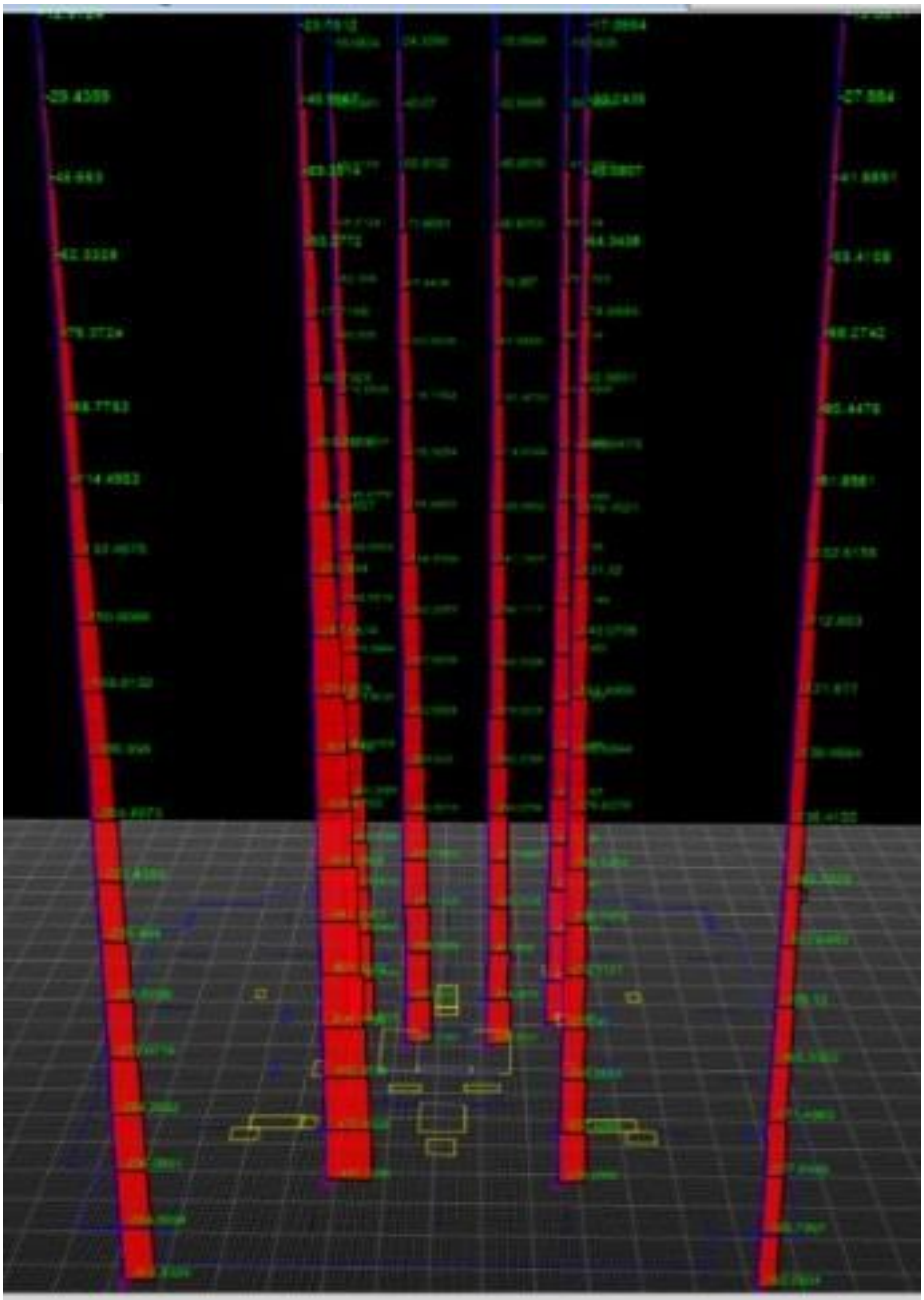


Figure A.3 Axial Forces all columns Iraqi seismic code (ISC 2019)

