Comparison of the Provisions of ACI318-19 Code and Eurocode on the Structural Design and Cost Analysis, of a High-rise Concrete Building Subjected to Seismic & Wind Forces

مقارنة بين بنود الكود الأمريكي ACI 318-19 و الكود الأوروبي Eurocode بشأن التصميم الإنشائي و تحليل التكلفة ، لمبنى خرساني شاهق يخضع لقوى الزلازل و الرياح

by

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of the requirements for the degree of

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Abstract

Engineers and scientists have developed many codes based on multiple experiments and analysis that assist them in making efficient design of safe and economical structures. These codes consist of many divisions, the significant and substantial divisions of these codes are related to the seismic and wind effects on the structure. Buildings codes are sets of specified principles and standards that serve as guidelines in structural design. Some of the famous design codes used nowadays are American Society of Civil Engineers Code (ASCE), American Concrete Institute (ACI), Eurocode (EC), International Building Code (IBC), Uniform building code (UBC) and British Standards code (BS).

This work aims to compare two codes; ACI 318-19 and Eurocode in terms of lateral effect on high-rise building, focusing on the seismic and wind provisions, and since these two forces effect the structural elements geometry, thus the cost comparison was included, by comparing the amount of reinforcement used. Since both of the codes have different standards and factors, therefor it is expected that there will be some differences in the structural design. This research will contain design and analyses of a high-rise building consisting of 50 storeys reinforced concrete structure and comparison of several provisions. This research study will investigate the difference between seismic and wind results between the two codes and if there might be any differences in the structural elements reinforcement amount, which will affect the cost of the building.

In order to illustrate a real and accurate comparison, both the designs have been compared in similar circumstances; both of them were assumed to be designed in Dubai, United Arab Emirates. It can infer from this comparison which design code is better to be used in Dubai specifically and in the United Arab Emirates generally. For the cost comparison it will include columns, shear walls and slabs. The analyses have been performed using ETABS software and Ram Concept software. The ETABS software will be used for the superstructure design and analyses while the Ram Concept will be used for the slabs design and analyses.

This study concluded that many seismic provisions data for both codes were close to each other such as time period and story drift, while few provisions were giving results far from each other such as P delta and response spectra. For the wind provision, the results were far relatively from another. For the cost comparison, results showed that the Eurocode is giving more economical design for column, shear walls and slabs.

ملخص البحث

بناءً على العديد من الإختبارات و التحاليل التي قام بها المهندسون و العلماء تم ابتكار العديد من قوانين البناء (المعايير) التي تساعدهم في التصميم الفعال لمبنى سليم و اقتصادي. تتضمن هذه القوانين العديد من الأقسام، و يعد قسم تأثيرات الزلازل و الرياح على المبنى من أهمها و أكثرها جوهرية. قوانين البناء هي مجموعة من المبادئ والمعايير المحددة التي تعمل كدليل إرشادي في التصميم الإنشائي. بعض رموز التصميم الإنشائي الشهيرة هي معيار معهد الخرسانة الأمريكي، معيار الجمعية الأمريكية للمهندسين المدنيين، المعيار الأوروبي، معيار البناء الموحد، معيار البناء الدولي و معيار المعايير البريطانية .

يتم تنفيذ هذا البحث لمقارنة الكودين الأمريكي ACI 318-19 و الأوروبي Eurocode من حيث تصميم القوة الجانبية المؤثرة على المبنى، مركزةً على تأثيرات الزلازل و الرياح وبما أن هاتين القوتين تؤثران على هندسة العناصر الهيكلية ، فقد تم تضمين مقارنة التكلفة من خلال مقارنة كمية الحديد المستخدمة. نظرًا لأن كلا المعيارين لهما معايير وعوامل مختلفة ، فمن الأرجح أن يكون هنالك بعض الاختلاف في التصميم الهيكلي. هذا البحث سيحتوي على تصميم و تحليل لمبنى شاهق الارتفاع مكون من ٥٠ طابقًا من الخرسانة المسلحة لمقارنة بعض من البنود. هذه الدراسة البحثية سوف تحقق من وجود أي اختلافات في نتائج الزلازل و الرياح بين الكودين و كذلك فيما إذا كان هنالك أي اختلاف في كمية الحديد التي سيتم إستخدامها في كلا التصميمين ، مما سيؤثر على التكلفة الإجمالية للمبنى.

للتأكد بأن المقارنة حقيقية و دقيقة تمت مقارنة كلا التصميمين في ظروف مماثلة ، حيث تم افتراض أن كلا التصميمين موجودان في دبي، دولة الإمارات العربية المتحدة. ويمكن أن يستنتج من هذه المقارنة أي من المعيارين يقود إلى التصميم الأفضل للإستخدام في دبي بشكل خاص وفي الإمارات العربية المتحدة بشكل عام. بالنسبة لمقارنة التكلفة ، ستشمل الأعمدة وجدران القص والألواح. لقد تم إجراء التحليل باستخدام برنامج الإيتابس و برنامج الرام كونسبت. سوف يتم استعمال برنامج الإيتابس لتصميم و تحليل البنية الفوقية، بينما سوف يتم إستخدام برنامج بنتلي لتصميم و تحليل الألواح .

استنتجت هذه الدراسة إلى أن العديد من بيانات الأحكام الزلزالية لكلا المعياريين كانت قريبة من بعضها البعض مثل الفترة الزمنية وانحراف الطوابق ، في حين أن بعض الأحكام كانت تعطي نتائج بعيدة عن بعضها البعض مثل P delta وresponse spectrum . بالنسبة لتقييم تأثير الرياح ، كانت النتائج بعيدة نسبيًا بين المعياريين. بالنسبة لمقارنة التكلفة ، أظهرت النتائج أن Eurocode يعطي تصميمًا أكثر اقتصادا للأعمدة وجدران القص والألواح.

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Symbols

Eurocode 1

q_k	Uniform distribution or liner load characteristic magnitude
α_A	Factor of reduction
α_n	Factor of reduction
φ	Dynamic magnification factor
ψ_0	Factor for combination value of a variable action

Eurocode 2

A_p	Area of a prestressing tendon or tendons
A_c	Area of the concrete's cross section
A_s	Area of the reinforcement's cross section
E_{cm}	Secant concrete's elasticity modulus
V	Shear force
e	Eccentricity
f_c	Compressive strength of concrete
f_{cm}	Mean value of concrete cylinder compressive strength
f_y	Yield strength of reinforcement
h	Height
t	Thickness
γ	Partial factor
μ	Tendons and ducts coefficient of friction
ν	Poisson's ratio

ψ Variable actions values factors:

- ψ_0 for combination values
- ψ_1 for frequent values
- ψ_2 for quasi-permanent values

Eurocode 8

S Soil factor

T System linear single degree of freedom vibration period

$v_{s,30}$ Average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear strain of 10^{-5} or less

γ_1 Importance factor

η Damping correction factor

ξ Viscous damping ratio (in percent)

$\psi_{2,i}$ Combination coefficient for the quasi-pern1anent value of a variable action i

$\psi_{E,i}$ Combination coefficient for a variable action i, to be used when determining the effects of the design seismic action

F_i Horizontal seismic force at storey i

F_b Base shear force

H Building height from the foundation or from the top of a rigid basement

T_1 Fundamental period of vibration of a building

d_r Design inter-storey drift

h Inter-storey height

θ	Inter-storey drift sensitivity coefficient
γ_C	Partial factor for concrete
γ_S	Partial factor for steel

ACI 318-19

E_c	Modulus of elasticity of concrete, psi
E_{cs}	Modulus of elasticity of slab concrete, psi
F_{ci}'	Specified compressive strength of concrete, psi
h	Overall thickness, height, or depth of member, in.
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.^{4}
H	Effect of service load due to lateral earth pressure ground water pressure, or pressure of bulk materials, Ib
W	Wind load effect
ϕ	Strength reduction factor
Ω_0	Amplification factor to account for overstrength of the seismic-force- resisting system determined in accordance with the general building code

ASCE 7-16

D	Dead load
Fx	A minimum design lateral force applied to level x of the structure and used for purposes of evaluating structural integrity
L	Live load.
E	Earthquake load

H	Load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
W	Wind load
C_d	Deflection amplification factor
F_a	Short-period site coefficient (at 0.2-s period)
F_i, F_n, F_x	Portion of the seismic base shear, V, induced at level i, n, or x, respectively
F_v	Long-period site coefficient (at 1.0-s period)
F_c’	Specified compressive strength of concrete used in design
f_y	Specified yield strength of reinforcement [psi (MPa)]
S_1	Mapped MCER, spectral response acceleration parameter, with 5% damping at a period of 1 second
S_{D1}	Design spectral response acceleration parameter, with 5% damping at a period of 1 second
S_{DS}	Design spectral response acceleration parameter, with 5% damping at short periods
S_{M1}	The MCER, spectral response acceleration parameter, with 5% damping at a period of 1 second
S_{MS}	The MCER, 5% damped, spectral response acceleration parameter at short periods adjusted for site class effects
S_S	Mapped MCER, 5% damped, spectral response acceleration parameter at short period
T	The fundamental period of the building
T_L	Long -period transition period
V	Total design lateral force or shear at the base or basic wind speed

Δ	Design storey drift
Δ_a	Allowable storey drift
δ_{max}	Maximum displacement at level x
θ	Stability coefficient for P-delta effects
Ω_0	Overstrength factor
R	The response modification factor
I_e	The Importance Factor
C_s	The seismic response coefficient
C_{vx}	Vertical distribution factor
h_{sx}	Storey height below level x [in. (mm)]
K_z	Velocity pressure exposure coefficient
K_{zt}	Topographic factor
K_d	Wind directionality factor
q_z	Velocity pressure at height z
P_{LX}, P_{LY}	Leeward face design pressure acting in the x, y principal axis, respectively
e (e_X; e_Y)	Eccentricity for the x, y principal axis of the structure, respectively

CHAPTER 1: INTRODUCTION

1.1. Background

As a structural engineer it is required to make a safe and adequate design for any structure, but as a professional and highly knowledgeable engineer, it is required to fully understand the various types of existing codes and to know which one is the best to use under any given circumstances. In the structural design there are two major requirements that must be fulfilled, which are the people's safety and the economic cost of the structure, both of these requirements are highly dependent on the seismic and wind behaviors. Since a low seismic zone area will not require a high concrete strength, larger cross-sections and amount of reinforcement similar to an area with a high seismic zone. Similar cases for the wind effects, a location with high wind force will most likely require a stronger building. Such loads, factors and conditions will affect the cost of the building.

This research will focus on lateral forces effect and economical comparison between two popular and frequently used codes, the ACI 318-19 and the Eurocode. Since the provisions of these codes are different, it would be very beneficial to compare them and to know the main difference between them. Since both designs will be simulated to be in surly safe, the economic aspect was considered as comparison parameter.

There are multiple researches that have been conducted regarding codes comparison, starting with Nandi and Guha who made a comparison for a reinforced concrete structure between IS456:2000, the Eurocode 2:1992 and BS8110:1985 codes, where they checked which code would be more economical based on the comparison done and the differences found within the structure's final design. Their conclusion was that the calculated steel area for slabs requirement is more in IS code compared to BS and Eurocode; for beams the steel area required is more in Eurocode compared to IS and BS codes and for columns the steel area required is more in BS compared to IS and Eurocode. (Nandi & Guha 2014)

Also, a comparative study was conducted by Dhanvijay, Telang and Nair to compare some of the international design codes including the Indian code IS 1893:2002, Eurocode and the American code (ASCE/IBC). The structural model was a G+10 special reinforced concrete moments-resisting frames, the building system was used in order to resist earthquakes. The columns and the beams were detailed in a way that enable them to resist axial, shear and flexural actions when the structure is being swayed in multiple displacement cycles during an earthquake. The analysis was made on STAAD-Pro software. This study main objective was to bring out the contributing factors which lead into poor performance of the structure during an earthquake and make recommendations on how to make structure perform better. To do so, they conducted a comparative analysis of displacement, base shear, axial load and moment at x and z directions for some selected columns, shear in y direction and moment in z direction for some selected beams. The conclusion for the columns analysis was that both the Indian code and the American code had very close results in both x and z directions while the Eurocode showed highly different result, while for displacements and moments in y and z, the obtained results were relatively different than each code. For beams only the moment in z direction for the American code and the Eurocode was close while for base shear, shear in y direction and torsion were all relatively different. (Dhanvijay, Telang & Nair 2015)

The main conclusion from previous mentioned studies is that the codes are differ from each other in many provisions, which make the comparison field huge.

This research will conduct a comparison of the seismic and wind analyses for different provisions between the ACI 318-19 code and the Eurocode, and by performing this analysis the final structural elements design will be compared in terms of the amount of reinforcement used. The study comparison for seismic assessment provisions will contain base Shear, storey shear, storey drift, torsional irregularity, P-Delta effect, response spectrum, mass participation and structural time period. While for the wind assessment provision comparison will consist of the storey drift due to wind effect. This study will be performed on a high-rise fifty storey building that is assumed to be built in Dubai, United Arab Emirates.

1.2. Problem Statement

The planning of a structural design in any process doesn't only relay on conceptual thoughts and basic calculations, it also important to fulfill the other sides of this complicated equation by applying the design codes, as well as following the country's design criteria and to make deep analysis of grounding and the surrounding environment. The codes are continuously being updated for seismic and wind design, and each code has its unique design criteria and specifications that should be followed for loads analysis. The two codes chosen for this study comparative analysis (ACI 318-19 and Eurocode) are unique and used widely worldwide. The main goal is to know what are the differences in ratio for seismic and wind provisions between these two codes and which one could possibly be more conservative. The seismic and wind forces are the main forces which have a great impact on the sizing's of the structural elements, and usage of shear walls, thus if one of the compared codes is giving less seismic force or wind force value then most likely the structural elements sizing and the amount of materials used in construction such as concrete and steel could be less. Such information is crucial and important when designing for a high-rise building, since a reduction in materials amount will have an impact on the total building cost.

The analysis will be assumed to be conducted in Dubai, United Arab Emirates. This country doesn't have its own design code, and though it's using other codes that are considered to be efficient; the most used codes in the country are the ACI 318 and ASCE 7 codes, while other codes such as Eurocode and BS are used less frequently. This dissertation will give a good insight for such country to either continue using the ACI code or to move with using other codes such as the Eurocode. The Eurocode is one of the highly used codes in many countries around the world since its known to be highly conservative based on many engineer's experience, and by putting these two codes into comparison, it will make it clearer which code would be more conservative for countries similar to UAE conditions.

The comparison will include the most critical seismic and wind assessment provisions effecting the building. Moreover, the chosen provisions are being checked by municipalities in the UAE, which makes this dissertation a useful reference of application.

1.3. Research Questions

The main purpose of this research is to find answers for the following questions:

1) What are the main differences of the selected seismic assessment provisions between the two codes? and is that difference ratio high?
2) What are the main differences of the selected wind assessment provisions between the two codes? and is that difference ratio high?
3) What could be the elements sizing's differences between them?
4) Which code could possibly give more economical design for the superstructure design?
5) To know which code is the best one to be used in locations such as Dubai, UAE for high-rise buildings?

1.4. Study Objectives

The main objectives of this study are:

1) Perform seismic design analysis comparison for significant provisions between ACI318-19 code and Eurocode and to check the main difference between them.
2) Perform wind design analysis comparison for significant provisions between ACI318-19 code and Eurocode and to check the main difference between them.
3) Check which code is more conservative by comparing the reinforcement ratio for the structural elements.
4) Recommend modifications or additional considerations when using ACI 318-19 code and Eurocode in regions similar to Dubai.
5) Check which of the codes is more suited to be used in the UAE since it does not have its own code.

1.5. Significance and Motivation of the study

This research investigates which of the two codes would provide the best design for a fifty-storey building under similar conditions. Some studies have made a comparison between these codes but most of them have focused on steel structures or on low-rise reinforced concrete structures, while just few studies have focused on comparing these two codes for high-rise buildings. In this research the results obtained will show which of the two codes is preferable in several parameters, moreover the findings will be compared with the previous studies that have been performed regarding the mentioned assessment provision.

Countries such as the United Arab Emirates have been using the ACI 318 code and ASCE 7-16 code in designing most of their buildings while they rarely used other codes such as Eurocode. Based on previous code's comparisons and many engineers experience in the design field, it has been stated that that Eurocode could give more conservative design which made it a great research topic to investigate this statement, and if this happened to be true, it will save enormous amount of money for a country like UAE which has cities such as Dubai that is full of high-rise buildings construction.

Also, it's essential to highlight that if the difference between the materials amount is considerably high, then the code that gives less structural materials could reduce the issue of raw materials depletion. It's important to raise the awareness to conserve those precious materials. From structural design point of view, this can be achieved by designing a building that uses minimal amount of materials and keeping it economical based on the chosen code.

CHAPTER 2: LITERATURE REVIEW

2.1- Introduction

2.1.1- Codes Importance

The building codes can be defined as a series of regulations and rules that highlight out the minimum requirement to construct a specific structure. The idea of making building code is to regulate and ensure the safety of people and the stability of structures. The codes are mainly used by engineers, construction managers and architects, it's also an interest for environmental engineers, safety and health inspectors, subcontractors, building materials manufacturers and others. It's true that building codes were done to enhance safety but no code can eliminate all risks, and the engineers should target to reduce risks to an acceptable level. (Banerjee 2015)

The design codes assist to reduce the impact of the natural disasters and increase the protection levels to create a more sustainable, flexible, safe and stable structures. Following the instructions and guidelines in the code will give the designer a clear path on how to build and function a building in the safest stable way. In addition, they give a baseline and directions in order to deal with the extreme weather conditions and manage their risks. (EESI 2013)

Building codes provides limitations and standard for the building against lateral loads like seismic and wind loads. They provide many provisions and requirements that need to be fulfilled in order to give more confidence about the building safety against such loads.

2.1.2- Codes Development History in Seismic & Wind

The recent building codes followed around the world are results of several tests and studies, as a result those codes were corrected and adjusted several times through the years which made it possible to reduce risks and develop a structural design in the safest way. The countries' authorities are requested to follow up with the codes update and use the latest published design codes. As there are several codes followed by different countries, this research will be focusing mainly on ACI code and Eurocode in seismic and wind fields.

ACI Code (History of seismic evaluation)

The ACI code was created as a result of lack in understanding and uniform in creating the concrete blocks mixture which caused many tragedies and collapses of building throughout history. The issue was initially highlighted by Charles Brown during September 1904 during the Municipal Engineering annual discussion, he pointed out the importance of having rules and standards to save the industry from its massive loses and putting people's life in danger and encouraged to form an organization to start testing and listing down standards for the practice. (Wilde 2004)

Based on that, in 1905 an institute was formed under the name of "National Association of Cement Users" which was responsible in creating the standards and regulations to be used in United States and worldwide. It was later on named as the "American Concrete Institute" (ACI) which the code was named after. (Lesley 2016)

The first published report was done on 1907, it discussed the safety factors and specification of designing members. After the San Francisco earthquake disaster in 1906, the association interest moved to study deeply the effect of earthquakes in structure. This was the main focus in the ACI publication in 1910, where they listed standards and regulations for resisting earthquakes and reduce their impacts on reinforced concrete structures. (Wilde 2004)

Furthermore, in 1971 a new publication under the name of ACI 318-71 was published which included an added section concerning thin shells as well as an appendix related to seismic design in which it explained the torsion effect and design provisions in concrete members. In addition to a unified analysis methodology for the designing of flat slabs, flat plates, and two-way slabs and basic statistics regarding the strength of concrete requirements. This publication was the milestone for the concrete codes. (Wilde 2004)

In 1989, the ACI design code witnessed a new addition which is adding a commentary section. The purpose of the commentary is to justify and explain further the idea behind a regulation or a rule within the code. Parallel to each rule is a commentary section specifically for that rule. This facilitate for the code users to understand where the rules came from, what was the background behind assigning it and what are the circumstances of not following it. In ten years later (1999), the code added a section for post tension requirements in anchorage zones to make it more rational in dealing with seismic design. (Wilde 2004)

The ACI publication in 2002 was substantially important as it added a whole new section of masonry standards for design and construction. ACI 318-02 added multiple changes from its latest publication ACI 318-99; the major changes were integrating more seismic design requirements, and modify the allowed design values for flexural tension, and the threshold of the speed of wind for the empirical design. This publication was seen as the most inclusive revised code publication since this code publication started. It's important to mention that this version showed a reduction in strength factors as well as the load factors compared to the previous ACI publications. (Wilde 2004)

The latest publication of the code is ACI 318-19 that added some adjustments in the resistance of earthquakes in structures and emphasized more with the design of structural walls. (Moehle 2019)

ACI Code (History of wind evaluation)

The ACI code related to wind loads had two major events in 1934 and 2002. In 1934, the building code updated the rules and proposed a revision regarding the wind loads that was published in 1928 after many concerns that was made regarding the factors assigned. Later in 2002, the Masonry Standards Joint organization (joint ACI-ASCE-TMS) published a new chapter of masonry regulations for constructing and design which include many major changes from its latest version 1999 some of them changes modifications in wind speed threshold for empirical design and reduction loads factor. (Moehle 2019)

Eurocode (History of seismic evaluation)

During 1975, Commission of the European Community initiated a new program related to the construction field that included a series of guidelines to be pursued from the construction design works. In the next fifteen years, this commission started the Eurocode program, the commission consisted of a member from different European state guiding the development of the codes based on their tests and study done in their countries, and in 1980 they announced the first generation of European codes. Later in 1989, the commission decided to transfer all the processes of preparing and publishing duties of the Eurocodes to the European Committee for Standardization (CEN) to enhance them with futuristic vision that suits all countries in Europe and was named later one the European Standard. ("Eurocodes - Wikipedia" 2020)

The EN Eurocode consists of 10 standards starting from the publication of 1990 till the publication of 1999 (EN 1990 - 1999) that highlighted several topics regarding construction, and point out many seismic design aspects. As a different approach for organizing this design code, the EN committee covered one design aspect in each chapter instead of distributing several topics like loadings, materials or geotechnical design in each chapter as it was published each year in between 1990 and 1999. The seismic design aspects were covered in (EN1998) chapter or as it’s called Eurocode 8. The Eurocode 8 title is “Design of Structures for Earthquake Resistance”. This code is following the most intellectual process for design compared to all seismic design codes currently applied around the world. (Fardis 2014)

The first generation of Eurocode was officially published in 2004, and it’s still been used till nowadays with minimal adjustments or additions from a year to another. (Booth 2014)

Eurocode (History of wind evaluation)

The wind loads were initially mentioned in the Eurocode in EN1991 as a part of chapter 1 “Actions on Structures” in section four (EN1991-1-4). This section covered a big number of different buildings’ shapes and dimensions. The wind load regulations and rules mentioned in the code is limited to the buildings and construction with height up to 200m, because it only deals with the loads effecting the gross structures, which made the code limited in giving alternatives or solutions, it only mentioned several actions such as: torsional vibrations. (Geurts & van Bentum 2007)

The first Eurocode that gave actions to deal with wind loads was the EN 1991-2-4 :1995 and followed by a revision in 2005 (EN 1991-1-4:2005) which made the Eurocodes more reliable and useful when it comes to wind loads regulations. (Owen et al. 2012)

2.1.3 Lateral Loads (Seismic & Wind)

Nowadays towers design became more slender and smaller in floor plates which increased the probabilities of the tower to be swayed and drifted. These designs created further challenges to the engineers to create a stable design which will withstand both gravity and lateral loads. In older designs, engineers were designing for gravity loads but due to the newly designs with extraordinary heights and within seismic zones, engineers need to account for lateral loads due to earthquake and wind forces. (Baikerikar & Kanagal 2014)

Examples on loads generated from gravity are dead load, live load, rain load, and snow load. In addition to previous loads, the structure is also facing lateral loads from earthquakes and winds. The lateral loads result in huge deflections and stresses. Due to that, building’s structure must accommodate necessary strength to counter vertical loads along with requirements of stiffness to withstand the lateral loads. (Kevadkar and Kodag, 2013)

Lateral loads can damage the building’s structure extensively, and it further increase with the increase in height. The stability of high-rise buildings depends on lateral loads considered and the choice of suitable system. The choice relays on multiple aspects such as structural system used, economic, feasibility and materials used. (Reddy & Eadukondalu 2018)

2.1.3.1- Seismic load

Earthquake engineering is a big field which revolves around seismic loads analysis resulted from seismic movements being transferred to structures. The transferring of loads occurs when structure is with direct contact with the surface or next to an adjacent structure. Seismic loads relays mainly on seismic hazard, the site geotechnical aspects and structure's natural frequency. (Lin & Yoda 2017)

An earthquake will generate random vibrations in all directions when it hits. It’s rarely to have a vertical motion from an earthquake, mostly it’s just horizontal movement. This horizontal movements vibrate the building structure and cause internal forces. Since the ground plates’ movement is a mixture of vibrations, it produces conflicting effects, where the tension stresses could turn into compression and vice versa. Due to that, the earthquake can cause concrete compression failure or yielding of reinforced steel. Adding to that, it might cause drifting of the floors and in very dramatic cases it can lead to building failure putting the occupants in high risks. (Kadhum & Saleem, 2018).

Earthquakes are considered a very destructive natural hazards which can lead to losses in properties and lives. Around 10,000 people die annually due to earthquakes. Furthermore, the economy faces massive in billions of dollars. (Yön, Sayın & Onat 2017)

The magnitude of the earthquake is the main aspect to define the earthquake strength and it characterizes the earthquake size. Nowadays, several scales are used to evaluate the earthquake size. After Richter (1935), several magnitude scales were proposed. (Kayal 2016)

The following table shows the earthquake magnitude and its effect based on Richter scale:

Table 1 : Richter magnitude comparison (Tendürüs, Wijngaarden & Kars 2010)

Richter magnitude	Modified Mercalli intensity	Description
1.0–3.0	I	Not felt except by a very few under especially favorable conditions.
3.0–3.9	II	Felt only by a few persons at rest, especially on upper floors of buildings.
	III	Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated.
4.0–4.9	IV	Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
	V	Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.
5.0–5.9	VI	Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.
	VII	Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.
6.0–6.9	VIII	Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.
	IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.
7.0 and higher	X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.
	XI	Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.
	XII	Damage total. Lines of sight and level are distorted. Objects thrown into the air.

Annually around 3,000,000 earthquakes happen worldwide. An estimation of 98% are lower than a magnitude of 3. Not more than 20 earthquakes which are generated yearly can be classified as extensive (magnitude 7.0–7.99) or great (magnitude 8 and greater) (Baxter 2000)

2.1.3.2- Wind load

Wind is a natural event involving complexities of several flow situations created by the interaction of the wind with the structural systems. Wind is made out of multiple tides from various sizes and rotational properties moving within an air path in relation with the earth's surface. These tides are the reason behind the wind's boisterous or turbulent characteristics. (Mendis et al. 2007)

High-rise buildings need strong structures to withstand the lateral loads effect which is a function of the amount of displacements caused by lateral and acceleration. Strong lateral displacements might lead to structural and non-structural damages, on the other hand strong acceleration can cause less comfort to the building users. (Park et al. 2008)

The method of measuring the wind loads specific intervals of the mean recurrence and uncertainties is complex and complicate to integrate from those loads. Loads factors and wind loads are explained in an analytical method provided with the design codes for typical buildings. But in case of high-rise buildings, methodology of calculations is not accurate and can mislead since it doesn't account for major aspects like the wind loads crossing, the instable aerodynamic, the shedding of vortex, and aerodynamic interactions. (Irwin, Denoon & Scott 2013)

2.1.4 Lateral Loads Resisting Systems

Lateral loads could cause serious damage to the building, and its impact increase with the raise in height. When it comes to the high-rise buildings, the stability of the structure relays on the lateral loads implemented and the selection of appropriate system. The selection is relaying on several components such as structural system used, economic, feasibility and materials used. The lateral structural systems give the stiffness to the structure that led to a dramatic decrease in the lateral displacements. (Reddy & Eadukondalu, 2018)

The structural systems in high seismic zones can be subjected to harsh damage. In addition to the gravity loads, structures have to handle the lateral loads that generate high stresses hitting the structure. Currently, one of the most famous techniques to reduce the impacts of lateral loads due winds or earthquakes is using the shear wall in reinforced concrete structures, it's one of the most popular and used techniques. Another technique is the steel bracing which has proven to resist the impacts of earthquake. (Kevadkar & Kodag, 2013)

In high-rise reinforced concrete structures, it's necessary to determine the lateral load-resisting model to create a stable structure and proper seismic design. Specific structural members are functioned to withstand the lateral loads which are generated from strong earthquakes and wind loadings. (Suresh, Rao & Rama 2012)

Several systems were created in order to resist these forces. Systems in which they resist lateral force absorbs the lateral forces generated from the earthquake and structure stiffness increase. Implementing lateral force resisting system is very important to make the structure earthquake resistant. When an earthquake hits significant horizontal forces hit the structures and generate serious damage to the structural elements which might lead in the end to structural failure. Lateral loads can generate high stresses, create displacement and shifting movements or generate vibration, all of these should be account to design stable structures and safe for the occupants. (Somasekharaiah, Sudhana & Muddasar 2016)

The main lateral loads resisting that will be discussed are moment resisting frame system, concrete shear walls system, bracing system and outrigger system.

2.1.4.1- Moment Resisting Frame System

Moment resisting frames are special type of structure designed particularly to handle the seismic loads since they have sufficient strength to withstand high yielding and plastic deformation. (Naqash, De Matteis & De Luca 2012)

Moment resisting frame is a structural system where the connecting joints between beam and column are fully restrained, and though allow for the flexural stiffness and frame members' strength to withstand horizontal forces. The advantage of using this system is that it has high ductility, which made it commonly used in the seismic areas and widely in applications of seismic criteria. (Asgarian, Sadrinezhad & Alanjari 2010)

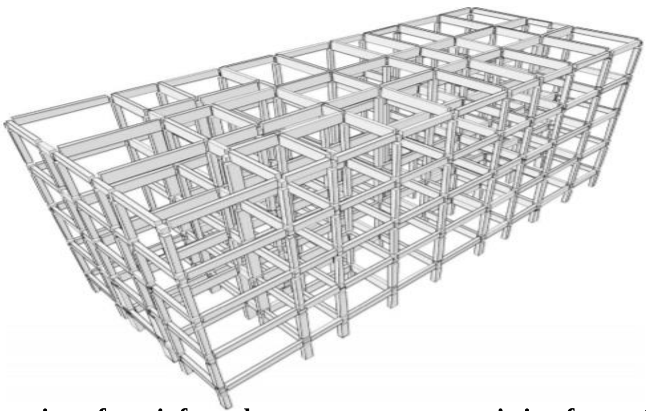


Figure 1: Illustration of a reinforced concrete moment-resisting frame (Smyth et al. 2004)

2.1.4.2- Concrete Shear Wall System

Shear walls are considered one of resistive systems of the horizontal forces, they are vertical structural elements that can counter for forces directly within the wall length. Shear walls have the ability of resisting horizontal forces by its strength and stiffness when designed properly. (Habibullah 2003)

Shear walls transfer the convenient lateral loads to withstand the received earthquakes' horizontal forces. If the shear walls were designed properly, they will facilitate transferring the horizontal loads to nearby load path elements directly underneath them. (Eusuf et al. 2012)

Figure 2: Illustration of shear walls design on site (Balkaya, Yuksel & Derinoz 2010)

2.1.4.3- Bracing System

One of the best and sufficient techniques for resisting lateral movements is structural bracing systems (Siddiqi, Hameed & Akmal 2014). The structural bracing function as a provider of lateral stiffness and stability to structures which is mainly used in tall and high-rise buildings. By using proper bracing arrangement, the building's structure lateral resistance raise up and the internal forces are reduced. (Yu, Ji & Zheng 2015). Due to that, the use of bracing system is spread worldwide since it's considered an economical solution. (Siddiqi, Hameed & Akmal 2014).

Figure 3: Structural bracing for john Hancock center building- Sample (Razak et al. 2018)

2.1.4.4- Outrigger System

Many structural systems are available in the markets which serves high rise buildings, one of the well-known and sufficient particularly for regular floor plan buildings is the outrigger system. Previously, outrigger systems were only utilized to add up to the structure’s stiffness and decrease drifting and deflection. Nowadays applications for outrigger systems developed and added damping characteristics in which decreases wind load and acceleration, as well as considered a structural fuse that strengthen the building during earthquakes. The main purpose of outriggers in building structures is to engage the perimeter and the internal structure as a whole to withstand lateral load. (Ho 2016)

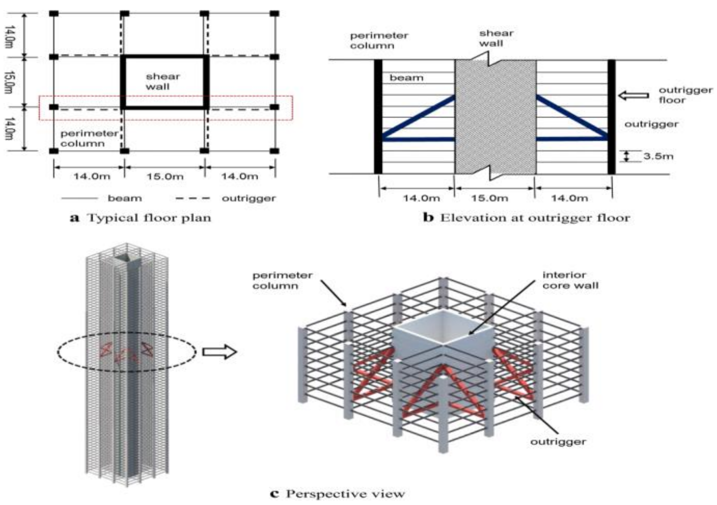


Figure 4: Outrigger building system illustration (Kim 2018)

2.2- Seismic & Wind Assessment Provisions

Each code has its own seismic and wind assessment provisions that vary according to the code's requirements. This research will explain the differences between some of these provisions in the ACI 318-19 code and the Eurocode in details, and it will include some of the previous studies conducted between these codes if any.

2.2.1- Seismic Base Shear & Scaling

Base shear is an estimation of the max conventional lateral force on the structure base when a seismic movement (earthquake) happens. The base shear value is determined as a combination of the following physical factors:

1- Site soil conditions
2- Closeness to prospect seismic movements sources (for example geological faults)
3- Eventuality of major seismic ground motion
4- Ductility Level and excessive strength integrated in the structure system and total weight
5- Primary (natural) period of structure vibration when subjected to dynamic loading. (Fragiadakis 2013)

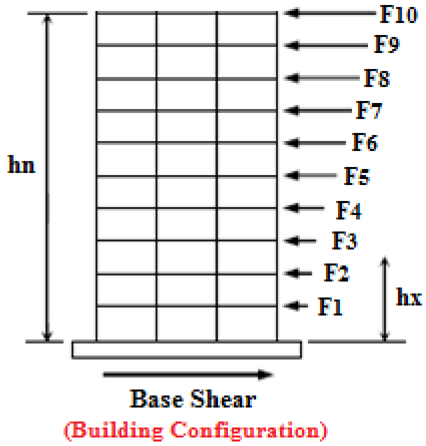


Figure 5: Seismic base shear & forces distribution illustration (Fragiadakis 2013)

A case study of seismic performance has been done by Khose, Singh & Lang for a ductile reinforced concrete frame building where it was designed by four design codes which are EN1998-1 (Europe), ASCE7 (US), IS 1893 (India), and NZS 1170.5 (New Zealand) to check the differences between the coefficient of the designed base shear for buildings with various ductile levels and several fundamental periods (to represent various heights). In order to compare the base shear, two values of peak ground acceleration of 2% exceedance probability (0.2g and 0.5g) were considered and design periods of 0.25, 1.5, 2.5, 3.5 and 4.5 seconds were chosen in order to cover acceleration, velocity and ranges of the design spectra controlled displacements of every tested code. Because every code uses a distinct classification, ASCE 7 corresponding site classes have been considered in their study. (Khose, Singh & Lang 2012)

Table 2: Different design codes consideration of design base shear coefficients (%) (With site classes corresponding to ASCE 7 class A) (Khose, Singh & Lang 2012)

Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	8.90	2.38	1.43	1.02	_1.00*_	22.23	5.93	3.56	2.54	_1.76_
	EC8 - DCL	16.32	5.13	2.47	1.53	1.53	40.74	12.80	6.13	3.87	3.87
	NZS - NDS	**17.83**	**5.77**	**3.48**	**3.00***	**3.00***	**44.58**	**14.43**	**8.66**	**5.25**	**3.40***
	IS 1893 - OMRF	_8.33_	_2.23_	_1.33_	_0.97_	--	_20.83_	_5.57_	_3.33_	_2.37_	--
II	ASCE 7 - IMRF	5.34	1.43	1.00*	1.00*	_1.00*_	13.34	3.56	2.14	1.53	_1.05_
	EC8 - DCM	6.28	1.97	0.95	0.59	0.59	15.67	4.92	2.36	1.49	1.49
	NZS - SLD	**8.58**	**3.00***	**3.00***	**3.00***	**3.00***	**28.33**	**6.01**	**3.61**	**3.40***	**3.40***
	IS 1893 - SMRF	_5.00_	_1.34_	_0.80_	_0.58_	--	_12.50_	_3.34_	_2.00_	_1.42_	--
III	ASCE 7 - SMRF	_3.34_	_1.00*_	1.00*	1.00*	1.00*	_8.34_	_2.23_	_1.34_	1.00*	1.00*
	EC8 - DCH	4.18	1.32	_0.63_	_0.39_	_0.39_	10.45	3.28	1.57	_0.99_	_0.99_
	NZS - DS	**5.28**	**3.00***	**3.00***	**3.00***	**3.00***	**17.43**	**3.40***	**3.40***	**3.40***	**3.40***

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

Table 3: Different design codes consideration of design base shear coefficients (%) (With site classes corresponding to ASCE 7 class B) (Khose, Singh & Lang 2012)

Category and Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	11.10	2.96	1.77	1.27	_1.00*_	27.77	7.40	4.44	3.17	_2.19_
	EC8 - DCL	16.32	5.13	2.47	1.53	1.53	40.74	12.80	6.13	3.87	3.87
	NZS - NDS	**17.83**	**5.77**	**3.48**	**3.00***	**3.00***	**44.58**	**14.43**	**8.66**	**5.25**	**3.40***
	IS 1893 - OMRF	_8.33_	_2.23_	_1.33_	_0.97_	--	_20.83_	_5.57_	_3.33_	_2.37_	--
II	ASCE 7 - IMRF	6.66	1.77	1.06	1.00*	_1.00*_	16.66	4.44	2.66	1.90	_1.32_
	EC8 - DCM	6.28	1.97	0.95	0.59	0.59	15.67	4.92	2.36	1.49	1.49
	NZS - SLD	**8.58**	**3.00***	**3.00***	**3.00***	**3.00***	**28.33**	**6.01**	**3.61**	**3.40***	**3.40***
	IS 1893 - SMRF	_5.00_	_1.34_	_0.80_	_0.58_	--	_12.50_	_3.34_	_2.00_	_1.42_	--
III	ASCE 7 - SMRF	_4.16_	_1.11_	1.00*	1.00*	1.00	_10.41_	_2.78_	1.67	1.19	1.00*
	EC8 - DCH	4.18	1.32	_0.63_	_0.39_	_0.39_	10.45	3.28	_1.57_	_0.99_	_0.99_
	NZS - DS	**5.28**	**3.00***	**3.00***	**3.00***	**3.00***	**17.43**	**3.40***	**3.40***	**3.40***	**3.40***

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

Table 4: Different design codes consideration of design base shear coefficients (%) (With site classes corresponding to ASCE 7 class C) (Khose, Singh & Lang 2012)

Category and Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	13.33	4.73	2.84	2.03	*1.40*	27.77	9.62	5.77	4.12	*2.85*
	EC8 - DCL	**19.55**	**7.67**	**3.67**	1.87	1.53	**48.90**	**19.20**	**9.20**	**4.67**	**3.87**
	NZS - NDS	17.83	5.77	3.48	**3.00***	**3.00***	44.58	14.43	8.66	5.25	3.40*
	IS 1893 - OMRF	*8.33*	*2.23*	*1.33*	*0.97*	--	*20.83*	*5.57*	*3.33*	*2.37*	--
II	ASCE 7 - IMRF	8.00	2.84	1.70	1.22	1.00*	16.66	5.77	3.46	2.47	1.71
	EC8 - DCM	7.52	2.95	1.41	0.72	*0.59*	18.81	7.38	3.54	1.79	*1.49*
	NZS - SLD	**8.58**	**3.00***	**3.00***	**3.00***	**3.00***	**28.33**	**6.01**	**3.61**	**3.40***	**3.40***
	IS 1893 - SMRF	*5.00*	*1.34*	*0.80*	*0.58*	--	*12.50*	*3.34*	*2.00*	*1.42*	--
III	ASCE 7 - SMRF	5.00	1.78	1.07	1.00*	1.00*	*10.41*	3.61	*2.17*	1.55	1.07
	EC8 - DCH	5.01	1.97	0.94	0.48	0.39	12.54	4.92	2.36	*1.20*	*0.99*
	NZS - DS	**5.28**	**3.00***	**3.00***	**3.00***	**3.00***	**17.43**	***3.40****	**3.40***	**3.40***	**3.40***

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

Table 5: Different design codes consideration of design base shear coefficients (%) (With site classes corresponding to ASCE 7 class D) (Khose, Singh & Lang 2012)

Category and Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	15.57	5.93	3.56	2.54	1.76	27.77	5.93	3.56	2.54	*1.76*
	EC8 - DCL	18.76	8.80	4.20	2.13	*1.53*	46.86	22.07	10.60	5.40	3.87
	NZS:C - NDS	**22.25**	7.18	**4.37**	**3.00***	**3.00***	**55.71**	**18.06**	**10.88**	**6.66**	**4.00**
	NZS:D - NDS	**28.28**	**11.77**	**7.03**	**4.29**	**3.00***	**70.74**	**29.30**	**17.61**	**10.80**	**6.51**
	IS 1893 - OMRF	*8.33*	*3.03*	*1.80*	*1.30*	--	*20.83*	*7.57*	*4.53*	*3.23*	--
II	ASCE 7 - IMRF	9.34	3.56	2.14	1.53	1.05	16.66	6.67	4.00	2.86	*1.05*
	EC8 - DCM	7.21	3.38	1.62	0.82	*0.59*	18.02	8.49	4.08	2.08	1.49
	NZS:C - SLD	**10.70**	3.00*	**3.00***	**3.00***	**3.00***	**35.40**	**7.52**	**4.53**	**3.40***	**3.40***
	NZS:D - SLD	**13.60**	**3.71**	**3.00***	**3.00***	**3.00***	**44.95**	**12.21**	**7.34**	**4.50**	**3.40***
	IS 1893 - SMRF	*5.00*	*1.82*	*1.08*	*0.78*	--	*12.50*	*4.54*	*2.72*	*1.94*	--
III	ASCE 7 - SMRF	5.84	2.23	1.34	1.00*	1.00*	*10.41*	*4.17*	*2.50*	*1.79*	*1.00**
	EC8 - DCH	*4.81*	2.26	*1.08*	*0.55*	*0.39*	12.02	5.66	2.72	1.38	0.99
	NZS:C - DS	**6.58**	**3.00***	**3.00***	**3.00***	**3.00**	**21.78**	**3.76**	**3.40***	**3.40***	**3.40***
	NZS:D - DS	**8.37**	**3.00***	**3.00***	**3.00***	**3.00**	**27.66**	**6.11**	**3.67**	**3.40***	**3.40***

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

Table 6: Different design codes consideration of design base shear coefficients (%) (With site classes corresponding to ASCE 7 class E) (Khose, Singh & Lang 2012)

Category and Ductility Class		PGA (2% PE in 50 years) = 0.2*g*					PGA (2% PE in 50 years) = 0.5*g*				
		Period *T*									
		0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*	0.25*s*	1.5 *s*	2.5 *s*	3.5 *s*	4.5 *s*
I	ASCE 7 - OMRF	*18.90*	*9.49*	*5.69*	4.07	2.81	*22.23*	*17.78*	*10.67*	*7.62*	5.27
	EC8 - DCL	21.99	11.73	6.60	*3.40*	*2.07*	54.97	29.35	16.53	8.47	*5.13*
	NZS - NDS	**24.64**	**18.20**	**10.95**	**6.66**	**4.07**	**61.64**	**45.51**	**27.31**	**16.72**	**10.14**
	IS 1893 - OMRF	8.33	3.70	2.23	1.60	--	20.83	9.27	5.57	3.97	--
II	ASCE 7 - IMRF	11.34	5.69	3.42	2.44	1.69	13.34	10.67	6.40	4.57	3.16
	EC8 - DCM	8.46	4.51	2.54	1.31	0.79	21.14	11.29	6.36	3.26	1.97
	NZS - SLD	**12.43**	**5.74**	**3.45**	**3.00***	**3.00***	**31.10**	**14.35**	**8.61**	**5.27**	**3.40***
	IS 1893 - SMRF	*5.00*	*2.22*	*1.34*	*0.96*	--	*12.50*	*5.56*	*3.34*	*2.38*	--
III	ASCE 7 - SMRF	7.09	**3.56**	2.14	1.53	1.05	*8.34*	*6.67*	*4.00*	2.86	1.98
	EC8 - DCH	*5.64*	*3.01*	*1.69*	*0.87*	*0.53*	14.09	7.53	4.24	*2.17*	*1.32*
	NZS - DS	**8.88**	3.00*	**3.00***	**3.00***	**3.00***	**22.21**	**7.18**	**4.31**	**3.40***	**3.40***

* minimum base shear requirements are governing; -- corresponding design spectral values are not available in the code
bold numbers indicate maximum and *italic numbers* indicate minimum base shear for a given category and design period

It is observed from Tables 2 to 6 that there are massive variations between the coefficients of the various design base shear from the tested design codes at different site classes. In nearly all cases, it was noted that the design base shear coefficients were ranking the codes as NZS 1170.5 being the highest, followed by Eurocode 8, ASCE 7 and IS 1893 being the least, excluding some cases where the smallest ASCE 7 base shear requirements were governing. The discrepancy wasn't only obvious in base shear, but also significant difference was noticed within the specific minimum design base shear. In cases of IS 1893 and Eurocode 8, the coefficient of base shear needed is quite low for taller buildings, since there is no limitation when it comes to the design base shear. The comparative study which was conducted conclude that the designing a building with different codes will result with different performance for the same level of hazard. For this reason, different codes compared risks should be unified in order to get comparable results to the several level of damage or failure for a certain seismic hazard. What was assumed is ASCE 7 code as a reference, and though it has been noticed that NZS results were the highest for design base shear, for approximately every case considered in this research. Following the NZS, Eurocode 8 results were the closest to NZS results, followed by IS 1893 results being the lowest for certain hazards. The design codes showed different results as well for the minimum design base shear needed, where IS1893 and Eurocode8 proven that the design base shear minimum limit can be neglected. (Khose, Singh & Lang 2012)

Another study done by Hassan, Anwar, Norachan & Najam to assess and compare the predictable performance during seismic when the design is conducted by several design codes for a 40-storeyhigh-rise building with an assumption that the building is within high active seismic area. The performance was evaluated using the non-linear response history analysis using several ground motions options on a binary structure system of moment-resisting frame and shear walls. This comparative analysis was conducted on three design codes ACI 318/ASCE 7-10, BS 8110 and Eurocode 2/8. (Hassan, Anwar, Norachan & Najam 2018)

The following are design codes along with the seismic codes used for each one:

Table 7: Seismic codes selection for each design code (Hassan, Anwar, Norachan & Najam 2018)

Design Codes	Seismic Codes
ACI 318-14	ASCE 7-10
BS 8110-1997	EURO-8
Euro Code-2-2004	EURO-8

It is observed that the base shear computed using ACI 318-14/ASCE 7-10 was 1.24 and 1.37 times higher than Eurocode8 in X and Y direction respectively as shown in following figure.