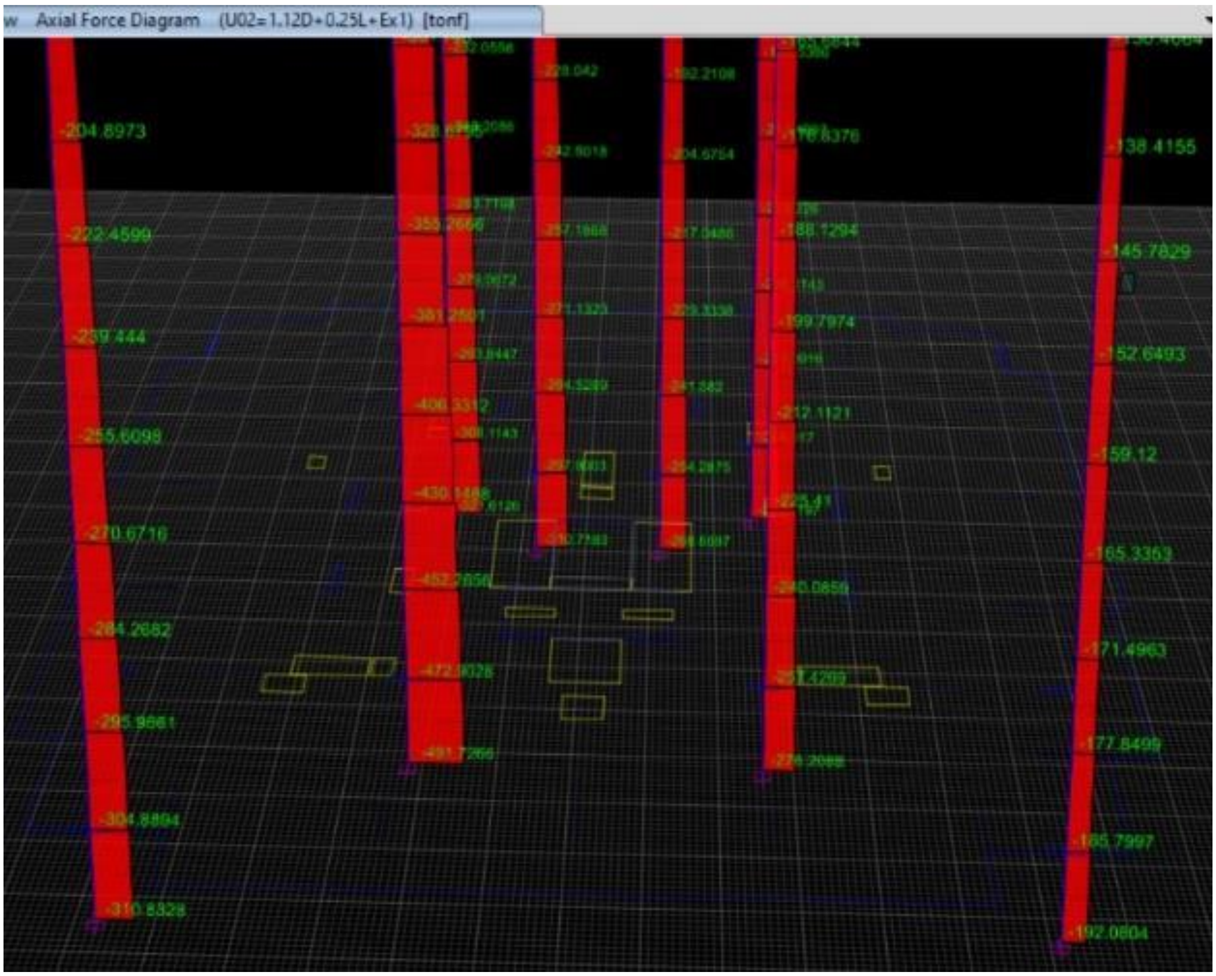


Figure A.4 Enlargement section for Axial Forces Iraqi seismic code (ISC 2019)

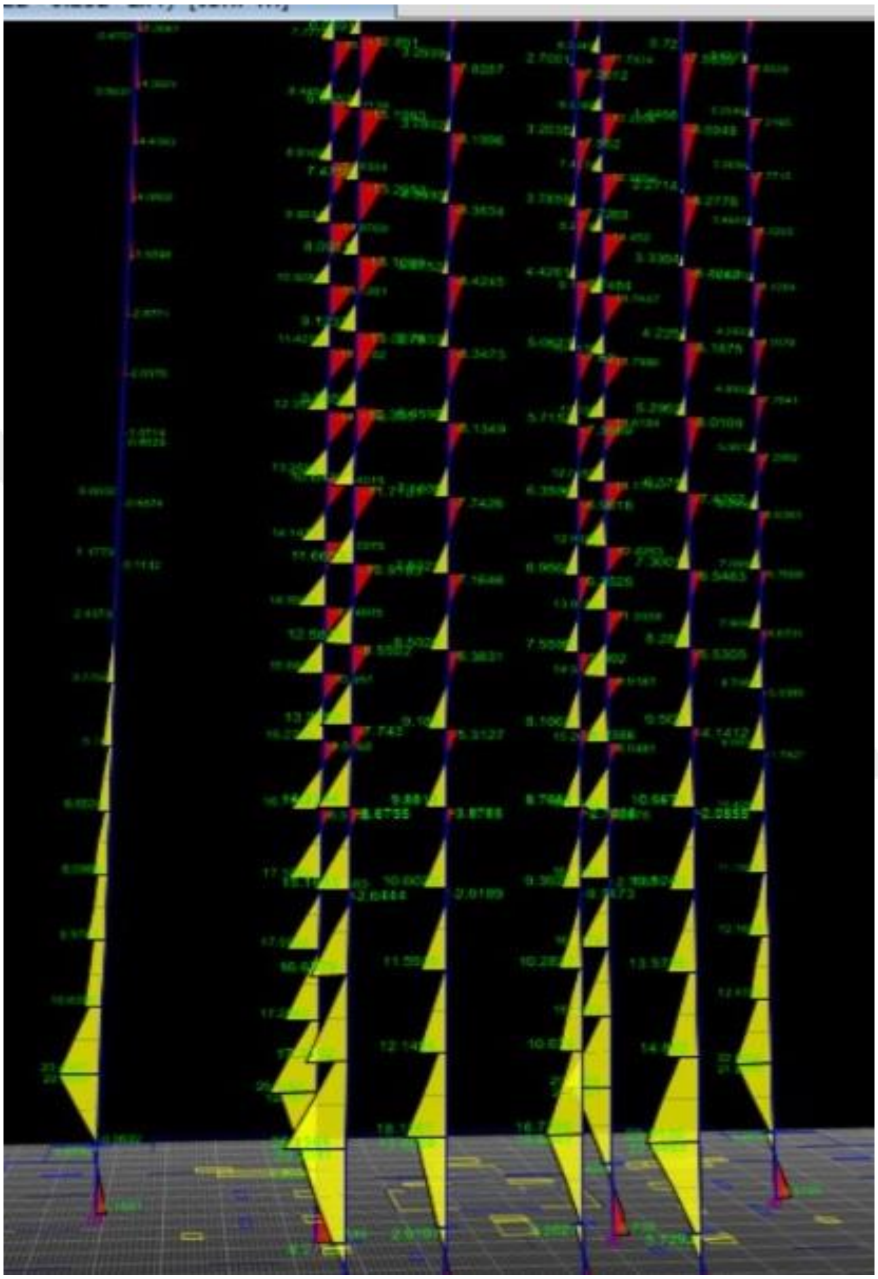
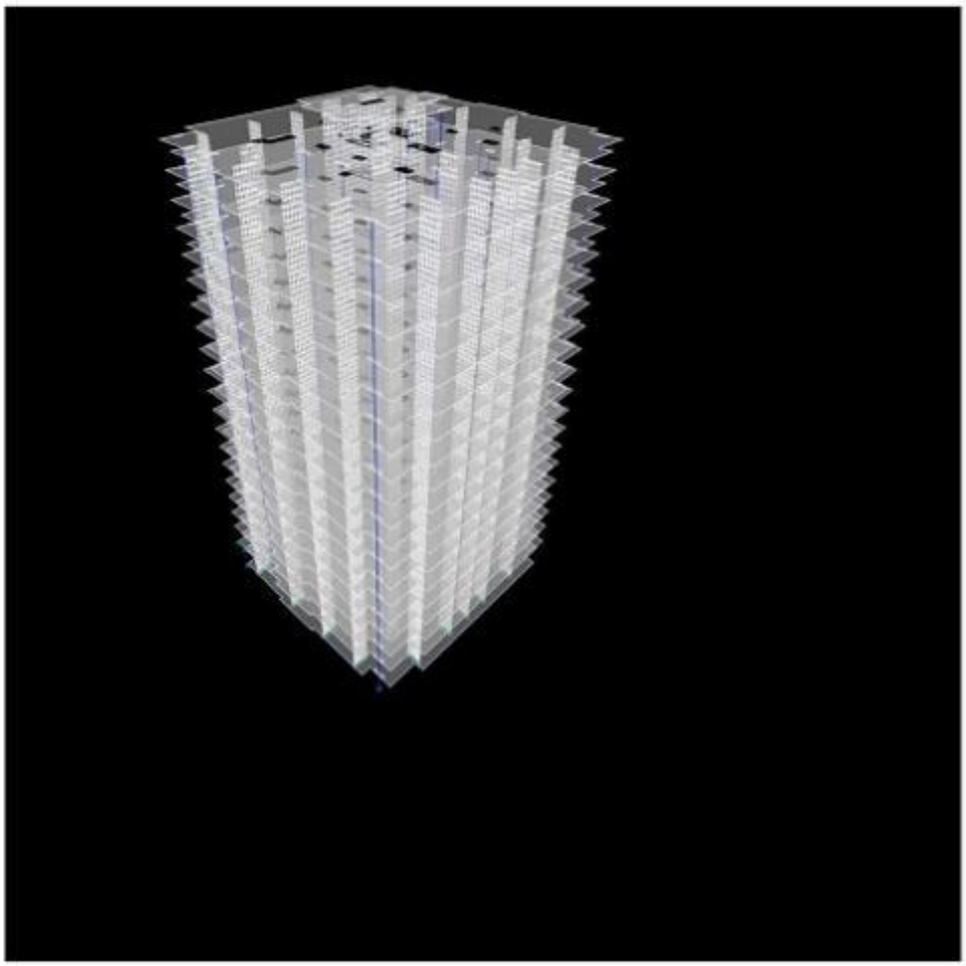


Figure A.5 Bending Moment in Building all columns Iraqi seismic code (ISC 2019)

A.1.1 Summary Analysis Report:

ETABS® version 18



Summary Report

1 Structure Data

This chapter provides model geometry information, including items such as story levels, point coordinates, and element connectivity.

1.1 Story Data

Table 1.1 - Story Definitions

Tower	Name	Height m	Master Story	Similar To	Splice Story	Splice Height m	Color
T1	Story19	3.2	Yes	None	No		Green
T1	Story18	3.2	No	Story1	No		Green
T1	Story17	3.2	No	Story1	No		Green
T1	Story16	3.2	No	Story1	No		Green
T1	Story15	3.2	No	Story1	No		Green
T1	Story14	3.2	No	Story1	No		Green
T1	Story13	3.2	No	Story1	No		Green
T1	Story12	3.2	No	Story1	No		Green
T1	Story11	3.2	No	Story1	No		Green
T1	Story10	3.2	No	Story1	No		Green
T1	Story9	3.2	No	Story1	No		Green
T1	Story8	3.2	No	Story1	No		Green
T1	Story7	3.2	No	Story1	No		Green
T1	Story6	3.2	No	Story1	No		Green
T1	Story5	3.2	No	Story1	No		Green
T1	Story4	3.2	No	Story1	No		Green
T1	Story3	3.2	No	Story1	No		Green
T1	Story2	3.2	No	Story1	No		Green
T1	Story1	3.2	Yes	None	No		Cyan
T1	Ground	3.4	Yes	None	No		Red
T1	Basement	3.2	Yes	None	No		Magenta

2 Loads

This chapter provides loading information as applied to the model.