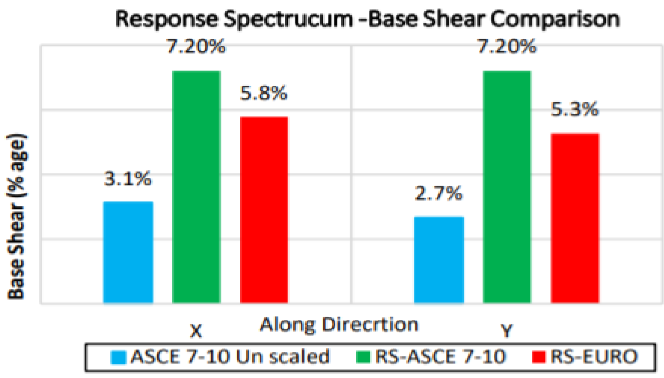


Figure 6: Base shear comparison of response spectrum - (Hassan, Anwar, Norachan & Najam 2018)

A study was conducted by Patil, Shiyekar & Ghugal to evaluate some seismic assessment parameters for Eurocode 8, ACI 318-08 and IS 1893:2016. A G+20 building is examined and analyzed for base shear, store forces, design force, design moments, storey-drift and also reinforcement requirement. The comparison for the base shear was performed in X-direction for the different codes, and it appeared based on their analysis that the calculated base shear in X direction compared to Indian code showed that Eurocode resulted with 16.70 % more base shear and ACI shows 10.05 % less base shear. In another words, Eurocode has the highest value of base shear followed by ACI then Indian code as shown in the figure 7.

This study concluded that comparing Eurocode with ACI when it comes to base shear, the Eurocode is higher by around 6.65%. (Patil, Shiyekar & Ghugal 2019)

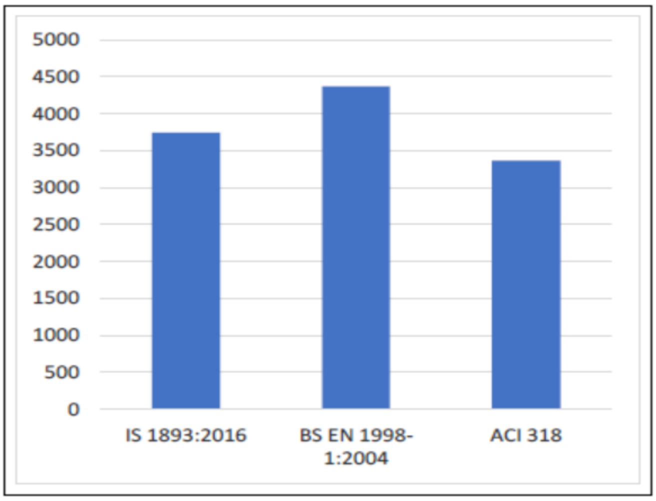


Figure 7: Base shear comparison in x-direction for Eurocode 8, ACI 318-08 and IS 1893:2016 codes (Patil, Shiyekar & Ghugal 2019)

2.2.2- Seismic Storey Drifts

Lateral deflection is defined as the expected structural movement when lateral loads are applied; and storey drift is the total variation in deflecting due to lateral among two adjacent stories. (Sigmund & Freeman 2004). Inter-storey drift is a significant index of structural behavior in performance-based seismic analysis. (Cai, Yang & Zuo 2014)

Drift is known as an essential control parameter for every design code; however, it differs from a code to another when it comes to the effective stiffness of reinforced concrete members, as well as the steps of drift calculation and the allowable limits on drift. Structure and non-structural components performance are controlled by internal storey drift. Internal storey drift effect the secondary effects (*P-Δ*) which is mentioned in section 2.2.5, and it's one of the essential design aspects, and roles the sizes of the members for several situations, especially for cases of high rise buildings. (Khose, Singh & Lang 2012)

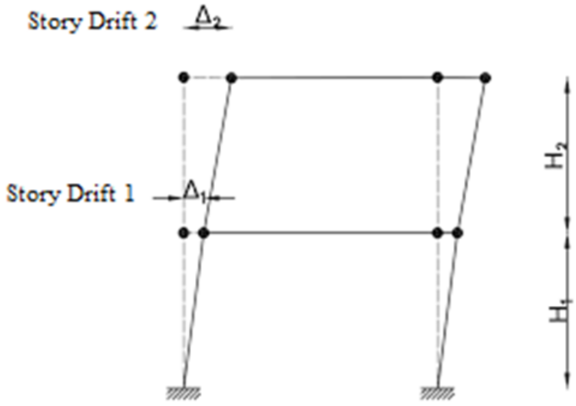


Figure 8: Storey drifts illustration (Newman 2004)

In ASCE 7, the drift in the level of floor is measured through an amplification factor of deflection which varies based on the building's typology. Limitations are given on amplified internal storey drifts that represent the building's rigid deformation. Eurocode 8 presents limits on the elastic displacements multiplied directly by the behavior factor. In ASCE 7, limits of the storey drift vary based on the occupancy category and gives until 2.5% drift for regular multistorey reinforced concrete frame buildings, on the other hand, the Eurocode8 allowed storey drift relays on non-structure elements type, and regarding the multistorey reinforced concrete framed buildings, the allowed drift is 1% for brittle non-structural elements, 1.5% for ductile non-structure elements, and 2% non-structure elements. (Khose, Singh & Lang 2012)

According to a study conducted by Khose, Singh & Lang where they compared the inter-storey drift for Eurocode8, NZS1170, ASCE 7 and IS 1893 in terms of design basis earthquake and max considerable earthquake, they concluded that the peak inter-storey drift ratio for all design codes in every case is more than 2.5% (the design drift maximum limit) for DBE, excluding the ASCE7 in longitudinal direction and IS1893 in transverse direction. For MCE, the highest inter-storey drift ratio got till or beyond 4% for the majority of the codes. (Khose, Singh & Lang 2012)

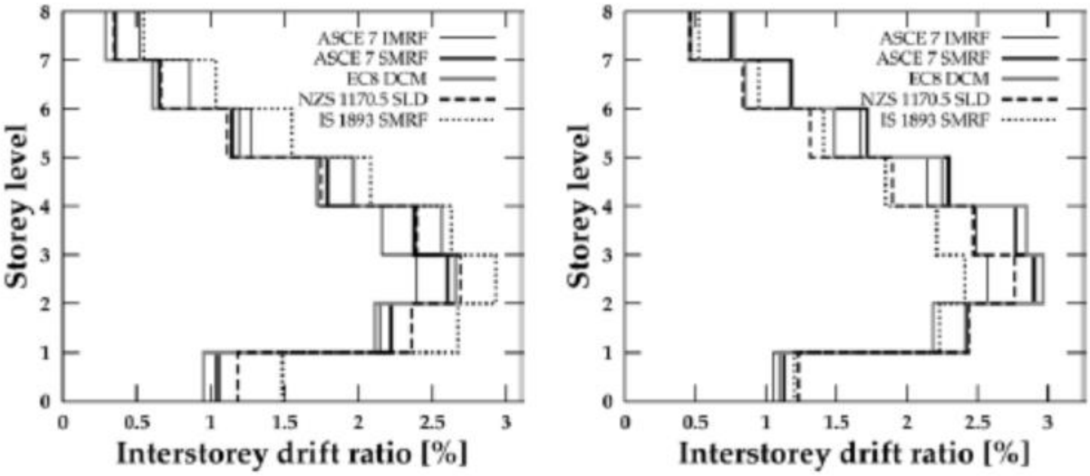


Figure 9: Drift ratio comparison with DBE in longitudinal (left) and transverse (right) directions when utilizing multiple design codes. (Khose, Singh & Lang 2012)

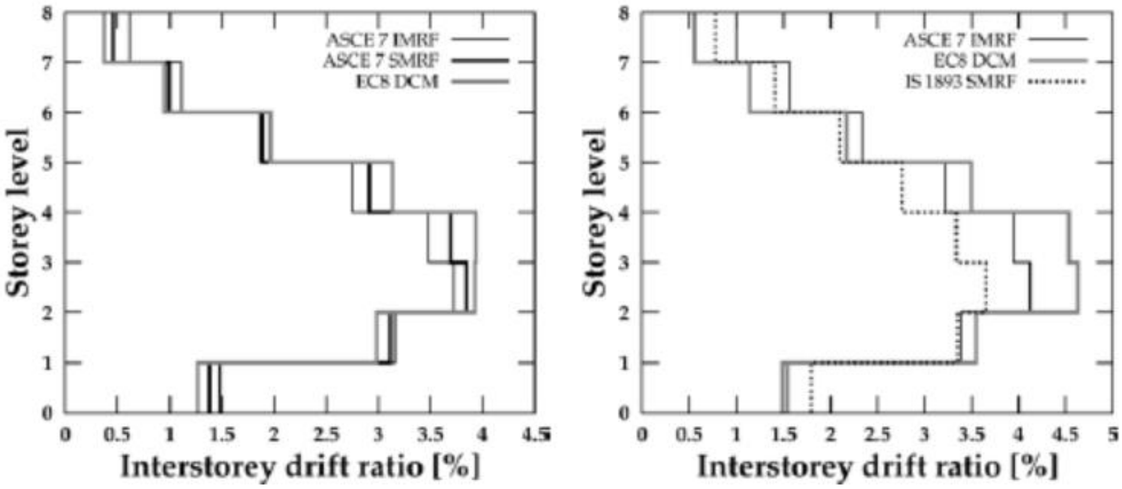


Figure 10: Drift ratio comparison with MCE in longitudinal (left) and transverse (right) directions when utilizing multiple design codes. (Khose, Singh & Lang 2012)

Following their analysis on storey drift for the seismic assessment provisions, Patil, Shiyekar & Ghugal concluded that the Eurocode had dramatic fluctuation values, on the other hand the Indian Code and ACI had less fluctuation values compared to Eurocode 8 as showing in the following figure:

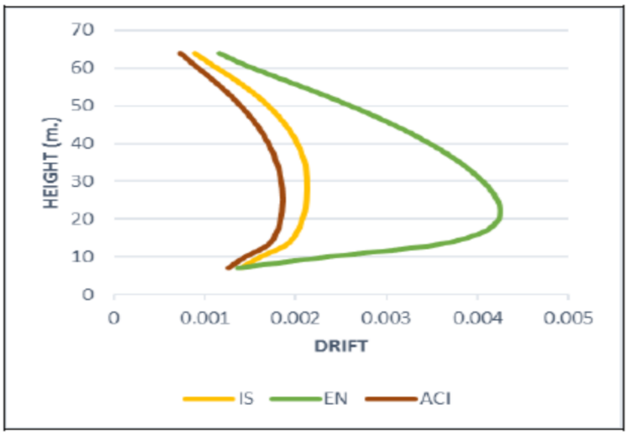


Figure 11: Storey drift comparison in x-direction for Eurocode 8, ACI 318-08 and IS 1893:2016 codes (Patil, Shiyekar & Ghugal 2019)

Another comparative analysis study was done by Karthik and Koti on dynamic loads carried out on a high-rise structure using ASCE 7-05, IS 1893:2016, IS 1893:2002 and Eurocode 8. The analysis was done on 25 floors reinforced concrete building (Karthik & Koti 2017). One of the analysis that have been performed was the storey drift comparison in the X and Y directions; the following data was obtained:

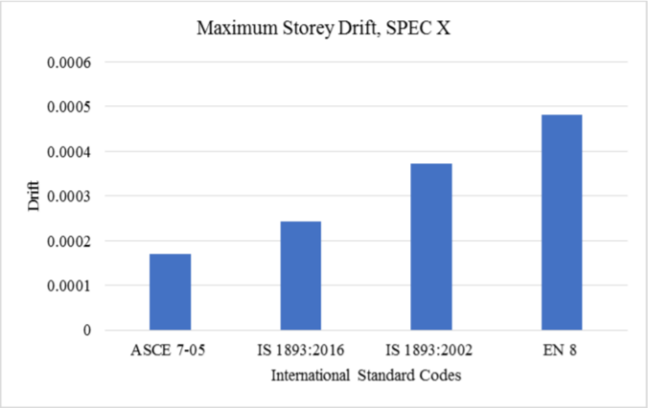


Figure 12: Storey drift comparison in x-direction for ASCE 7-05, IS 1893:2016, IS 1893:2002 and Eurocode 8 (Karthik & Koti 2017)

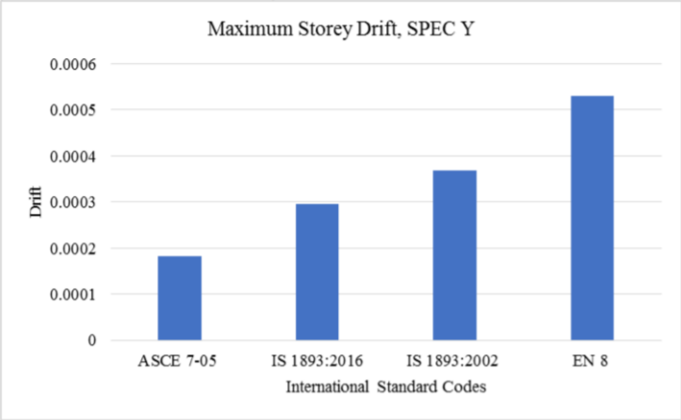


Figure 13: Storey drift comparison in y-direction for ASCE 7-05, IS 1893:2016, IS 1893:2002 and Eurocode 8 (Karthik & Koti 2017)

Based on the previous chart and the study analysis, they have found that the Eurocode model gives more drift in both directions when compared to the other codes. Furthermore, they also concluded that since Eurocode exceeded all the other codes in the compared structural provisions like storey drift, then the Eurocode design will need more reinforcement compared to other codes.

2.2.3- Storey Force

The storey shear force distribution of most seismic design codes reflects impacts of higher vibration modes based on the structure elastic deformation. Even though, the seismic design permits for the elastic structure behavior, the storey shear force distribution should be efficient after its yielding because of the earthquake movements. Generally, the storey shear forces applied horizontally on structure is measured in reference to the height and weight applied on every floor. That’s why, seismic design code in many countries analyze the load distribution characteristics resulted from higher vibration modes in different ways and apply several forms of the storey modification factor for shear force distribution accordingly. (Oh & Jeon 2014)

In many design codes related to seismic design such as Uniform Building Code 1997, NEHRP 1994 and ATC 1978 the distribution of lateral forces can be calculated with consideration of building's height from the following equation:

$$\boldsymbol{F_i = V.\frac{w_i\,.\,h_i^k}{\sum_{j=1}^{n} w_j\,.\,h_j^k}}$$

In other codes like ANSI/ASCE 7-95, the 'k' value rises from 1 up to 2 as period differs from 0.5 to 2.5 s. (Hajirasouliha & Moghaddam 2009); and till nowadays the latest ASCE code which is ASCE 7-16 is still using the same equation to measure seismic forces vertical distribution. (ASCE/SEI 7-16 2016)

Distribution of the horizontal seismic forces can be calculated using two methods:

a) With reference to the height of masses

b) With reference to absolute masses horizontal displacement

Distribution of the horizontal seismic forces according to Eurocode 8 is calculated using the height of masses and it's done using the following equation: (Patil, Shiyekar & Ghugal 2019)

$$\boldsymbol{F_i = F_b\cdot\frac{z_i\,.\,m_i}{\sum z_j\,.\,m_j}}$$

In Patil, Shiyekar & Ghugal comparative analysis on the storey force for Eurocode 8, ACI 318-08 and IS 1893:2016; their results showed that the storey forces obtained from Eurocode has larger range than the same obtained from Indian Standard and ACI. (Patil, Shiyekar & Ghugal 2019)

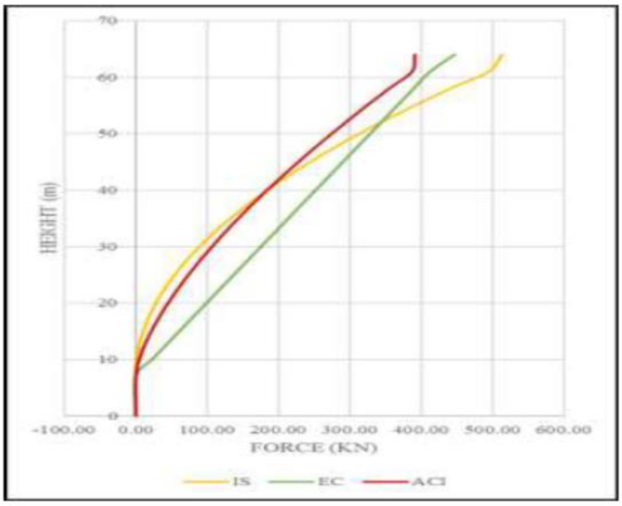


Figure 14: Storey forces comparisons Eurocode 8, ACI 318-08 and IS 1893:2016 codes (Patil, Shiyekar, Ghugal 2019

2.2.4- Torsional irregularity

In many cases, when a building faces an earthquake, it can be subjected to torsional rotations along with lateral displacement. The irregularity in structural design of the building may lead to an increase of the earthquake damages compared to uniform structural distribution especially when it comes to torsional irregularity. Those building could face different displacement amount in different stories in addition to excessive torsion. (Mehana, Mohamed & Isam 2019)

To evaluate torsional irregularity, two terms need to be determined which are center of mass and center of rigidity. The center of mass is the spot in which the whole floor mass acts and the center of rigidity is where the entire building's stiffness acts. (Burhanuddin 2017)

Seismic loads effects are applied at the floor's center of mass while lateral loads are applied at the floor's center of rigidity. Many research papers highlighted the relation between torsion on concrete structures and the building structure irregularities. They all indicated that seismic load will be affecting all foundation points at once with same amount. Therefore, if the centers of mass and centers of rigidity were hitting at same point, a horizontal component of ground motion will produce translational motion without rotation. However, if they were not, a horizontal component of earthquake motion will produce both translational and rotational movements on the vertical axis. Figure 15(a) illustrates the situation where the center of mass and center of rigidity are not happening at one point and Figure 15(b) illustrates the situation where they are. This gives a conclusion that location of vertical lateral force must consider the center of mass in order to reduce potential of torsional effects. (Mehana, Mohamed & Isam 2019)

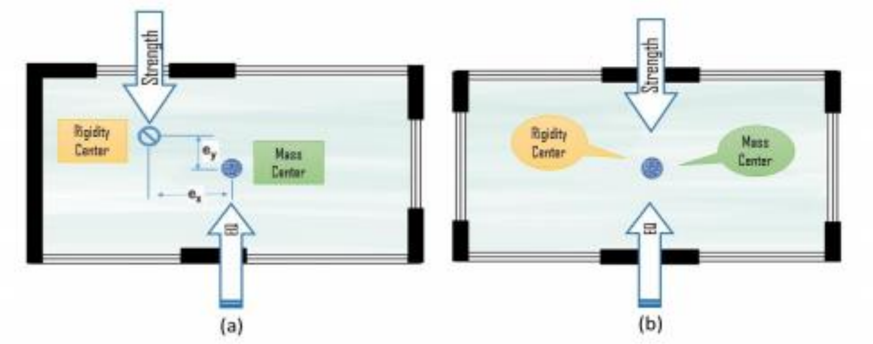


Figure 15: (a) Eccentric structure system. (b) Regular structure system (Mehana, Mohamed & Isam 2019)

The studies done in earthquake field approve that non-regular structures face higher damage compared to regular structures. Torsional irregularity is an essential factors that leads to heavy damages or even collapse for the structures. (Özmen, Girgin & Durgun 2014)

The variation between the Eurocode 8 and ACI318-19/ASCE 7-16 in defining the torsional irregularity is that, Eurocode 8 rates buildings based on their floor plan by qualitative and parametric criteria like stiffness, radius of torsion and eccentricity which are majorly effective when it comes to multistorey buildings, thus, these plan-related characteristics need improvement while for ASCE, the characteristics related to plan or torsional irregularity are only used when max storey drift is occurring, specifically in cases when one of the structure's ends is over 1.2 times the storey drifts average at the two structure ends under the same static analysis. Although, ASCE methodology of having threshold value at 1.2 require to be backed up by trustworthy background researches, and also by investigating it using nonlinear seismic analysis. (Coseenaz, Manfredi & Realfonzo 2000)

The torsional irregularity can affect other parameters like the storey drift and the structure period of vibration. A study done by Hussein, Eid & Khaled, concluded that as the floor plan irregularity becomes higher, the structure period of vibration reduces, this statement proven that the period of vibration isn't depending only on the buildings' height similar to the traditional methodology of calculations but also depends on the building's shape. On the other hand, regarding the storey drift, it has been concluded that the total storey drift ratio becomes higher when the floor plan irregularity constantly increases. (Hussein, Eid & Khaled 2019)

The below table shows the different criteria required and limitations specified by the ASCE code and the Eurocode:

Table 8: Torsional irregularity as defined in several seismic codes. (İlerisoy 2019)

EUROCODE-8	At each level and for each direction of analysis *x* and *y*, the structural eccentricity *eo* and the torsional radius *r* shall be in accordance with the two conditions below, which are expressed for the direction of analysis *y*: $eox \leq 0.30.\ rx;\ rx \geq ls$ where; *eox* is the distance between the centre of stiffness and the centre of mass, measured along the *x* direction, which is normal to the direction of analysis considered; *rx* is the square root of the ratio of the torsional stiffness to the lateral stiffness in the *y*-direction; *ls* is the radius of gyration of the floor mass in plan
ASCE/SEI 7-10	*Torsional irregularity* is defined to exist where the maximum story drift, computed including accidental torsion with $Ax = 1.0$, at one end of the structure transverse to an axis, is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid. *Extreme torsional irregularity* is defined to exist where the maximum story drift, computed including accidental torsion with $Ax = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.

2.2.5- P-delta Effect

P-delta effect is the secondary moment that is generated by wind or seismic actions. For tall reinforced concrete building, the P-delta impacts have a major effect on stability and ductility when designing the building for seismic or wind considerations. (Pouya 2019)

All structures deflect transversely under seismic loading. When the structure is loaded, it will deform or deflect to release the stress, which is referred to as first-order effect. If this first-order effect was the cause of any further stress on the structure, it is then called the second order effect or P-Delta effect. (Abbas & Abdulhameed 2016)

The vertical loads are concentric with the members' base, when lateral loads applied on structure, it starts to displace and the vertical loads become irregular with reference to the base. The overturning moment is referred to as primary moment, when the total vertical loads are concentric with the structure's base. The magnitude of this moment is as shown in Figure 16, where F is the lateral load and h is the height of the structure. When the vertical loads become irregular with reference to the base, the overturning moment supplements an additional bending stress to the members, this moment is called secondary moment (Ms) and the magnitude of it is PΔ where P is the weight of the building include the vertical load and Δ is the drift. Accordingly, P-Delta is the added deflection and overturning moments generated from vertical loads applied through the relative transverse displacement of the member ends. (Abbas & Abdulhameed 2016)

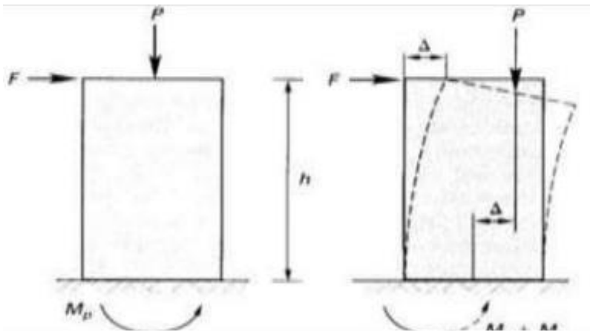


Figure 16: P-Delta effect (Abbas &Abdulhameed 2016)

The Eurocode 8 consist of an exaggeration for the displacements effects and forces internally using an amplification factor α, that can be calculated using the equation:

$$\boldsymbol{\alpha = \frac{1}{1 - \theta}}$$

Where θ is the coefficient of stability per story. In Eurocode 8 methodology for the second order effects, the coefficient of stability is the most essential parameter driving the analysis. It is measured using the followed equation:

$$\boldsymbol{\theta = \frac{P_{TOT}\ x\ d_r}{V_{TOT}\ x\ H}}$$

where P_{TOT} is the vertical load applied on the storey, V_{TOT} the storey shear, d_r the design drift and H is the height. In case of Eurocode 8, second order effects can be ignored whenever θ is less than 0.10, the factor of amplification α need to be considered if θ was in-between 0.10 - 0.20. If the structure coefficient of stability exceeded 0.30 then the design is unacceptable. (De Stefano, Nudo & Viti 2004)

The ACI code uses the ASCE code as a reference for seismic and wind design and calculation for several provisions. The ASCE stated that P-Delta must be considered when estimating a modal response spectrum analysis for seismic design, it mentions as well that P-Delta doesn't have to be included when the coefficient of stability (θ) is equal or less than 0.10 as showing in the following equation:

$$\boldsymbol{\theta = \frac{P_x\ x\ \Delta\ x\ I_e}{V_x\ x\ h_{sx}\ x\ C_d}}$$

The code mentions that θ shouldn't be higher than θ_{max} or 0.25 as stated in the equation below, otherwise the structure is considered unsafe and need to be redesigned.

$$\boldsymbol{\theta_{max} = \frac{0.50}{\beta\ x\ Cd} \leq 0.25}$$

Furthermore, the code mentions that when $0.10 \leq \theta \leq \theta_{max}$, all displacement and member forces should be multiplied by a factor of $\frac{1}{1-\theta}$. Alternatively, P-Delta can be added in an automated analysis. (ASCE/SEI 7-16 2016)

2.2.6- Response Spectrum

Response spectrum analysis is a simplified analysis methodology from the model analysis which is used in many buildings' design. It's mainly used to get a rapid estimation of the peak response in short dynamic events without needing to go into much details of response history analysis which will need figuring out different equation of motion over time. The response spectrum is a function of the natural oscillator frequency and its damping. Regardless that this method relays on proximity; this analysis is considered very helpful and convincing when it comes to seismic loads calculations. Every design code around the world defines seismic hazards by means of a code-compliant, response spectrum can change facilely based on the seismic hazard of the site. The spectra are capable to give a rough idea if the structures are well designed to reduce seismic impacts by deforming elastically. (Fragiadakis 2013)

Eurocode 8 and ACI codes utilizes a 5% elastic acceleration response spectrum as the reference design spectrum. They indicate basic spectral shapes to several site classes that are resized with peak ground acceleration and adjusted based on site settings and return period to predict the design spectra. These codes capture the variation impact of soil in the short and long-period ranges of the spectrum. (Lang, Singh & Khose 2012)

For Eurocode 8, it has two spectral shapes sets for several site classifications, type (1) for the higher seismicity areas, and type (2) to be used in the least active areas. The Type 1 spectrum is related to earthquakes with values near to 7.5 while the Type 2 spectrum is used mainly in cases where Ms 5.5 as shown in figure 17. (Elghazouli 2019)

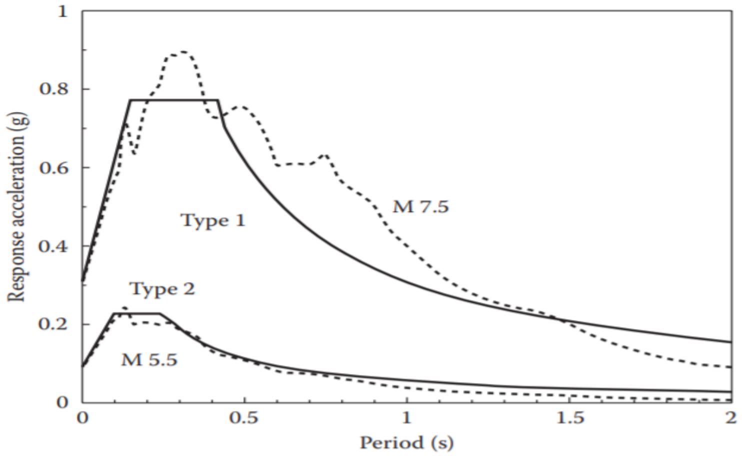


Figure 17: Predicted median spectral ordinates based on the European ground motion equations predicted of Ambraseys et al. (1996) for rock sites (Elghazouli 2019)

The following are the different types of response spectrum for different site classes according to Eurocode 8:

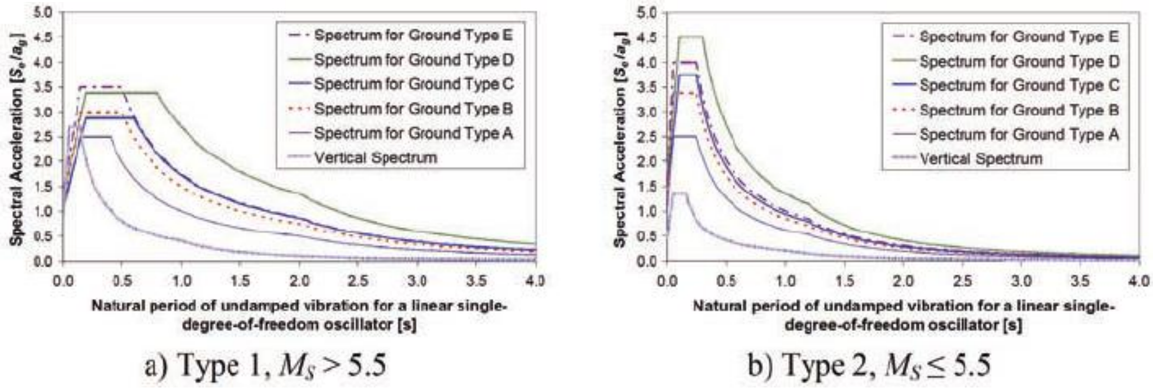


Figure 18: Response spectrum types 1 & 2 for multiple site classes based on Eurocode8 (Elghazouli 2019)

The elastic acceleration spectrum of 5% in Eurocode 8 is presented in figure 19. It includes some parameters that generate the spectrum shape; the area of stable spectra acceleration within the periods T_B and T_C with a magnitude of 2.5 multiplied by the max soil acceleration a_gS, which is pursued with an area of constant spectral velocity at interval of the periods T_C and T_D in which the spectra acceleration is proportion to 1/T, and an area of constant spectra displacement having the spectra acceleration proportion to $1/T^2$. (Eurocode 8 2004)

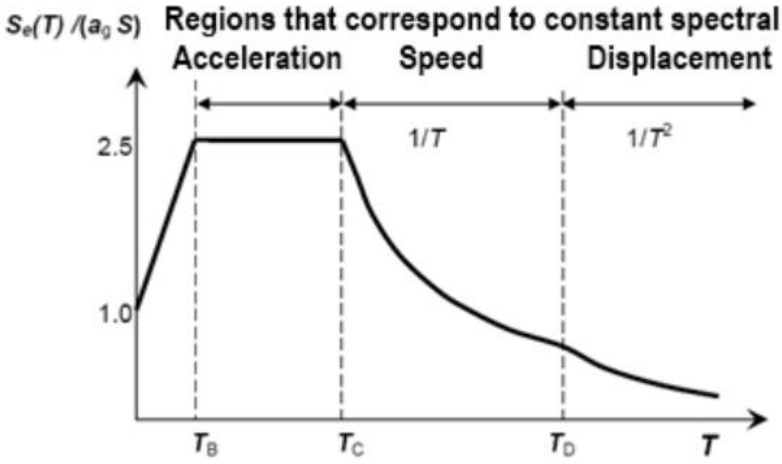


Figure 19: Horizontal direction for elastic spectrum in Eurocode with 5% damping (Elghazouli 2019)

As mentioned previously, the ACI 318-19 code follows ASCE 7-16 in the seismic analysis, thus the response spectrum procedure is generated directly from ASCE 7-16 code. The code relays fundamentally on spectral acceleration parameters in creating the response spectrum graph which are “Mapped Risk-Targeted Maximum Considered Earthquake” with 5% damped spectra response acceleration parameter at a period of 1 second similar parameter at short period which are denoted as S_1 and S_S respectively.

The values of building site spectral acceleration parameters S_S and S_1 are generated from the interactive web application based on every site class. After that, Fa and Fv coefficients are calculated from tables 11.4-1 and 11.4-2 respectively in ASCE standard and based on the coefficients of 1-second-long period, 0.2-second-short period and soil class, then the spectral acceleration of short period S_{MS} and 1 second period S_{M1} are calculated. (Aksoylu et al. 2020)

Other essential parameters needed to find out the response spectrum graph are highlighted in the ASCE 7-16 code which are risk category, response modification coefficient, long transition period and period of vibration. (ASCE/SEI 7-16 2016)

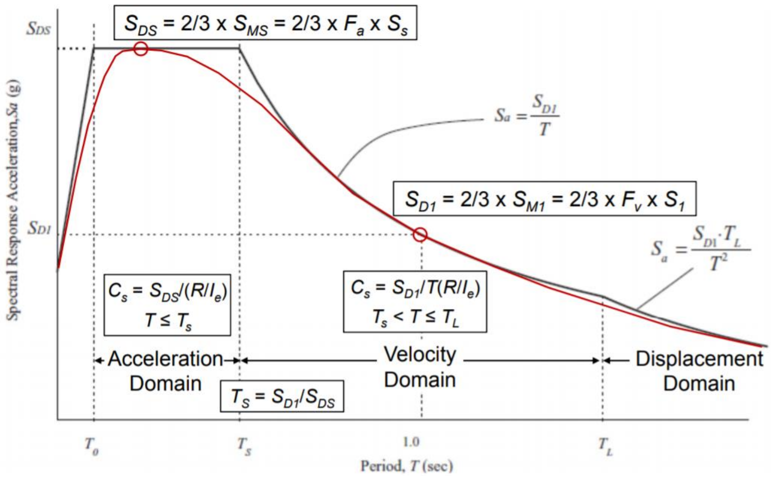


Figure 20: Response spectrum for ASCE 7-16 code for a damping of 5% (Kircher 2017)

2.2.7- Mass participation

Every structure has the capability to vibrate at a given set of natural frequencies. Every frequency is integrated with a shape, called a mass shape in which it indicates when vibration could happen to a mass at a certain frequency. The mass participation factor represents how strong a mass would take a part in the reaction of the structure when facing force/displacement in a certain direction. (Fragiadakis 2013)

The criteria most frequently used in Eurocode 8, demands that the modes number accounted produce a total effective modal mass within every seismic action components x, y or z, used in the design should be of minimum 90% of the total structure mass. Another method to do the calculation in Eurocode if the previous method was hard to achieve, the overall value analysis must count for every mode with effective modal mass in every seismic action component x, y or z within the design of larger than 5% of the total. ("Modal response spectrum analysis - Earthquake Resistance Eurocode" 2021)

While according to the ASCE 7-16 code, analysis should be done to figure out the structure vibration natural modes. The analytical study should have an adequate bunch of modes to generate a total modal mass participated combination of 100% of the structure’s overall mass. Although, an exception should be considered in which the analysis should account a minimal modes number to generate a modal mass participated combination of minimum 90% of the overall mass in every orthogonal horizontal response direction accounted in the model. (ASCE/SEI 7-16 2016)

2.2.8- Time period of Vibration

The building's natural period defines the time needed to go through one fully completed oscillation cycle. It's an inseparable building characteristic derived by its mass and stiffness. The reverse of the building natural period represents the natural frequency with measurement unit of Hertz. (Bhuskade, Meghe & Sagane 2017)

The structures design should accommodate for natural hazards like earthquakes and hurricanes, and the structures safety considerations which can be measured from knowing the natural frequencies and damping value in every vibration mode. Frequency applied on the building and its damping have a noticeable impact on its response value. The natural period of building vibration is an essential property to access the seismic base shear. It relays on basic properties like building's height or floors number; however, building's height is not enough to define period variability. The period of a reinforced concrete frame structure varies based on the consideration of structure longitudinal or transverse direction. (Bhuskade, Meghe & Sagane 2017)

The structural fundamental period (T) is an essential aspect when it comes to earthquake design since the design spectra acceleration is very critical to it. Approximately all seismic design codes are using empirical equations to roughly calculate the fundamental period. A basic simplified equation for framed buildings is:

$$\mathbf{T} = \boldsymbol{C_t} \,.\, \boldsymbol{h_n^x}$$

It is substantial to highlight that every design code provides equations that drive to nearly similar values of fundamental period T. (Khose, Singh & Lang 2012)

A study has been performed by Singh, Khose & Lang that investigated the fundamental periods for 4, 8, and 12-storeyreinforced concrete frame buildings based on several design codes as shown in the figure below:

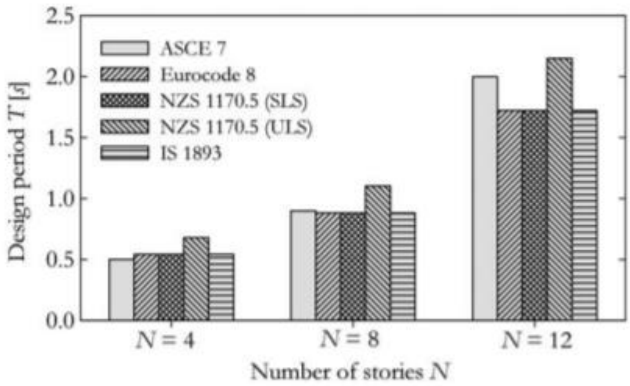


Figure 21: Comparative fundamental periods for reinforced concrete frame buildings based on several design codes (Khose, Singh & Lang 2012)

It was concluded from the analysis done in this study that the Eurocode give slightly more time period when it is compared to ASCE 7 code.

2.2.9- Wind Inter-Storey Drifts

One of the major issues in high rise building structure design is the serviceability characteristics associated with the wind loads including lateral deflection and acceleration limits. Deflections under wind loading are named as the “design wind speed”. (Smith 2011)

The control of the internal storey drifting by producing adequate lateral stiffness is essential to make sure less elastic deformation in structural members is occurring and less damage in non-structural elements. It has an obvious impact when it comes to seismic design but rarely impacting or considered in wind related analysis for high rise design, as the internal storey drifting generated by wind loads are usually lower than the specified limit. (Fu et al. 2015)

The ASCE 7-16 standard doesn’t state an allowable drift limit for wind design similarly to seismic design but referring to the non- compulsory *Appendix CC* (Serviceability Considerations) of ASCE 7-16, the usual use for building design is on the order of 1/600 to 1/400 of the building or storey height excluding more details. The typical wind drift limits used range between H/100 to H/600 for overall building drift and h/200 to h/600 for internal storey drift, based on building type and cladding type and even partitions’ materials integrated. (ASCE/SEI 7-16 2016)

While the Eurocode doesn’t state a limitation for storey drift against wind, but according to Rob Smith it that the inter-storey drift ratio in Eurocode 8 is between 1/200 and 1/100 (Smith 2011). The full table of inter-storey drift ratios for different codes are shown in table 96 in appendix C.

2.3- Structural Elements Cost and Sizing

2.3.1 Columns

It’s rarely for columns to be exposed to only pure axial loads. Usually, reinforced concrete columns are exposed to a combined axial and lateral loads as well as deformations, generated from several complex load’s patterns due to earthquakes which in the end cause lateral deflection that leads into the horizontal stiffness to be affected. According to a research based on case studies comparison conducted by Rafaa & Akram, their study concluded that axial load index and longitudinal reinforcement ratio are the major parameters affecting directly the column actions. In addition, the column capacity strength can be enhanced by the increase of concrete compressive strength and reduction of the column aspect ratio. Furthermore, raising lateral reservation by transverse reinforcement at the column ends will lead into an increase of flexure-controlled reinforced concrete columns flexural strength. (Abbas & Awazli 2017)

In Eurocode 2 the maximum reinforcement area for columns outside laps is 4%. Although, this could be higher in case more concrete is being placed and compacted adequately (Bond 2006), while for ACI 318-19, the maximum reinforcement ratio for columns is 8% of the total column area (Guide 2021).

Several studies have been done to compare the Eurocode and ACI code in terms of reinforcement amount in columns. Patil, Shiyekar & Ghugal study comparative analysis for Eurocode, ACI and Indian code IS for columns reinforcement requirement was conducted by analysing the need of column reinforcement of one column at Storey1 in the 25th storey as a study case, which resulted with variation in the column reinforcement amount within the three codes. The study concluded that Eurocode reinforcement needed for the column is higher than other two codes, the amount of column reinforcement generated by the Eurocode is higher by 26.58% compared to ACI code (Patil, Shiyekar & Ghugal 2019).

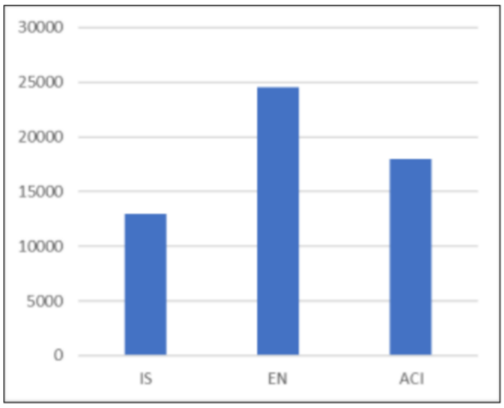


Figure 22: Column reinforcement amount comparison for Eurocode, ACI and Indian code IS 1893:2016 (Patil, Shiyekar & Ghugal 2019)

Referring to the study conducted by Hassan, Anwar, Norachan & Najam, they have compared the structural elements reinforcement for Eurocode, ACI and BS codes on 10,20 and 30 levels. They found out that concrete columns reinforcement of the Eurocode needs slightly less reinforcement than the ACI code (Hassan, Anwar, Norachan & Najam 2018)

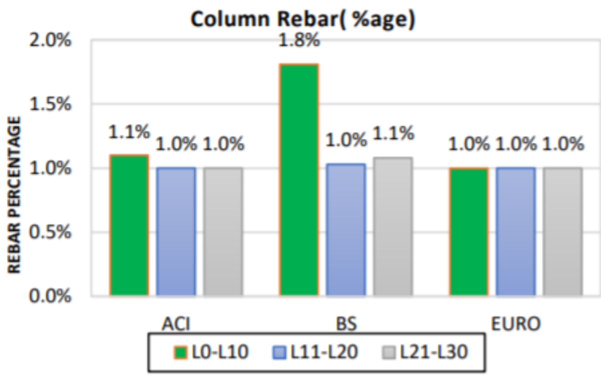


Figure 23: Column reinforcement amount comparison for Eurocode, ACI and BS codes (Hassan, Anwar, Norachan & Najam 2018)

It can be concluded from the previous figure that for the 20 levels, all codes requirements for the column reinforcement are close to each other, the significant difference is obvious for the levels 0 to 10 where the reinforcement required by the ACI code is higher by around 0.10% compared to Eurocode (Hassan, Anwar, Norachan & Najam 2018). The total amount of longitudinal reinforcement was compared between three design codes in terms of reinforcement weight and found that Eurocode needs less reinforcement compared to ACI as shown in the following table:

Table 9: Reinforcement comparison (Hassan, Anwar, Norachan & Najam 2018)

Reinforcement Comparison (Weight in Kg)			
	ACI	BS	EURO
Columns	67,598	81,484	64,362
Beams	280,522	334,593	387,196
Coupling Beams	73,784	86,949	60,791
Shear Walls	480,852	412,804	204,048
Total Sum (Kg)	902,756	915,830	716,396

2.3.2 Shear Walls

Larger strength and stiffness is provided to the building's structure within shear walls direction of orientation in which decreases majorly the lateral drift of the building and though decreases the damage to structure and its contents. (Kevadkar & Kodga 2013)

Shear walls are easily constructed since the reinforcement details are mainly straight-forward and easier to implement on site. Shear walls are quite sufficient when it comes to construction cost and reducing earthquakes damages in structural and non-structural elements (Agrawal & Charkha 2012).

The concept behind shear wall structures is to provide a high lateral rigidity. That's why shear walls are one of the essential structural elements which are resistible to earthquake forces because of their high lateral rigidity and load bearing capacities. It is mentioned in Eurocode 2 that the minimum reinforcement ratio on RC shear walls can be 0.004 and the maximum longitudinal reinforcement ratio could be 0.04. While for the ACI 318-19 the ratio of horizontal and vertical reinforcement should be greater than 0.0025. (Maali 2020)

A study done by Hassan, Anwar, Norachan & Najam (2018) comparing the shear walls reinforcement of different codes have shown that the Eurocode is giving shear walls reinforcement lower than half of the ACI reinforcement requirement for shear walls reinforcement. In addition, based on figure 24 it can be noticed that the ACI model has more reinforcement than the Eurocode mode by around 57.57 % (Hassan, Anwar, Norachan & Najam 2018).

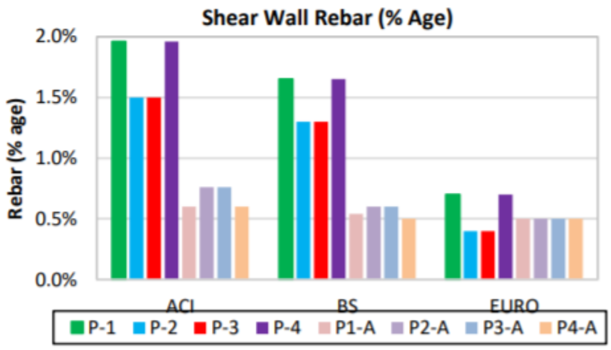


Figure 24: Shear walls reinforcement amount comparison for Eurocode, ACI and BS codes (Hassan, Anwar, Norachan & Najam 2018)

2.3.3 Slabs

To enhance the lateral stiffness and structural strength, the reinforced concrete walls will be carrying out the largest amount of the horizontal loads produced from the earthquakes while for every slab-column connection has to have the ability to move with the lateral displacement produced from the seismic loads and in the same time withstand the ability to move vertical loads from slabs to columns. If this combination didn’t work properly, severe damage or slab failure will occur. In another words, the deformation capacity of the whole structure is subjected to the deformation capacity of the slab-column connection. (Drakatos, Beyer & Muttoni 2014)

Nowadays in many European countries, flat slab systems are spreading widely in the construction of reinforced concrete buildings. This structural type is mainly used in Southern European countries for office and residential buildings. Several design codes have included this type of structures in their sections; however, it wasn’t mentioned in the latest draft of Eurocode 8. This structural system showed many benefits when it comes to earthquake resistance, but also their drawbacks should be kept in mind which include the major seismic response non-dissipative features. In addition, flat slab building structures shows high flexibility compared to the traditional concrete frame structures, which makes it more exposed to second order P-Δ impacts during seismic movements. Due to that, the seismic behavior characteristics of flat slab buildings will need some extra measures for directing the concept and structural design in seismic areas. (Coelho et al. 2004)

2.4- Major Different Provisions Comparison

The different seismic and wind provisions and consideration in the codes are not the only reason for obtaining different results, there are some other provisions and codes characteristics that can affect the results. In this section some of these characteristics and provisions will be discussed.

According to Lang, Singh and Khose, they stated that different codes differ in some considerations such as materials factors (or strength reduction factors) for the members design and that the codes don’t follow the same pattern, thus this made a direct impact on the estimated designed building performance using multiple design codes. They also stated that such characteristics might affect the seismic assessment provisions being compared, they also gave an example which is for the drift. (Lang, Singh & Khose 2012)

Results between the codes are different due to the effective stiffness of reinforced concrete members (Lang, Singh & Khose 2012). Moreover, Karthik and Koti have pointed in their comparison study for different codes that the different results obtained from each code was due to the different independent constants, loading and load combinations of their respective International Standard codes. (Karthik & Koti 2017)

In this study many resources have referred to ACI 318-14 code, especially in Bakhoum, Mourad and Hassan study and it is essential to highlight that the ACI 318-19 is following the same requirement on the different provisions they mentioned.

2.4.1- Materials

Nowadays, the concrete is not used solely in construction, it’s used as part of the reinforced concrete where steel is embedded in concrete and the two materials work together to resist forces. A special type of this material is the “High Strength Concrete” that is used extensively in high-rise buildings to minimize the huge width of columns needed to support such tall buildings’ structure especially in lower floors. What makes this material special is its elastic property, as it has high elastic modulus that reduces the deflection amount. (Brown 2016)

What makes the high strength concrete special is its elastic modulus. Based on the ACI 318-19 the equations used to calculate the modulus of elasticity in Mpa is $4700\sqrt{Fc'}$ (ACI 318-19 2019), while for the Eurocode it is 22,000 $(\frac{Fcy}{10})^{0.30}$ (Eurocode 2 2004); Concluding from the previous equations that when calculating the elastic modulus for the same amount of concrete strength, the Eurocode result would be higher compared to the ACI code.

A study was done by Neville supports this statement, he compared the equation used to calculate elastic modulus in different design codes and had them in one chart. He found out that the Eurocode considers the highest elastic modulus factor compared to other codes. Comparing Eurocode with ACI codes, the Eurocode2 had higher elastic modulus than ACI 318-95 until the compressive strength reached 65MPa in which ACI 318-95 elastic modulus value became higher, but comparing with ACI 363R-92, the Eurocode 2 will always be higher as shown in the figure below: (Jurowski & Grzeszczyk 2018)

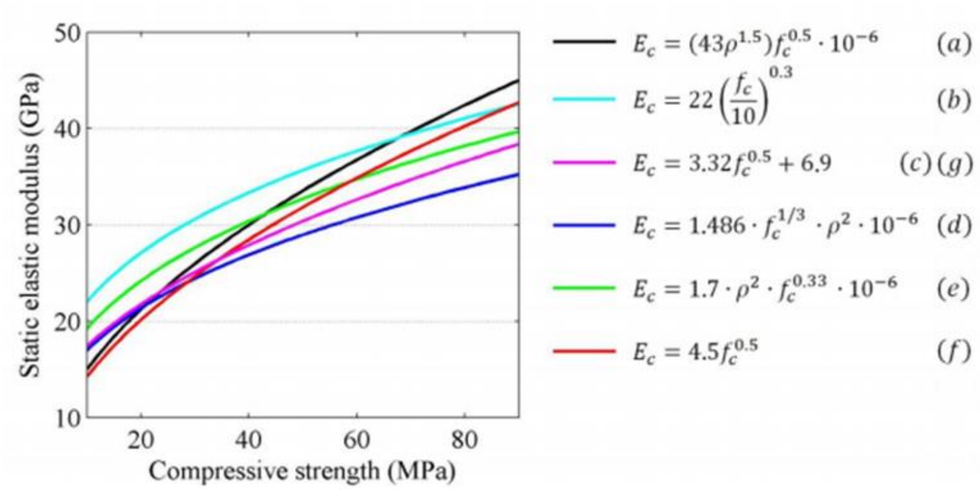


Figure 25: The chosen relationships between concrete compressive strength and elastic modulus: (a) ACI 318-95; (b) Euro code 2; (c) ACI 363; (d) Noguchi et al. (2009); (e) BS 8110-2:1985; (f) CSA A23.3-04; (g) NZS 3101-2006 (Jurowski & Grzeszczyk 2018)

2.4.2- Safety Factors

The term 'safety factor' refers to the additional carrying loads assumed for the structure as precautious step to ensure that the structure would withstand the loads assigned to. During the design phase of the structural system, the engineers use the safety factor to make sure the structure would stay stable in the overload cases. Assigning this factor reduces the risk that might happen such as failure of the structure and the harm that could happen to the occupants. Many design codes assume higher value for safety factor in the critical situations and when the failure possibility is high. The safety factor is mainly a measure of the materials safety factor and the loading safety factor. (Wilhite 2018)

Materials Safety Factors

Due to manufacture errors, the building material may not be exactly with the properties it's made for, some could be lower in strength handling which could lead to failure in many cases. The main structural material used worldwide is the mixture of concrete and steel "reinforced concrete" in which many of the design codes assign safety factor for both materials. Since concrete strength is an essential measure when it comes to structure stability and safety, the building design codes assign a test named ‘compressive strength test’, both ACI 318- 14 code and the Eurocode 2 use the cylinder compressive strength characteristic while the rest of the codes use the cube compressive strength characteristic. To calculate the safety factors the design strength and the characteristic strength should be used as per the following equation:

$$\textbf{Partial Material safety factor } (\boldsymbol{\gamma m}) = \frac{\textbf{Characteristic Strength of the material } (\mathbf{f})}{\textbf{Design Strength of the material } (\mathbf{fd})}$$

The design strength represents the maximum stress that the material could handle while the characteristic strength is the amount of strength in which more than 95% of the tests results are expected to pass. The materials safety factor should be more than 1, otherwise if it was less, then the structure is more likely to express failure or will cause no viability (Staff 2016). Table 10 shows the strength reduction factors of ACI 318-14, which is similar to ACI 318-19.

Table 10: Strength reduction factor form ACI318-19 ("Strength reduction factor Ø" 2019)

Stress Condition	**Φ Factors**
Flexural	0.90
Axial tension	0.90
Shear and Torsion	0.75
Compression member spirally reinforced	0.70
Compression member tied reinforced	0.65
Bearing	0.65

Loading safety factor

It's imperatively by most of the design codes to apply the loading safety factor in the load calculations, the load calculated is an additional load assumption to be added to the member load to eliminate the uncertainties in the design and reduce the possibility of any error, mistakes or extra loads occurring on members, this total load is named "Ultimate Design Loads".

The design load can be calculated by multiplication of the characteristic loads and the safety factor (γf), stated by several design codes (Scott, Kim & Salgado 2003).

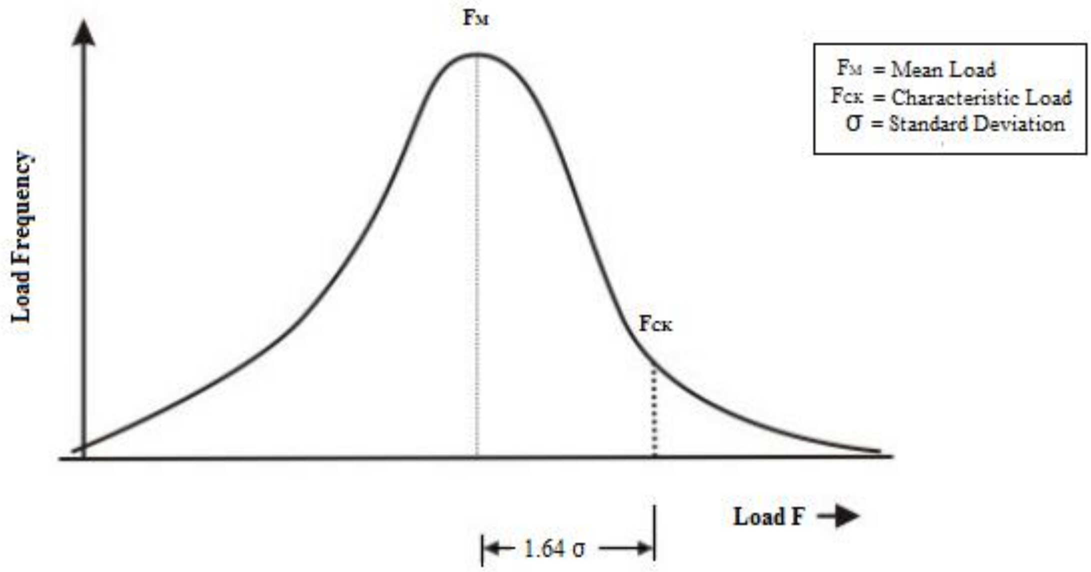


Figure 26: Characteristic load (Scott, Kim & Salgado 2003)

The characteristic values of loads are based on statistical data done on many experiments and studies related to materials load tolerance. As shown in figure above, 95% of cases the characteristic loads of structures did not exceed their limit nor failed during their life (Scott, Kim & Salgado 2003). However, in real life situations structures are exposed to overloading. Due to that, the structures of the building must be designed based on loads calculated from multiplying the characteristic loads with appropriate safety factors (γf) based on the load types applied on the member, and the limit condition being assigned to it.

The design loads can be determined by using the following equation:

Design load = Characteristic Load x Safety factor for load (γf)

The following table shows a comparison of the safety factors for the dead loads (DL) and the live loads (LL) according to ACI code and Eurocode.

Table 11: Load safety factor comparison (ACI Code & Eurocode)

Code	Dead Load (DL)	Live Load (LL)
ACI 318	1.20	1.60
Eurocode2	1.35	1.50

As concluded from the above table, the Eurocode2 considers lower live load safety factor compared to the ACI 318 code, but in case of the deadload the Eurocode2 considers higher value. Based on research and studies done on this topic, considering a high safety factor will result with a member that needs a higher shear force and design moments which leads to a need for a bigger section, and in many cases, this is considered to be uneconomical. The ACI 318 and Eurocode2 have relatively low dead load safety factor since their live load safety factor is considerably high. (Bakhoum, Mourad and Hassan, 1996)

2.4.3- Load Combinations

Loads can be defined as the forces that acts on the structure and cause its deformation. There are several types of loads impacting the buildings such as dead loads, live loads, snow load, rain load, seismic loads and wind loads. Live loads and dead loads have significant effect on the structure functionality since they are continuous load applied during its life time, while the wind loads and seismic loads are considered high forces that could cause the structure to collapse. Mentioning also the rain load and snow loads, these loads ae season related and have less effect compared to the previous mentioned loads.

The design codes are used to get an estimation of loads amount needed to be into consideration while design a building. The design codes use the maximum expected loads that the building would face and then give an estimation for the passing or failing structural elements which could be considered as a safety regulation. The codes give different way of calculation depending on the type of the load, and codes act differently in relation to building occupancy to ensure the structure safety under various expected loading and at most severe scenarios. The definition “loads combination’ refers to a group of loads applied on structure simultaneously or together and they must be considered in any building. These load combinations are varying from a code to another due to jurisdiction. ("Load Combinations of Concrete Design" n.d.)

The ACI 318-19 code follows the ASCE 7-16 in determining the loads combinations used in the building. The ASCE 7-16 code takes into account multiple load combinations.

Table 12: Load combinations for strength design (ASCE/SEI 7-16 2016)

Load Combinations for Strength Design
1.4D
1.2D + 1.6L + 0.5(L_r or S or R)
1.2D + 1.6(L_r or S or R) + (L or 0.5W)
1.2D ± 1.0W + 1.0L + 0.5(L_r or S or R)
1.2D ± 1.0E + 1.0L + 0.20S
0.90D ± 1.0W
0.90D ± 1.0E

While for the Eurocode2 the following are the load combinations used:

Table 13: Eurocode load combinations (Jawad, 2006)

Load combination	Permanent load		Variable load: Imposed		Variable load: Wind	Prestress
	Adverse	Beneficial	Adverse	Beneficial	Wind	Prestress
Load combination (6.10)						
Permanent + imposed	1.35	1.00	1.50	0	–	1.00
Permanent + wind	1.35	1.00	–	–	1.50	1.00
Permanent + imposed + wind	1.35	1.00	1.50	0	$1.50 \times 0.5 = 0.75$	1.00
Load combination (6.10a)						
Permanent + imposed	1.35	1.0	ψ_0^* 1.5	–	–	1.0
Permanent + wind	1.35	1.0	–	–	ψ_0 1.5 = 0.9	1.0
Permanent + imposed + wind	1.35	1.0	ψ_0^* 1.5	–	ψ_0 1.5 = 0.9	1.0
Load combination (6.10b)						
Permanent + imposed	ξ 1.35 = 1.25	ξ 1.0 = 0.925	1.5	–	–	1.0
Permanent + wind	ξ 1.35 = 1.25	ξ 1.0 = 0.925	–	–	1.5	1.0
Permanent + imposed + wind	ξ 1.35 = 1.25	ξ 1.0 = 0.925	1.5	–	ψ_0 1.5 = 0.9	1.0

(1) It is assumed that wind is not the leading action.
(2) ψ_0^* will vary with the use of the building.

As shown from the previous load combinations for the codes, both of them have different combinations and modifiers used, which will give different results for the seismic and wind assessment provisions.

CHAPTER 3: METHODLOGY

3.1- Overview and Data Analysis Method

Multiple research papers have been selected and analyzed according to their methodology which will be discussed shortly. Many codes comparisons have followed the simulation methodologies which provide fast data analysis, concise and precise results. The researchers have used different software for analysis and comparison such as ETABS and Midas for the superstructure design, RAM Concept and ADAPT Builder for post tension analysis and many other different software in their studies.

Most of the research papers concerning codes comparison prefer more the simulation methods in the evaluations, for example a research which is done by Dhanvijay, Telang and Nair that investigated and tested different codes in seismic assessment provisions, the performed study compared the Eurocode, IBC/ASCE and Indian code IS 1893:2002. The study has given a better understanding to the main factors that lead to the structure poor performance during an earthquake, thus they can determine how to achieve an adequate safe behaviour for buildings under future earthquakes. The analysis was made for a G+10 special reinforced concrete moment-resisting frame using STAAD pro. V8i software (Dhanvijay, Telang & Nair 2015). Such comparison is way easier to be done using a software, the data are more accurate and it saves time, which makes the simulation methodology the best route for such comparisons.

Different building codes have been used by many engineers worldwide to assist them to make safe and efficient structural designs. This paper conducts a study on lateral forces affecting a high-rise building and the cost difference of the structural elements. The lateral forces considered are seismic and wind forces. It compares and discusses the different seismic and wind assessment provisions and criteria of two famous building codes that are being used worldwide which are ACI code and Eurocode. For the seismic effects the following criteria's will be compared, base shear, storey force, storey drift, torsional irregularity, P-Delta effect, response spectrum, mass participation and structural time period. All the previous mentioned criteria will be determined and extracted using the ETABS software. For the wind comparison, it will consist of the storey drift effect which will extracted from ETABS as well.

After finalizing the model and comparing the seismic and wind provisions between the codes, another type of comparison will be done which is the cost comparison. This comparison is based on the reinforcement amount being used in the vertical elements and the slabs. For columns and shear walls the data will be obtained from the ETABS model, while for the slab design the data will be obtained from the RAM concept models to compare the amount of post-tension and reinforcement required.

3.1.1- List of Research Variables

Multiple variables can affect the results of the design analysis and they are limited in this research into the following:

Table 14: Research variables list

Controlled Variables (Variables that can change)	Fixed Variables	Calculated/Measured Variables
Live loads value	Seismic parameters in the site location	Wind storey drift
Super imposed dead loads value	Wind parameters in the site Location	Seismic storey drift
Initial beams sizing	Number of stories	Storey force
Slabs thickness	Areas of the plans	Torsional irregularity
Initial vertical elements sizing (Columns and Walls)	Plans shape	P-delta effect
Number of vertical elements in the building		Seismic base shear
Storey height		Response spectrum
Concrete strength		Mass participation
Steel strength		Structural time period
Post tensioning strand sizing		Final structural elements sizing
Number of strands in tendons		Cost of structural Elements

The controlled variables are considered as the variables that can be freely changed based on the designer and they are the magnitudes of live loads and super imposed dead loads, number and sizing of vertical elements and beams, slabs thickness, materials strength and each storey height. These previous variables may be changed based on the results obtained from the analysis at the initial stage which most likely will have some failures that makes it necessary to modify the building accordingly.

The fixed variables are considered as variables that cannot be changed. Some are related to the building geometry such as number of stories, plans area and the plan shape which should be fixed in reality, while other variables are related to the building location such as seismic parameters and wind parameters at the selected location.

The Measured/Calculated Variables are the variables and provisions that will be compared between the codes and they are classified mainly into three main comparisons, and they are seismic effect study, wind effect study and cost difference study.

For seismic comparison it will include base shear, storey force, storey drift, torsional irregularity, P-Delta effect, response spectrum, mass participation and structural time period. For wind comparison it will include wind storey drift. While for the cost, the comparison will be based on the final structural elements (columns, shear walls and slabs) reinforcement amount used for both codes.

3.1.2- List of Assumptions

1- The building's location will be in Dubai, United Arab Emirates at the following coordinates (25.18°N, 55.27°E).

2- The analysis will be performed on a 50-storey high rise building.

3- Lateral forces affecting the building are seismic loads and wind loads only, other types of lateral loads won't be considered in the analysis.

4- The concrete strengths of columns and walls are varying being higher at the bottom floors and decreasing is strength for upper floors which implies a real-life situation and they are Fcu = 60 Mpa for the first 23 floors and Fcu = 50 Mpa for the other floors in the building.

5- The concrete strengths of slabs are varying and they are Fcu = 45 Mpa for the first 15 floors while it is Fcu = 40 Mpa for the rest of the upper floor's slabs in the building. Having a ratio of vertical Elements concrete strength/ slabs concrete strength not more than 1.40 in order to avoid punching problems in the slab.

6- The steel yield strength used is 460 Mpa with modulus of 200,000 Mpa.

7- The types of slabs used will be post tension slabs.

8- The strands grade used are Grade 270 [1860].

9- The strands that will be used will have diameter of 12.7 mm and modulus of elasticity of 195,000 Mpa, since it is used widely in the UAE.

10- For parking areas, the building won't consist of any basement; it will consist of podiums instead.

11- The heights of columns and walls will be 5.00 meters from the ground floor to the first floor, 3.00 meters for all podium floors, while it's 3.60 meters for the typical floors.

12- No swimming pool will be considered in the design analysis for simplicity purposes.

13- Wind tunnel won't be considered since it requires laboratory tests.

3.2- Dubai Site Analysis Study

In this section the historical data for the site location will be discussed in order to investigate more about the possible behaviour of the structure against seismic loads and wind loads in that area. The seismic data were obtained from Dubai Municipality website while the wind data were obtained from WindAlert website.

3.2.1- Seismic Data Analysis in Dubai - UAE

The seismic activity in the United Arab Emirates is considered to be very low. Through its history, there were no indication or sign for any huge seismic activity or a catastrophic earthquake. Ambraseyes has published the seismicity history for the country and surroundings in the periods between 1899-1963 as shown in figure 27, and ISC Bulletin has published the seismicity history in the periods between 1964-1973 as shown in figure 28. At the previous periods, it can be noticed that most of the earthquake events are occurring in the southern part of Iran along the plate boundary. The USGC has published the seismicity history in the periods 1973-2006 as shown in figure 29, it shows one earthquake that occurred in Masafi which is in northern UAE, with a magnitude around 5. ("Seismicity of United Arab Emirates" n.d.)

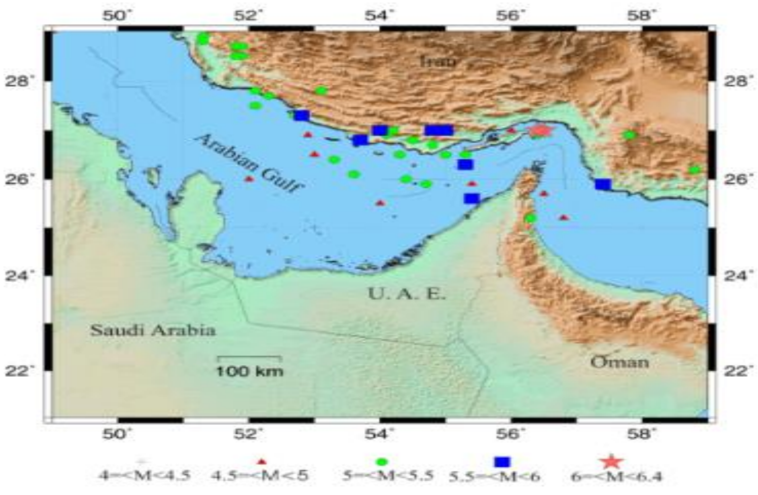


Figure 27: Seismicity in & around UAE during the period from 1899-1963. ("Dubai Municipality" 2021)

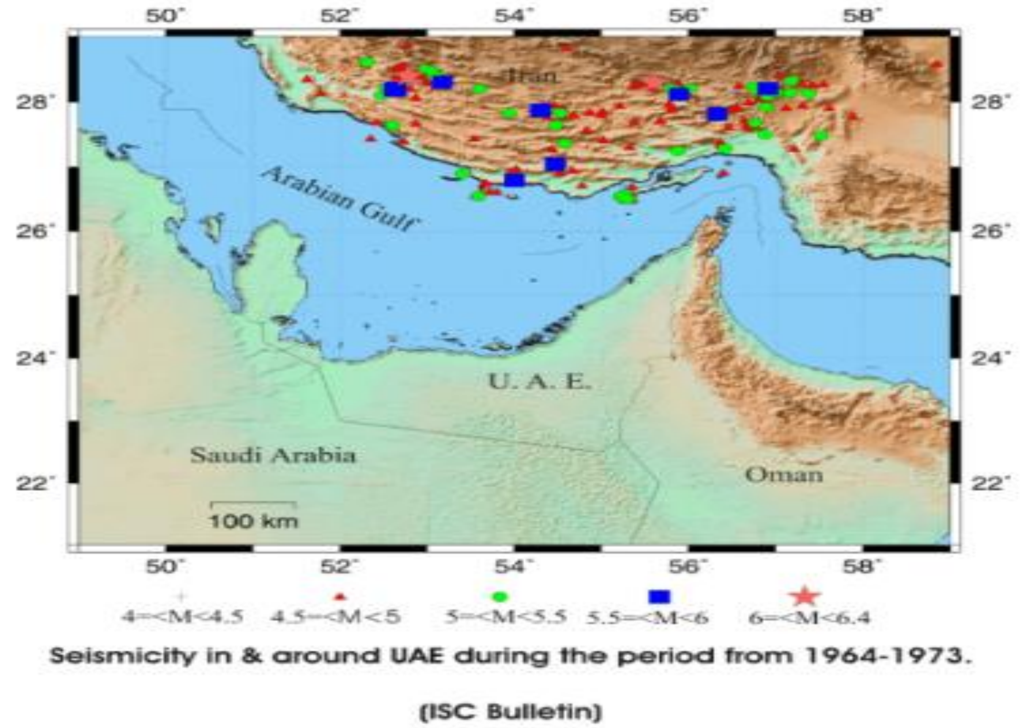


Figure 28: Seismicity in & around UAE during the period from 1964-1973. ("Dubai Municipality" 2021)

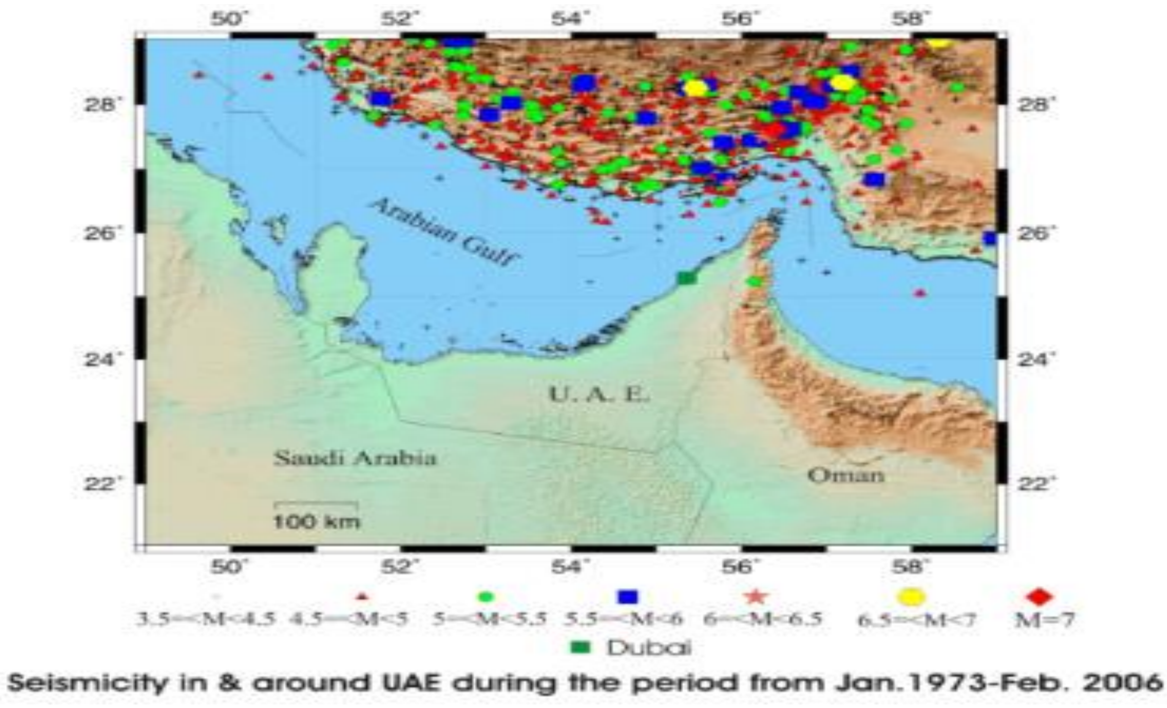


Figure 29: Seismicity in & around UAE during the period from Jan. 1973-Feb. 2006 ("Dubai Municipality" 2021)

A country like UAE is undergoing huge construction development in high-rise buildings making it the main feature, which means a strong earthquake can cause significant economic loss for major cities. The seismic activity in the UAE has been recognized to be one of the lowest in the world, the earthquakes occurred in the past are considered to be moderate. The

UAE location is at the eastern coast of the Arabian Peninsula, facing the subduction boundary across the Arabian Gulf water, and the country lies opposite the Hurmuz Straits at the tip as shown in figure 30. (Barakat, Shanableh & Malkawi 2008)

Figure 30: Location of the United Arab Emirates within its region, (Google Earth, Downloaded January, 2021)

The design of high-rise buildings is considered a major challenge due to the seismic activities which might cause large earthquakes. These earthquakes might cause high force distribution on the structure that must be considered and calculated carefully in the design. (Sigbjornsson and Elnashai 2006)

Since Dubai is not at a fault line, the risk of earthquake is considered to be very low, but this doesn't necessarily mean that Dubai residence don't feel the earthquakes. UAE is close to Iran, and Iran is known of having large earthquakes; and though when an earthquake effect reaches Dubai, the people typically feel a mild to moderate shaking, same goes for northern emirates. The earthquakes tend to be felt more strongly in Dubai's high-rise buildings due to the long heights. In April 2013, a strong earthquake of magnitude of 7.8 hit the Iran-Pakistan border, this earthquake has transferred with lower force and its effect have reached Dubai, its effect made thousands of people to evacuate the high-rise buildings as a precaution, this was the biggest earthquakes to be felt in Dubai, but it didn't cause any damage. According to this event, the Dubai municipality have updated its requirements, stating that any building with more than 10 storeys should be able to withstand an earthquake of 5.9 magnitude. Previously, the towers were designed to withstand moderate earthquakes of magnitudes between 5.0 and 5.5. (Downes 2017)

3.2.2- Wind Data Analysis in Dubai - UAE

The effects of wind loads are considered as one of the critical loads that must be considered in the high-rise building design. High-rise buildings must be built to withstand the wind loads during its service life. The municipality in Dubai states that structures should withstand a basic wind speed of 38 meters per second for a three-second gust, which is equivalent to 137km/h (85 mi/h).

The following are some data collected from the "WindAlert" website, which gives average weather source of different locations at different time spans; and in this analysis it was considered the time period from 2007 to 2020 in Dubai.

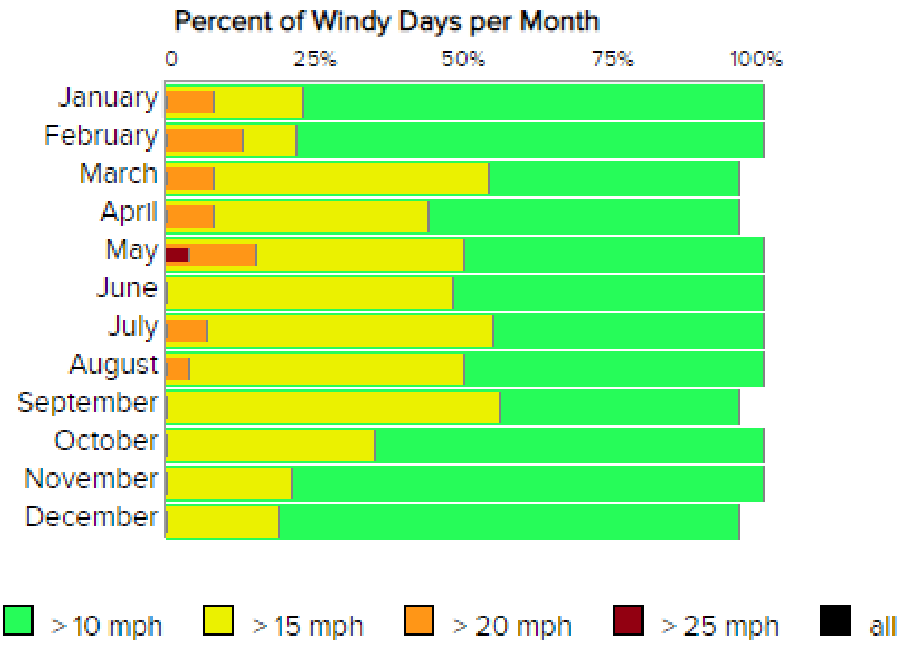


Figure 31: Maximum average wind speed for different months from the periods 2007 to 2020 ("WindAlert" 2020)

As shown from the previous data the maximum average wind speed for each month from the period 2007 to 2020 is slightly more than 25 miles/hour in May for a small-time duration while for the rest they are slightly more than 20 miles/hour during small time period, and the vast majority is between 10 miles/hour and 15 miles/hour. These average wind speeds are way less than the wind speed considered for high-rise buildings which is 85 miles/hour. This gives a good indication about the high resistance of the high-rise buildings in Dubai against the wind effects.

The following data shows the wind direction distribution for the previous data at each month, from year 2007 to 2020.

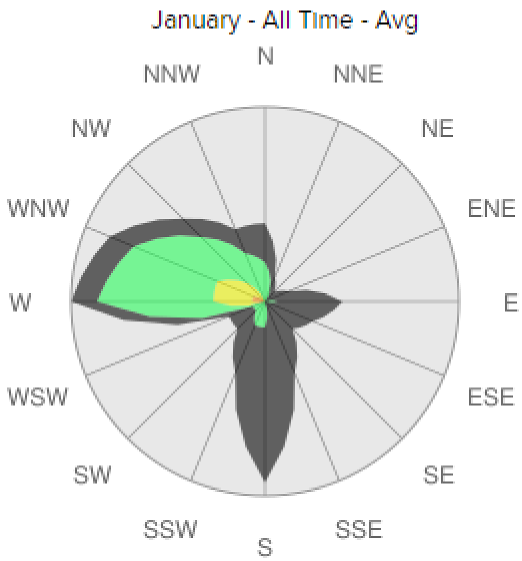


Figure 33: Wind speed average in January (2007 to 2020) ("WindAlert" 2020)

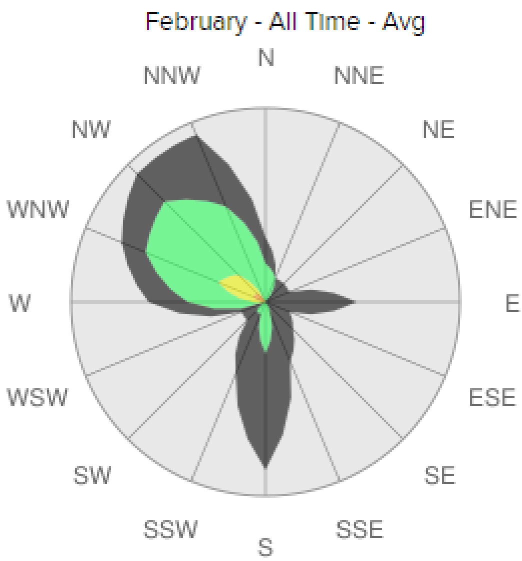


Figure 32: Figure: Wind speed average in February (2007 to 2020) ("WindAlert" 2020)

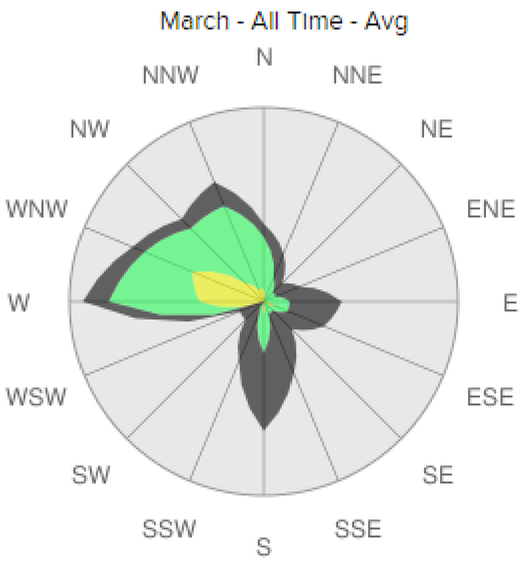


Figure 35: Wind speed average in March (2007 to 2020) ("WindAlert" 2020)

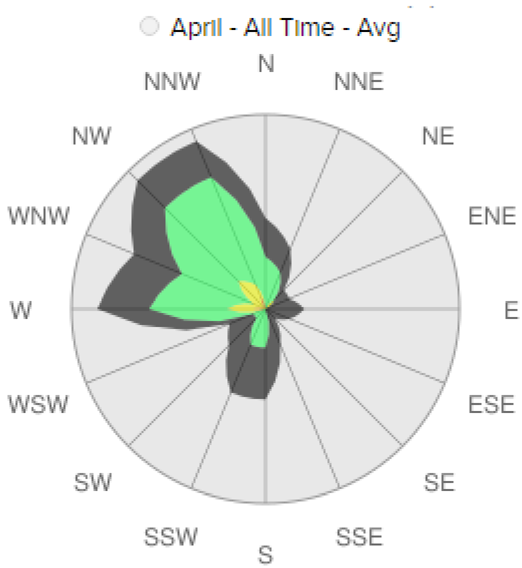


Figure 34: Wind speed average in April (2007 to 2020) ("WindAlert" 2020)

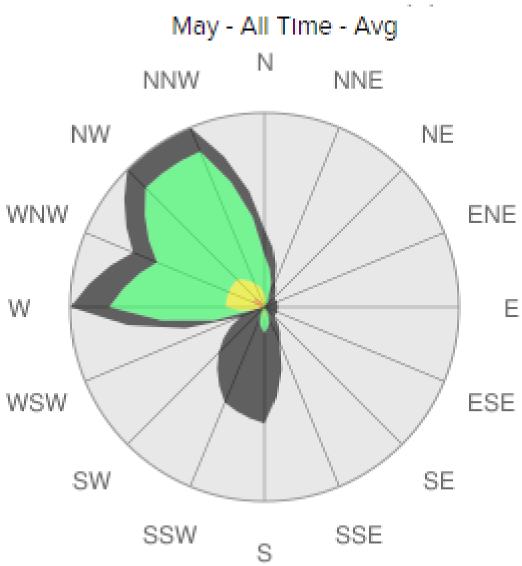


Figure 37: Wind speed average in May (2007 to 2020) ("WindAlert" 2020)

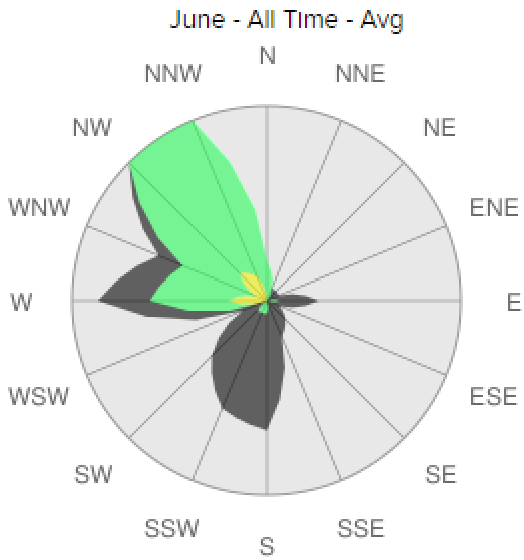


Figure 36: Wind speed average in June (2007 to 2020) ("WindAlert" 2020)

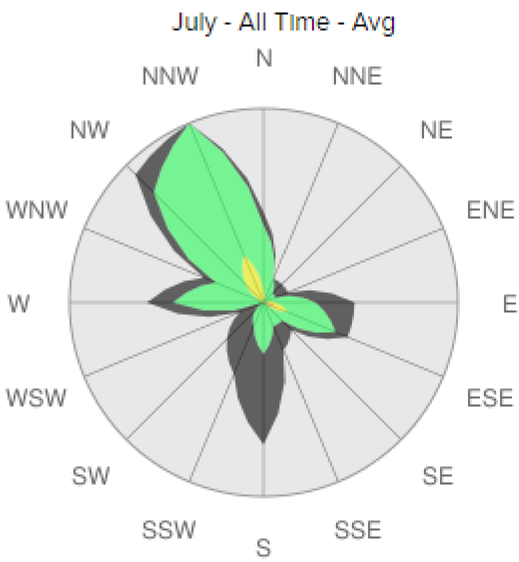


Figure 39: Wind speed average in July (2007 to 2020) ("WindAlert" 2020)

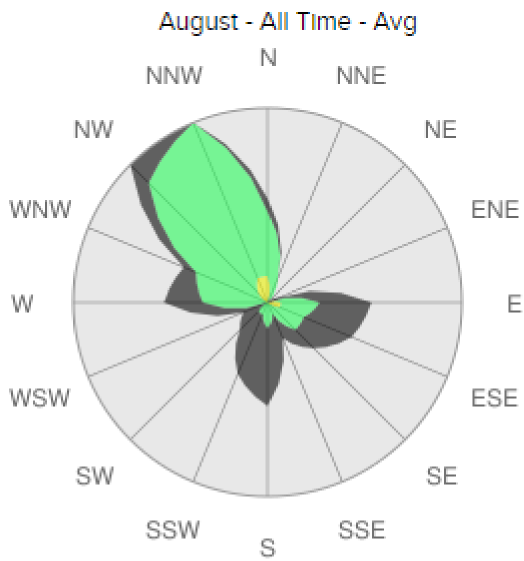


Figure 38: Wind speed average in August (2007 to 2020) ("WindAlert" 2020)

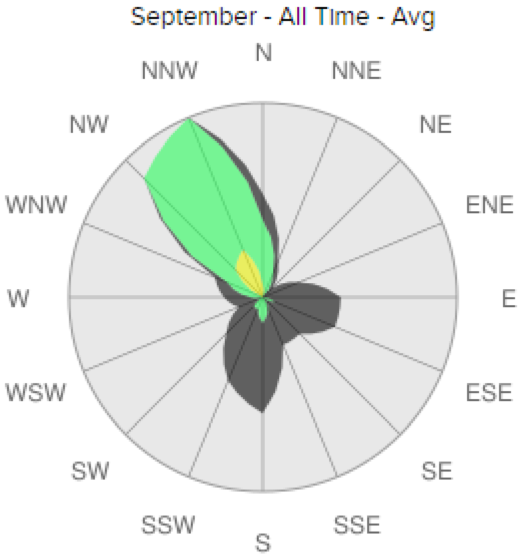


Figure 40: Wind speed average in September (2007 to 2020) ("WindAlert" 2020)

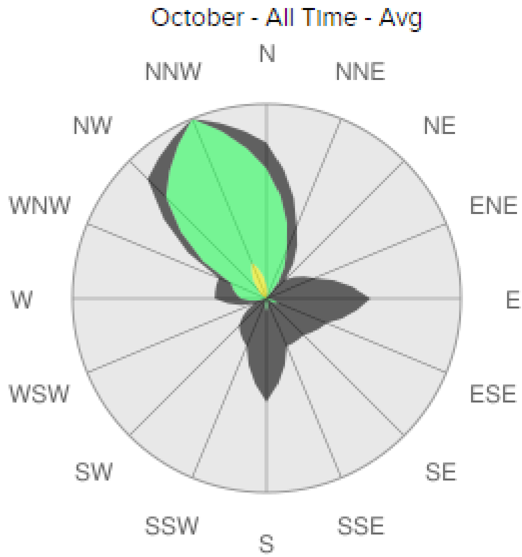


Figure 41: Wind speed average in October (2007 to 2020) ("WindAlert" 2020)

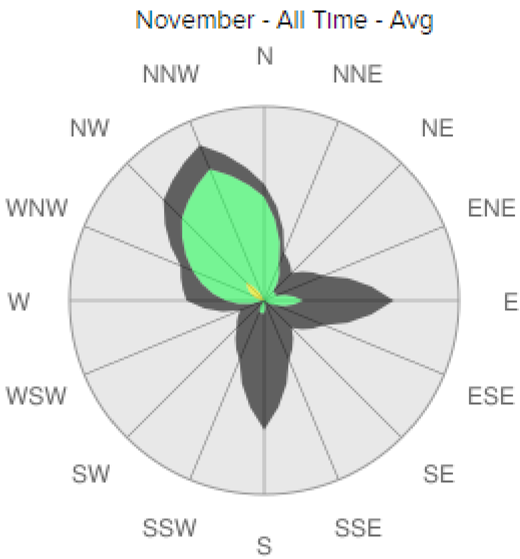


Figure 43: Wind speed average in November (2007 to 2020) ("WindAlert" 2020)

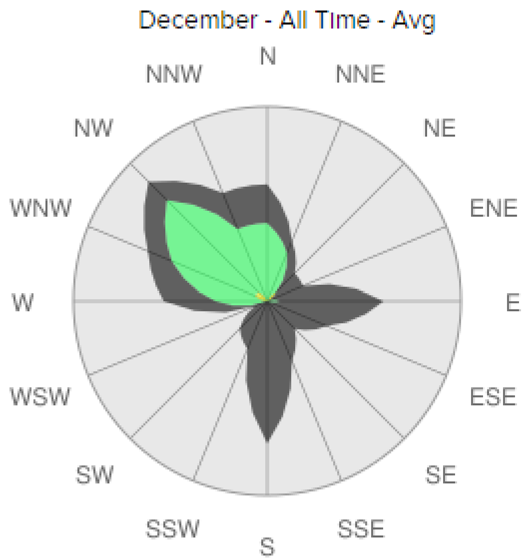


Figure 42: Wind speed average in December (2007 to 2020) ("WindAlert" 2020)

3.3- Seismic Parameters Determination

3.3.1- Seismic Parameters Determination for ACI 318-19/ASCE7-16 codes

In the United Arab Emirates, the main code used in the design analysis is the ACI code, and when it comes to seismic and wind assignment in the modelling design, the procedure followed in the analysis is related to the American Society of Civil Engineers (ASCE) as it is referred by the ACI code as shown in figure 97 in appendix B.