2.1 Load Patterns

Table 2.1 - Load Pattern Definitions

Name	Is Auto Load	Type	Self Weight Multiplier	Auto Load
~DYNxECC	Yes	Other	0	
~DYNyECC	Yes	Other	0	
~LLRF	Yes	Other	0	
Dead	No	Dead	1	
Ex1	No	Seismic	0	ASCE 7-10
Ex2	No	Seismic	0	ASCE 7-10
Ey1	No	Seismic	0	ASCE 7-10
Ey2	No	Seismic	0	ASCE 7-10
FC	No	Super Dead	0	
Live	No	Live	0	
PLC	No	Super Dead	0	
WALLS	No	Super Dead	0	

Story17	Dead	LinStatic				Top	1612.4471	0	0
Story17	Dead	LinStatic				Bottom	1853.3271	0	0
Story17	Live	LinStatic				Top	527.7507	0	0
Story17	Live	LinStatic				Bottom	527.7507	0	0
Story17	FC	LinStatic				Top	392.1753	0	0
Story17	FC	LinStatic				Bottom	392.1753	0	0
Story17	PLC	LinStatic				Top	39.2175	0	0
Story17	PLC	LinStatic				Bottom	39.2175	0	0
Story17	WALLS	LinStatic				Top	784.3506	0	0
Story17	WALLS	LinStatic				Bottom	784.3506	0	0
Story17	Ex1	LinStatic				Top	0	-110.7485	0
Story17	Ex1	LinStatic				Bottom	0	-110.7485	0
Story17	Ex2	LinStatic				Top	0	-110.7485	0
Story17	Ex2	LinStatic				Bottom	0	-110.7485	0
Story17	Ey1	LinStatic				Top	0	0	-110.7485
Story17	Ey1	LinStatic				Bottom	0	0	-110.7485
Story17	Ey2	LinStatic				Top	0	0	-110.7485
Story17	Ey2	LinStatic				Bottom	0	0	-110.7485
Story17	DYNx	LinRespSpec	Max			Top	0	136.5759	1.7822
Story17	DYNx	LinRespSpec	Max			Bottom	0	136.5759	1.7822
Story17	DYNy	LinRespSpec	Max			Top	0	2.8432	150.3733
Story17	DYNy	LinRespSpec	Max			Bottom	0	2.8432	150.3733

Figure A.6 Example of Story Forces

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (m)	Centroid Y (m)	Length (m)	Thickness (m)	LLRF
Basement	P6	38356.01156	27537.59593	4.2	0.3	0.4

Material Properties

E_c (tonf/m²)	f'_c (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{ys} (tonf/m²)
2619160.17	2800	1	36000	36000

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 m	Left Y_1 m	Right X_2 m	Right Y_2 m	Length m	Thickness m
Top	Leg 1	38366.33156	27536.54593	38366.33156	27538.64593	2.1	0.3
Top	Leg 2	38345.69156	27536.54593	38345.69156	27538.64593	2.1	0.3
Bottom	Leg 1	38366.33156	27536.54593	38366.33156	27538.64593	2.1	0.3
Bottom	Leg 2	38345.69156	27536.54593	38345.69156	27538.64593	2.1	0.3

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (m²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u tonf	M_{u2} tonf-m	M_{u3} tonf-m	Pier A_g m²
Top	0.00315	0.0025	0.0057	U09=1.12D+0.25L-Ey2	1186.312	174.7567	-59.2915	1.26
Bottom	0.00315	0.0025	0.0057	U09=1.12D+0.25L-Ey2	1200.424	175.8483	76.4715	1.26

Shear Design

Station Location	ID	Rebar m²/m	Shear Combo	P_u tonf	M_u tonf-m	V_u tonf	ΦV_c tonf	ΦV_n tonf
Top	Leg 1	0.00075	U07=1.12D+0.25L-Ey1	598.7791	-34.5361	24.9345	49.5156	83.5356
Top	Leg 2	0.00075	U09=1.12D+0.25L-Ey2	601.6404	-32.3909	23.5437	49.5156	83.5356
Bottom	Leg 1	0.00075	U07=1.12D+0.25L-Ey1	605.8351	45.2542	24.9345	49.5156	83.5356
Bottom	Leg 2	0.00075	U09=1.12D+0.25L-Ey2	608.6964	42.949	23.5437	49.5156	83.5356

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (m)	Governing Combo	P_u tonf	M_u tonf-m	Stress Comp tonf/m²	Stress Limit tonf/m²	C Depth m	C Limit m
Top–Left	Leg 1	0.47939	U07=1.12D+0.25L-Ey1	598.7791	-34.5361	1107.07	560	0.68939	0.46667
Top–Right	Leg 1	0.37645	U07=1.12D+0.25L-Ey1	536.8557	-6.2924	823.82	560	0.58645	0.46667
Top–Left	Leg 2	0.47972	U09=1.12D+0.25L-Ey2	601.6404	-32.3909	1101.88	560	0.68972	0.46667
Top–Right	Leg 2	0.37692	U09=1.12D+0.25L-Ey2	538.9974	-6.4192	826.44	560	0.58692	0.46667

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4) (continued)