For the seismic loading based on the ASCE 7-16, there are some parameters that need to be determined first, in order to proceed for the comparisons stated in the previous chapters. The main parameters that need to be determined are the seismic coefficients, seismic factors and the approximate period parameters. For the seismic coefficients, there are two important parameters which are response spectral acceleration parameter at 0.2 seconds period (Ss) and response spectral acceleration parameter at 1 second period (S1) where both are dependent on the location of the building; other coefficient will be mentioned in details below. Moreover, the site class should be determined to proceed in the seismic load's analysis. For the seismic factors, the main factors are the response modification (R), system over strength, deflection amplification (Cd) and the occupancy importance (I). Last parameter to consider is the approximate period parameters Ct and x of the building.

The following will explain the procedure and calculations method for the building designed in Dubai, United Arab Emirates based on the requirements and notes provided in the ACI 318-19/ASCE7-16 codes.

For determining response spectral acceleration parameter at 0.2 seconds period (S_s) and response spectral acceleration parameter at 1 second period (S_1), the building's location should be determined first and later by following figures from the ASCE 7-16 code, the values of S_1 and S_s can be determined easily. The figures/maps (figure 22-1 up to figure 22-8) in the ASCE 7-16 code were prepared by the United States Geological Survey (USGS) with the cooperation of Building Seismic Safety Council (BSSC) Provisions Update Committee and the American Society of Civil Engineers Seismic Subcommittee and have been updated for the ASCE 7-16 standard.

The assumed building location is in Business Bay, Dubai, United Arab Emirates as shown below:

Figure 44: A Satellite image for the Business Bay Area in Dubai ("Google Maps" 2020)

The following is a closer view for the area with the exact location of the building:

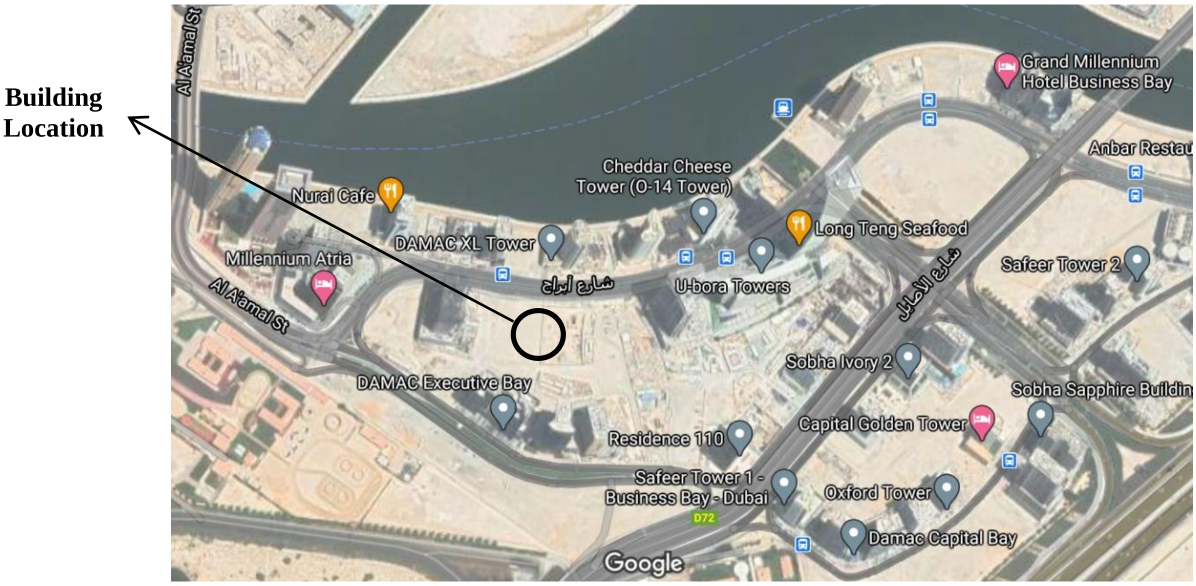


Figure 45: A closer satellite image for the business bay area in Dubai ("Google Maps" 2020)

One of the first steps to be done in the ASCE 7-16 code is to determine the seismic values of the two following parameters S_s and S_1. These two values depend on the location of the building being studied, and on the soil type according to the soil report. The maps available in the code (From figure 22-1 up to figure 22-8) are for Conterminous United States, Alaska, Hawaii, Puerto Rico, Guam, the Northern Mariana Islands, and American Samoa, so in order to determine these two values in the specified location for this analysis, the values of S_s and S_1 have been taken from the Dubai municipality recommendation. It recommends to use an average value stated by them for S_s and S_1 based on multiple tests conducted for the soil in Dubai, thus the values were taken in the design analysis as shown in table 15.

The location of the building must be defined; the coordinates are as the following:

25.18º N   55.27º E

Ss parameter from the code is the maximum mapped earthquake ground motion considered, with damping of 5 percent for a response spectral acceleration parameter at a period equal to 0.2 second, and it was considered as **0.51g**.

S_1 parameter from the code is the maximum mapped earthquake ground motion considered, with damping of 5 percent for a response spectral acceleration parameter at a period of 1 second, and it was considered as **0.18g**.

Both the values of S_s and S_1 were obtained based on Dubai municipality recommendation as shown below:

Table 15: Dubai municipality recommended values for Ss and S1 ("Dubai Municipality - Document" 2021)

PGA (g)	S_S (g)	S_1 (g)	T_L (s)
0.2	0.51	0.18	24

After determining S_s and S_1 parameters it is required to determine the site class by considering the soil properties in the site. The classification is one of six categories according to section 20.3-1 from ASCE 7-16 code, as shown below form the code.

Table 16: Site classification according to ASCE 7-16 code

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50 blows/ft	>2,000 lb/ft^2
D. Stiff soil	600 to 1,200 ft/s	15 to 50 blows/ft	1,000 to 2,000 lb/ft^2
E. Soft clay soil	<600 ft/s	<15 blows/ft	<1,000 lb/ft^2
	Any profile with more than 10 ft of soil that has the following characteristics: — Plasticity index $PI > 20$, — Moisture content $w \geq 40\%$, — Undrained shear strength $\bar{s}_u < 500$ lb/ft^2		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

Note: For SI: 1 ft = 0.3048 m; 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.

The site class considered is **C**, since the soil is a very dense soil.

By defining the site class, the site coefficients F_a and F_v are required. The parameters Ss and S_1 along with the site class determined above are used to obtain the adjustment factors which are required to obtain the maximum earthquake response spectral acceleration considered parameters, S_{MS} and S_{M1}.

By using both tables 11.4-1 and 11.4-2 from the code:

Table 17: Site coefficient values Fa from ASCE 7-16 code

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

Table 18: Values of site coefficient Fv from ASCE 7-16 code

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.
[a]Also, see requirements for site-specific ground motions in Section 11.4.8.

The F_a and F_v values are:

- F_a value (using interpolation): $\frac{1.3-1.2}{0.50-0.75} = \frac{1.3-Fa}{0.50-0.51}$ **Fa = 1.296**

- F_v value (using interpolation): $\frac{1.5-1.5}{0.10-0.20} = \frac{1.5-Fv}{0.10-0.15}$ **Fv = 1.50**

Based on the previous data the values of the maximum earthquake spectral response acceleration considered parameters S_{MS} and S_{M1} can be determined. The purpose is to use the site coefficients in order to adjust the mapped acceleration parameters for site effects. The coefficients are obtained by applying equations 11.4-1 and 11.4-2 from the ASCE 7-16, and they are as the following:

- S_{MS} = Fa x Ss =1.296 x 0.51g **S_{MS} = 0.661g**
- S_{M1} = Fv x S_1 =1.50 x 0.18g **S_{M1} = 0.27g**

After obtaining the S_{MS} and S_{M1} parameters, the design earthquake response spectral acceleration parameter at short period, S_{DS} and S_{D1} can be determined according to equations 11.4-3 and 11.4-4 respectively for the ASCE 7-16.

- $S_{DS} = \frac{2}{3} \times S_{MS} = \frac{2}{3} \times 0.661g$ **S_{DS} = 0.4406g**

- $S_{D1} = \frac{2}{3} \times S_{M1} = \frac{2}{3} \times 0.270g$ **S_{D1} = 0.18g**

After calculating the previous parameters, the building's risk category is obtained by using table 1.5-1 from the ASCE 7-16. The main purpose from getting the risk category is to know the structural failure based on the occupancy. According to table 19, the risk category has been found to be **(Risk Category II)**

Table 19: Building and other structures risk categories ASCE 7-16 code

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	$\boxed{\text{II}}$
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

According to the risk category obtained previously, the seismic importance factor (I_e) is determined. It is used to increase the safety margins against any possible collapse. As the risk category changes from low to high for structures, the importance factor increases, since because they are directly proportional. The seismic importance factor is obtained as shown in table 20, and it was found as: $\mathbf{I_e = 1.0}$

Table 20: Seismic importance factor from ASCE 7-16 code

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	$\boxed{1.00}$	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Based on risk category and design earthquake response spectral acceleration parameters S_{DS} and S_{D1}, the seismic design category can be obtained. It is obtained according to sections 11.6-1 and 11.6-2 from ASCE 7-16 code, as shown in tables 21 and 22. The seismic design categories have been found to be as the following:

- Seismic design category response acceleration based on short period = **C**
- Seismic design category response acceleration based on 1 second period = **C**

Table 21: Short period seismic design category from ASCE 7-16 code

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \le S_{DS} < 0.33$	B	C
$0.33 \le S_{DS} < 0.50$	C	D
$0.50 \le S_{DS}$	D	D

Table 22: 1 Second period seismic design category from ASCE 7-16 code

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \le S_{D1} < 0.133$	B	C
$0.133 \le S_{D1} < 0.20$	C	D
$0.20 \le S_{D1}$	D	D

Then the response modification coefficient or the R factor should be obtained. According to the code it takes in to account the structure's stiffness and ductility, the more the ductility of the building the better it performs during a seismic activity since it's related to the performance of dissipating the energy. As it can be noticed from table 85, the ductile buildings are given higher value of response modification coefficient. In this analysis, the value was determined from Table 12.2-1 in the ASCE 7-16 code, the value depend on the bearing wall system and it was found to be equal to: **R = 4**

Other important factors that need to be determined are the system over strength (Ω_0) and the deflection amplification (Cd). These two parameters are obtained from Table 12.2-1 in the ASCE 7-16 code, and their values are dependent on the bearing wall system, and the type that will be used in the design is ordinary reinforced concrete shear walls, thus the values of over strength (Ω_0) and the deflection amplification are as the following (Table 85 in appendix B):

$$\mathbf{\Omega_0 = 2.50} \qquad\qquad \mathbf{Cd = 4}$$

Based on section 12.8-2 from the ASCE 7-16 code, the approximate fundamental period parameters value of the building $\mathbf{C_t}$ and **X** can be obtained as shown below:

Table 23: Approximate period parameters Ct & x values according to ASCE 7-16 code

Table 12.8-2 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	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

[a]Metric equivalents are shown in parentheses.

$$\mathbf{C_t = 0.0488} \qquad\qquad \mathbf{X = 0.75}$$

Last thing to be determined is the Long Transition Period, T_L. The value of the long transition period can be obtained directly from the Dubai Municipality code as shown in the table below:

Table 24: Dubai municipality values for TL ("Dubai Municipality - Document" 2021)

PGA (g)	S_S (g)	S_1 (g)	T_L (s)
0.2	0.51	0.18	24

3.3.2- Seismic Parameters Determination for Eurocode

The Eurocode comprises of many different standards and each standard is generally consisting of number of parts, each part has its own sections. The Eurocode standard related to the earthquake is Eurocode 8 which will be stated in this section to be used in the analysis later. Eurocode 8 is used to design the structures for earthquake resistance. The obtained data from this section will be used in the ETABS model to proceed in the design analysis.

For the seismic loadings based on the Eurocode 8, there are some parameters that need to be determined first in order to proceed for the comparisons stated in the Chapter 2. The main parameters that need to be determined are the ground acceleration (ag/g), type of spectrum, type of the ground, soil factor (S), periods of the spectrum Tb, Tc and Td, Lower bound factor (Beta), correction factor (Lambda) and the behaviour factor (q). The procedure of obtaining these parameters will be discussed in details.

Based on the requirements and notes provided in the Eurocode 8 and on the location specified for the building the spectral acceleration can be determined by knowing the seismic zone. The ground acceleration has been taken as 0.10g based on previous studies conducted.

Then it is required to determine the site class by taking into account the soil properties for the site, it is classified into one of the categories mentioned in table 3.1 in Eurocode as shown in table 25.

Table 25: Ground Types from the Eurocode 8

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Since the soil is a very dense soil, then the site class is **B**.

To get the values of the soil factor and the periods T_b, T_c and T_d, the type of spectra should be known. There are a total of two choices to be considered for the spectra, and they are Type 1 and Type 2. In order to differentiate between both of them, if the earthquakes have a magnitude of surface-wave that is less than 5.5, the recommended spectra type is type 2 spectrum, otherwise the spectra taken is type 1. For the five ground types the parameters values S, T_B, T_C and T_D are shown in table 3.2 from the Eurocode for the type 1 spectrum, while table 3.3 from the Eurocode shows the values for type 2 Spectrum as shown below.

Table 26: Values of the parameters describing the recommended type 1 elastic response spectra from Eurocode 8

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 27: Values of the parameters describing the recommended type 2 elastic response spectra from Eurocode 8

Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

After determining the ground type and the recommended type of the elastic response spectra, the following table 3.3 from the Eurocode will be used in order to determine the soil factor, and spectrum periods T_b, T_c and T_d, since the magnitude of surface-wave in Dubai is less than 5.5, the soil factor and periods T_b, T_c and T_d are as the following:

Table 28: Values of the parameters describing the recommended type 2 elastic response spectra from Eurocode 8

Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

Based on the previous table the following parameters values are as the following:

- Soil factor (S) = 1.35
- Spectrum period (Tb) = 0.05
- Spectrum period (Tc) = 0.25
- Spectrum period (Td) = 1.20

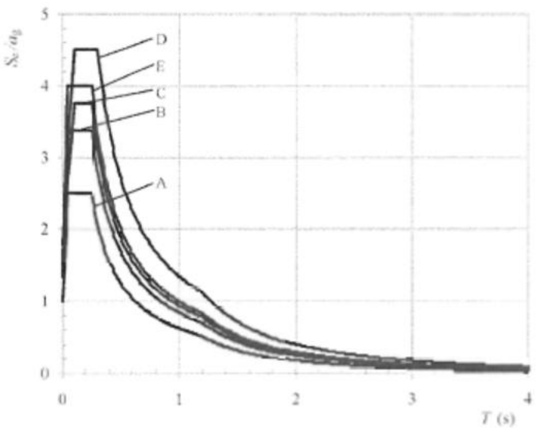


Figure 3.3: Recommended Type 2 elastic response spectra for ground types A to E (5% damping)

Figure 46: Recommended Type 2 Elastic response spectra for ground types A to E (5% damping) from Eurocode 8

Next is determining the correction factor since it’s required for the base shear analysis. As mentioned in the code, if the building has more than two storeys then the correction factor should be taken as 1. As shown in the figure 93 in appendix B.

After determining the correction factor, the last two parameters to be determined are the lower bound factor and the behavior factor. In order to avoid any explicit inelastic structural design analysis, the structure capacity to dissipate the energy into mainly ductility behavior of the elements and other mechanisms is taken into consideration by applying an elastic analysis. This is achieved by introducing the behavior factor (q), it is basically an approximation of the ratio of the structure seismic forces that it may experience taking into account complete elastic response with 5% viscous damping to the design seismic forces that is used in the design.

Those two parameters along with some parameters obtained previously in this section are used for calculating the horizontal components of the seismic action in the design spectrum. The equations are shown in figure 94 in appendix B.

According to the code the lower bound factor for the horizontal design spectrum can be taken as 0.20 as shown in figure 95 in appendix B.

While for the behavior factor it is taken as 1.50, based on the value provided in section 5.3.3 from the code, as shown in figure 96 in appendix B.

3.4- Wind Parameters Determination

As mentioned in section 3.3.1, the ACI 318-19 code refer to the ASCE 7-16 code when it comes to seismic and wind assignment in the modelling design, thus the procedure followed in the wind analysis is based on chapter 26 form the ASCE 7-16 code which refers to the wind loads as shown in figure 111 in Appendix B.

For the wind loading based on the ASCE 7-16, there are some parameters that need to be determined first in order to proceed for the comparisons stated in chapter 2. The main parameters that need to be determined are the wind coefficients, exposure height and wind exposure parameters. For the wind coefficients, the parameters required for the analysis are wind speed, exposure type, ground elevation factor, topographical factor, gust factor and directionality factor. For the exposure height, it basically refers to the height limits which the wind load will be applied on the building from the ground floor up to the last floor which was considered in this analysis. On the other hand, for the wind exposure parameters, they can be obtained directly from figure 27.3-8 from the ASCE 7-16 code.

The following procedure and calculations followed in this section is for the building designed in the Dubai, United Arab Emirates based on the requirements and notes provided in the ACI 318-19/ASCE7-16 code.

The wind load in most of the cases is considered as a horizontal load caused due to the air movement relatively to the earth. Wind load must be taken into account for the building analysis and design, especially when the height of the building is high which is the case for the considered high-rise building. The horizontal forces applied by the wind's components must be considered while designing the building. The wind loads calculation majorly depends on two factors and they are wind velocity and building size.

3.4.1- Wind Parameters Determination for ACI 318-19/ASCE7-16 code

One of the first steps that are followed for wind load analysis is determining the wind speed. The wind speed can be different based on the location, and as mentioned in the previous sections, the building location will be in Dubai. According to Dubai Municipality the wind speed based on their recommendation can be taken as 85 mile/hour which equal to 137 kilometer/hour (38 meters/second), as shown below:

Table 29: Wind speed for strength deign based on Dubai municipality recommendation ("Dubai Municipality - Document" 2021)

سرعات الرياح التصميمية (Strength Design)

Mean Recurrence Interval (Years)	Wind Speed (m/s, 3-sec gust, 10 m, open terrain)	
	Strength	Load Factor
50[II]	38	1.6*
100[III,IV]	40	1.6*
300[I]	44	1**
700[II]	47	1**
1000	49	1**
1700[III]	51	1**
3000[IV]	53	1**

After getting the wind speed for the site, some coefficients and parameters are required for determining the wind loads on the building as will be shown in this section.

One of the first steps is to get the risk category of the building, which is already obtained in section 3.3.1. It was found to be **II**.

Table 30: Building and other structures risk categories from ASCE 7-16 code

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

Then the building's exposure type should be determined. For each considered direction for the wind, the upwind exposure should be according to the roughness of the ground surface that is determined from constructed facilities, natural topography and vegetation.

Based on section 26.7.2 from the ASCE 7-16 code, the surface roughness that should be considered is B, since the location of the building is in an urban area as shown in figure 98 in appendix B, while for checking the exposure category, section 26.7.3 from the ASCE 7-16 is followed, as shown in figure 99. Since both Exposures B and D are not fulfilling the requirements, then Exposure C is followed in the design.

Then determining the wind directionality factor which has some effects in measuring the wind loads. It is obtained directly from table 26.6.1 in the ASCE 7-16 code.

Table 31: Wind directionality factor, Kd from ASCE 7-16 code

Table 26.6-1 Wind Directionality Factor, K_d

Structure Type	Directionality Factor K_d
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Circular Domes	1.0[a]
Chimneys, Tanks, and Similar Structures	
Square	0.90
Hexagonal	0.95
Octagonal	1.0[a]
Round	1.0[a]
Solid Freestanding Walls, Roof Top Equipment, and Solid Freestanding and Attached Signs	0.85
Open Signs and Single-Plane Open Frames	0.85
Trussed Towers	
Triangular, square, or rectangular	0.85
All other cross sections	0.95

[a]Directionality factor $K_d = 0.95$ shall be permitted for round or octagonal structures with nonaxisymmetric structural systems.

Since the structure type is a building then the directionality factor is taken as 0.85.

Next in the analysis is to determine the topographical factor (Kzt), which is dependent on the topography of the area the building will be constructed at. Based on section 26.8.1 which is mainly discussing the hill height near the building and its effect, and since there is no hill in the area the building being designed at, the topographical factor will be taken as 1 based on section 26.8.2, as shown in figure 100.

Next is to determine the ground elevation factor, which is dependent on the height of the building above the sea level, and since the structure is assumed to be at the mean sea level, then the ground elevation factor will be taken as 1.

Table 32: Ground elevation factor, Ke from ASCE 7-16 code

Table 26.9-1 Ground Elevation Factor, K_e

Ground Elevation above Sea Level		Ground Elevation Factor K_e
ft	m	
<0	<0	See note 2
0	0	$\boxed{1.00}$
1,000	305	0.96
2,000	610	0.93
3,000	914	0.90
4,000	1,219	0.86
5,000	1,524	0.83
6,000	1,829	0.80
>6,000	>1,829	See note 2

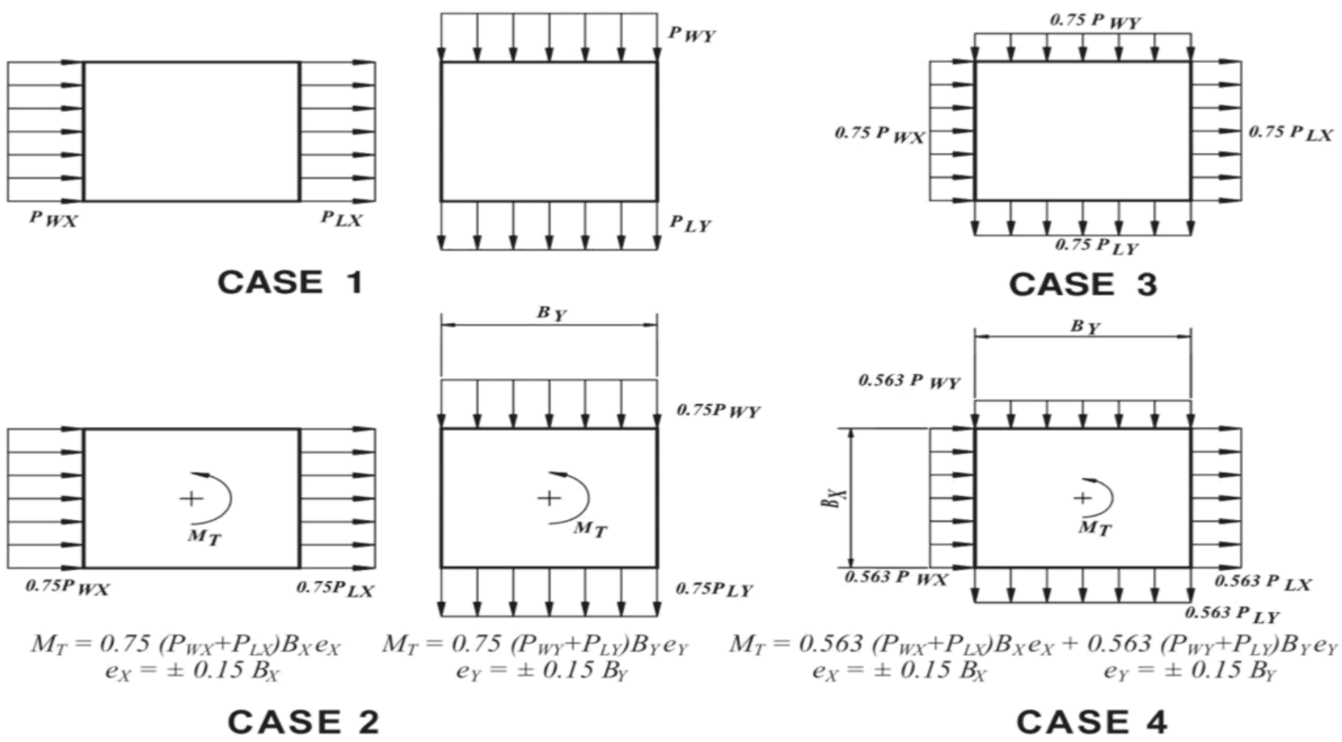
Notes

1. The conservative approximation $K_e = 1.00$ is permitted in all cases.
2. The factor K_e shall be determined from the above table using interpolation or from the following formula for all elevations:
   $K_e = e^{-0.0000362 z_g}$ (z_g = ground elevation above sea level in ft).
   $K_e = e^{-0.000119 z_g}$ (z_g = ground elevation above sea level in m).
3. K_e is permitted to be take as 1.00 in all cases.

Last coefficient to be determined in the wind load analysis in the gust factor, which can be taken from section 26.11.1 as 0.85, as shown in figure 101.

For the Wind exposure parameter, it takes the eccentricity ratio e1 and e2 based on figure 47 as shown below:

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-8	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   e (e_X. e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Figure 47: Design wind load cases from the ASCE 7-16 code

3.4.2- Wind Parameters Determination for Eurocode

In this section the wind effect on the building will be discussed based on the steps provided in the Eurocode. The part of Eurocode used in this section is “Eurocode 1: Actions on structures - Part 1-4: General actions Wind actions”. Since the code explains the procedure to be followed for wind calculation analysis.

For the wind loading based on the Eurocode 1, there are some parameters that need to be determined in order to proceed for the comparisons stated in chapter 2. The main parameters that need to be determined are the wind coefficients, exposure height and wind exposure parameters. For the wind coefficients the parameters required for the analysis are wind velocity (V_0), orography factor ($C_o(z)$), structural factor (CsCd), air density, terrain category and turbulence factor (K_1).

For the exposure height it basically refers for the height limits that the wind will be affecting the building, which is basically considered from the ground floor up to the last floor and in this analysis, it is considered up to the 50th floor, while the wind exposure parameters required are the windward coefficient and the leeward coefficient.

The following procedure and calculations followed in this section is for the building designed in the Dubai, United Arab Emirates based on the requirements and notes provided in Eurocode1.

One of the first steps that are followed for wind load analysis is determining the wind velocity. The wind velocity has already been obtained in section 3.4.1 and its value has been taken as 85 mile/hour which equal to 137 kilometre/hour (38 meters/second), After getting the wind velocity for the site, some coefficients and parameters are required for determining the wind loads on the building. The procedure taken in the Eurocode is a bit different than the ACI318-19/ASCE7-16 code.

First is to determine the terrain category of the site. There are a total of five representative terrain categories, 0, I, II, III and IV as shown in the table below:

Table 33: Terrain categories and terrain parameters from Eurocode 1

Table 4.1 — Terrain categories and terrain parameters

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

As shown from the previous table, the terrain category that is considered for the site location is **IV**.

Next is to determine the orography factor ($C_0(z)$). Orography is like hills and cliffs and in order to determine its value, the Appendix A has been considered from the code. The main controlling factor is the upwind slope H/Lu in the wind direction which is denoted as Φ.

Based on figure A.2 and figure A.3 from the Eurocode, H/Lu can be taken as Zero in the site chosen for analysis as shown in figure 102 and figure 103 respectively.

Based on figures 102 and 103 taking H/Lu in the site being analyzed, the upwind slope will be equal to 0, and by following the equation A.1 in appendix A in t Eurocode 1 it can be figured out that Co is equal to 1, as shown in figure 104.

For the turbulence factor (K1), it can be taken directly from section 4.4 form the code, and the recommended value is 1, as shown in figure 105.

After that the structural factor CsCd is to be determined. The structural factor takes in consideration the effect on wind actions from the non-simultaneous occurrence of peak wind pressures on the surface (Cs) and the effect of the vibration of the building due to turbulence. In this analysis it will be assumed to be 1 and for the air density it will be assumed to be 1.25 kg/m^3 as mentioned in the code recommendation, as shown in figure 106, appendix B.

3.5- Software and Computer Simulation Information

The computer software used to illustrate and model the building for the two codes were ETABS 2019 and RAM Concept; both of the software are used for design and analyses purposes. They are highly used by engineers worldwide since they provide very accurate results. Both of these software are considered easier and simple compared to other relevant software which is the reason of choosing them in this dissertation.

3.5.1- ETABS 2019

ETABS is a software used in engineering to assist in the design and analyse multi-storey buildings. It is consist of modelling tools, code-based loads consideration, analysing methods and many others. Interoperability with a series of design and documentation platforms makes ETABS a coordinated and productive tool for designs for simple 2D frames up to high-rises buildings. ("Home - Dashboard - Computers and Structures, Inc." 2013).

This software was created by Computers and Structures, Inc. (CSI) which is an engineering Software Company for earthquakes and structures, founded in 1975. ("Structural Software | Computers and Structures, Inc." 2011)

USE IN DISSERTATION: The ETABS software will be used to design and analyse a 50-storey building. The main purpose is to check the lateral effects on this building using both the ACI318-19 and Eurocode and compare the mentioned provisions. The lateral forces considered are only the seismic and wind; the main seismic assessment provisions obtained from the software are storey drift for seismic and wind loads, torsional irregularity, P-Delta effect, response spectrum, base shear, storey force, mass participation and structural time period. The software will also be used to compare the reinforcement amount for some structural elements based on the design analysis to determine the reinforcement differences between the two models.

3.5.2- RAM Concept (Version 8)

RAM Concept is a software used for reinforced and post-tensioned concrete floors, mats, and rafts analysis and design. It is used to design a wide variety of different floor systems due to its advanced features. It uses the finite element modeling for the slab design. It has many advantages as it can predict the elastic slab behavior way more accurately than frame models. Moreover, the method of finite element used guarantees that the performed analysis fulfills all equilibrium. ("RAM Concept - RAM | STAAD | OpenTower Wiki - RAM | STAAD | OpenTower - Bentley Communities" 2021)

USE IN DISSERTATION: The Ram Concept will be used in this dissertation to design the Post tension slab for the building. Since the building consists of different floors occupancy and different geometry, a total of five models will be provided, one model for the podium floor, one model for the 1st Office floor, two models for the office floors and one model for the residential floors. Apparently, there will be five designs for each model which leading in to having a total of ten post tension slabs model, each relative floor from the two codes will be compared with each other in terms of total amount of reinforcement required due to seismic and wind effects, and in the post tension rate being used, which shows which one can give less cost, thus being more economical.

It is important to mention that the RAM Concept higher version of ACI code is ACI 318-14, but this is not considered an issue since there is no difference occurred in the post tension slab requirement in ACI 318-19 code compared to ACI 318-14 version.

CHAPTER 4: ANALYSIS RESULTS AND DISCUSSION

This chapter comprises of seismic and wind provisions comparison of different parameters on a G + 50 building using a ETABS commercial software; Moreover, the final design of the vertical elements and slabs will be compared in order to make a cost analysis of the buildings using both ACI 318-19 and Eurocode provisions and to give recommendation which code could be more economical. The proposed building that has been analyzed is a 180.2 meters high rise building. This chapter will provide enough evidence through the results obtained from the software to support the available findings in the literature given in chapter 2.

Moreover, in the end of this chapter there will be a comparison between the data obtained from the two codes as well as a comparison between the data obtained from previous analysis made by others as mentioned previously in the literature review.

4.1- Study Limitations

The study has some limitations. These limitations must be considered for any future research similar to this, to be more aware and to obtain more accurate results.

1. The research area is huge and many factors can affect it, like building's height and location. The building height and location were chosen randomly and these two can have an impact on the final ratios and results that were obtained in this study comparison, since a different location and a different building height means that the seismic forces and wind forces are different which can have some impact on the results.
2. The response spectral acceleration parameters S_s and S_1 were taken directly from the Dubai municipality recommendation, and in order to get more accurate values of these results a soil study should be performed by using bore holes and analyzing it in the laboratory. Same for the ground acceleration that is used in Eurocode model analysis.
3. The wind speed at the chosen location was taken directly from the Dubai municipality recommendation, and this value at the chosen location might be different than this value, so in order to get more accurate data the previous wind data at that location should be obtained or other tests should be done.
4. The software can generate variety of results and it can make many analyses. Some software errors might occur in analysis or it may have slightly inaccurate percentage. Also using one software may give assumptions which are not 100% compatible with real life since designing similar building using 2 different software will never give exact same results.

4.2- Computer Simulation

This paper will be testing several seismic and wind provisions using the computer simulations. The seismic provisions that will be compared are base shear, storey force, storey drift, torsional irregularity, P-Delta effect, response spectrum, mass participation and structural time period; All of these provisions will be compared in the ETABS software. For the wind comparison, the provision that will be compared is the inter-storey drift and it will be compared using ETABS software as well. For the final comparison which is the cost, the ETABS model will be used to compare the columns and shear walls sizing and reinforcement for each code model, while for the slabs, the RAM Concept will be used to check the amount of steel reinforcement used in each model due to lateral loads effect as well as the post tension rate.

The analyzed building has rectangular plan shapes, it consists of podium floors from the 1st floor to the 8th floor and office floors from the 9th floor to the 31st floor and residential floors from the 32nd floor up to the 50th floor. This building was assumed to be designed in Dubai, United Arab Emirates and all the seismic and wind parameters for each code mentioned in chapter 3 have been considered in the buildings design for each model.

The type of building occupancy chosen is a mixed-use building, consisting of office floors and residential floors. Since there are different occupancies, the design of vertical elements and slabs like reinforcement, structural element dimension and amount of post tension might be changed since the gravity loads based on each occupancy is different.

4.2.1- Initial & Final Computer Simulation Designs

The following is a 3D view for the initial model design which have been modeled using the ETABS software. The vertical elements in the model have been assumed and assigned for the whole building.

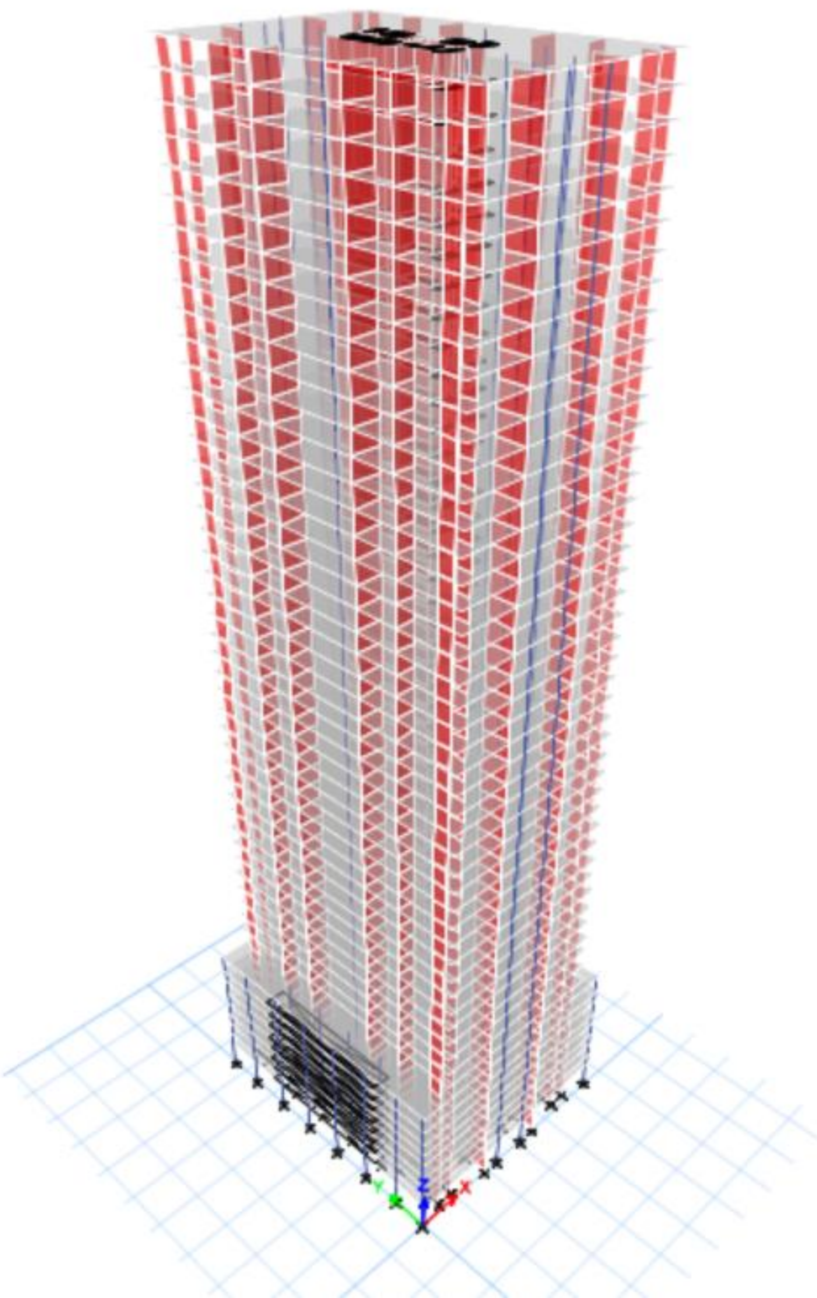


Figure 48: 3D view of the initial building design in ETABS

The following are the initial plans for each occupancy. These initial plans are before applying the gravity, seismic and wind loads. The vertical elements initially have similar sizing all along the building which most likely will be changed due to the gravity loads and due to the effects of seismic and wind loads.

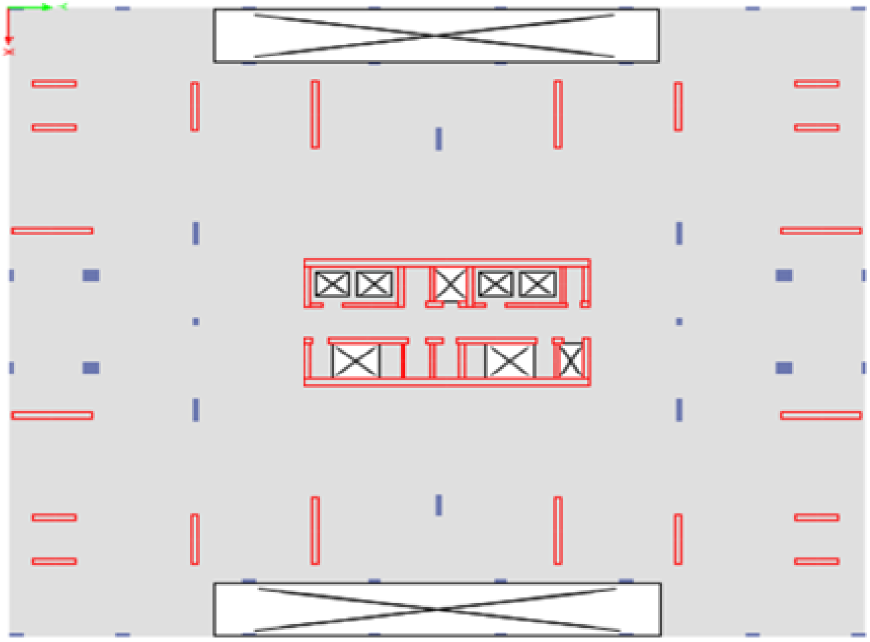


Figure 49: Typical initial podiums floor plan

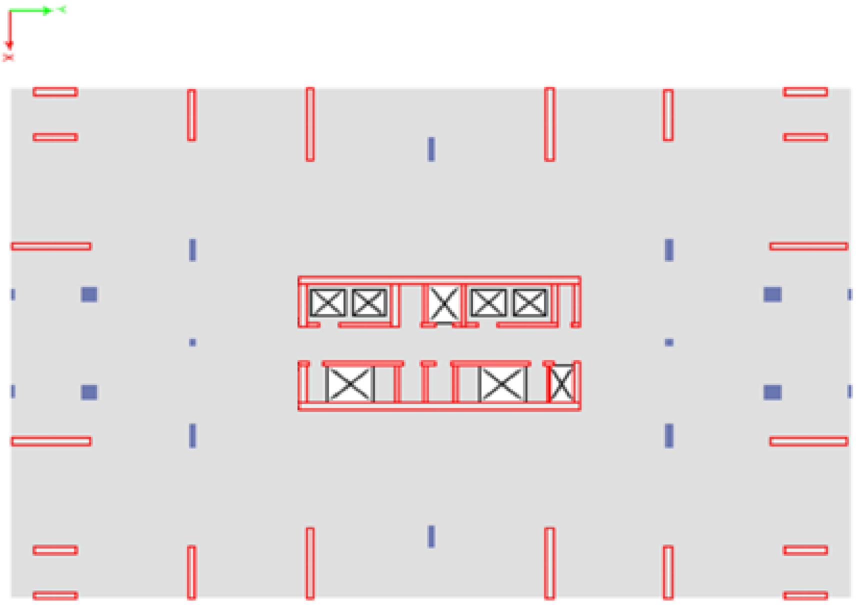


Figure 50: Typical initial office and residential floors plan

Two different building designs were made, one model following the ACI 318-19 parameters while the other model following the Eurocode parameters. As mentioned previously, the buildings have typical plans for the podium floors, office floors and residential floors. The typical podium floors are rectangular having a dimension of (50 meters x 43 meters), while for the other floors the plans are rectangular but having dimension of (43 meters x 33 meters) as shown in the appendix A.

- **Initial Sizing's for the Structural Elements**

Preliminary elements sizing for columns and walls have been assigned based on the gravity loading applied on them. It is significant to note that not only the gravity loads can affect the sizing, but also the seismic loads and wind loads applied on them. The building consists of core walls, shear walls and columns as vertical elements and initially no beams will be assigned in the design analysis except for the beams at the ramp.

- **Initial Sizing's for Columns**

The columns sizing's have been assumed as shown in the table below. Each column has been unified through the height of the building for easier work.

Table 34: Initial columns sizing

<table>
<tr><th>Floors Range</th><th>Initial Columns Sizing (All Floors)</th></tr>
<tr><td>From the Ground floor to the 10th Floor</td><td rowspan="5">1.50 meters x 0.40 meters
1.00 meters x 0.90 meters,
0.80 meters x 0.25 meters
&
0.50 meters x 0.40 meters,</td></tr>
<tr><td>From the 11th floor to the 20th floor</td></tr>
<tr><td>From the 21st floor to the 30th floor</td></tr>
<tr><td>From the 31st floor to the 40th floor</td></tr>
<tr><td>From the 41st floor to the 50th floor</td></tr>
</table>

- Initial Sizing's for Shear Walls

The shear walls sizing's have been assigned in the models as well and similarly to the columns, the shear walls will have similar sizing through the height of the building.

Table 35: Initial shear walls sizing

Floors Range	Initial Shear Walls Sizing (All Floors)
From the Ground floor to the 10^{th} Floor	4.60 meters x 0.40 meters,
From the 11^{th} floor to the 20^{th} floor	4.55 meters x 0.40 meters,
From the 21^{st} floor to the 30^{th} floor	3.30 meters x 0.40 meters,
From the 31^{st} floor to the 40^{th} floor	&
From the 41^{st} floor to the 50^{th} floor	2.50 meters x 0.40 meters

- Initial Beams Design

As mentioned earlier initially there will be no beams assigned in the design of the building, except for the ramp beams.

- Initial thicknesses of Slabs

For the post tensioned slab system there will be a total of five slabs design for each code. All the podium floors will have similar slab thickness of 220 mm while for the other floors they will have slab thickness of 200 mm.

Table 36: Initial slabs thickness

Floor	Level	Initial Slab Thickness
Podium Floors	From 1^{st} Floor to 8^{th} Floor	220 mm
Office Floors	From 9^{th} Floor to 31^{st} Floor	200 mm
Residential floors	From 32^{nd} Floor to 50^{th} Floor	200 mm

4.2.2 Building Design Checking

The seismic and wind parameters data and gravity loads have been assigned for each model design in order to check the building stability and the validity of the structural elements if they are failing or not. All the parameters shown below have been assigned based on the data obtained in chapter 3.

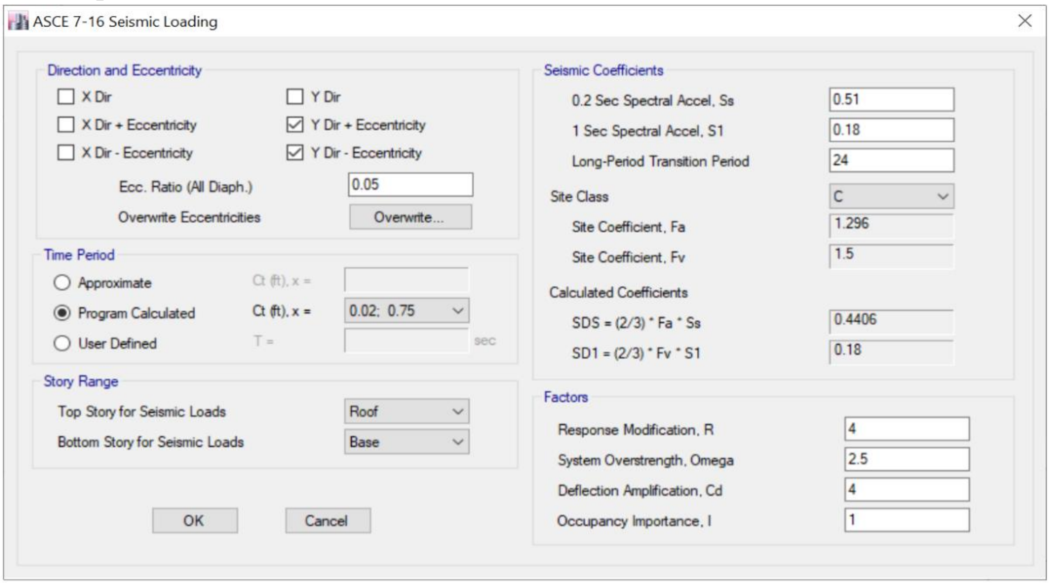


Figure 51: Seismic parameters assigned in ETABS model in ACI Model (Sample: y-direction)

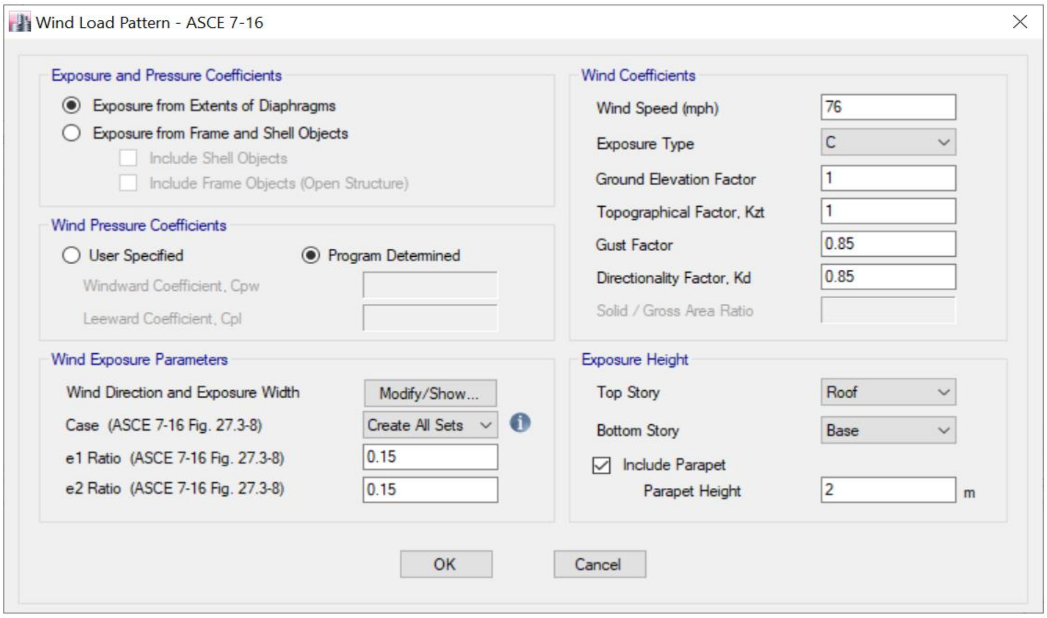


Figure 52: Wind parameters assigned in ETABS model in ACI model

While for the Eurocode model design the following parameters, data were assigned as mentioned in chapter 3.

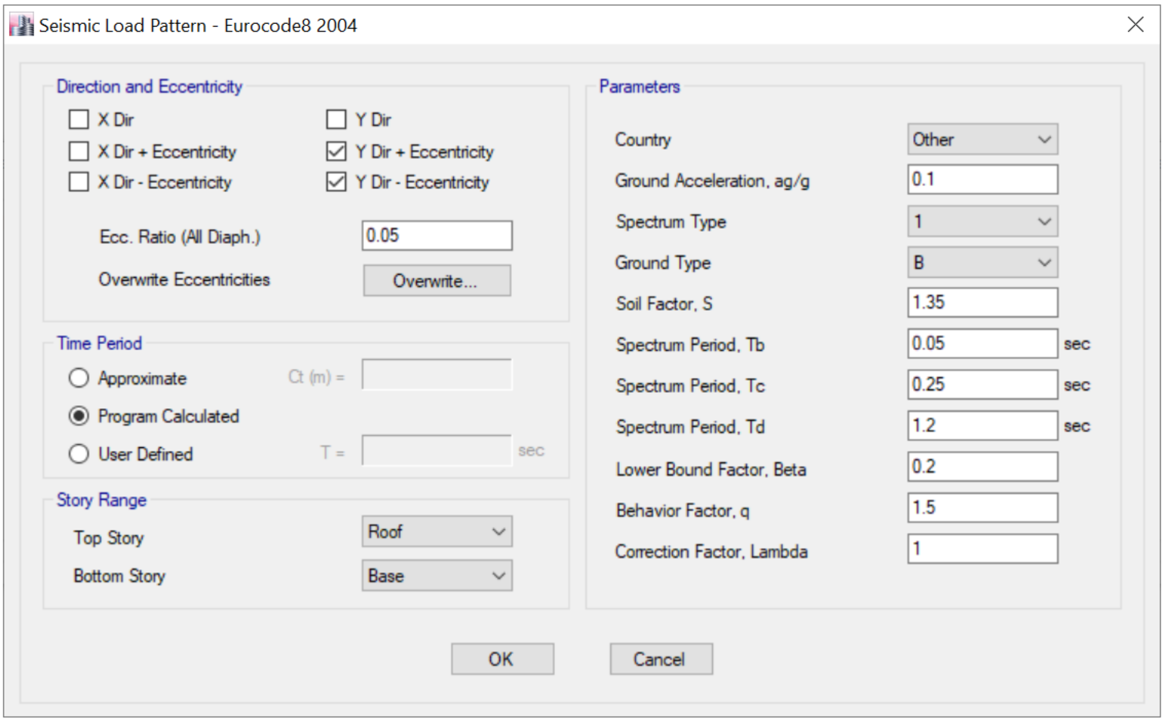


Figure 53: Seismic loading parameters assigned in the ETABS model in Eurocode model (Sample: y-direction)

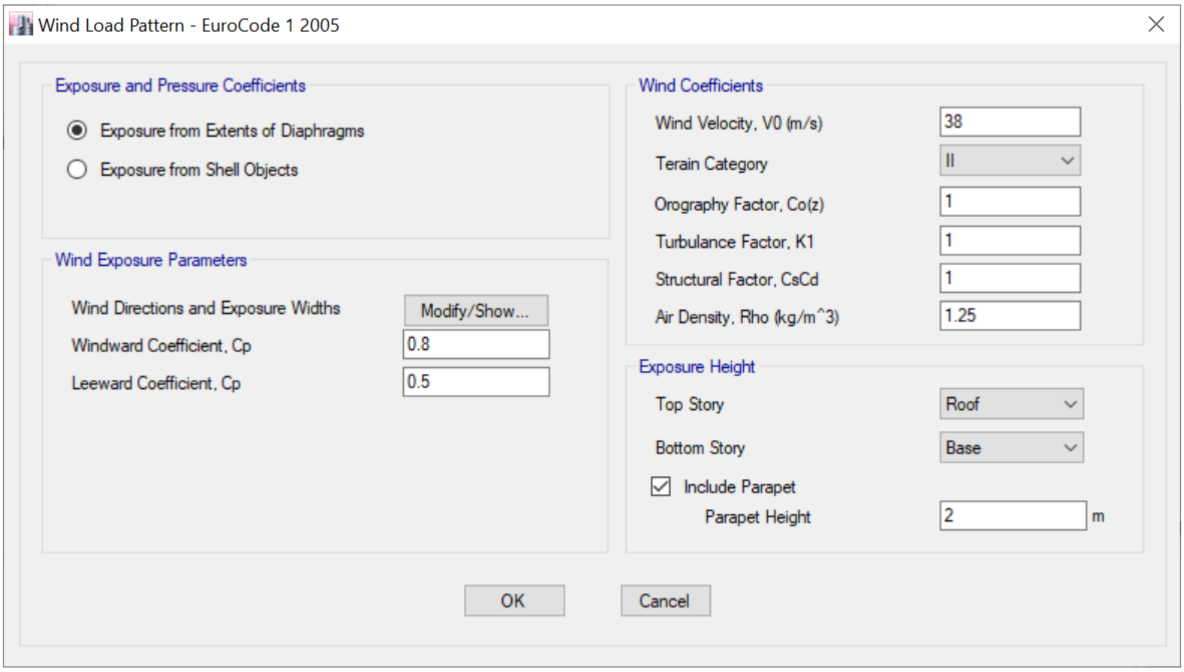


Figure 54: Wind loading parameters assigned in the ETABS model in Eurocode model

4.2.3- Final Sizing's for the Structural Elements

After applying the gravity loads, seismic and wind effects on the initial building design, there were many failures in the columns and walls design also the time period of the building was high. In order to solve the previous issues, the columns and shear walls sizes have been increased to withstand the applied loads as well as the lateral loads effects, and some reinforcement was increased in some of them. In order to solve the building time period issue, beams have been added in the building.

- **Final Sizing's for Columns**

The columns sizing's have been adjusted based on the results obtained from the model. Some of the columns at the lower levels had increase in sizing which is logical since the applied gravity load at these columns will be the highest. While at the upper floors, the columns were overdesigned which leaded to decrease their sizing in order to obtain logical results which reflects a real-life situation.

Table 37: Final columns sizing

Floors Range	Final Columns Sizing for each floors range
From the 1st Podium Floor to the 1st Offices Floor	1.80 meters x 1.60 meters, 1.70 meters x 1.50 meters, 1.50 meters x 1.20 meters, 1.50 meters x 1.10 meters, 1.50 meters x 0.80 meters, 1.30 meters x 1.20 meters & 0.90 meters x 0.40 meters
From the 2nd Offices Floor to the 5th Offices Floor	1.80 meters x 1.60 meters, 1.70 meters x 1.50 meters, 1.50 meters x 1.20 meters, 1.50 meters x 1.10 meters, 1.50 meters x 0.80 meters & 1.30 meters x 1.20 meters
From the 6th Offices Floor to the 15th Offices Floor	1.70 meters x 1.50 meters, 1.50 meters x 1.20 meters, 1.50 meters x 1.10 meters, 1.35 meters x 1.20 meters, 1.30 meters x 1.20 meters & 1.30 meters x 0.80 meters
From the 16th Offices Floor to the 31st Residential Floor	1.50 meters x 1.20 meters, 1.10 meters x 0.80 meters, 1.00 meters x 1.00 meters & 1.00 meters x 0.60 meters
From the 32nd Residential Floor to the 36th Residential Floor	1.10 meters x 0.70 meters, 0.90 meters x 0.40 meters, 0.80 meters x 0.60 meters & 0.60 meters x 0.60 meters
37th Residential Floor	1.10 meters x 0.70 meters, 0.90 meters x 0.90 meters, 0.90 meters x 0.40 meters, 0.80 meters x 0.60 meters, 0.80 meters x 0.50 meters & 0.60 meters x 0.60 meters
From the 38th Residential Floor to the Roof Floor	1.10 meters x 0.70 meters, 0.90 meters x 0.90 meters, 0.80 meters x 0.60 meters, 0.80 meters x 0.50 meters, 0.70 meters x 0.40 meters & 0.60 meters x 0.60 meters

Discussion of Columns Sizing:

As shown from the previous table, the columns sizing's are larger at the lower floors in the building, since the cumulative gravity loads at the lower floors are more. The axial force keeps increasing as going down along the building height, which explains why the columns sizing are increasing as going down along the building.

- **Shear Walls**

The shear walls sizing's have been adjusted based on the failures obtained from the initial model, there were many failures in flexural and shear reinforcement, which lead to increase in the thickness of the shear walls and reinforcement in some shear walls. While at the upper levels since many were overdesigned, the sizing's have been decreased to obtain results which are realistic.

Table 38: Final shear walls sizing

Floors Range	Shear Walls Sizing
From the Ground floor to the 15th Office	4.60 meters x 0.60 meters 4.55t meters x 0.80 meters 3.30 meters x 0.80 meters & 2.50 meters x 0.60 meters
From the 16th floor to the Roof floor	4.60 meters x 0.50 meters 4.55 meters x 0.50 meters 3.30 meters x 0.50 meters & 2.50 meters x 0.50 meters

Discussion of Shear Walls Sizing:

As shown from the previous table, all the shear walls have increased in thickness due to the lateral loads effect that caused too many failures. Similarly, the shear walls sizing is decreasing as going up in the building, since the lateral force is decreasing as going up along the building.

- **Beams**

Initially there were only the ramp beams in the design but in order to improve the results some beams have been added in all the floors.

Table 39: Final beams sizing

Floors Range	Beams Sizing
From the 1st Podium Floor to the 1st Office Floor	0.30 m x 0.60 m, 0.60 m x 1.20 m, 0.60 m x 1.10 m & 0.60 m x 1.00 m
From the 2nd Office Floor to the 15th Office Floor	0.60 m x 1.20 m, 0.60 m x 1.10 m & 0.60 m x 1.00 m
From the 16th Office Floor to the 31st Residential Floor	0.60 m x 1.20 m & 0.50 m x 1.10 m
From the 32nd Residential Floor to the Roof Floor	0.60 m x 1.20 m & 0.40 m x 1.00 m

Discussion of Adding Beams:

Initially there were no beam in the model design, except for the ramp beams, but after performing the analysis, it appeared that the seismic and wind drift results are exceeding the limits, also the time period for the structure was high, so in order to solve these issues the beams in table 50 were added, which made both the seismic and wind drift below the limit and enhanced the building's time period.

- **Slabs**

For the post tension design, there were no change in their thicknesses, all the models have been modeled in Ram concept and the results are satisfying all the codes requirements which is clarified in section 4.5.3.

Table 40: Final slabs thickness

Floor	Level	Initial Slab Thickness
Podium Floors	From 1st Floor to 8th Floor	220 mm
Office Floors	From 9th Floor to 22nd Floor	200 mm
Residential floors	From 23rd to 42nd Floor	200 mm

<u>Discussion of Slabs Thickness:</u>

There was no change in the slabs thickness, since the design of the post tension slabs depend mainly on the post tension amount and reinforcement used. Any deflection and stresses issues in the slabs design have been solved by using enough amount of post tension.

4.2.4 Codes Loading

The significant loads in the models are super imposed dead loads (SDL), dead loads (DL) and live loads (LL). The dead loads are directly measured in the ETABS models as well as in the Ram Concept models, thus the assigned values were the super imposed dead loads and the live loads. The values for each floor in each designed model were mentioned in tables 69 and 70.

The following are the final plans for the building. Some plans are shown below, which are the podiums, 1st office floor, other office floors (Which have different plan than the 1st office floor) and the residential floors.

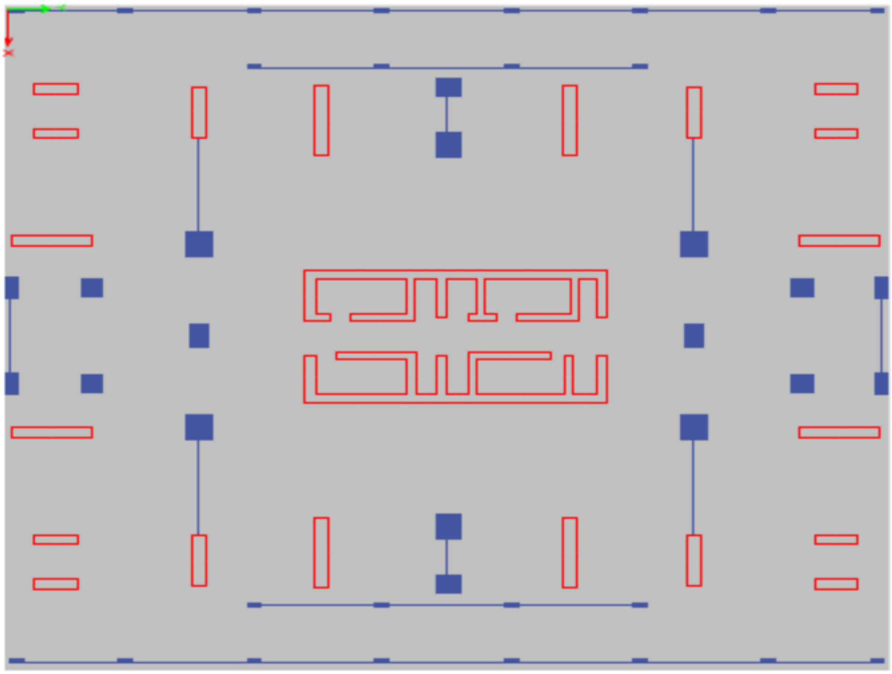


Figure 55: Figure: Final podiums floor plan (Sample: 1st Podium)

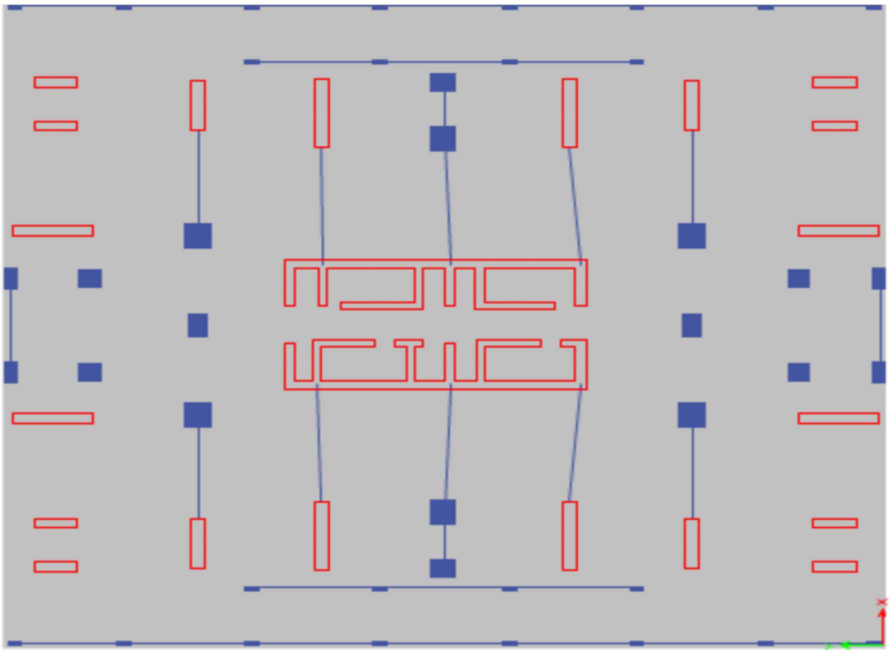


Figure 56: Final 1st Office floor plan

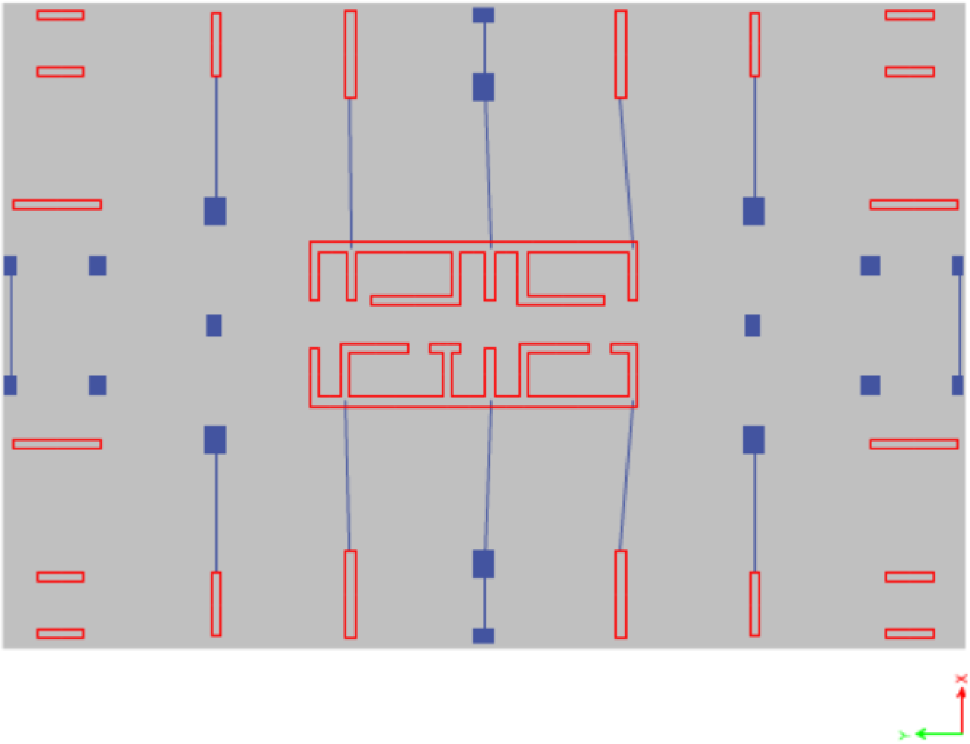


Figure 57: Final offices floor plan (Sample: 20th office's floor)

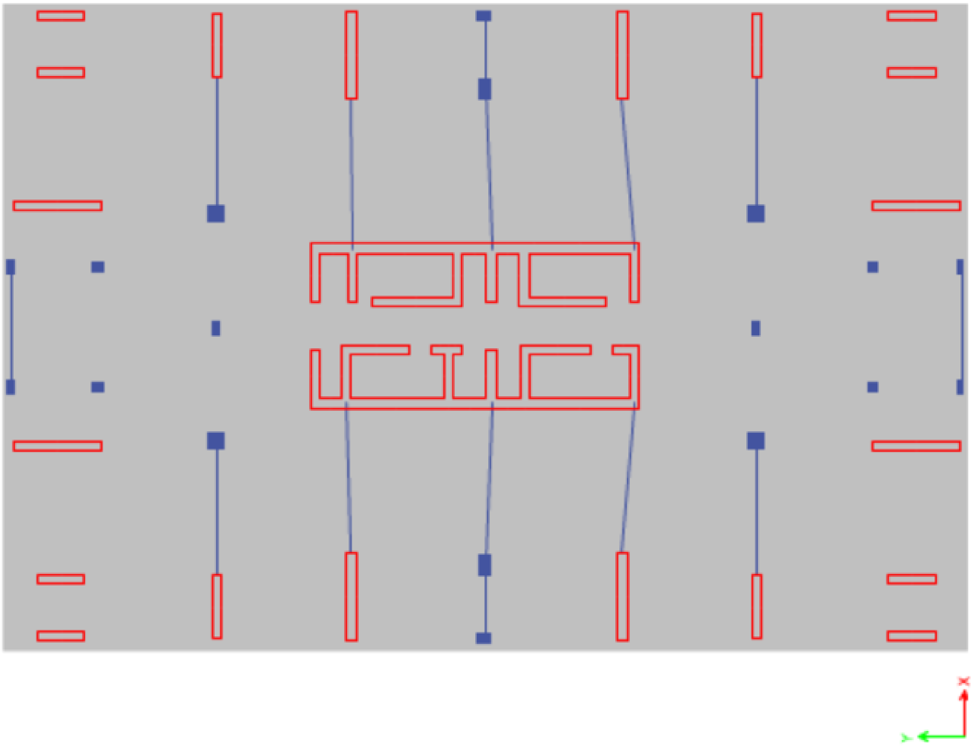


Figure 58: Final Residential floor plan (Sample: 40th residential floor)

4.2.5 Materials Characteristics

The main materials that will be used in any building are concrete and steel. For both models the same concrete and steel strength have been used for each structural element, but the main difference is that the calculation of modulus of elasticity in the ACI code and in Eurocode are not the same.

Since the steel reinforcement and Tendons don't have similar type of differences, then main discussion in this section will be on the concrete. The below table shows the concrete strengths that have been used for all the structural elements in the building.

Table 41: Concrete strength assigned for each level for ACI and Eurocode models

Material's Characteristic	Floors Range	Concrete Strength
Vertical Elements Concrete Strength for the first 23 floors (MPa)	1st Podium Floor to 15th Office Floor	C60/C48
Vertical Elements Concrete Strength for the other 27 floors (MPa)	16th Floor to Roof Floor	C50/C40
Slabs Concrete Strength for the first 15 floors (MPa)	1st Floor to 15th Floor	C45/C36
Slabs Concrete Strength for the other 35 floors (MPa)	16th Floor to Roof Floor	C40/C32

Based on the modulus of elasticity calculations mentioned before, the following table is obtained:

Table 42: Modulus of elasticity difference between the codes

Concrete Strength	Modulus of Elasticity Based on ACI 318-19	Modulus of Elasticity Based on Eurocode	Modulus of Elasticity Ratio (ACI 318 − 19 /Eurocode)
C60/C48	32562	37278	87.35%
C50/C40	29725	35220	84.40%
C45/C36	28200	34077	82.75%
C40/C32	26587	33346	79.73%

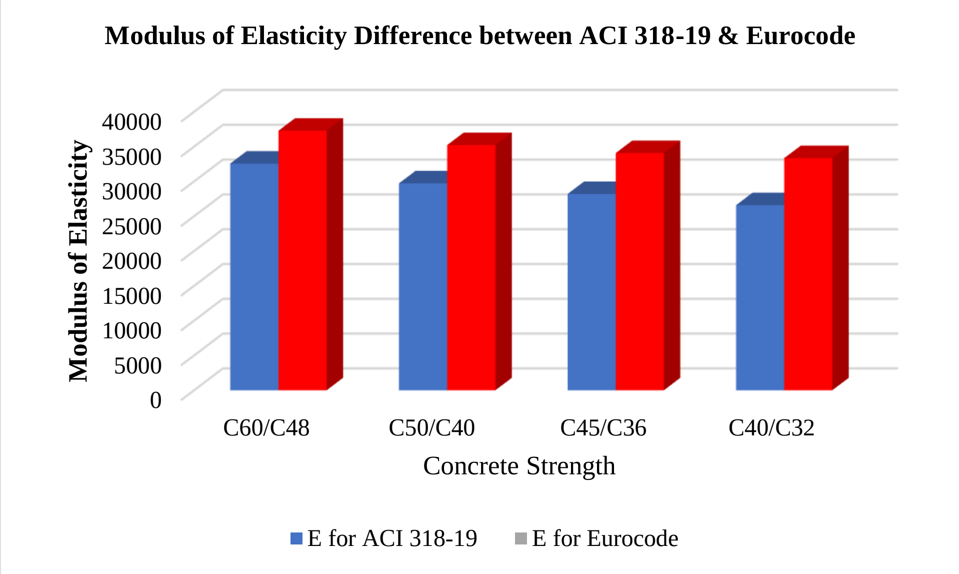


Figure 59: Comparison for the modules of elasticity between the codes

<u>Discussion of Materials Characteristics Difference:</u>

As it can be concluded from the previous comparison, even though the concrete strength is exactly the same but the modulus of elasticity for different strengths are different. It can be noticed that as higher the concrete strength as the gap between the ratio difference decreases.

Based on the such assumptions from the codes the obtained results will be different. According to the previous data of modulus of elasticity, it is assumed that the Eurocode model would give less storey drifts and time periods, which gives the Eurocode more flexibility in the design compared to the ACI.

4.3 Seismic Analysis Comparison

The buildings in both codes have found to be safe against seismic and wind loads and no structural element was failing based on the previous adjustments made. Thus, this allows to compare the two codes model in seismic and wind provisions mentioned before.

In this section the seismic and wind provisions comparison will be discussed based on the achieved results.

4.3.1- Seismic Base Shear

The base shear in both directions have been obtained from the ETABS models. The base shear from dynamic analysis has been scaled to match at least 85% in the x-direction and y-direction of the equivalent static base shear at the ground level. The floor considered was the 1st Podium floor and the scales for both models found to be more than 85% as shown below:

Table 43: Seismic base shear for the ACI 318-19 Model

Story	Load	Location	VX	VY
1st Podium	EQX	Top	28,041.13	0.0001
1st Podium	EQX	Bottom	28,041.13	0.0001
1st Podium	EQY	Top	0.000008	28,041.13
1st Podium	EQY	Bottom	0.000008	28,041.13
1st Podium	SPECX MAX	Top	26,242.24	582.20
1st Podium	SPECX MAX	Bottom	26,242.24	582.20
1st Podium	SPECY MAX	Top	248.45	24,765.18
1st Podium	SPECY MAX	Bottom	248.45	24,765.18

Table 44: Scaling of base shear for the ACI 318-19 Model

Equation	Scaling in X-Direction	Scaling in Y-Direction
SPECX/EQX	0.936	-
SPECY/EQY	-	0.883
Ratio (Percentage)	93.60 %	88.3%

Discussion of the Base Shear in the ACI model:

According to the previous table, the base shear was obtained from the 1st Podium floor. The static base shear in x-direction and y-direction have been found to be equal to 28041.13 kN. The total base shear was scaled for both directions and found to be more than 85% in both directions. The scaling in the x-direction was 93.60%, while in the y-direction was 88.30%.

The same procedure has been applied for the Eurocode model, and the result obtained are as the following:

Table 45: Seismic base shear for the Eurocode Model

Story	Load	Location	VX	VY
1st Podium	EQX	Top	28888.786	0.0001
1st Podium	EQX	Bottom	28888.786	0.0001
1st Podium	EQY	Top	0.0001	28888.786
1st Podium	EQY	Bottom	0.0001	28888.786
1st Podium	SPECX MAX	Top	27039.741	269.270
1st Podium	SPECX MAX	Bottom	27039.741	269.270
1st Podium	SPECY MAX	Top	305.788	28918.529
1st Podium	SPECY MAX	Bottom	305.788	28918.529

Table 46: Scaling of base shear for the Eurocode model

Equation	Scaling in X-Direction	Scaling in Y-Direction
SPECX/EQX	0.936	-
SPECY/EQY	-	1.00
Ratio (Percentage)	93.60%	100%

Discussion of the Base Shear in the Eurocode model:

According to the previous table, the base shear was obtained from the 1st podium. The static base shear in x-direction and y-direction have been found to be equal to 28888.786 kN. The total base shear was scaled for both directions and found to be more than 85% in both directions. The scaling in the x-direction was 93.60%, while in the y-direction was 100%.

Discussion of Base Shear results comparison for Both codes:

Based on the previous data obtained it can be noticed that the Eurocode is showing higher base shear in the x-direction by 797.50 kN having a percentage difference of 2.95%, also for the y-direction the base shear for the Eurocode is more by 4153.35 kN having a percentage difference of 14.36%. Moreover, by comparing the scaling of the two models it can be noticed that the scaling of the Eurocode is more which means that the dynamic base shear for the Eurocode model is more.

The following are comparisons for the static and dynamic base shears based on the obtained results:

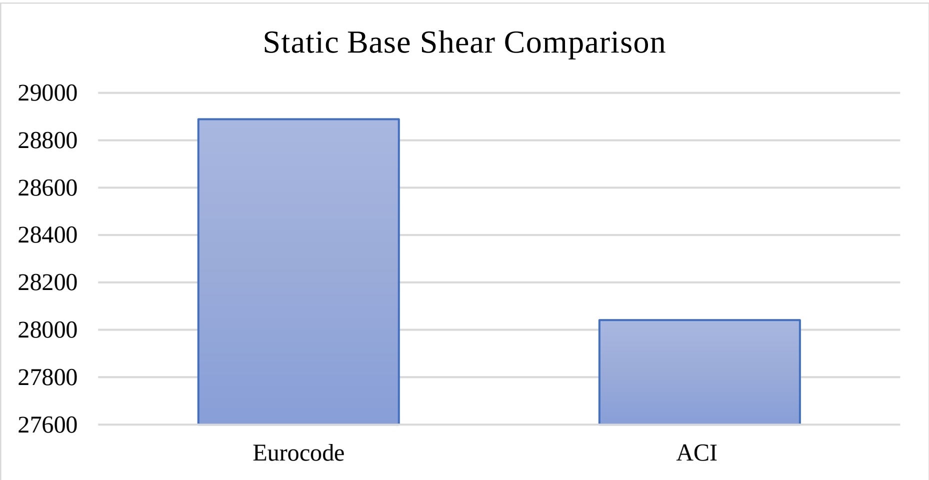


Figure 60: Static base shear comparison between the ACI code and Eurocode

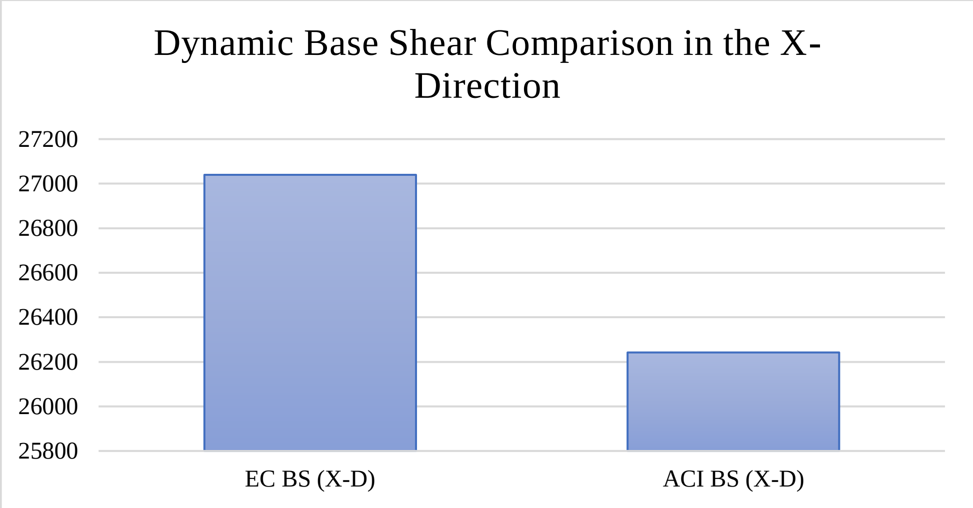


Figure 61: Dynamic base shear comparison between the ACI code and Eurocode in x-direction

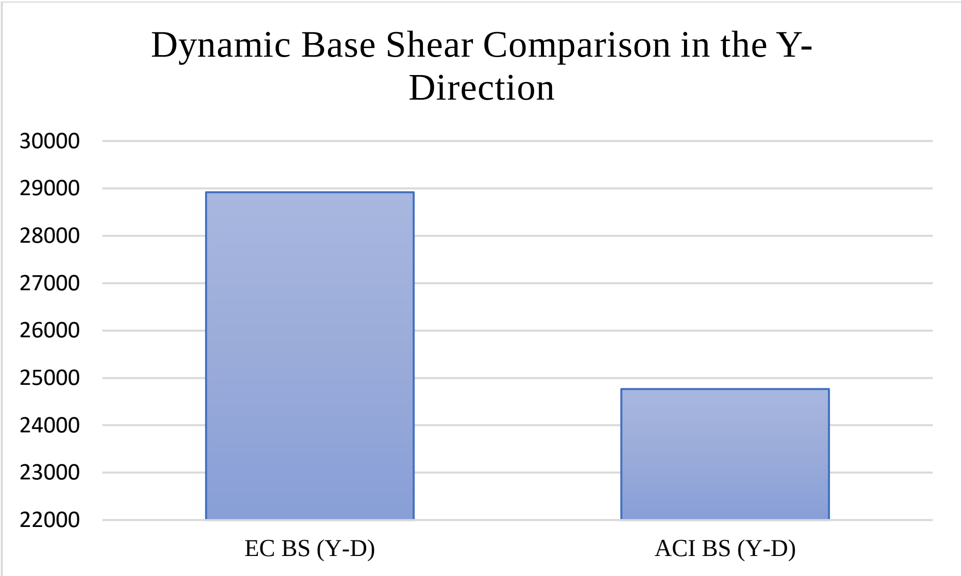


Figure 62: Dynamic base shear comparison between the ACI code and Eurocode in y-direction

4.3.2- Storey Forces Vertical Distribution

As was shown in Figure 5 in chapter 2, the base shear is distributed along each story. The summation of the lateral seismic load applied on each storey must be equal to the static base shear which were obtained from section 4.3.1.

Table 47: Vertical distribution of storey force for every 5 storeys for both codes

Floor	Forces from the ACI Code Model (kN)	Forces from the Eurocode Model (kN)	Forces Difference (kN)
Ground Floor	1.51	36.85	-35.34
5th Floor	21.67	132.31	-110.64
10th Floor	74.01	249.68	-175.67
15th Floor	165.07	371.93	-206.86
20th Floor	292.93	495.46	-202.53
25th Floor	421.00	569.91	-148.91
30th Floor	601.37	678.84	-77.47
35th Floor	818.10	791.78	26.32
40th Floor	1024.46	867.68	156.78
45th Floor	1295.22	975.27	319.95
50th Floor	1369.49	928.58	440.91

The previous table shows the vertical distribution of storeys force for every five storeys, the full table is available in appendix D.

<u>Discussion of Vertical Distribution of Storey Force Results Comparison:</u>

As shown in the previous table, the seismic forces are increasing as going into upper floors and vice versa, which is logical since the building is considered fixed at the base and it won't have a high storey force applied on it, but for the upper floors the seismic force is higher.

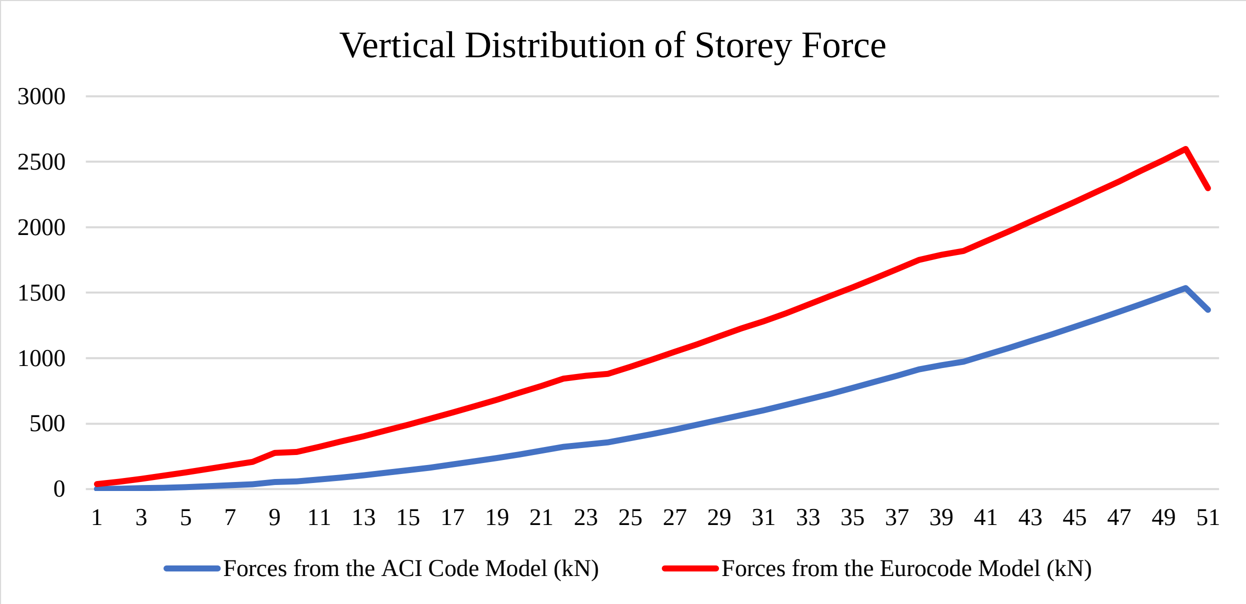


Figure 63: Vertical distribution of storey force between the ACI code and Eurocode

After analyzing the previous data in Table 47, the above graph was obtained, and as shown in the previous section that the base shear in the Eurocode model was higher than the base shear in the ACI model which means that the distribution of this base shear along the building (Storey force) for the Eurocode will be higher and this is proved in the graph shown above.