Station Location	ID	Edge Length (m)	Governing Combo	P_u tonf	M_u tonf-m	Stress Comp tonf/m²	Stress Limit tonf/m²	C Depth m	C Limit m
Bottom–Left	Leg 1	0.37855	U02=1.12D+0.25L+Ex1	543.9117	2	854.28	560	0.58855	0.46667
Bottom–Right	Leg 1	0.50074	U02=1.12D+0.25L+Ex1	605.8351	45.2542	1166.88	560	0.71074	0.46667
Bottom–Left	Leg 2	0.38126	U03=1.12D+0.25L-Ex1	546.0534	2.3136	856.26	560	0.59126	0.46667
Bottom–Right	Leg 2	0.50087	U03=1.12D+0.25L-Ex1	608.6964	42.949	1160.96	560	0.71087	0.46667

ETABS Shear Wall Design

BS 8110-97 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (m)	Centroid Y (m)	Length (m)	Thickness (m)	LLRF
Basement	P9	38356.01156	27537.59593	4.2	0.3	0.4

Material Properties

E_c (tonf/m²)	f_{cu} (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{ys} (tonf/m²)
2619160.17	2800	1	36000	36000

Design Code Parameters

γ_C	γ_S	γ_M	IP_{MAX}	IP_{MIN}	P_{MAX}
1.5	1.15	1.25	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 m	Left Y_1 m	Right X_2 m	Right Y_2 m	Length m	Thickness m
Top	Leg 1	38359.86156	27538.22593	38359.86156	27538.64593	0.42	0.3
Top	Leg 2	38359.86156	27537.80593	38359.86156	27538.22593	0.42	0.3
Top	Leg 3	38359.86156	27537.38593	38359.86156	27537.80593	0.42	0.3
Top	Leg 4	38359.86156	27536.96593	38359.86156	27537.38593	0.42	0.3
Top	Leg 5	38359.86156	27536.54593	38359.86156	27536.96593	0.42	0.3
Top	Leg 6	38352.16156	27538.22593	38352.16156	27538.64593	0.42	0.3
Top	Leg 7	38352.16156	27537.80593	38352.16156	27538.22593	0.42	0.3
Top	Leg 8	38352.16156	27537.38593	38352.16156	27537.80593	0.42	0.3
Top	Leg 9	38352.16156	27536.96593	38352.16156	27537.38593	0.42	0.3
Top	Leg 10	38352.16156	27536.54593	38352.16156	27536.96593	0.42	0.3
Bottom	Leg 1	38359.86156	27538.22593	38359.86156	27538.64593	0.42	0.3
Bottom	Leg 2	38359.86156	27537.80593	38359.86156	27538.22593	0.42	0.3
Bottom	Leg 3	38359.86156	27537.38593	38359.86156	27537.80593	0.42	0.3
Bottom	Leg 4	38359.86156	27536.96593	38359.86156	27537.38593	0.42	0.3
Bottom	Leg 5	38359.86156	27536.54593	38359.86156	27536.96593	0.42	0.3
Bottom	Leg 6	38352.16156	27538.22593	38352.16156	27538.64593	0.42	0.3
Bottom	Leg 7	38352.16156	27537.80593	38352.16156	27538.22593	0.42	0.3
Bottom	Leg 8	38352.16156	27537.38593	38352.16156	27537.80593	0.42	0.3
Bottom	Leg 9	38352.16156	27536.96593	38352.16156	27537.38593	0.42	0.3
Bottom	Leg 10	38352.16156	27536.54593	38352.16156	27536.96593	0.42	0.3

Flexural Design for N, M_2 and M_3

Station Location	Required Rebar Area (m²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	N tonf	M_2 tonf-m	M_3 tonf-m	Pier A_g m²
Top	0.00315	0.0025	0.0096	U09=1.12D+0.25L-Ey2	1161.3837	-22.9587	-98.697	1.26
Bottom	0.00315	0.0025	0.0096	U09=1.12D+0.25L-Ey2	1175.4957	-21.2651	140.1802	1.26

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (m)	Centroid Y (m)	Length (m)	Thickness (m)	LLRF
Story18	P10	38356.01156	27537.59593	2.1	0.3	0.896

Material Properties

E_c (tonf/m²)	f'_c (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{ys} (tonf/m²)
2619160.17	2800	1	36000	36000

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 m	Left Y_1 m	Right X_2 m	Right Y_2 m	Length m	Thickness m
Top	Leg 1	38356.01156	27536.54593	38356.01156	27538.64593	2.1	0.3
Bottom	Leg 1	38356.01156	27536.54593	38356.01156	27538.64593	2.1	0.3

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (m²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u tonf	M_{u2} tonf-m	M_{u3} tonf-m	Pier A_g m²
Top	0.001575	0.0025	0.0025	U09=1.12D+0.25L-Ey2	17.2785	0.7879	-1.0779	0.63
Bottom	0.001575	0.0025	0.0025	U09=1.12D+0.25L-Ey2	22.9233	-0.5882	10.7819	0.63

Shear Design

Station Location	ID	Rebar m²/m	Shear Combo	P_u tonf	M_u tonf-m	V_u tonf	ΦV_c tonf	ΦV_n tonf
Top	Leg 1	0.00075	U01=1.2D+1.6L	22.4	-11.8426	5.5901	58.1583	92.1783
Bottom	Leg 1	0.00075	U01=1.2D+1.6L	28.448	8.2456	5.5901	59.6129	93.6329