4.3.3- Seismic Storey drift (In X-Direction)

The seismic storey drifts have been checked for both models taken in to account the limitations in each code. For the ACI 318-19 code the inter storey drift should not exceed 2.5% while for the Eurocode the storey drift should not exceed 1%. Both the models are fulfilling the requirements stated.

The table below shows the maximum seismic storey drifts in the x-direction for both models:

Table 48: Seismic storey drift in x-direction (for both codes)

Storeys with Highest Seismic Drift (ACI code)	Drift in ACI Model (Percentage)	Limit Satisfaction (ACI Model)	Storeys with Highest Seismic Drift (Eurocode)	Drift in Eurocode Model (Percentage)	Limit Satisfaction (Eurocode Model)
OFFICE 18	0.731	Satisfied	OFFICE 16	0.556	Satisfied
OFFICE 17	0.730	Satisfied	OFFICE 17	0.556	Satisfied
OFFICE 19	0.730	Satisfied	OFFICE 18	0.555	Satisfied
OFFICE 16	0.728	Satisfied	OFFICE 19	0.552	Satisfied
OFFICE 20	0.727	Satisfied	OFFICE 20	0.548	Satisfied
OFFICE 21	0.722	Satisfied	OFFICE 15	0.547	Satisfied
OFFICE 22	0.716	Satisfied	OFFICE 14	0.547	Satisfied
OFFICE 15	0.716	Satisfied	OFFICE 13	0.546	Satisfied
OFFICE 14	0.713	Satisfied	OFFICE 12	0.543	Satisfied
OFFICE 13	0.709	Satisfied	OFFICE 21	0.543	Satisfied
RESIDENTIAL 23	0.709	Satisfied	OFFICE 11	0.541	Satisfied

<u>Discussion of Seismic Storey Drift Results Comparison in X-Direction:</u>

As shown from the previous table the maximum seismic storey drifts for both codes are generally at similar floors. The maximum storey drift in the x-direction for the ACI model is 0.731% which is less than the limit specified in the code which is 2.50% while for the Eurocode model the maximum storey drift in the x-direction is 0.556% which is less than 1%. As it can be analyzed from the data that even though the Eurocode model has more storey forces applied as mentioned in previous section, the seismic storey drift is less compared to the ACI code; The reason of such difference could be due to many reasons such as the load combination difference, the modulus of elasticity difference in concrete, the distribution of the shear walls in the x-direction in the building or due to other reasons. Overall based on the data it can be concluded that the Eurocode model can resist more seismic base shear (In the x-direction).

4.3.4- Seismic Storey drift (In Y-Direction)

Following the same procedure performed in the previous analysis the following data was obtained:

Table 49: Seismic storey drift in y-direction (for both codes)

Storeys with Highest Seismic Drift (ACI code)	Drift in ACI Model (Percentage)	Limit Satisfaction (ACI Model)	Storeys with Highest Seismic Drift (Eurocode)	Drift in Eurocode Model (Percentage)	Limit Satisfaction (EC Model)
RESIDENTIAL 27	0.692	Satisfied	RESIDENTIAL 25	0.704	Satisfied
RESIDENTIAL 28	0.692	Satisfied	RESIDENTIAL 24	0.704	Satisfied
RESIDENTIAL 26	0.691	Satisfied	RESIDENTIAL 26	0.703	Satisfied
RESIDENTIAL 29	0.691	Satisfied	RESIDENTIAL 23	0.702	Satisfied
RESIDENTIAL 25	0.690	Satisfied	RESIDENTIAL 27	0.702	Satisfied
RESIDENTIAL 30	0.689	Satisfied	OFFICE 22	0.700	Satisfied
RESIDENTIAL 24	0.688	Satisfied	RESIDENTIAL 28	0.700	Satisfied
RESIDENTIAL 31	0.687	Satisfied	OFFICE 21	0.697	Satisfied
RESIDENTIAL 32	0.685	Satisfied	RESIDENTIAL 29	0.697	Satisfied
RESIDENTIAL 23	0.684	Satisfied	RESIDENTIAL 30	0.693	Satisfied

Discussion of Seismic Storey Drift Results Comparison in Y-Direction:

As shown from the previous table the maximum seismic storey drifts for both codes are generally at similar floors, same as shown in the x-direction. The maximum storey drift in the y-direction for the ACI model is 0.692% which is less than the limit specified in the code which is 2.50% while for the Eurocode model the maximum storey drift in the y-direction is 0.704% which is less than 1%. As it can be analyzed from the previous data that the Eurocode model has slightly more seismic storey drift in the y-direction compared to the ACI model. Based on these data it can be concluded that the main reason affecting the storey drift could be the way if shear walls distribution along the building since, they are considered as the main structural elements that resist the seismic loads as well as the drift.

4.3.5- Torsional Irregularity

In this section the torsional irregularity for both codes will be compared to check the differences if any, based on each code's provisions.

Table 50: Torsional irregularity for ACI code model in x-direction

Storey	Direction	Max Drift (mm)	Avg Drift (mm)	Torsional Irregularity in ACI Model
P1	X-Direction	6.061	4.16	1.457
P2	X-Direction	6.112	4.59	1.331
P3	X-Direction	6.485	5.275	1.229
OFFICE 1	X-Direction	10.653	8.806	1.21
P8	X-Direction	10.292	8.53	1.207
RESIDENTIAL 41	X-Direction	10.324	8.598	1.201
RESIDENTIAL 40	X-Direction	10.672	8.889	1.201
RESIDENTIAL 39	X-Direction	11.002	9.17	1.2
RESIDENTIAL 42	X-Direction	9.977	8.323	1.199
RESIDENTIAL 38	X-Direction	11.32	9.442	1.199

Table 51: Torsional irregularity for ACI code model in y-direction

Storey	Direction	Max Drift (mm)	Avg Drift (mm)	Torsional Irregularity in ACI Model
P1	Y-Direction	1.272	1	1.272
P2	Y-Direction	2.852	2.281	1.25
P3	Y-Direction	4.846	3.889	1.246
P4	Y-Direction	7.169	5.762	1.244
P5	Y-Direction	9.783	7.886	1.241
P6	Y-Direction	12.662	10.242	1.236
P7	Y-Direction	15.796	12.818	1.232
P8	Y-Direction	19.161	15.599	1.228
OFFICE 1	Y-Direction	22.602	18.506	1.221
OFFICE 2	Y-Direction	25	21.24	1.177

Discussion of Torsional Irregularity for the ACI318-19 Code Model:

As shown in the previous table the maximum storey drift at one of the ends of the building is exceeding more than 1.2 times the average storey drifts at the two ends of the building which means that there is torsional irregularity at the floors shown. This irregularities in the building can have an impact of other parameters such as the storey drift as well as the fundamental period of the building.

The Eurocode provisions for torsional irregularity check are that at each level and in x and y directions the building eccentricity *eo* and the torsional radius *r* shall be in accordance with the following: $eox \leq 0.30.\ rx$ and $rx \geq ls$ for Y-Direction & $eoy \leq 0.30.\ ry$ and $ry \geq ls$ for x-Direction.

Table 52: Torsional irregularity for Eurocode model in x-direction

Storey	Direction	$eox \leq 0.30$	$rx \geq ls$	Torsional Irregularity (Required or Not)
1st OFFICE	X-Direction	0.214< 0.30	280.18>228.45	Not Required
8th Podium	X-Direction	0.216< 0.30	233.935> 228.45	Not Required
7th Podium	X-Direction	0.216< 0.30	189< 228.45	Required
6th Podium	X-Direction	0.216< 0.30	148.27< 228.45	Required
5th Podium	X-Direction	0.216< 0.30	111.89< 228.45	Required
4th Podium	X-Direction	0.216< 0.30	79.93< 228.45	Required
3rd Podium	X-Direction	0.216< 0.30	52.6< 228.45	Required
2nd Podium	X-Direction	0.216< 0.30	30.14< 228.45	Required
1st Podium	X-Direction	0.216< 0.30	13.25< 228.45	Required

Table 53: Torsional irregularity for Eurocode model in y-direction

Storey	Direction	$eox \leq 0.30$	$rx \geq ls$	Torsional Irregularity (Required or Not)
OFFICE 1	Y-Direction	0.2509<0.30	377.11> 228.45	Not Required
8th Podium	Y-Direction	0.2507<0.30	319.43> 228.45	Not Required
7th Podium	Y-Direction	0.2508<0.30	264.31> 228.45	Not Required
6th Podium	Y-Direction	0.2508<0.30	211.875< 228.45	Required
5th Podium	Y-Direction	0.2508<0.30	162.825< 228.45	Required
4th Podium	Y-Direction	0.2508<0.30	118< 228.45	Required
3rd Podium	Y-Direction	0.2508<0.30	78.325< 228.45	Required
2nd Podium	Y-Direction	0.2508<0.30	44.945< 228.45	Required
1st Podium	Y-Direction	0.2508<0.30	19.46< 228.45	Required

Discussion of Torsional Irregularity for the Eurocode Model:

According to the previous data, torsional Irregularity in the Eurocode model exists in the 1st Podium up to the 7th Podium in x-direction, while it exists in the 1st Podium up to the 6th Podium in y-direction.

<u>Discussion of Torsional Irregularity Comparison between the Codes:</u>

Based on the two codes provisions in determining the torsional irregularity in each floor, it have been found that for the torsional irregularity in y-direction, there are more floors in the ACI model compared to the Eurocode. In the ACI model the floors having torsional irregularity in the y-direction are from the 1^{st} Podium up to the 1^{st} Office floors while for the Eurocode model the floors having torsional irregularity are from the 1^{st} Podium up to the 6^{th} Podium floor.

In the x-direction the data shows that there are torsional irregularities at different floors levels for the ACI model. For the number of floors that have torsional irregularities, the Eurocode is having less floors compared to the ACI code.

The irregularity in structural design of the building may lead to an increase of the earthquake damages and since the number of floors having torsional irregularities in the ACI model is more than Eurocode, this means that the ACI code gives more attention for such cases which makes the designer more aware of it and can make a safer design.

4.3.6- P-Delta (In X-Direction and Y Direction)

P-delta effect is the secondary moment that is produced by wind or seismic effects. This type of checking is related with external forces or loading application upon the displaced configuration of the building.

The main check is to check if the P-delta effect is required or not for the building. The data for P-delta in the below tables are for each five floors, the full table is available in appendix D.

Table 54: P-Delta due to EQx in the ACI 318-19 model

Storey	Height	Force	Shear (Vx)	Δx	θx	Consideration of P-Delta
	m	kN	kN	m		
50	3.6	28882.98	2910.871	0.002863	0.0020	No P-delta
45	3.6	173699.9	9064.924	0.003372	0.0045	No P-delta
40	3.6	318615.2	11090.32	0.00377	0.0075	No P-delta
35	3.6	468038.3	12644.5	0.004018	0.0103	No P-delta
30	3.6	618586.6	13852.05	0.004227	0.0131	No P-delta
25	3.6	773183.9	14951.69	0.004314	0.0155	No P-delta
20	3.6	934944.2	16216.85	0.00423	0.0169	No P-delta
15	3.6	1101480	17619.44	0.004091	0.0178	No P-delta
10	3.6	1268476	19246.76	0.003686	0.0169	No P-delta
5	3	1434854	21580.38	0.00293	0.0162	No P-delta
Base	5	1601402	26242.24	0.001238	0.0038	No P-delta

Table 55: P-Delta due to EQy in the ACI 318-19 model

Storey	Height	Force	Shear (Vy)	Δy	θy	Consideration of P-Delta
	m	kN	kN	m		
50	3.6	28882.98	2472.617	0.006356	0.0052	No P-delta
45	3.6	173699.9	8805.818	0.006596	0.0090	No P-delta
40	3.6	318615.2	9104.613	0.006814	0.0166	No P-delta
35	3.6	468038.3	9872.667	0.006916	0.0228	No P-delta
30	3.6	618586.6	11029.49	0.006842	0.0266	No P-delta
25	3.6	773183.9	11421.47	0.006501	0.0306	No P-delta
20	3.6	934944.2	12416.6	0.00585	0.0306	No P-delta
15	3.6	1101480	13626.87	0.004972	0.0279	No P-delta
10	3.6	1268476	15789.99	0.003738	0.0209	No P-delta
5	3	1434854	20696.89	0.002374	0.0137	No P-delta
Base	5	1601402	24765.18	0.00046	0.0015	No P-delta

Discussion of P-Delta Results (X-Direction):

Based on the obtained data from the software it can be noticed that the value of $\boldsymbol{\theta}$ in x-direction and y-direction are less than 0.10 so the building will be designed with no P-delta because it is not needed, based on the ACI 318-19 code requirements; The second order effects can be neglected.

The same procedure will be applied for the Eurocode model to verify if the effect of P-delta is required or not for the buildings.

Table 56: P-Delta due to EQx in the Eurocode model

Floor	Height	Force	Shear (Vx)	Δx	$\boldsymbol{\theta x}$	Consideration of P-Delta
	m	kN	kN	m		
50	3.6	28911.2	1013.795	0.003148	0.025	No P-delta
45	3.6	173728.1	6272.704	0.003655	0.028	No P-delta
40	3.6	318643.4	10680.4	0.004318	0.036	No P-delta
35	3.6	468066.6	14511.09	0.00484	0.043	No P-delta
30	3.6	618614.9	17779.16	0.005298	0.051	No P-delta
25	3.6	773212.1	20519.85	0.005548	0.058	No P-delta
20	3.6	934972.5	22848.74	0.005456	0.062	No P-delta
15	3.6	1101508	24753.86	0.005247	0.065	No P-delta
10	3.6	1268504	26104.69	0.004649	0.063	No P-delta
5	3	1434882	26875.94	0.00351	0.062	No P-delta
Base	5	1601430	27131.48	0.00084	0.010	No P-delta

Table 57: P-Delta due to EQy in the Eurocode model

Floor	Height	Force	Shear (Vy)	Δy	$\boldsymbol{\theta y}$	Consideration of P-Delta
	m	kN	kN	m		
50	3.6	28911.2	1309.099	0.006226	0.0382	No P-delta
45	3.6	173728.1	7701.882	0.006539	0.0410	No P-delta
40	3.6	318643.4	12627.65	0.006821	0.0478	No P-delta
35	3.6	468066.6	16689.19	0.006996	0.0545	No P-delta
30	3.6	618614.9	20038.24	0.007024	0.0602	No P-delta
25	3.6	773212.1	22780.18	0.006798	0.0641	No P-delta
20	3.6	934972.5	25046.78	0.006233	0.0646	No P-delta
15	3.6	1101508	26837.19	0.005408	0.0617	No P-delta
10	3.6	1268504	28051.99	0.00417	0.0524	No P-delta
5	3	1434882	28736.41	0.002723	0.0453	No P-delta
Base	5	1601430	28999.31	0.000608	0.0067	No P-delta

Discussion of the P- Delta Results (Y-Direction):

Based on the obtained data from the software it can be noticed that the value of $\boldsymbol{\theta}$ in x-direction and y-direction are less than 0.10 so the building will be designed with no P-delta since it won't be required, based on the Eurocode requirements; The second order effects can be neglected.

When comparing the values of P- delta in both directions, the data shows that the Eurocode is giving more values which is logical, based on the equation given by the ASCE 7-16 the stability coefficient (θ) is obtained through the following expression:

$$\boldsymbol{\theta = \frac{P_x \; x \; \Delta \; x \; I_e}{V_x \; x \; h_{sx} \; x \; C_d}}$$

While for Eurocode the stability coefficient is obtained through the following expression:

$$\boldsymbol{\theta = \frac{P_{TOT} \; x \; d_r}{V_{TOT} \; x \; H}}$$

As it can be noticed that both codes have similar parameters in this equation except that the ACI code is adding the deflection amplification factor ($\boldsymbol{C_d}$) and the importance factor ($\boldsymbol{I_e}$). The importance factor is not making any effect in the equation since its value is 1, while for the deflection amplification factor have been considered to be 4 in this study and by substituting it in the equation the value of P-delta will be reduced compared to the Eurocode which does not have such parameter in the equation, which explains the reason of having higher values in p-delta in the Eurocode.

4.3.7- Response Spectrum

The response spectrum diagrams have been obtained from the ETABS model based on the seismic parameters assigned in it. The following are the response spectrums for both models.

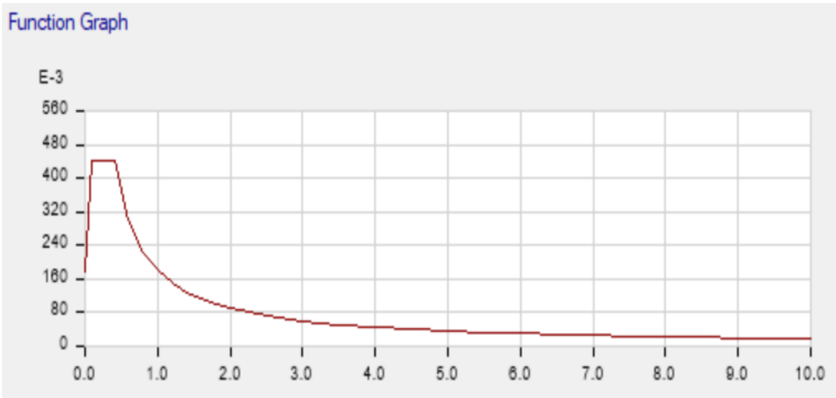


Figure 64: The response spectrum for the ACI model from ETABS

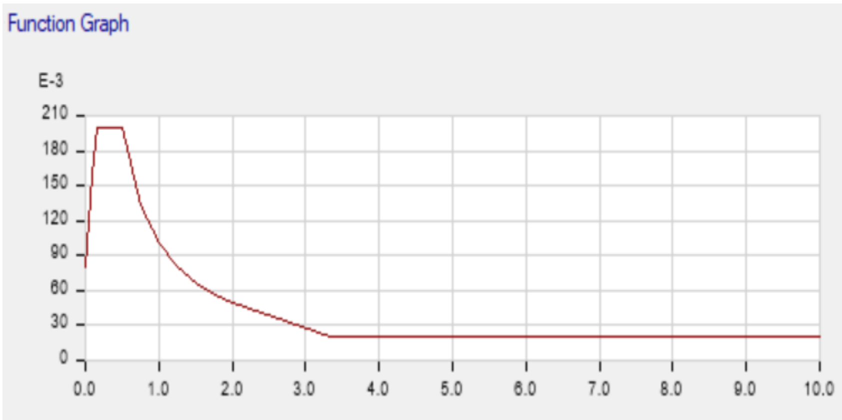


Figure 65: The response spectrum for the Eurocode model from ETABS

After obtaining the response spectrum for both models, they have been combined in one graph to check the differences as shown below:

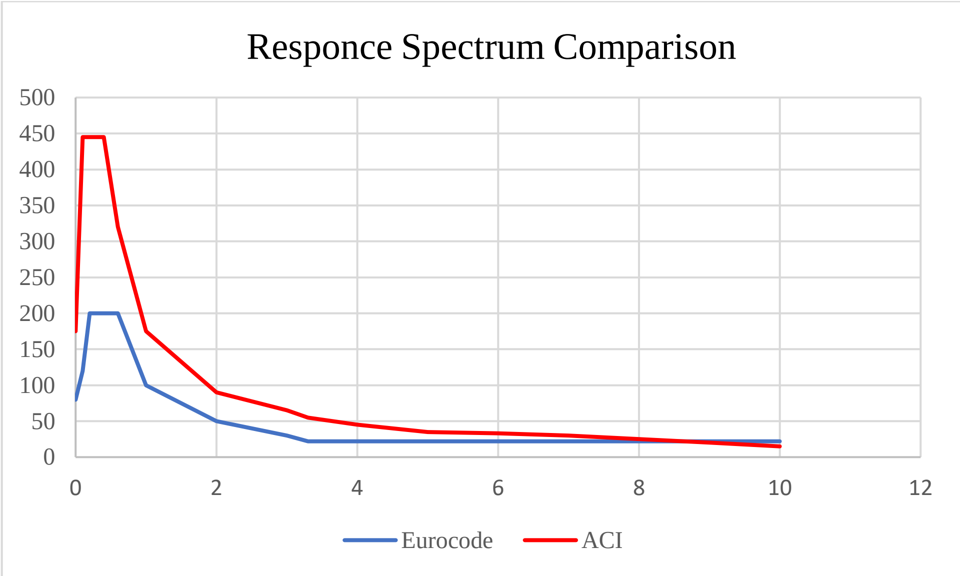


Figure 66: Response spectrum comparison between ACI Code and Eurocode Models

<u>Discussion of the Response Spectrum:</u>

The response spectrum is a function of the natural frequency of the oscillator and of its damping. This method relays on proximity. As shown in figure 65, the response spectral acceleration for the ACI model is more than the Eurocode model.

4.3.8- Mass Participation Ratio (For Ux and Uy)

The tables show the mass participation for both models in x-direction and y-direction.

The results confirm the analytical model validity in this study and shows that the stiffness of the building is homogenous and the mass participation ratio reaching more than 90% for both models. For the ACI model, the minimum modes number considered to achieve the mass participation ratio were 19 modes, while for the Eurocode model, the minimum number of modes considered to achieve the mass participation ratio were 17 modes to reach mass participation ratio greater than 90% in both directions.

Table 58: Mass participation for the ACI 318-19 model in x and y directions

Mode	UX	UY	SUM UX	SUM UY
1	0.0031	0.001	0.0031	0.001
2	0.6923	0.0005	0.6954	0.0015
3	0.0006	0.6123	0.696	0.6138
4	0.0007	0.0001	0.6967	0.6138
5	0.1393	0.000001625	0.836	0.6138
6	0	0.1899	0.836	0.8037
7	0.0002	0.0003	0.8362	0.804
8	0.0505	0	0.8867	0.804
9	0.0001	0	0.8868	0.804
10	0.029	0.00001394	0.9158	0.804
11	0.000006228	0.0769	0.9158	0.881
12	0.0001	0.00002264	0.9158	0.881
13	0.0163	0.00000169	0.9322	0.881
14	0.0019	0	0.9341	0.881
15	0.0001	0.000004869	0.9342	0.881
16	0.0004	0	0.9346	0.881
17	0.0006	0.0019	0.9352	0.8829
18	0.0113	0.0004	0.9465	0.8833
19	0.0003	0.039	**0.9468**	**0.9223**
20	0.0004	0	0.9472	0.9223
21	0.0004	0	0.9476	0.9223
22	0.00001458	0	0.9476	0.9223
23	0.0087	0.000001013	0.9563	0.9223
24	0.000001818	0.0229	0.9563	0.9452
25	0.0072	0	0.9635	0.9452
26	0.0003	0.00002829	0.9638	0.9452
27	0.000004154	0.0134	0.9638	0.9586
28	0.007	0.000001739	0.9708	0.9586
29	0.00001031	0.0097	0.9708	0.9683
30	0.0086	0.00001024	0.9794	0.9683
31	0.00001489	0.0111	0.9794	0.9794
32	0.0096	0.000009232	0.989	0.9794
33	0.00001425	0.0106	0.989	0.99
34	0.0092	0.00001147	0.9982	0.99
35	0.00001226	0.0086	**0.9982**	**0.9987**

Table 59: Mass participation for the Eurocode model in x and y directions

Mode	UX	UY	SUM UX	SUM UY
1	0.002	0.0021	0.002	0.0021
2	0.00001414	0.6182	0.002	0.6203
3	0.6922	0.00003375	0.6942	0.6204
4	0.0004	0.0001	0.6947	0.6204
5	0.1534	0.000007982	0.848	0.6204
6	0.000005029	0.1865	0.848	0.8069
7	0.0001	0.0001	0.8481	0.807
8	0.0487	0.000001294	0.8968	0.807
9	0.0000332	0	0.8968	0.807
10	0.0006	0.0735	0.8974	0.8805
11	0.0256	0.0019	0.923	0.8824
12	0.00002173	0.000003861	0.9231	0.8824
13	0.0159	0.000005012	0.939	0.8824
14	0.0001	0.000002941	0.939	0.8824
15	0.0001	0	0.9391	0.8824
16	0.0005	0	0.9395	0.8824
17	0.000003889	0.0408	**0.9395**	**0.9232**
18	0	0.00002173	0.9395	0.9232
19	0.0107	0.000001115	0.9503	0.9232
20	0.0002	0	0.9504	0.9232
21	0.000002119	0	0.9504	0.9232
22	0.0032	0	0.9536	0.9232
23	0.0054	0.00003889	0.9589	0.9233
24	0.00001852	0.0227	0.959	0.9459
25	0.007	0.000002265	0.966	0.9459
26	0.000009187	0.0132	0.966	0.9591
27	0.0059	0.0001	0.9718	0.9592
28	0.0014	0.0034	0.9732	0.9626
29	0.0009	0.0068	0.974	0.9694
30	0.0073	0.0007	0.9813	0.9701
31	0.0003	0.0101	0.9816	0.9802
32	0.0084	0.0009	0.99	0.9811
33	0.0006	0.0094	**0.9905**	**0.9906**

Discussion of the Mass Participation Results:

As shown from the previous tables, the mass participation in both directions for both models are more than 90% at different modes, whoever the mass participation reaches more than 90% for the Eurocode model at mode 17 while for the ACI model it is reached at mode 19. These mass participation results represent how strong a mass would take a part in the reaction of the building when facing force/displacement in a certain direction and based on the previous data the Eurocode model can face more force/displacement than the ACI model in case of any reaction of the building.

4.3.9- Structural Time Period

The time periods have been extracted from the ETABS models, taking into consideration the service model in this analysis. Only the first 3 modes have been considered in the analysis.

Table 60: Building time period for the ACI 318-19 code and the Eurocode models

Mode	Time Period from the ACI 318-19 code (Seconds)	Time Period form the Eurocode (Seconds)	Time Period Difference (Seconds)	The Code with Higher Time Period
1	8.081	7.109	0.972	ACI Code
2	6.843	6.948	0.105	Eurocode
3	6.662	6.021	0.641	ACI Code

The following are the modes shapes for the first three modes:

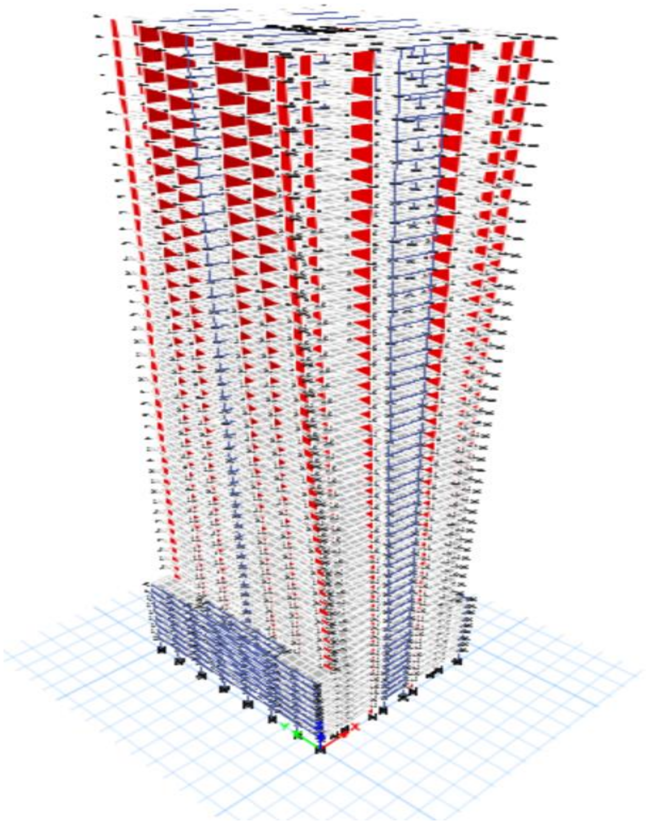


Figure 67: Mode Shape 1 (ACI Model): 8.08 seconds

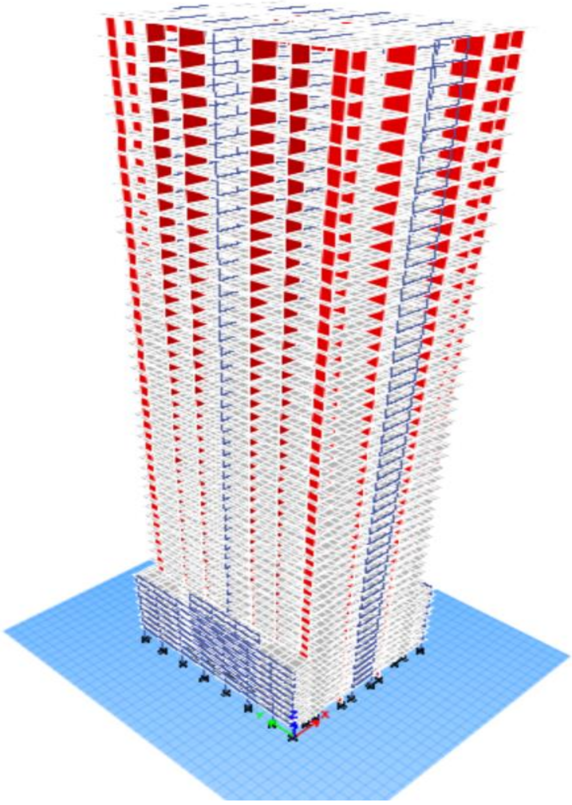


Figure 68: Mode Shape 1 (Eurocode Model): 7.11 seconds

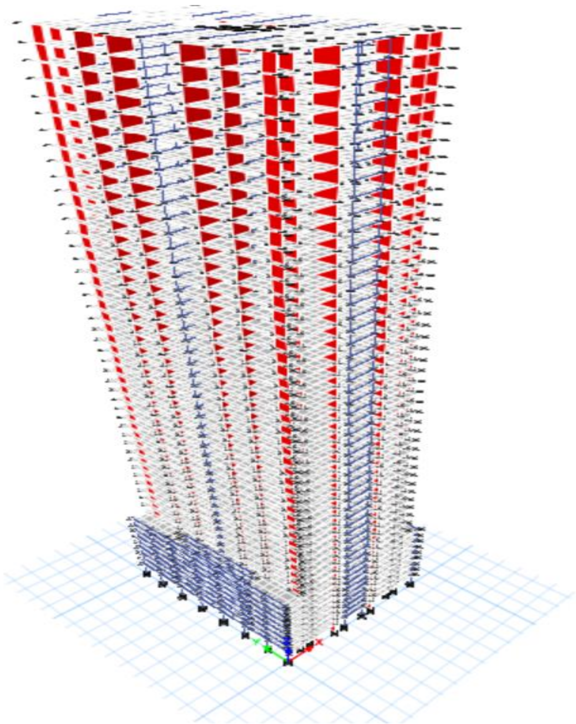


Figure 69: Mode Shape 2 (ACI Model): 6.84 seconds

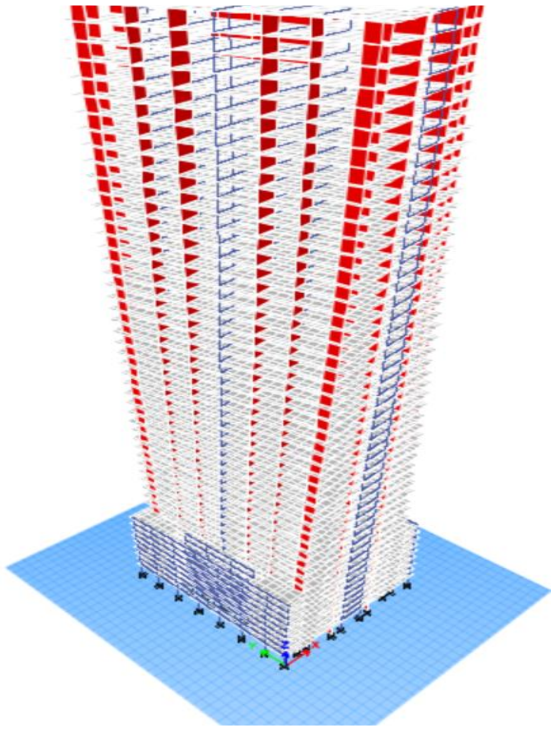


Figure 70: Mode Shape 2 (Eurocode Model): 6.95 seconds

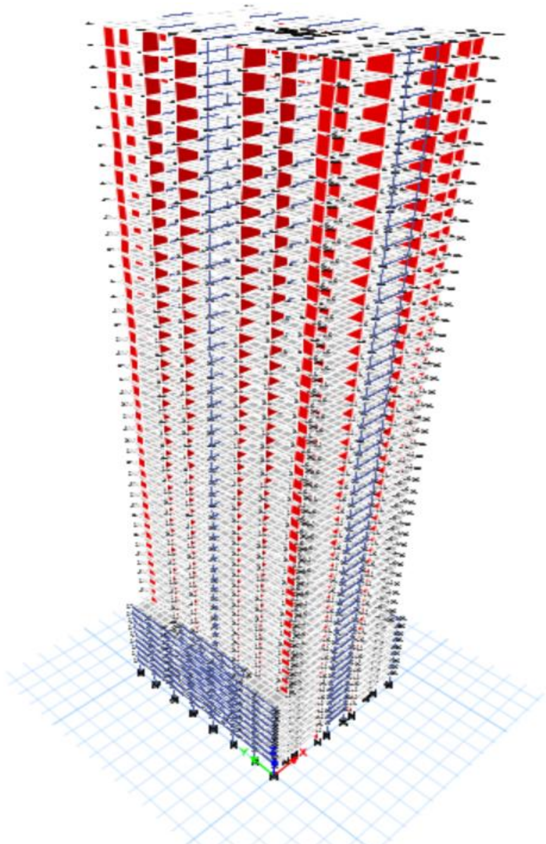


Figure 72: Mode Shape 3 (ACI Model): 6.66 seconds

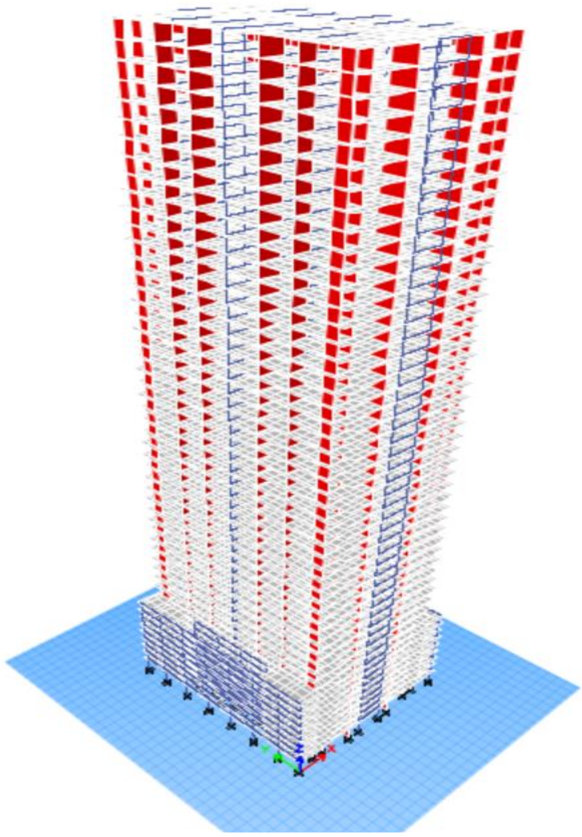


Figure 71: Mode Shape 3 (Eurocode Model): 6.02 seconds

Discussion of the Time Period Results:

For the data comparison, the service models have been taken into account. The first 3 time periods have been taken in the analysis and as noticed the ACI model has a time period which is more than the Eurocode for modes 1 and 3 by 0.972 seconds and 0.641 seconds respectively, while for mode 2, the time period is very close to each other with difference of 0.105 seconds only. The data obtained in this analysis is logical since there is a difference in the modulus of elasticity which basically reduce the time period of the structure as it gets higher which is the case in the Eurocode model. Moreover, the time period depends on many different parameters such as the concrete strength, modulus of elasticity and the building height; as well as the consideration of the building's longitudinal or transverse direction as mentioned by Bhuskade, Meghe & Sagane, the difference between the time periods between these 2 codes for different structures will vary and the data obtained here will give an idea about the time period difference between the ACI318-19 code and the Eurocode.

4.4- Wind Analysis

The buildings in both codes have found to be safe against the wind drift based on the adjustments made in the initial models. Carrying on for the wind design analysis the drifts form both codes have been compared and each model have satisfied the limitations specified in each code. In this section the wind comparison will be discussed through demonstrating the structural characteristics of the building and summary of the achieved results.

4.4.1- Wind Storey Drift (In X-Direction)

Table 61: Wind storey drift comparison in x-direction

Storeys with Highest Wind Drift (ACI code)	Drift in ACI Model (Percentage)	Storeys with Highest Wind Drift (Eurocode)	Drift in Eurocode Model (Percentage)
12th OFFICE	0.002269	2nd PODIUM	0.003947
11th OFFICE	0.002269	3rd PODIUM	0.003661
16th OFFICE	0.002268	1st OFFICE	0.003436
13th OFFICE	0.002265	4th PODIUM	0.003383
10th OFFIEC	0.002264	8th PODIUM	0.003369
14th OFFICE	0.002259	7th PODIUM	0.003269
9th OFFICE	0.002253	5th PODIUM	0.003237
15th OFFICE	0.002252	6th PODIUM	0.003211
17th OFFICE	0.00225	16th OFFICE	0.003084
8th OFFICE	0.002234	17th OFFICE	0.003076
18th OFFICE	0.002228	18th OFFICE	0.003072

Discussion of Inter-storey Wind Drift Results Comparison in the X-Direction:

As shown from the previous table the maximum inter-storey wind drift in the x-direction for the ACI model is 0.002269 which is less than the limit specified in the code 1/400 (0.0025), while for the Eurocode the maximum inter-storey wind drift in the x-direction have found to be 0.003947 which is less than the limit specified in the code 1/200 (0.0050). Based on the previous table the Eurocode shows more inter-storey wind drift compared to the ACI code, the reason mainly can be referred to the different load combinations assigned for both codes, since the Eurocode has larger wind modifiers when it is compared to the ACI modifiers for wind loads combinations.

Following the same procedure in the wind drift determination in the x-direction, the wind drifts have been determined in the y-direction as shown in the table below:

Table 62: Wind storey drift comparison in y-Direction

Storeys with Highest Wind Drift (ACI code)	Drift in ACI Model (Percentage)	Storeys with Highest Wind Drift (Eurocode)	Drift in Eurocode Model (Percentage)
OFFICE 19	0.001621	OFFICE 21	0.002899
OFFICE 18	0.00162	OFFICE 20	0.002899
OFFICE 20	0.001618	OFFICE 22	0.002895
OFFICE 17	0.001615	OFFICE 19	0.002894
OFFICE 21	0.001612	RESIDENTIAL 23	0.002886
OFFICE 22	0.001604	OFFICE 18	0.002883
OFFICE 16	0.001601	RESIDENTIAL 24	0.002874
RESIDENTIAL 23	0.001593	OFFICE 17	0.002868
OFFICE 19	0.001592	RESIDENTIAL 25	0.002858
RESIDENTIAL 24	0.001581	OFFICE 16	0.002848
OFFICE 15	0.00158	RESIDENTIAL 26	0.002839

Discussion of Wind Storey Drift Results Comparison in the Y-Direction:

As shown from the previous table the maximum inter-storey wind drift in the y-direction for the ACI model is 0.001621 which is less than the limit specified in the code 1/400 (0.0025), while for the Eurocode the maximum inter-storey wind drift in the y-direction have found to be 0.002899 which is less than the limit specified in the code 1/200 (0.0050). Based on the previous table the Eurocode shows more inter-storey wind drift compared to the ACI code similar to the condition in the x-direction. It can be concluded that the Eurocode gives more permittivity for the wind drift when it is compared to ACI code.

4.5- Cost Analysis

The structural elements cost analysis is considered as one of the most important analysis in the design. Based on the data provided by the ETABS and RAM Concept software, the most conservative code can be determined. The columns and shear walls will be compered in terms of the amount of reinforcement used. Moreover, based on the reinforcement ratio comparison, it can be determined which code would give less cross-sectional area. The columns and walls cost will be compared using the ETABS software, the cost here will be compared through the amount of reinforcement used in vertical element, while for the slabs cost comparison, the RAM concept will be used to check the amount of reinforcement used as well as the amount of post tension rate in each model after taking into account the seismic and wind effects.

4.5.1- Cost Comparison for Columns using ETABS

In this section the columns will be compared based on the reinforcement ratio, since both of the models have been designed with the same columns cross section area (Sizing).

The most conservative code can be known by determining the reinforcement ratio difference between the two codes.

The following are the reinforcement ratios obtained from each model.

Table 63: Column reinforcement ratio comparison for 1st podium floor

Columns	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1- 0.80 m x1.50 m	0.705	0.694	0.011	Eurocode
C2-1.20 m x 1.30 m	0.737	0.628	0.109	Eurocode
C3-1.70 m x 1.50 m	0.725	0.678	0.047	Eurocode
C3A-1.80 m x 1.60 m	0.842	0.822	0.02	Eurocode
C4-1.10 m x1.50 m	0.809	0.682	0.127	Eurocode
C4A-1.20 m x 1.50 m	0.836	0.987	0.151	ACI Code
C5-0.40 m x 0.90 m	0.531	0.805	0.247	ACI Code

Discussion of the Conservative Code for Concrete Columns Design:

Based on the table above, the conservative code for concrete columns design in the 1st podium floors is the Eurocode. As shown previously most of the columns designed according to Eurocode require less reinforcement compared to the ACI code. The following is a calculation showing the reinforcement difference:

Table 64: Amount of reinforcement comparison between the codes for columns

Columns	Number of Columns	Ratio Difference	Number of Reinforcement Bars	Amount of Reinforcement Difference (mm^2)
C1- 0.80 m x1.50 m	4	0.011	42 Bars of T32	1,485.792
C2-1.20 m x 1.30 m	4	0.109	36 Bars of T32	12,619.584
C3-1.70 m x 1.50 m	2	0.047	56 Bars of T32	4,232.256
C3A-1.80 m x 1.60 m	4	0.02	60 Bars of T32	3,859.2
C4-1.10 m x1.50 m	2	0.127	40 Bars of T32	8,168.64
C4A-1.20 m x 1.50 m	2	0.151	44 Bars of T32	10,683.552
C5-0.40 m x 0.90 m	24	0.247	18 Bars of T16	21,447.504

- The amount of reinforcement saved by Eurocode is 1,485.792 mm^2 + 12,619.584 mm^2 + 4,232.256 mm^2 + 3,859.2 mm^2 + 3,859.2 mm^2 + 8,168.64 mm^2 + 10,683.552 mm^2 = 41,049.02 mm^2.
- The amount of reinforcement saved by ACI code is 21,447.504 mm^2
- The difference in reinforcement saving is 41,049.02 mm^2 – 21,447.504 mm^2 = 19,601.516 mm^2.

Based on the previous calculation difference it can be concluded that the Eurocode saves more reinforcement compared to the ACI code, and just in this 1st podium floor the amount of reinforcement saved was 19,601.516 mm^2, which is almost equivalent to 98 bras of T16.

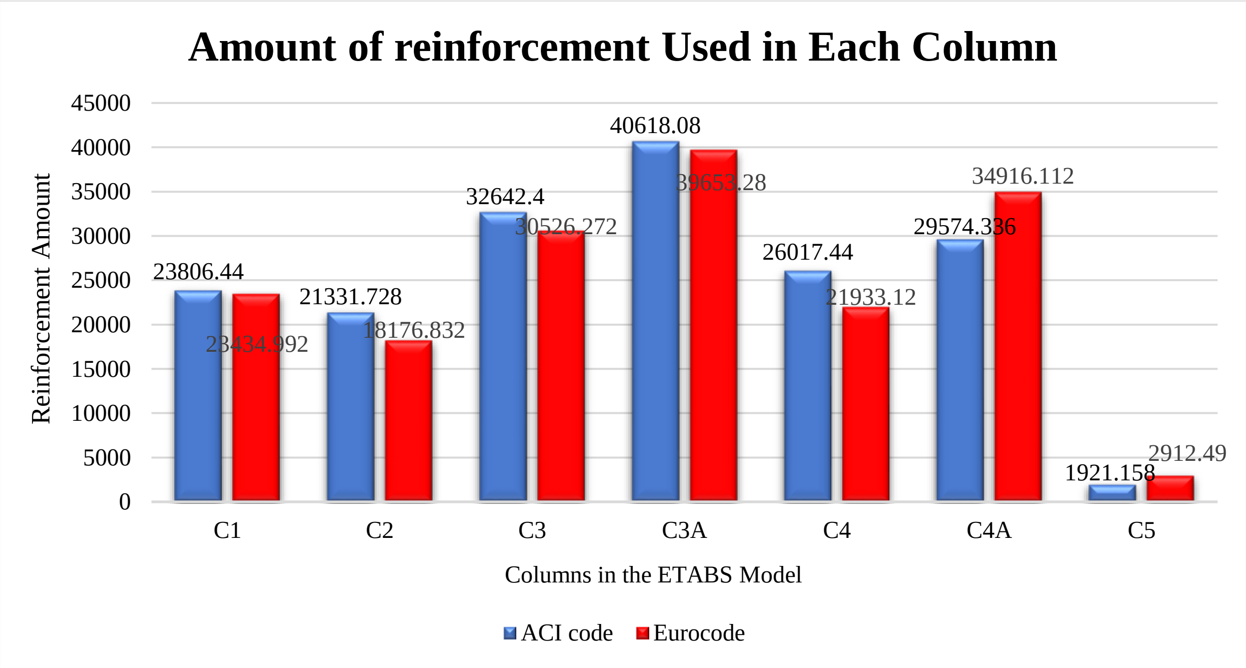


Figure 73: Amount of reinforcement difference for each column

In order to confirm the outputs of the previous data, more columns have been compared at different levels of the building. Since the first comparison was at the 1st podium floor, there will be one more comparison at the middle floors of the building and one at the higher floors.

The considered floors are the 25th floor and the 49th floor. The rest of the floors are provided in Appendix E.

Table 65: Column reinforcement ratio comparison for 25th floor (17th Office)

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1-0.60 m x 1.0 m	0.743	0.647	0.096	Eurocode
C2-1.0 m x 1.0 m	0.732	0.643	0.089	Eurocode
C3-1.50 m x 1.20 m	0.781	0.858	0.077	ACI Code
C4-0.80 m x 1.10 m	0.914	0.964	0.05	ACI Code

Table 66: Column reinforcement ratio comparison for 49th floor (42nd Residential)

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1A-0.40 m x 0.70 m	0.374	0.246	0.128	Eurocode
C2A-0.60 m x 0.60 m	0.144	0.135	0.009	Eurocode
C3-1.10 m x 0.70 m	0.212	0.362	0.15	ACI Code
C3A- 0.90 m x 0.90 m	0.224	0.288	0.064	ACI Code
C4-0.60 m x 0.80 m	0.188	0.168	0.02	Eurocode
C4A-0.50 m x 0.80 m	0.167	0.405	0.238	ACI Code

<u>Discussion of the Columns Reinforcement Comparison Results:</u>

As it can be concluded from all the previous data, the Eurocode model is giving less reinforcement for many column sections when they are compared to the sections in the ACI model, which make the Eurocode more conservative in the concrete columns design.

4.5.2- Cost Comparison for Shear Walls using ETABS

In this section the shear walls will be compared in terms of reinforcement ratio for each corresponding shear wall in each model since both of the models have been designed with the same shear walls cross sectional area (Sizing). By knowing the reinforcement ratio difference between the two codes, the most conservative code can be determined.

The following is the plan view for the shear walls being compared in this dissertation:

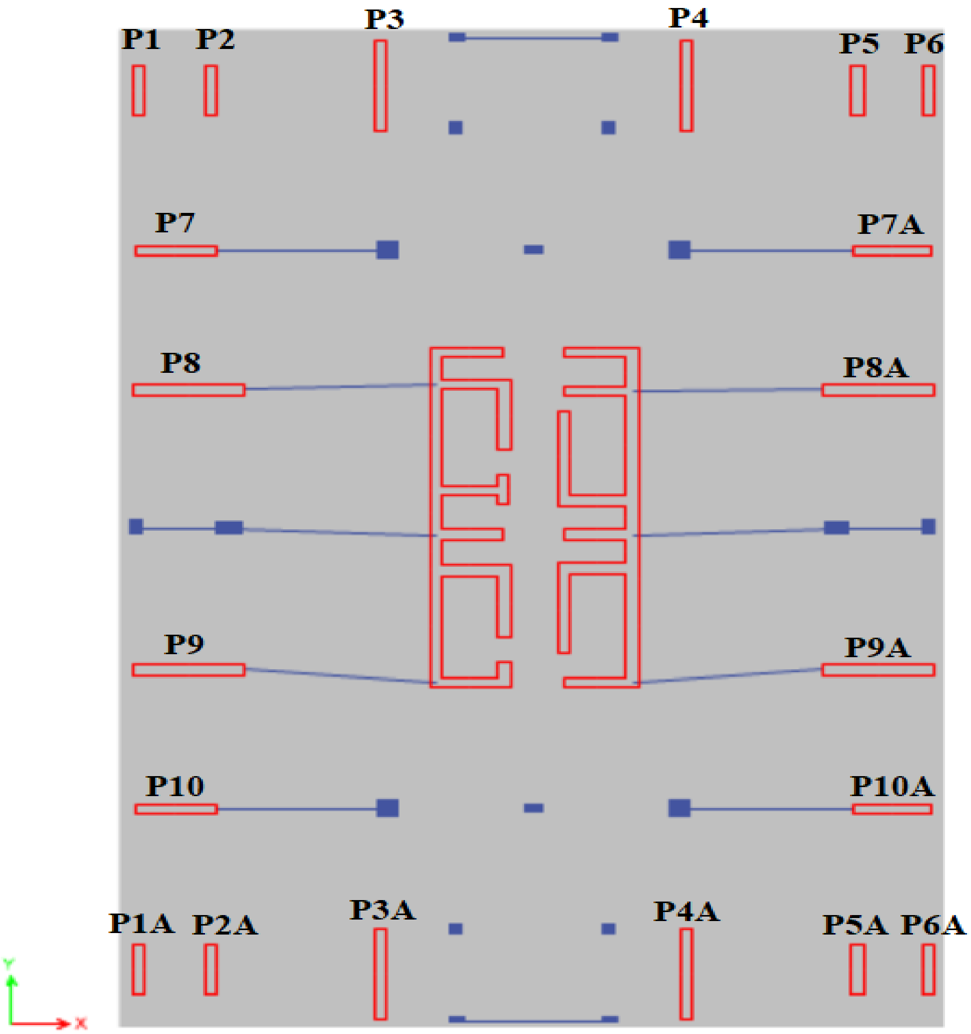


Figure 74: Shear walls naming in the ETABS models

Table 67: shear walls reinforcement ratio comparison for 1st podium floor

Shear Wall Name	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T32@100cm	0.687	0.544	0.143	Eurocode
P1A	T32@100cm	0.685	0.555	0.13	Eurocode
P2	T32@100cm	0.727	0.543	0.184	Eurocode
P2A	T32@100cm	0.735	0.55	0.185	Eurocode
P3	T32@100cm	0.562	0.427	0.135	Eurocode
P3A	T32@100cm	0.564	0.437	0.127	Eurocode
P4	T32@100cm	0.561	0.425	0.136	Eurocode
P4A	T32@100cm	0.579	0.45	0.129	Eurocode
P5	T32@100cm	0.718	0.536	0.182	Eurocode
P5A	T32@100cm	0.755	0.565	0.19	Eurocode
P6	T32@100cm	0.672	0.533	0.139	Eurocode
P6A	T32@100cm	0.650	0.529	0.121	Eurocode
P7	T32@80cm	0.773	0.717	0.056	Eurocode
P7A	T32@80cm	0.764	0.678	0.086	Eurocode
P8	T32@100cm	0.744	0.728	0.016	Eurocode
P8A	T32@100cm	0.731	0.719	0.012	Eurocode
P9	T32@100cm	0.764	0.756	0.008	Eurocode
P9A	T32@100cm	0.751	0.742	0.009	Eurocode
P10	T32@100cm	0.807	0.741	0.066	Eurocode
P10A	T32@100cm	0.793	0.735	0.058	Eurocode

Discussion of the Shear Walls Reinforcement Comparison Results:

As shown from the previous table it can be concluded that the Eurocode model require less reinforcement than the ACI code model, which makes the Eurocode more conservative than ACI code in the concrete shear walls design.

The following is a calculation showing the reinforcement difference. The reinforcement below is only for 1 meter strip.

Table 68: Amount of reinforcement comparison between the codes for shear walls

Shear Wall Name	ACI Code Reinforcement (mm^2)	Eurocode Reinforcement (mm^2)
P1	5523.48	4373.76
P1A	5507.4	4462.2
P2	5845.08	4365.72
P2A	5909.4	4422
P3	4518.48	3433.08
P3A	4534.56	3513.48
P4	4510.44	3417
P4A	4655.16	3618
P5	5772.72	4309.44
P5A	6070.2	4542.6
P6	5402.88	4285.32
P6A	5226	4253.16
P7	7768.65	7205.85
P7A	7678.2	6813.9
P8	5981.76	5853.12
P8A	5877.24	5780.76
P9	6142.56	6078.24
P9A	6038.04	5965.68
P10	6488.28	5957.64
P10A	6375.72	5909.4

Based on the previous table it can be concluded that the Eurocode gives less reinforcement when it is compared to the ACI code. The previous table shows the reinforcement in 1 meter strip.

The following is a graph for the previous table:

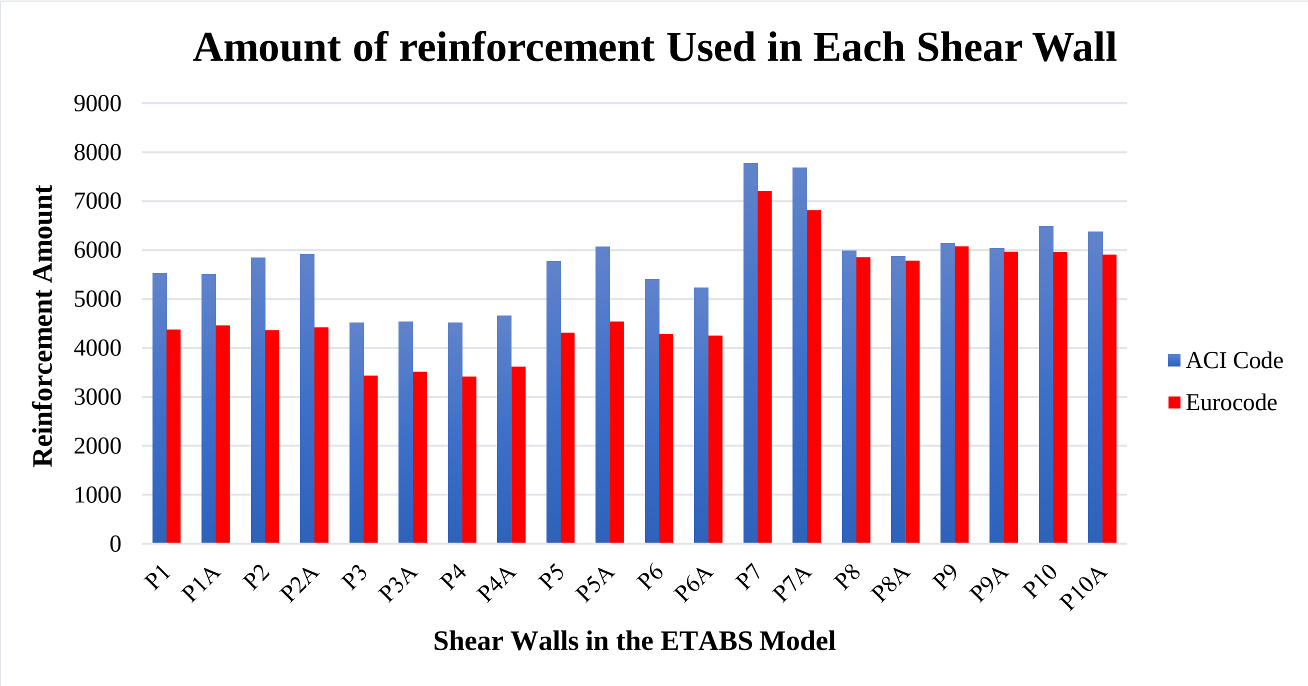


Figure 75: Amount of reinforcement difference for each shear wall

For other shear walls in the other floors the same analysis has been done for each one, and it found that the Eurocode is the conservative code when it comes for the shear walls design. The shear walls tables for other floors are in Appendix F.

4.5.3- Cost Comparison for Slabs using RAM Concept

For cost checking for the slabs, the slabs have been designed as post tension slabs using RAM Concept software. The method of cost checking in this section will be divided into two part, the first parts will be about reinforcement comparison for both codes which is done by taking seismic and wind moments in the ETABS model at each column and compare it with the moment capacity at each column in the RAM Concept. Then based on the obtained results, it will be required to check the reinforcement difference between both the models (ACI model and Eurocode model) and determine which code is giving less reinforcement. The second part will be a post tension rate comparison, which can be obtained directly from the software.

There was a total of five post tension slab designs for each proposed code, so ending up with a total of ten post tension slabs design models. The below table summarizes the inputs assigned in each model. The cost checking in this section will be based on the amount of post tension used in each RAM Concept model as well as the amount of reinforcement used, the amount of concrete won't be checked since both of the slabs have exactly same dimensional plan as well as same thickness.

The table below shows the loading assigned for each model based on each code

Table 69: Summary of the loads input for the ACI 318-19 code models

Floors	Concrete Strength (Mpa)	Occupancy	Live load (kN/m^2)	Dead Load (kN/m^2)	Balconies LL (kN/m^2)	Corridors LL (kN/m^2)
Podiums	45	Podium	1.92	3.00	-	4.79
9th	45	Office	2.40	5.00	-	4.79
10th to 15th	45	Office	2.40	5.00	-	4.79
16th to 30th	40	Office	2.40	5.00	-	4.79
31st to 50th	40	Residential	1.92	5.00	4.79	4.79

Table 70: Summary of the loads input for the Eurocode models

Floors	Concrete Strength (Mpa)	Occupancy	Live load (kN/m^2)	Dead Load (kN/m^2)	Balconies LL (kN/m^2)	Corridors LL (kN/m^2)
Podiums	45	Podium	1.92	3.00	-	3.00
1st	45	Office	2.40	5.00	-	3.00
2nd to 7th	45	Office	2.40	5.00	-	3.00
8th to 22nd	40	Office	2.40	5.00	-	3.00
23rd to 42nd	35	Residential	1.92	5.00	4.00	3.00

Discussion of Assigned Loadings:

For the live loads considered in the ACI models, they have been taken directly from the code, while for the Eurocode models, a range of live loads loading was provided from the code as shown in table 88 in Appendix B. In order to get a better comparison, the live loads values for the Eurocode models have been taken similar to the loads in the ACI code since the values recommended by the ACI code are falling within the loading range specified by the Eurocode, while in some cases the Eurocode model have recommended less live loads for some areas like balconies and corridors, thus such values from the Eurocode were taken as they are. For the dead load's values, an approximate value has been taken depending on each occupancy.

Starting with the analysis of the podium floor slab, the design has been checked against the lateral loads as well as the gravity loads. The required checking has been applied for each slab to verify its adequacy and safety. The checking of the slab included the following: precompression checking, punching checking, long-term deflection limit, incremental deflection limit and design status checking.

Precompression Checking:

The following are the outputs from the models regarding the previous mentioned checking, which shows that the slabs are safe and adequate, which allow for safe reinforcement calculation and post tension rate determination. The data are for the 1st Podium Floor.

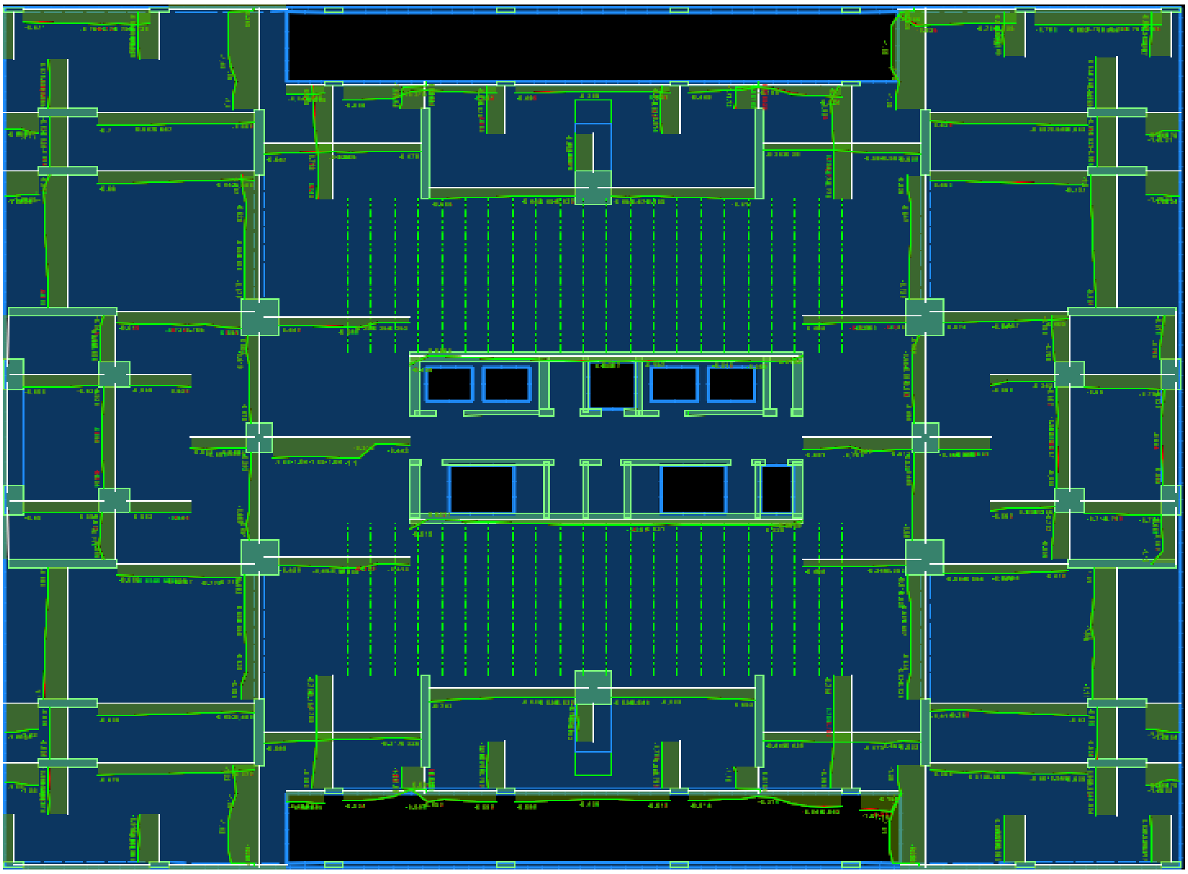


Figure 76: Precompression for the podium floor in ACI code model

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

Punching Checking:

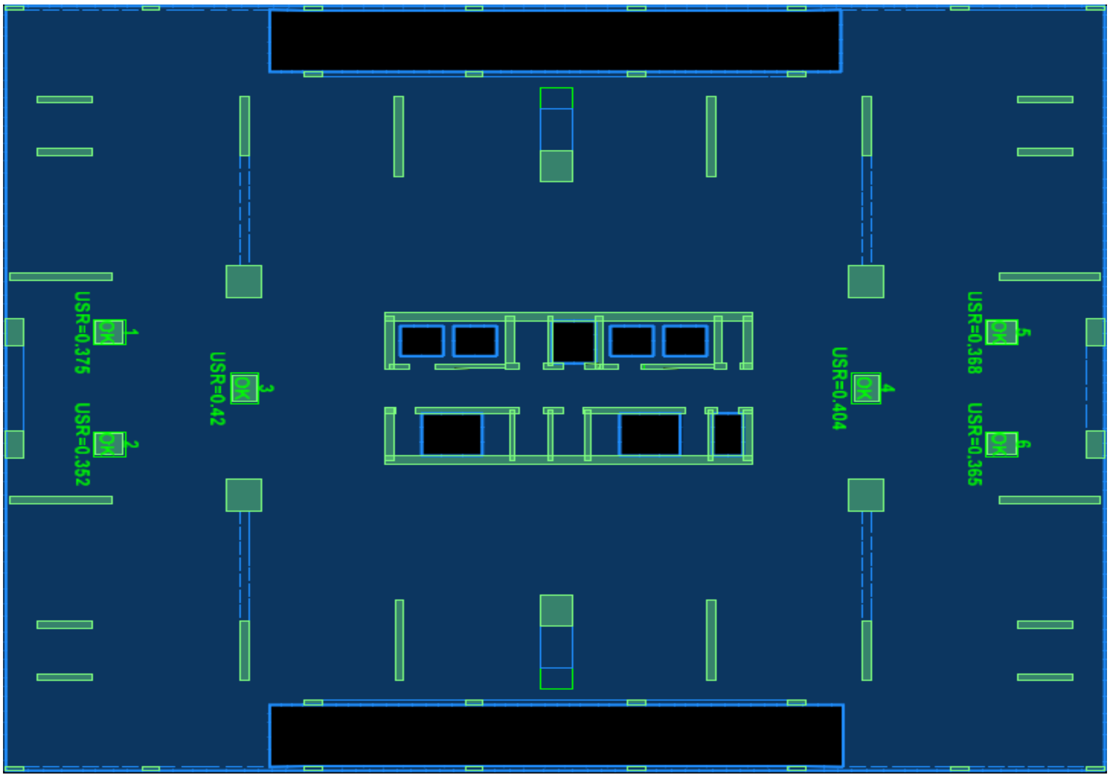


Figure 77: Punching for the podium floor in ACI code model

As shown in the figure above, the slabs are safe against punching.

Deflections Checking

For the deflections, there are mainly two types of deflections that need to be checked and the are the long-term deflection and the incremental deflection. As mentioned in the ACI code, the long-term deflection should not exceed L/240, where L is the longest span, while for the incremental defelction the deflection should not exceed L/480. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 31.67 mm, while the limit for the incremental deflection is 15.83 mm.

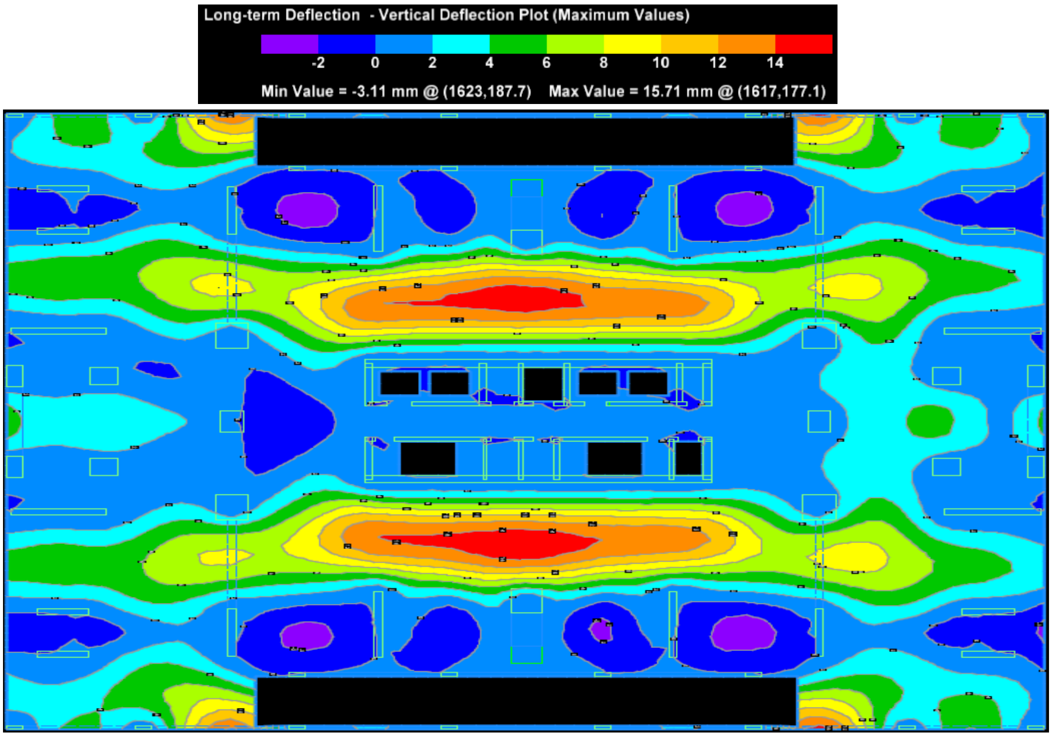


Figure 78: Long-term deflection for podium floors in ACI code model

As shown from the figure 77, the long-term deflection is 15.71 mm, which is less than the limit 31.67 mm, so the slab is safe in the long-term deflection checking.

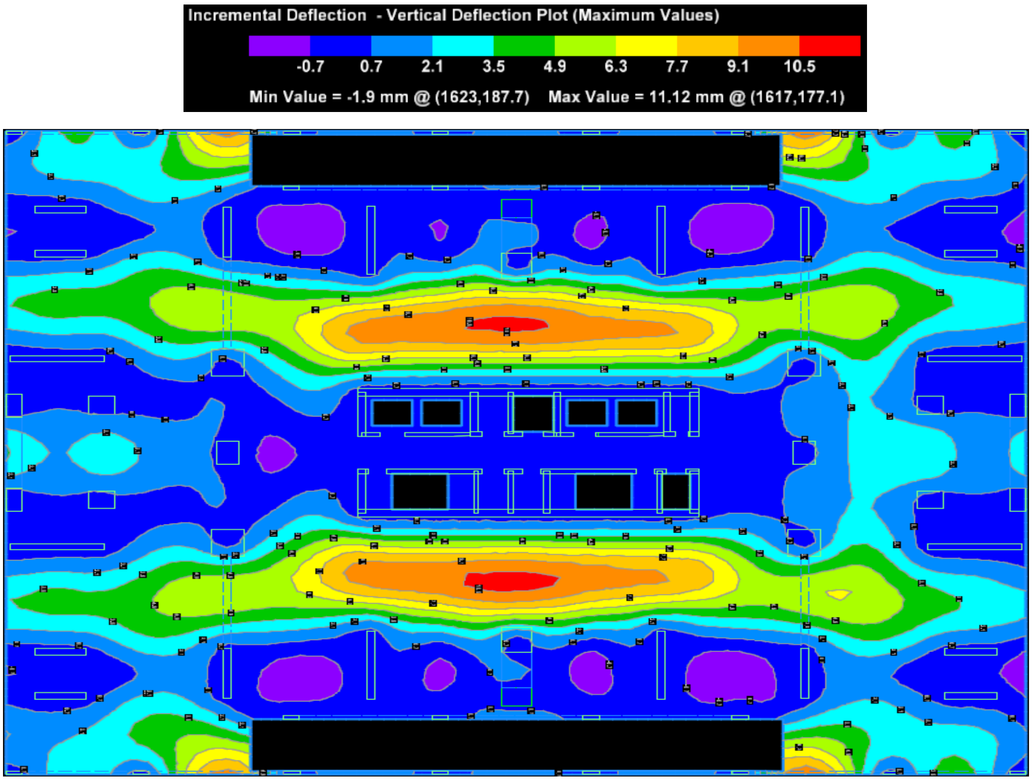


Figure 79: Incremental deflection for podium floors model in ACI code model

As shown from the figure 78, the incremental deflection is 11.12 mm, which is less than the limit 15.83 mm, so the slab is safe in the incremental deflection checking.

<u>Design Status Checking</u>

For the last checking which is the design status. Based on it, the stresses are checked if they are within the limit or not, and based on figure 79, the design status are all passing, which means that the stresses are all within the limit and the design is safe.

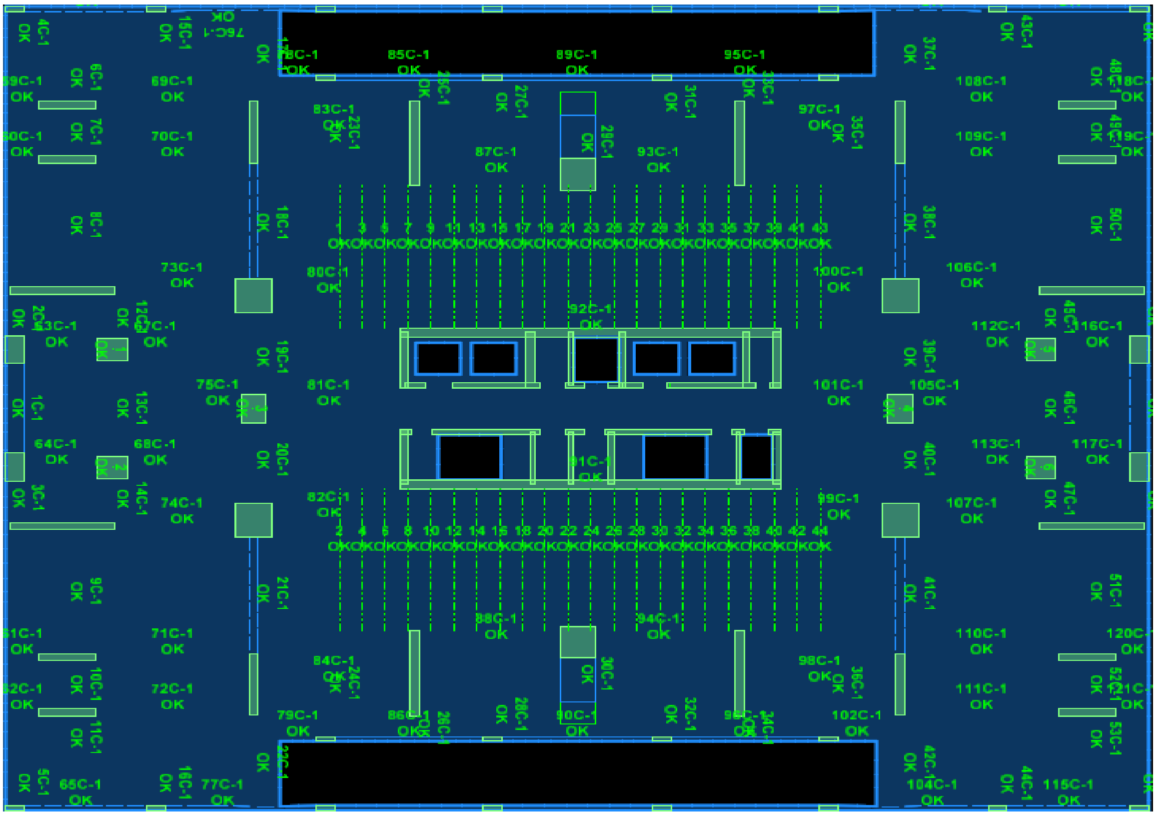


Figure 80: Design status for podium floors model in ACI code model

According to all the previuos checking in the post tension slab, the conclusion is that the slab is safe, the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model. All other slabs have followed the same procedure as for the 1st podiums slab. The data for the other slabs are available in APPENDIX G.

Following the procedure in the ACI models, the Eurocode results for the 1st podium floor are as the following. The only difference is that in Eurocode it considers the long-term deflection or the maximum deflection under quasi-permanent loads.

The outputs shows that the slabs are safe in all of the checking mentioned previously, which allow for safe reinforcement calculation and post tension rate determination.

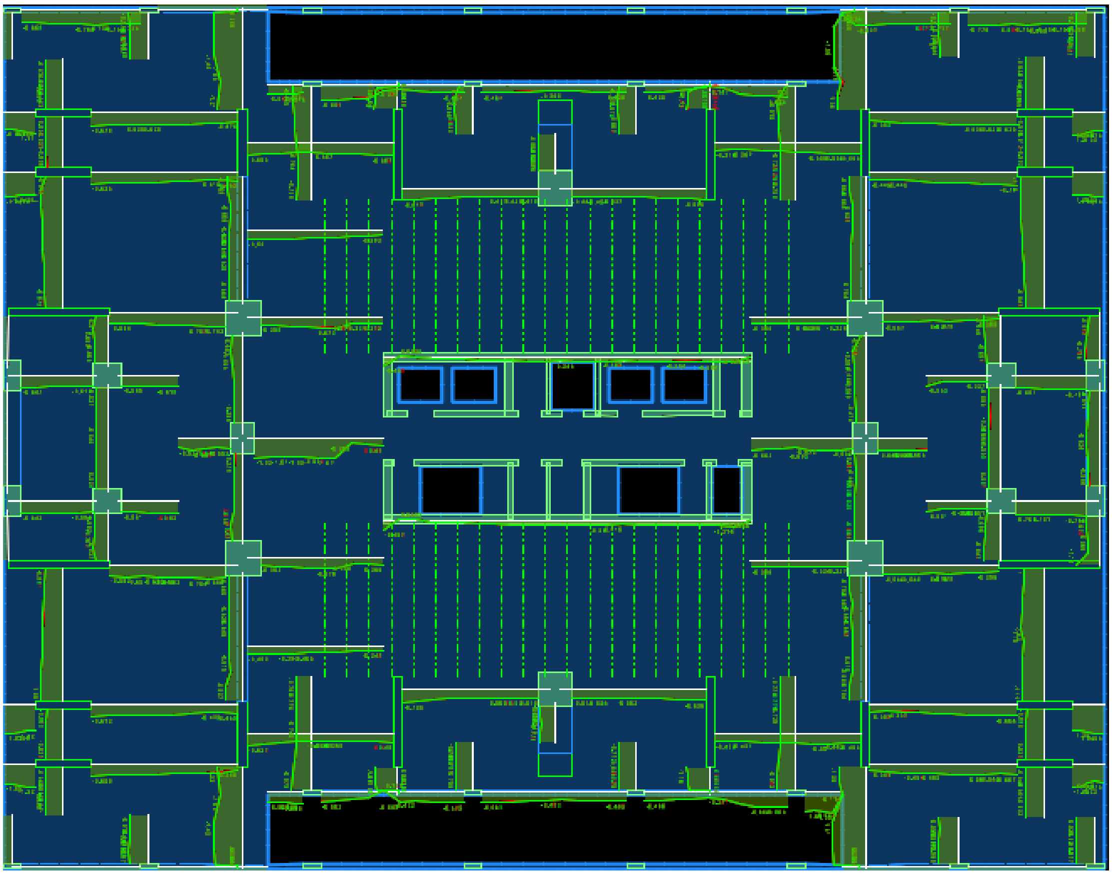


Figure 81: Precompression for 1st podium floor model in Eurocode model

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

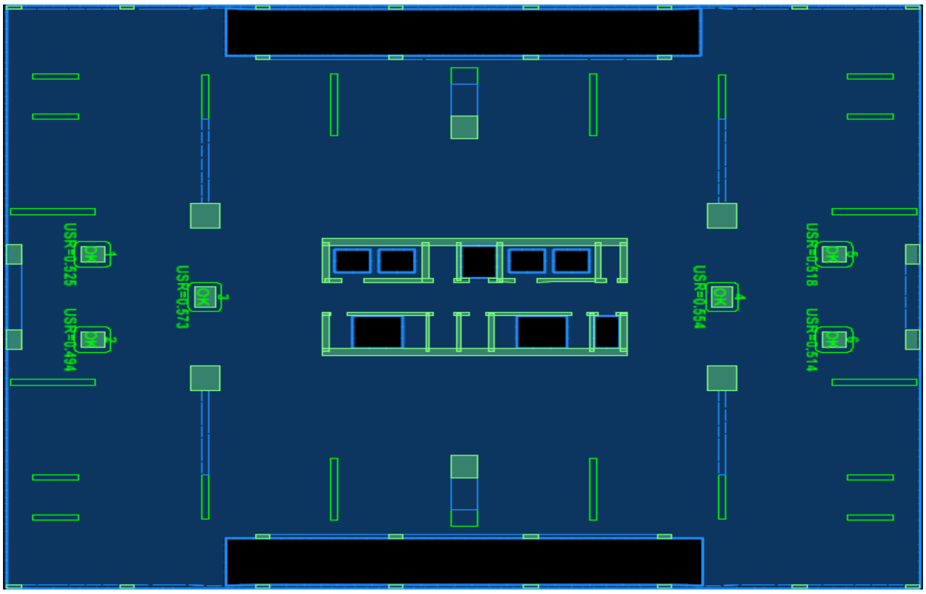


Figure 82: Punching for 1st podium floor model in Eurocode model

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long-term deflection or the maximum deflection under quasi-permanent loads should not exceed L/250, where L is the longest span. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 30.40 mm

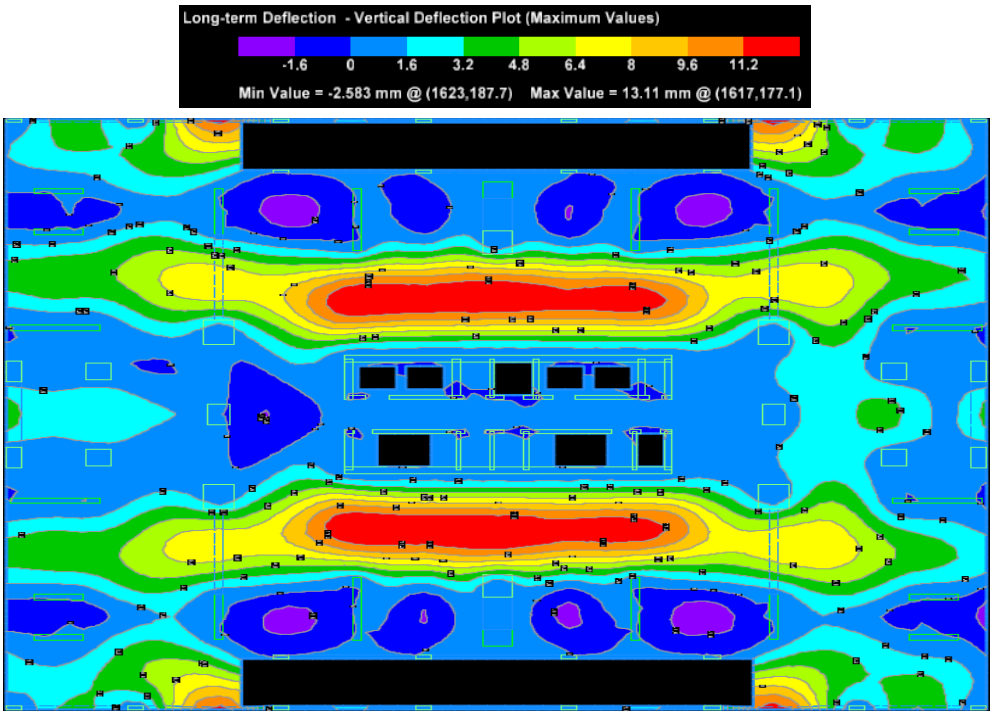


Figure 83: Long-term deflection for 1st podium floors model in Eurocode model

As shown from the figure 82, the long-term deflection is 13.11 mm, which is less than the limit 31.67 mm, so the slab is safe in the long-term deflcetion checking.

For the last checking which is the design status. Based on it, the stresses are checked if they are within the limit or not, and based on figure 83, the design status are all passing and all the stresses are within the limit.

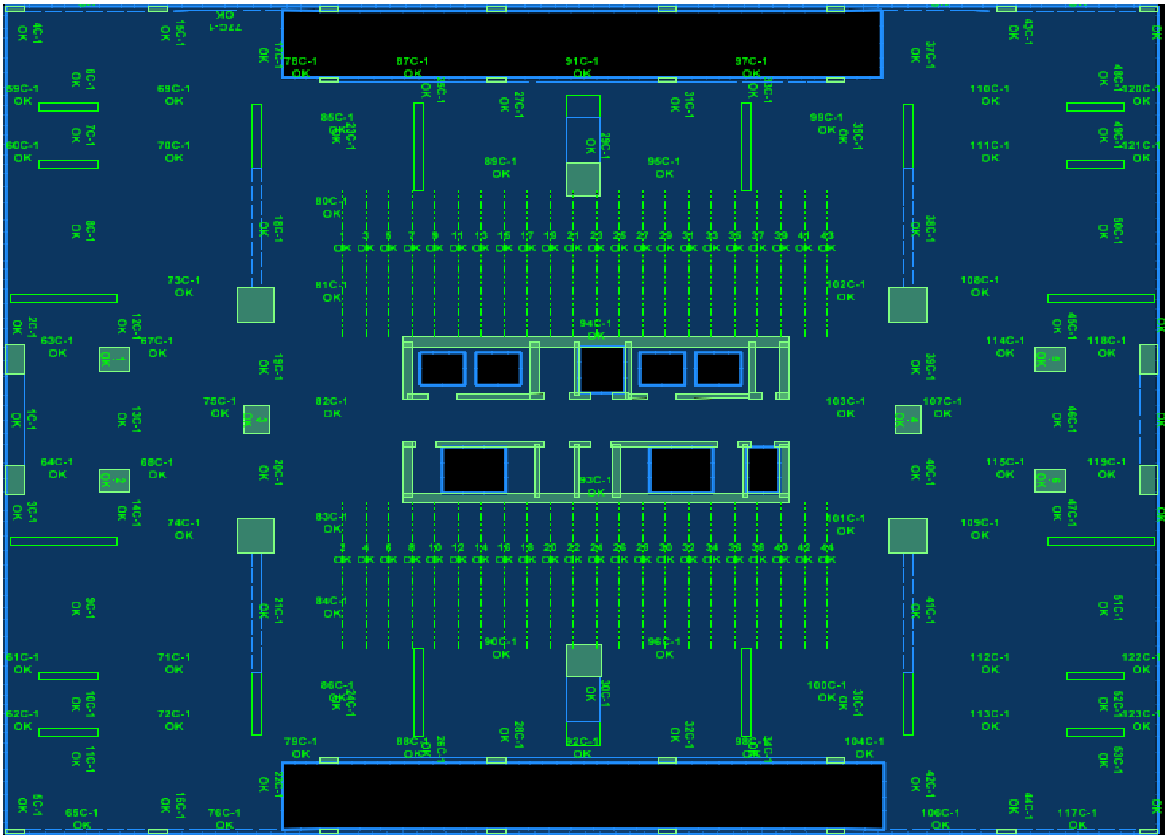


Figure 84: Design status for the 1st podium floors in Eurocode model

According to all the previuos checking in the post tension slab, the conclusion is that the slab is safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.