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (m)	Governing Combo	P_u tonf	M_u tonf-m	Stress Comp tonf/m²	Stress Limit tonf/m²	C Depth m	C Limit m
Top–Left	Leg 1	Not Required	U07=1.12D+0.25L-Ey1	17.2798	-1.0738	32.3	560		
Top–Right	Leg 1	Not Stressed	U07=1.12D+0.25L-Ey1	0	0	0	0		
Bottom–Left	Leg 1	Not Required	U08=1.12D+0.25L+Ey2	17.9991	-2.1379	38.27	560		
Bottom–Right	Leg 1	Not Required	U08=1.12D+0.25L+Ey2	22.9248	10.7626	85.2	560		

ETABS Shear Wall Design

ACI 318-14 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (m)	Centroid Y (m)	Length (m)	Thickness (m)	LLRF
Basement	P10	38356.01156	27537.59593	2.1	0.3	0.4

Material Properties

E_c (tonf/m²)	f'_c (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{ys} (tonf/m²)
2619160.17	2800	1	36000	36000

Design Code Parameters

Φ_T	Φ_C	Φ_V	Φ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
0.9	0.65	0.75	0.6	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 m	Left Y_1 m	Right X_2 m	Right Y_2 m	Length m	Thickness m
Top	Leg 1	38356.01156	27536.54593	38356.01156	27538.64593	2.1	0.3
Bottom	Leg 1	38356.01156	27536.54593	38356.01156	27538.64593	2.1	0.3

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (m²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P_u tonf	M_{u2} tonf-m	M_{u3} tonf-m	Pier A_g m²
Top	0.001575	0.0025	0.0025	U09=1.12D+0.25L-Ey2	476.2417	-0.2275	-25.2454	0.63
Bottom	0.001575	0.0025	0.0025	U09=1.12D+0.25L-Ey2	483.2977	0.2843	36.7813	0.63

Shear Design

Station Location	ID	Rebar m²/m	Shear Combo	P_u tonf	M_u tonf-m	V_u tonf	ΦV_c tonf	ΦV_n tonf
Top	Leg 1	0.00075	U07=1.12D+0.25L-Ey1	476.2729	-25.3863	19.4797	49.5156	83.5356
Bottom	Leg 1	0.00075	U07=1.12D+0.25L-Ey1	483.3289	36.9488	19.4797	49.5156	83.5356

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (m)	Governing Combo	P_u tonf	M_u tonf-m	Stress Comp tonf/m²	Stress Limit tonf/m²	C Depth m	C Limit m
Top–Left	Leg 1	0.87727	U07=1.12D+0.25L-Ey1	476.2729	-25.3863	871.12	560	1.08727	0.46667
Top–Right	Leg 1	0.60342	U07=1.12D+0.25L-Ey1	417.8203	-2.8459	650.3	560	0.81342	0.46667
Bottom–Left	Leg 1	0.63615	U05=1.12D+0.25L-Ex2	424.8763	1.612	666.19	560	0.84615	0.46667
Bottom–Right	Leg 1	0.91199	U05=1.12D+0.25L-Ex2	483.3289	36.9488	934.76	560	1.12199	0.46667

ETABS Shear Wall Design

BS 8110-97 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (m)	Centroid Y (m)	Length (m)	Thickness (m)	LLRF
Story18	P9	38356.01156	27537.59593	4.2	0.3	0.673

Material Properties

E_c (tonf/m²)	f_{cu} (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{ys} (tonf/m²)
2619160.17	2800	1	36000	36000

Design Code Parameters

γ_C	γ_S	γ_M	IP_{MAX}	IP_{MIN}	P_{MAX}
1.5	1.15	1.25	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 m	Left Y_1 m	Right X_2 m	Right Y_2 m	Length m	Thickness m
Top	Leg 1	38359.86156	27538.22593	38359.86156	27538.64593	0.42	0.3
Top	Leg 2	38359.86156	27537.80593	38359.86156	27538.22593	0.42	0.3
Top	Leg 3	38359.86156	27537.38593	38359.86156	27537.80593	0.42	0.3
Top	Leg 4	38359.86156	27536.96593	38359.86156	27537.38593	0.42	0.3
Top	Leg 5	38359.86156	27536.54593	38359.86156	27536.96593	0.42	0.3
Top	Leg 6	38352.16156	27538.22593	38352.16156	27538.64593	0.42	0.3
Top	Leg 7	38352.16156	27537.80593	38352.16156	27538.22593	0.42	0.3
Top	Leg 8	38352.16156	27537.38593	38352.16156	27537.80593	0.42	0.3
Top	Leg 9	38352.16156	27536.96593	38352.16156	27537.38593	0.42	0.3
Top	Leg 10	38352.16156	27536.54593	38352.16156	27536.96593	0.42	0.3
Bottom	Leg 1	38359.86156	27538.22593	38359.86156	27538.64593	0.42	0.3
Bottom	Leg 2	38359.86156	27537.80593	38359.86156	27538.22593	0.42	0.3
Bottom	Leg 3	38359.86156	27537.38593	38359.86156	27537.80593	0.42	0.3
Bottom	Leg 4	38359.86156	27536.96593	38359.86156	27537.38593	0.42	0.3
Bottom	Leg 5	38359.86156	27536.54593	38359.86156	27536.96593	0.42	0.3
Bottom	Leg 6	38352.16156	27538.22593	38352.16156	27538.64593	0.42	0.3
Bottom	Leg 7	38352.16156	27537.80593	38352.16156	27538.22593	0.42	0.3
Bottom	Leg 8	38352.16156	27537.38593	38352.16156	27537.80593	0.42	0.3
Bottom	Leg 9	38352.16156	27536.96593	38352.16156	27537.38593	0.42	0.3
Bottom	Leg 10	38352.16156	27536.54593	38352.16156	27536.96593	0.42	0.3

Flexural Design for N, M_2 and M_3

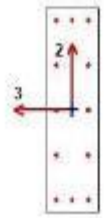
Station Location	Required Rebar Area (m²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	N tonf	M_2 tonf-m	M_3 tonf-m	Pier A_g m²
Top	0.00315	0.0025	0.0096	U09=1.12D+0.25L-Ey2	42.1544	3.4554	-9.4152	1.26
Bottom	0.00315	0.0025	0.0096	U09=1.12D+0.25L-Ey2	53.444	-0.5269	39.6837	1.26

Shear Design

Station Location	ID	Rebar m^2/m	Shear Combo	N tonf	M tonf-m	V tonf	V_c tonf	V[illegible] tonf
Top	Leg 1	0.00039	U08=1.12D+0.25L+Ey1	-10.0047	0.909	1.7282	0	4.1115
Top	Leg 2	0.00039	U08=1.12D+0.25L+Ey1	-2.3611	0.6333	1.4206	2.4927	6.6042
Top	Leg 3	0.00039	U01=1.2D+1.6L	2.7236	0.8225	2.274	4.5715	8.683
Top	Leg 4	0.00039	U01=1.2D+1.6L	7.0658	1.4137	3.7372	5.7876	9.8991
Top	Leg 5	0.00039	U01=1.2D+1.6L	18.1059	1.8968	4.1678	8.0823	12.1937
Top	Leg 6	0.00039	U08=1.12D+0.25L+Ey2	-9.9795	0.8962	1.7067	0	4.1115
Top	Leg 7	0.00039	U08=1.12D+0.25L+Ey2	-2.3861	0.6344	1.4223	2.4781	6.5896
Top	Leg 8	0.00039	U01=1.2D+1.6L	2.7299	0.8302	2.2953	4.5735	8.685
Top	Leg 9	0.00039	U01=1.2D+1.6L	7.1023	1.4157	3.744	5.7917	9.9032
Top	Leg 10	0.00039	U01=1.2D+1.6L	18.1419	1.9011	4.1766	8.0887	12.2002
Bottom	Leg 1	0.00039	U08=1.12D+0.25L+Ey1	2.1138	0.7251	1.8997	4.3746	8.4861
Bottom	Leg 2	0.00039	U01=1.2D+1.6L	8.2237	0.6354	1.971	6.0649	10.1764
Bottom	Leg 3	0.00039	U01=1.2D+1.6L	8.096	0.7933	2.4465	6.0344	10.1459
Bottom	Leg 4	0.00039	U01=1.2D+1.6L	6.5365	1.0911	3.0782	5.6489	9.7604
Bottom	Leg 5	0.00039	U01=1.2D+1.6L	-0.3092	1.3135	2.9223	3.4714	7.5829
Bottom	Leg 6	0.00039	U08=1.12D+0.25L+Ey2	2.087	0.7145	1.8779	4.3658	8.4773
Bottom	Leg 7	0.00039	U01=1.2D+1.6L	8.2203	0.6478	2.0035	6.0641	10.1758
Bottom	Leg 8	0.00039	U01=1.2D+1.6L	8.0588	0.8009	2.4692	6.0255	10.137
Bottom	Leg 9	0.00039	U01=1.2D+1.6L	6.5026	1.0938	3.0886	5.6402	9.7517
Bottom	Leg 10	0.00039	U01=1.2D+1.6L	-0.3545	1.319	2.9352	3.451	7.5625

ETABS Concrete Frame Design

ACI 318-14 Column Section Design



Column Element Details (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (m)	LLRF	Type
Basement	C14	54	C30*100	U03=1.12D+0.25L-Ey2	3.2	3.2	0.4	Sway Special

Section Properties

b (m)	h (m)	dc (m)	Cover (Torsion) (m)
0.3	1	0.06	0.0273

Material Properties

E_c (tonf/m²)	f'_c (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{ys} (tonf/m²)
2519160.17	2800	1	36000	36000

Design Code Parameters

ϕ_T	ϕ_{CTied}	$\phi_{CSpiral}$	ϕ_{Vns}	ϕ_{Vs}	ϕ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u tonf	Design M_{u2} tonf-m	Design M_{u3} tonf-m	Minimum M2 tonf-m	Minimum M3 tonf-m	Rebar Area m²	Rebar % %
179.6667	-4.3551	1.4538	4.3551	8.1281	0.003	1

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Effective Length m
Major Bend(M3)	0.51967	1	1	1	3.2
Minor Bend(M2)	0.225833	1	1	1	3.2

Shear Design for V_{u2} , V_{u3}

	Shear V_u tonf	Shear ϕV_c tonf	Shear ϕV_s tonf	Shear ϕV_p tonf	Rebar A_v /s m^2/m
Major, V_{u2}	0.5454	21.4093	0	0	0
Minor, V_{u3}	0.6503	18.2208	0	0	0

Joint Shear Check/Design

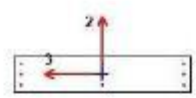
	Joint Shear Force tonf	Shear $V_{u,Top}$ tonf	Shear $V_{u,Tot}$ tonf	Shear ϕV_c tonf	Joint Area m^2	Shear Ratio Unitless
Major Shear, V_{u2}	N/A	N/A	N/A	N/A	N/A	N/A
Minor Shear, V_{u3}	N/A	N/A	N/A	N/A	N/A	N/A

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/A	N/A

ETABS Concrete Frame Design

BS 8110-97 Column Section Design



Column Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (m)	LLRF
Basement	C9	36	FSec4	U01-1.2D+1.6L	3.2	3.2	0.4

Section Properties

b (m)	h (m)	dc (m)	Cover (Torsion) (m)
1.5	0.3	0.06	0.03

Material Properties

E_c (tonf/m²)	f_{cu} (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{yv} (tonf/m²)
2619160.17	2800	1	36000	36000

Design Code Parameters

E_c (tonf/m²)	f_{cu} (tonf/m²)	Lt.Wt Factor (Unitless)
2619160.17	2800	1

Axial Force and Biaxial Moment Design For N , M_2 , M_3

Design N tonf	Design M_2 tonf-m	Design M_3 tonf-m	Minimum M2 tonf-m	Minimum M3 tonf-m	Rebar Area m²	Rebar % %
403.0255	-9.7765	13.9267	9.7765	7.2324	0.002016	0.45

Axial Force and Biaxial Moment Factors

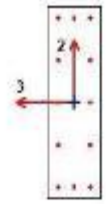
	M_i Moment tonf-m	M_{add} Moment tonf-m	β Factor Unitless	Length m
Major Bend(M3)	2.2343	8.3426	1	3.2
Minor Bend(M2)	6.8765	-1.6585	1	3.2

Shear Design for V_2 , V_3

	Shear V tonf	Shear V_c / γ_M tonf	Shear V_s / γ_M tonf	Rebar A_v /s m^2/m
Major, V_2	2.617	59.0351	14.684	0.00195
Minor, V_3	0.9101	94.6574	17.6207	0.00039

ETABS Concrete Frame Design

ACI 318-14 Column Section Design



Column Element Details (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (m)	LLRF	Type
Story18	C14	815	C30*100	UD9-1.12D+0.25L-Ey2	3.2	3.2	0.654	Sway Special

Section Properties

b (m)	h (m)	dc (m)	Cover (Torsion) (m)
0.3	1	0.06	0.0273

Material Properties

E_c (tonf/m²)	f'_c (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{ys} (tonf/m²)
2619160.17	2800	1	36000	36000

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}	Ω_0
0.9	0.65	0.75	0.75	0.6	0.85	2

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u tonf	Design M_{u2} tonf-m	Design M_{u3} tonf-m	Minimum M2 tonf-m	Minimum M3 tonf-m	Rebar Area m²	Rebar % %
0.0959	-2.3511	5.1342	0.1962	0.3863	0.003	1

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Effective Length m
Major Bend(M3)	0.33074	1	1	1	3.2
Minor Bend(M2)	0.241038	1	1	1	3.2

Shear Design for V_{u2}, V_{u3}

	Shear V_u tonf	Shear ϕV_c tonf	Shear ϕV_s tonf	Shear ϕV_p tonf	Rebar A_v/s m²/m
Major, V_{u2}	2.6844	15.3028	0	0	0
Minor, V_{u3}	1.394	0	1.394	0	0.00027

Joint Shear Check/Design

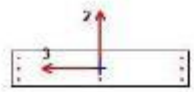
	Joint Shear Force tonf	Shear $V_{u,Top}$ tonf	Shear $V_{u,Tot}$ tonf	Shear ϕV_c tonf	Joint Area m²	Shear Ratio Unitless
Major Shear, V_{u2}	N/A	N/A	N/A	N/A	N/A	N/A
Minor Shear, V_{u3}	N/A	N/A	N/A	N/A	N/A	N/A

(6/5) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/A	N/A

ETABS Concrete Frame Design

BS 8110-97 Column Section Design



Column Element Details

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (m)	LLRF
Story18	C9	227	FSec4	U01=1.2D+1.6L	2.2	3.2	0.734

Section Properties

b (m)	h (m)	dc (m)	Cover (Torsion) (m)
1.5	0.3	0.06	0.03

Material Properties

E_c (tonf/m²)	f_{cu} (tonf/m²)	Lt.Wt Factor (Unitless)	f_y (tonf/m²)	f_{ys} (tonf/m²)
2619160.17	2800	1	35000	35000

Design Code Parameters

E_c (tonf/m²)	f_{cu} (tonf/m²)	Lt.Wt Factor (Unitless)
2619160.17	2800	1

Axial Force and Biaxial Moment Design For N , M_2 , M_3

Design N tonf	Design M_2 tonf-m	Design M_3 tonf-m	Minimum M2 tonf-m	Minimum M3 tonf-m	Rebar Area m²	Rebar % %
21.3308	9.5116	18.0368	0.4278	0.3209	0.005281	1.17

Axial Force and Biaxial Moment Factors

	M_i Moment tonf-m	M_{add} Moment tonf-m	β Factor Unitless	Length m
Major Bend(M3)	7.0807	0.8651	1	3.2
Minor Bend(M2)	3.7754	3.073	1	3.2

Shear Design for V_2 , V_3

	Shear V tonf	Shear V_c / γ_M tonf	Shear V_s / γ_M tonf	Rebar A_{sv} /s m²/m
Major, V_2	9.3514	26.5631	14.684	0.00195
Minor, V_3	4.9251	33.4033	17.6207	0.00039