Few locations were taken for the reinofrcemnt analysis and they are as shown:

- **For Vertical Elements**

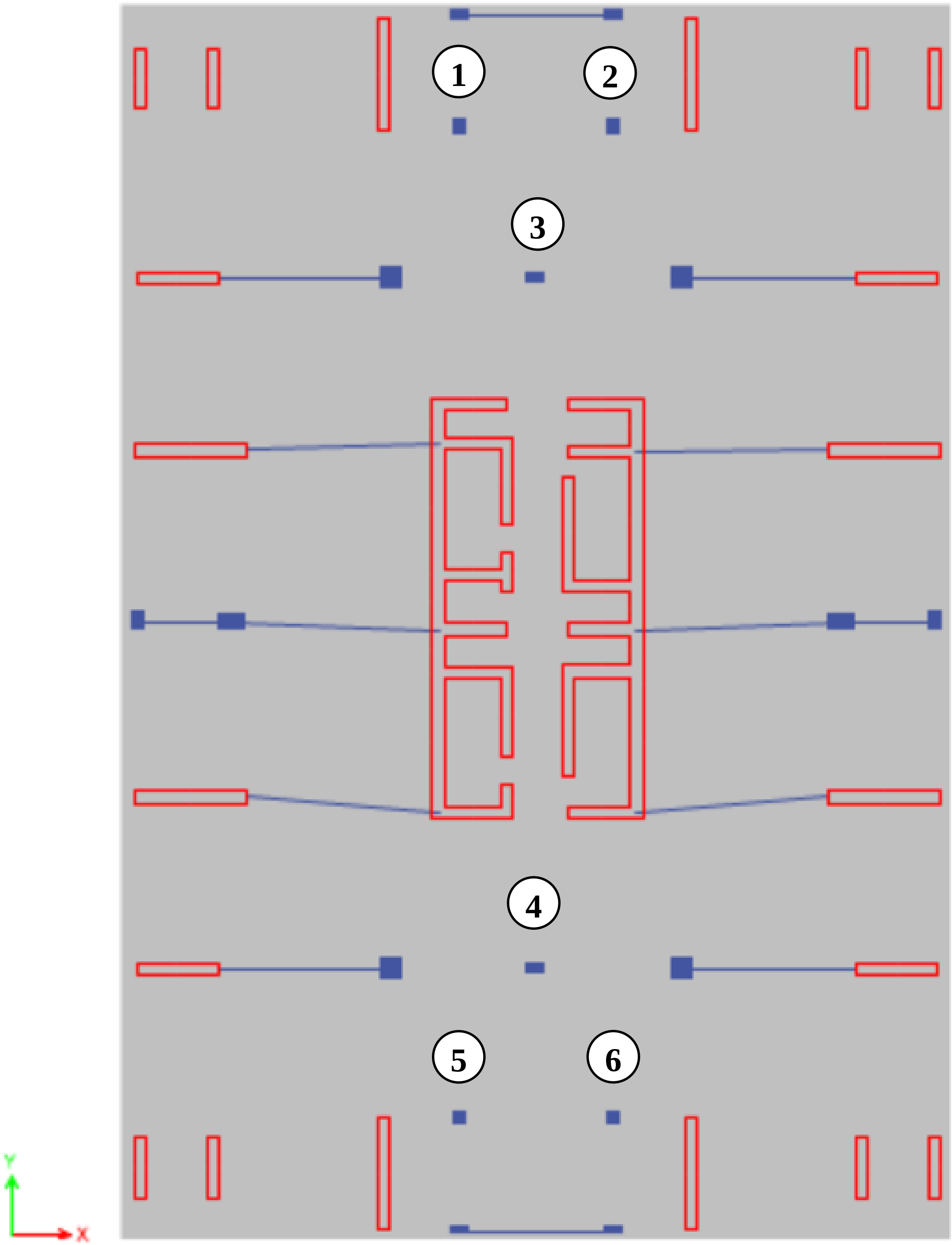


Figure 85: Final residential floor plan (Sample: 40th residential floor)

- **For Midspans**

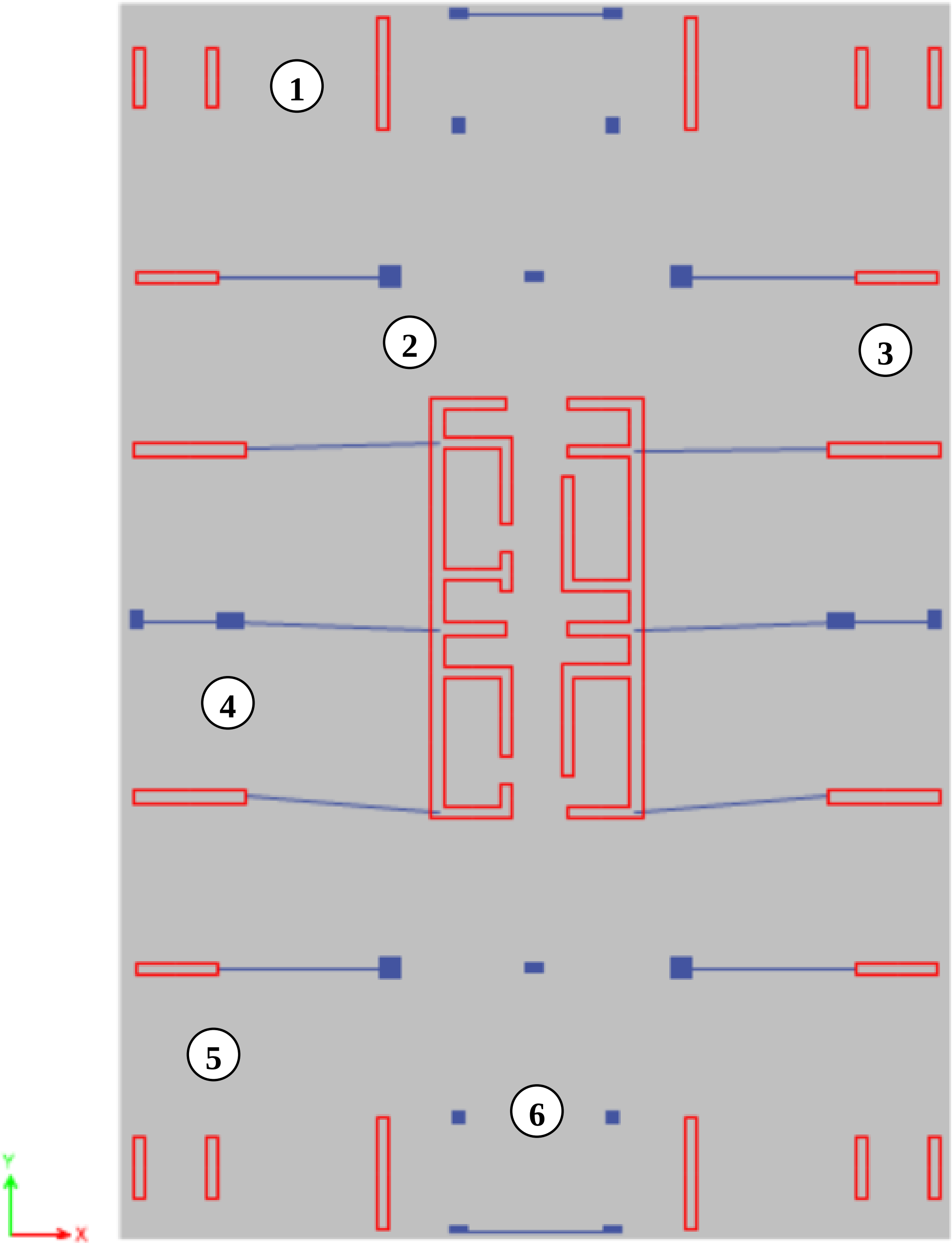


Figure 86: Final residential floor plan (Sample: 40th residential floor)

The following table shows the seismic and wind loads checking for ACI code:

Table 71: Slab reinforcement at the support's comparison due to lateral loads on the ACI slabs

Column / Shear Wall	Laterl Load Envelope Maimum Moment from ETABS ACI Model (kN.m)	Maximim Moment Capacity from RAM (kN.m)	Moment Difference	Required Reinorcement if any (mm^2)	Approximate Equivalent Reinforcement
1	1510.2	1378	132.2	1596.618	8T16
2	1469.6	1337	132.6	1601.449	9T16
3	566.2	374.6	191.6	2314.01	12T16
4	477.2	328.3	148.9	1798.309	10T16
5	1437.7	1238	199.7	2411.836	12T16
6	1449.6	1283.2	166.4	2009.662	10T16

Table 72: Slab reinforcement at the midspans comparison due to lateral loads on the ACI slabs

Midspan	Laterl Load Envelope Maimum Moment from ETABS ACI Model (kN.m)	Maximim Moment Capacity from RAM (kN.m)	Moment Difference	Required Reinorcement if any (mm^2)	Approximate Equivalent Reinforcement
1	675.8	526.1	149.7	1807.97	9T16
2	361.6	392.5	-30.9	No Reinforcemnt Required	-
3	267.3	871.4	-604.1	No Reinforcemnt Required	-
4	504.6	576.8	-72.2	No Reinforcemnt Required	-
5	592.2	904.3	-312.1	No Reinforcemnt Required	-
6	640.7	839.2	-198.5	No Reinforcemnt Required	-

The following table shows the seismic and wind loads checking for Eurocode

Table 73: Slab reinforcement at the support's comparison due to lateral loads on the Eurocode slabs

Column / Shear Wall	Laterl Load Envelope Maimum Moment from ETABS Eurocode Model (kN.m)	Maximim Moment Capacity from RAM (kN.m)	Moment Difference	Required Reinorcement if any (mm^2)	Approximate Equivalent Reinforcement
1	1333.7	1104.72	228.98	2765.46	14T16
2	1332.7	1102.56	230.14	2779.49	14T16
3	847.2	617.6	229.6	2772.95	14T16
4	841.20	546.7	294.5	3556.76	18T16
5	1595	1395.66	199.34	2407.49	12T16
6	1829.6	1605.2	224.4	2710.14	14T16

Table 74: Slab reinforcement at the midspans comparison due to lateral loads on the Eurocode slabs

Midspan	Laterl Load Envelope Maimum Moment from ETABS Eurocode Model (kN.m)	Maximim Moment Capacity from RAM (kN.m)	Moment Difference	Required Reinorcement if any (mm^2)	Approximate Equivalent Reinforcement
1	367	270.8	96.2	1161.836	6T16
2	255.70	206.6	49.1	593	3T16
3	173.20	90.51	82.69	998.6715	5T16
4	253.1	1042	-788.9	No Reinforcemnt Required	-
5	248.10	508.8	-260.7	No Reinforcemnt Required	-
6	271.1	178	93.1	1124.397	6T16

Table 75: Reinforcement at the support's comparison between the ACI code slab & Eurocode slab

Column / Shear Wall	Reinforcement at the Support for the ACI 318-19 code	Reinforcement at the Support for the Eurocode	Bars Difference	Code with More Reinforcemnt
1	8T16	14T16	6T16	Eurocode
2	9T16	14T16	5T16	Eurocode
3	12T16	14T16	2T16	Eurocode
4	10T16	18T16	8T16	Eurocode
5	12T16	12T16	-	Both are same
6	10T16	14T16	4T16	Eurocode

Table 76: Reinforcement at the midspan comparison between the ACI code slab & Eurocode slab

Midspan	Reinforcement at the Midspan for the ACI 318-19 code	Reinforcement at the Midspan for the Eurocode	Bars Difference	Code with More Reinforcemnt
1	9T16	6T16	3T16	ACI Code
2	-	3T16	3T16	Eurocode
3	-	5T16	5T16	Eurocode
4	-	-	-	-
5	-	-	-	-
6	-	6T16	6T16	Eurocode

<u>Discussion of Slab Reinforcement Comparison:</u>

For the post tension rate comparison in the 40th Residential floor, the ACI code model have given a rate of 4.09 while for the Eurocode model the rate was 3.69. The results show the Eurocode require less post tension than the ACI code.

The 40th Residential floor slab has been taken into account for the reinforcement comparison. As shown from the previous results, the Eurocode require more reinforcement at the supports and at the midspans. The amount of reinforcement difference between the code can be different since it depends on how the designer is assigning the tendons in the slab, but the previous data gives insight that the Eurocode generally require more reinforcement. The results are logical since the post tension rate in the Eurocode is less which means it is most likely to have more reinforcement required and this is what was obtained in the results.

Table 77: Summary of post tension rate & reinforcement requirement comparison between the codes for all the floors

Floors	ACI Code PT Rate	Eurocode PT Rate	PT Rate Difference	Code with more PT Rate	Code with more Reinforcement Requirement
Podiums	3.82	3.54	0.28	ACI Code	Eurocode
1st Floor Office	3.86	3.64	0.22	ACI Code	Eurocode
2nd to 7th Office Floors	4.01	3.85	0.16	ACI Code	Eurocode
8th to 22nd Office Floors	4.09	3.73	0.37	ACI Code	Eurocode

CHAPTER 5: CONCLUSIONS, RECOMMENDATIONS & FUTURE RESEARCHES

5.1 Conclusion of Research

This research has investigated the differences between the ACI318-19 codes and the Eurocode analysis in seismic and wind provisions. Also, it has compared the final design of both codes based on the conservativity of cost/materials for columns shear walls and slabs.

5.1.1. Conclusion of Codes Provisions Comparison

A detailed analysis for the seismic and wind provisions of ACI318-19 code and Eurocode have been performed for a G+50 high-rise building, and based on the comparison made in chapter 4 and the findings in chapter 2, the following have been concluded for each provision:

5.1.1.1- Seismic Base Shear

- The base shear scaling requirement for response spectrum analysis is required in the ACI318-19 code to be at least 85%, while the Eurocode does not show a requirement to implement base shear scaling.
- The base shear calculation for both codes are different, since in the ACI318-19 code the base shear calculation depends on the seismic response coefficient and the building effective seismic weight (V = CsW), while for Eurocode, the base shear calculation depends on the the design spectrum at period T_1, total mass of the building and the correction factor ($F_b = S_d(T_1).m.\lambda$). Such difference in the equations and data considered will lead to different results between the codes.

5.1.1.2- Storey Shear

For storey shear determination, both codes follow very close equations in determining the storey shear. The ASCE7-16 depends on the seismic base shear, height from the building base to a certain level and portion of the total effective structure seismic weight at a certain level

($\boldsymbol{F_i = V.\frac{w_i .h_i^k}{\sum_{j=1}^{n} w_j .h_j^k}}$), while the Eurocode, it depends on seismic base shear, displacements of masses and the storey masses ($\boldsymbol{F_i = F_b .\frac{z_i .m_i}{\sum z_j .m_j}}$), the main difference is that the ASCE 7-16 uses an exponent (k) which is related to the structure period.

5.1.1.3- Storey Drift

- The storey drift limits for the ACI 318-19 and Eurocode are different, since for the ASEC 7-16/ACI 318-19 allows 2.5% drift for ordinary multi-storey reinforced concrete frame buildings but for Eurocode it allows drift up to 1% for multi-storey reinforced concrete framed buildings having brittle non-structural elements, 1.5% for buildings having ductile non-structural elements, and 2% for buildings having non-structural elements. Such difference shows that the Eurocode limitations for Drift is stricter than the ACI318-19.
- For the wind storey drift the ASCE 7-16 standard does not give a suggestion for an allowable drift limit for wind design as it does with a seismic design but it gives a common usage for building design on the order of 1/600 to 1/400 of the building or storey height, without more details regarding it, while the Eurocode does not specify any limitations for storey drift against wind. As it can be concluded that those codes don't take the storey drift against wind as a critical requirement since it's effect can be way less compared to the storey drift due to seismic load, so by fulfilling the requirement of the storey drifts against the seismic affect means that the building will be safe against the wind drift since its force is less than seismic in most cases.

5.1.1.4- Torsional Irregularity

- Both codes follow different criteria in determining the torsional irregularity. The ASCE 7-16 states that a torsional irregularity exists in a building when the maximum storey drift at one end of the structure exceeds more than 1.2 times the average of the storey drifts at the two ends of the structure under equivalent static analysis, while the Eurocode provides two requirements to be fulfilled, it states that the building eccentricity *eo* and the torsional radius *r* shall be in accordance with the following: $eox \leq 0.30.\ rx$ and $rx \geq ls$ for Y-Direction & $eoy \leq 0.30.\ ry$ and $ry \geq ls$ for X-Direction. Based on that it can be concluded that the Eurocode requirement for torsional irregularity determination is more complicated than the ASCE7-16 since many aspects are taken in to account.

5.1.1.5- P-Delta Effect

- Both codes require almost similar data to determine the P-delta effect in the building, the main difference is that the ASCE 7-16 code adds the importance factor as well as the deflection amplification factor in calculating P-delta which will make changes in the results produced compared to the Eurocode.

- Both codes recommend to neglect P-Delta when θ is below 0.10.

5.1.1.6- Response Spectrum

- For the response spectrum graph, the two code follow different requirements since the ASCE 7-16 depends mainly on the measured spectral acceleration parameters S_S and S_1, while in the Eurocode the main considerations are the ground acceleration magnitude and the Surface wave magnitude.

5.1.1.7- Mass Participation

- The most commonly used criterion which is adopted by Eurocode 8 that the total number of modes considered in the analysis have to provide together a total effective modal mass in any of the seismic action components x, y or even z, considered in the design to be at least equal to 90% of the total mass of the structure. While the ASCE 7-16 code requires a sufficient number of modes in order to get a combined modal mass participation of 100% of the structure's mass, but some exceptions in the analysis is permitted to include a minimum number of modes to obtain a combined modal mass participation of at least 90% of the actual mass in each orthogonal horizontal direction. Overall, it can be stated that both codes require at least 90% of the mass participation to be considered in the building design.

5.1.1.8- Structural Time Period

- Every seismic design code almost provides empirical equations to approximately estimate the building's fundamental period. The ASCE 7-16 provides the following equation $\mathbf{T} = \boldsymbol{C_t} \cdot \boldsymbol{h_n^x}$, while the Eurocode provides $\mathbf{T_1} = \boldsymbol{C_t} \cdot \boldsymbol{H^{0.75}}$ for structures up to 40 meters or to use the alternative equation $\mathbf{T_1} = \mathbf{2}\,\sqrt{\boldsymbol{d}}$. The values considered for Ct and x can be different for both codes which is the reason for having different results for time period.

5.1.1.9- Columns Reinforcement Limitations

- The Eurocode gives more modulus of elasticity of a concrete strength when it is compared to ACI 318-19 code, which will affect the results obtained as reinforcement and other seismic and wind provisions.
- In Eurocode 2 the maximum nominal reinforcement area for columns outside laps is 4% and it can be increased if the concrete can be placed and compacted sufficiently. While in ACI code 318-19, the maximum reinforcement ratio for columns is 8% the gross area of the column. Which means that ACI 318-19 generally allows more use of reinforcement.

5.1.1.10- Shear Walls Reinforcement Limitations

- The Eurocode gives more modulus of elasticity of a concrete strength when it is compared to ACI 318-19 code, which will affect the results obtained as reinforcement and other seismic and wind provisions.
- In the Eurocode 2 that the minimum reinforcement ratio on reinforced concrete shear walls can be 0.004 and the maximum longitudinal reinforcement ratio could be 0.04. While for the ACI 318-19 the ratio of horizontal and vertical reinforcement should be greater than 0.0025. The minimum reinforcement required by the Eurocode is more than ACI318-19 which means it would probably require more reinforcement for Eurocode compared to ACI code in some conditions like small seismicity areas where few reinforcements is required.

5.1.1.11- Cost Comparison for Slabs using RAM Concept

- The Eurocode gives more modulus of elasticity of a concrete strength when it is compared to ACI 318-19 code, which will affect the results obtained as reinforcement and other seismic and wind provisions.
- Both codes almost have similar post tension slab verifications, like checking the deflection of the slab, the precompression value, post tension rate, etc.

5.1.2 Conclusions Obtained from Data Analysis

After following each code requirements and designing the building on the ETABS model, the results for the mentioned seismic and wind provisions have been obtained and compared for the high-rise building and the following was obtained.

5.1.2.1- Seismic Base Shear

- In the statice base shear comparison, the Eurocode is showing higher base shear than the ACI 318-19 in the X-direction by 2.95% and 14.36% in the Y- direction.
- The scaling in both codes is more than 85%. They both have scaling of 93.60% in the X-direction but in the Y-direction the Eurocode is having a scaling of 100% which is 11.7% more compared to the ACI code scaling in that direction, which means that the dynamic base shear for the Eurocode model is more.

5.1.2.2- Storey Force

-The seismic storey force distribution in the Eurocode is more than the ACI318-19 code.

5.1.2.3- Storey drift (In X-Direction and Y Direction)

- In the X-direction, the maximum seismic storey drift obtained in the Eurocode model is 0.556%, while the maximum seismic storey drift obtained in the ACI model is 0.731%, having a difference of 0.175%. This shows that even though the storey force applied at the floors in

Eurocode model are more, they have less drift applied on them which can be related to the higher modulus of elasticity in the Eurocode, different load combinations between the codes and the distribution of vertical elements in the building.

- In the Y-direction, the maximum seismic storey drifts obtained in the Eurocode model are very close to the ACI model, having a maximum seismic drift of 0.704% in the Eurocode while its 0.692% in the ACI code, having a difference of 0.012%.

- In the X-direction, the difference of seismic drift is less compared to the difference in the seismic drift in the Y-direction, the reason can be related to the different load combinations assigned for each code or even the way of distribution of the vertical elements along the structure.

- In the wind drift comparison, it shows that the Eurocode model is giving more drift, which can be related to the load combinations differences between the codes and the different values of modifiers used.

5.1.2.4- Torsional Irregularity

- The number of floors having a torsional irregularity in the ACI model are more compared to the Eurocode for in both directions.

5.1.2.5- P-Delta (In X-Direction and Y Direction)

- According to the designed building, it shows that in both models there are no P-delta required, since both of them are giving $\boldsymbol{\theta}$ value in X-direction and Y-direction that is less than 0.10.

5.1.2.6- Response Spectrum

- The response spectral acceleration for the ACI model is more than the Eurocode model due to the different parameters of both codes in drawing the response spectrum.

5.1.2.7- Mass Participation (For Ux and Uy)

- The mass participation in both directions for both models are more than 90%.

- The mass participation reaches more than 90% for the Eurocode model at mode 17 while for the ACI model it is reached at mode 19, which indicates that Eurocode can face more force/displacement than the ACI in case of any reaction of the building.

5.1.2.8- Structural Time Period

- For almost all different modes, it shows that the ACI model is having more time period compared to the Eurocode.

5.1.2.9- Cost Comparison for Columns using ETABS

- Most of the columns in Eurocode model required less reinforcement when they are compared to the columns in the ACI code model, which means that the Eurocode is more conservative and can be designed with less reinforcement and/or less section compared to the ACI.

5.1.2.10- Cost Comparison for Walls using ETABS

- All the shear walls in Eurocode model required less reinforcement when they are compared to the shear walls in the ACI code model, which means that the Eurocode is more conservative and can be designed with less reinforcement and/or less section compared to the ACI.

5.1.2.11- Cost Comparison for Slabs using RAM Concept

- All the floors required less post tension rate in the Eurocode models compared to the floors designed using the ACI model. The reason can be related to the higher modulus of elasticities of the Eurocode and for other provisions in the Eurocode.

The amount of reinforcement in the ACI models were less once it is compared to the Eurocode models, this can be related to the less post tension rate in the Eurocode models which is covered then by additional reinforcement.

5.1.3 Conclusions Comparison with the findings

In this section the data obtained from this study will be compared with the previous researches studies that have been done. The comparison here will only be on the research studies that have been found.

5.1.3.1- Seismic Base Shear

According to a previous study done by Patil, Shiyekar & Ghugal, they have compared to seismic base shear in x-direction and found out that the base shear in the Eurocode is more than the ACI code by 10.05%. In this study it has also shown that the base shear in the x-direction is more in the Eurocode and the difference between them have been found to be around 2.95%. The percentage value will vary from a building to another since there are many factors that may contribute in it, one of the most significant factors is the weight of the building; In this study the building is G+50 floors while in the Patil, Shiyekar & Ghugal study the building was G+20. Moreover, the plans are different in both studies.

5.1.3.2- Storey Drift (In X-Direction and Y Direction)

Based on the study done by Karthik N. and Varuna Koti in comparing the storey drift in the x and y directions, they have found that the Eurocode is giving more seismic drift, while in this study it has found that the Eurocode is giving more drift in the y-direction, but in the x-direction the ACI 318-19 model is higher.

5.1.3.3- Storey Force

Based on the study done by Patil, Shiyekar & Ghugal in comparing the storey forces of Eurocode 8 and ACI 318-08, they have found that the Eurocode is giving more storey force. In this study the comparison was between Eurocode 8 and ACI 318-19 and the results have shown that the Eurocode is giving more storey force, which is similar to the previous study.

5.1.3.4- Structural Time Period

Lang & Khose have studied the time period for different buildings, one of them had 12 floors which can be considered as a high-rise building, and based on it has shown that the ASCE 7 is giving slightly more time period compared to the Eurocode, which is similar to the conclusion obtained in this study. It is important to note that the difference in time period will differ based on the number of stories in the building.

5.1.3.5- Cost Comparison for Columns

Too many studies have been done for columns reinforcement comparison many had different results but most of them have shown that the Eurocode require less reinforcement. Taking in to account the study done by Hassan, Anwar, Norachan & Najam, was found that the Eurocode gives slightly less reinforcement than the ACI code, which is similar to this study. In this study the ratio difference varies from columns in a floor to another but the most important conclusion is that the columns in the Eurocode require less amount of reinforcement which will save materials and cost in construction.

5.1.3.6- Cost Comparison for Shear Walls

According to the study done by Hassan, Anwar, Norachan & Najam for the shear walls reinforcement comparison they have found that the Eurocode is giving less shear walls reinforcement compared to the ACI code, which similar to the outputs obtained in this study.

5.2 Research Questions Answers and Recommendations

In this section the researches questions will be answered based on the data obtained from the design analysis, and some recommendations are given.

What are the differences in the selected seismic assessment parameters between the two codes? and is that difference ratio high?

- After following the provisions for each code and performing the design analysis, it has been found that there are differences in the seismic assessment parameters between the ACI 318-19 code and the Eurocode. The Eurocode model have shown higher values for base shear in both directions, storey shear, seismic storey drift in y-direction, mass participation and wind drift, while the ACI318-19 model have given more seismic storey drift in x-direction, torsional irregularity, Response spectrum and time period. While both of them gave a same conclusion for P-delta effect.
- The main differences between the codes are noticed in the seismic drifts in x-direction, storey forces and in P-delta effects since they both gave results which aren't close relatively, while the rest comparisons are close to each other relatively.

What are the differences of the selected wind assessment parameters between the two codes? and is that difference ratio high?

- The comparison of the codes has been performed on the wind storey drift, and it have found that the drift in both directions is different than each other, where ACI 318-19 gives less drift compared to Eurocode.

Which code could possibly give more economical design for the superstructure design?

- According the performed study, most of the columns showed that the Eurocode require less reinforcement compared with the ACI 318-19.
- In the concrete shear walls design almost all the shear walls have shown that the Eurocode require less amount of reinforcement, and the amount of reinforcement is very less compared to the ACI code.
- In the post tension slabs, the rate of post tension in the Eurocode models are less compared to the ACI model, while it requires a bit more reinforcement.

Based on the previous study it is recommended to go for Eurocode model design if the target was to have an economical design with less materials and costs of the building.

What could be the elements sizing's differences between them?

- Based on the data obtained, it has shown that the amount of reinforcement ratio required in the Eurocode is less for most of the vertical elements, which means that to get a similar reinforcement ratio for both codes, the vertical elements sizing for the Eurocode model can be reduced. So, over all the Eurocode is more likely to have less sizing compared to the ACI code.

To know which code is the best one to be used in locations such as Dubai, UAE for high-rise buildings?

- Dubai is one of the cities in the world that has huge construction of high-rise buildings, it is location is located in a low seismicity area, which means that by following the Eurocode design there will be much saving in terms of the cost of the whole building, and the building itself will be able to withstand the applied lateral forces on it. It is recommended to use the Eurocode instead of the ACI318-19/ ASCE 7-16 for such wise.

5.3 Scope for Future Research

This research work has focused mainly on comparing two famous codes, the ACI code and the Eurocode in differences of seismic and wind assessment provisions and their effects on a high-rise building as well as the cost difference for columns, shear walls and slabs based on these forces. Future work could include researches for some wind assessment provisions since this study focused only on the inter storey wind drift, and to compare other seismic assessment provisions that were not mentioned in this research. Another study can be done which is to compare these codes in terms of code design provisions in ultimate limit state like flexural and shear, as well as for the service limit state like deflection. A further research that can be done is to make a seismic and wind assessments comparison for these codes in case if there are any updates for any of them; The ACI code is being updated every 5 years which make it a good research topic to see also the difference occurred between the two mentioned codes in this research as well as any different versions for these codes. This research deals with concrete code design and it can be done on other materials like steel or composite materials in future scope. Hence, this research is only a drop of the vast ocean of researches possibilities.

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Appendix A: Structural sections & layouts

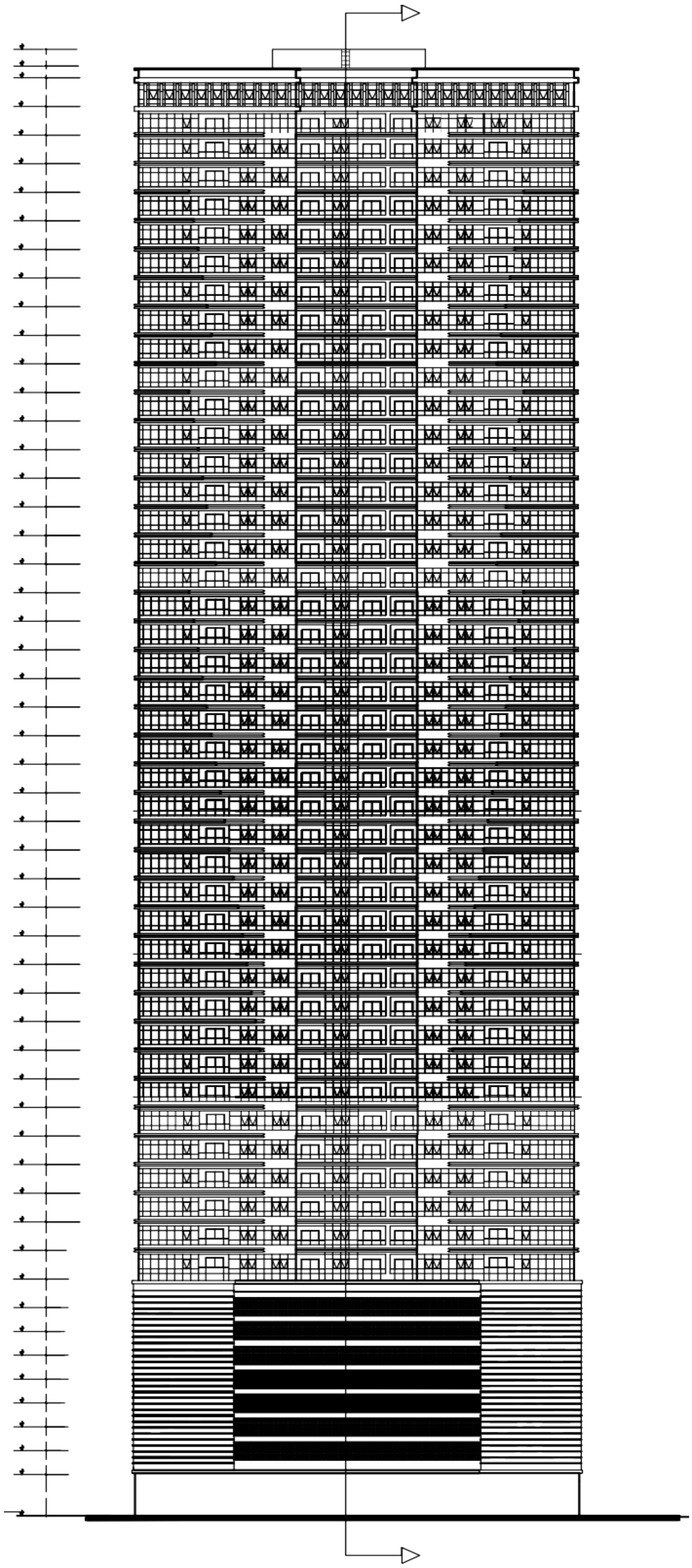

Figure 87: Front view of the building

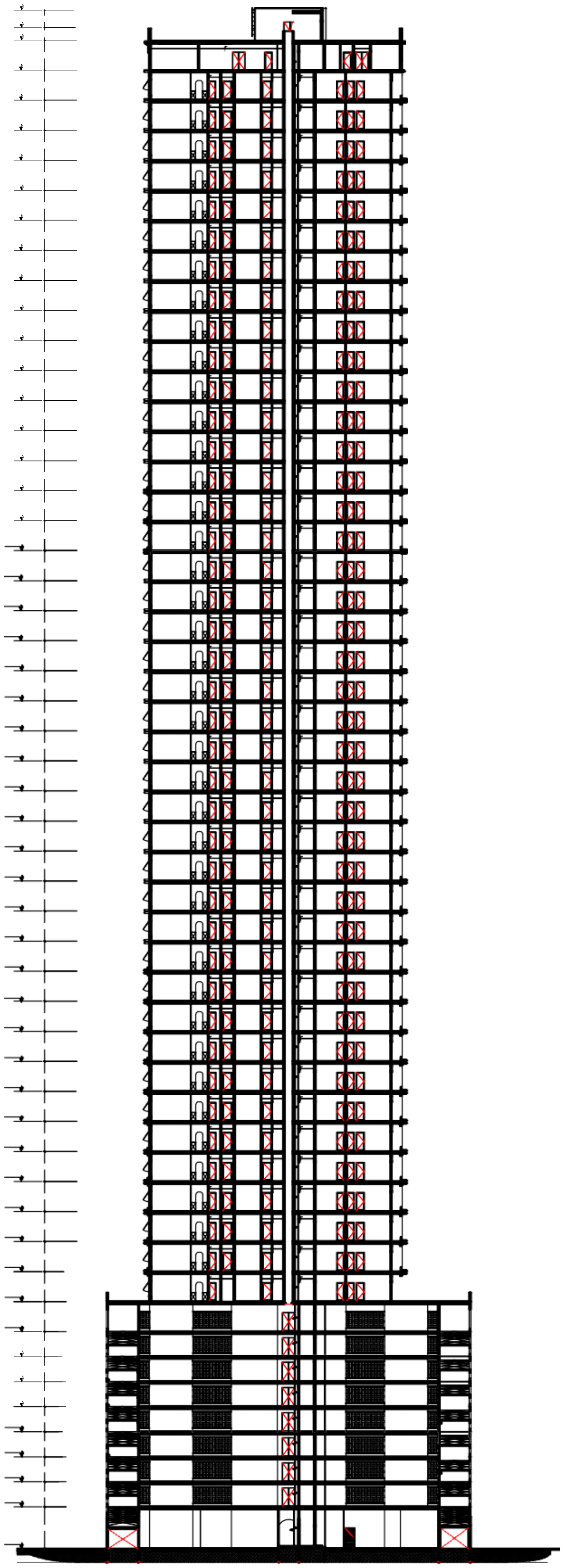

Figure 88: Cross section of the building

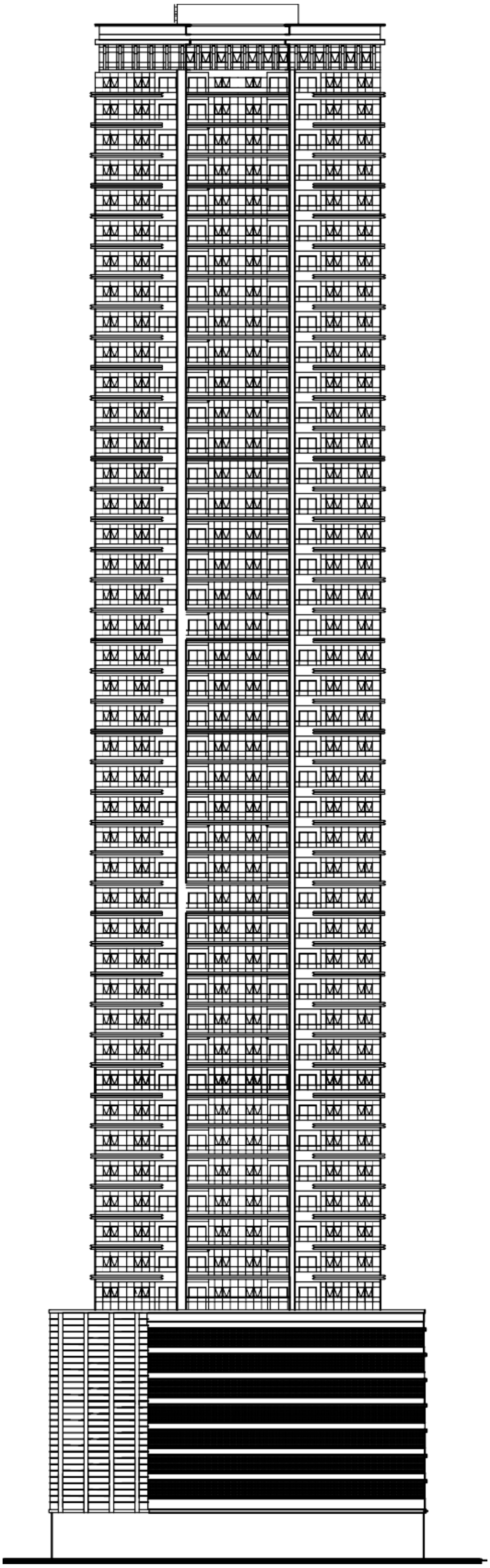

Figure 89: Side view of the building

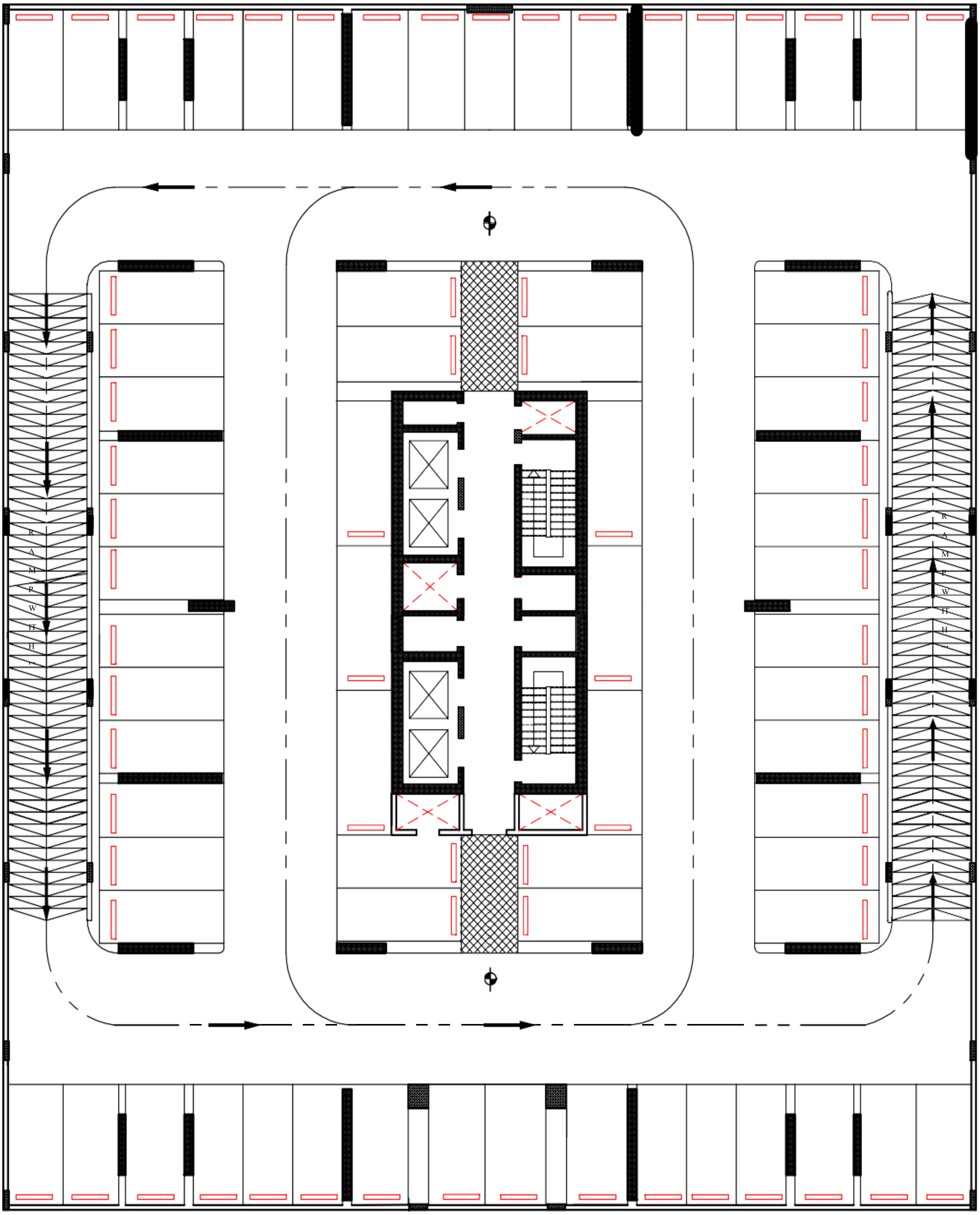

Figure 90: Initial sizing and geometry for podium floors plan

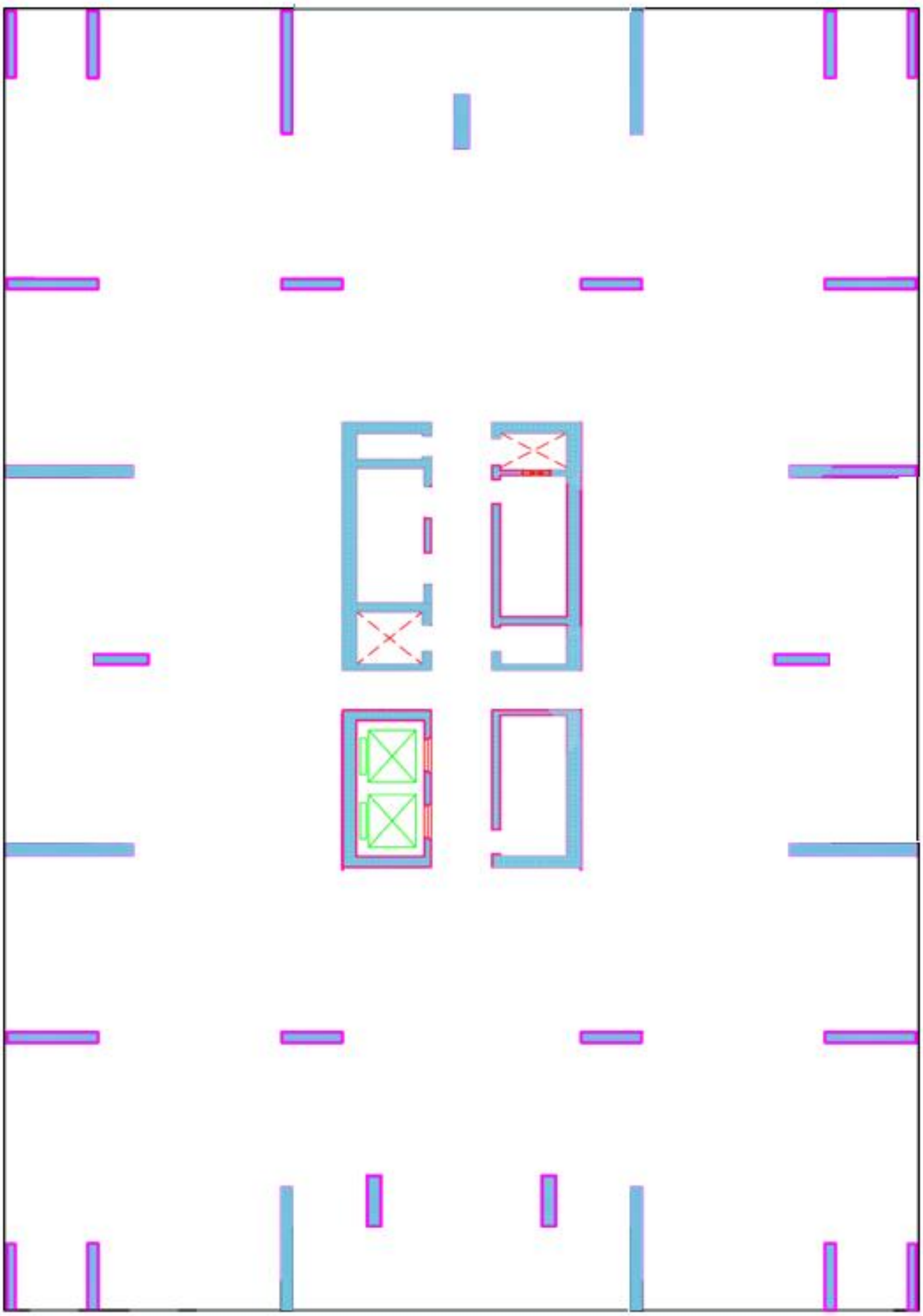

Figure 91: Initial sizing and geometry for typical floors plan

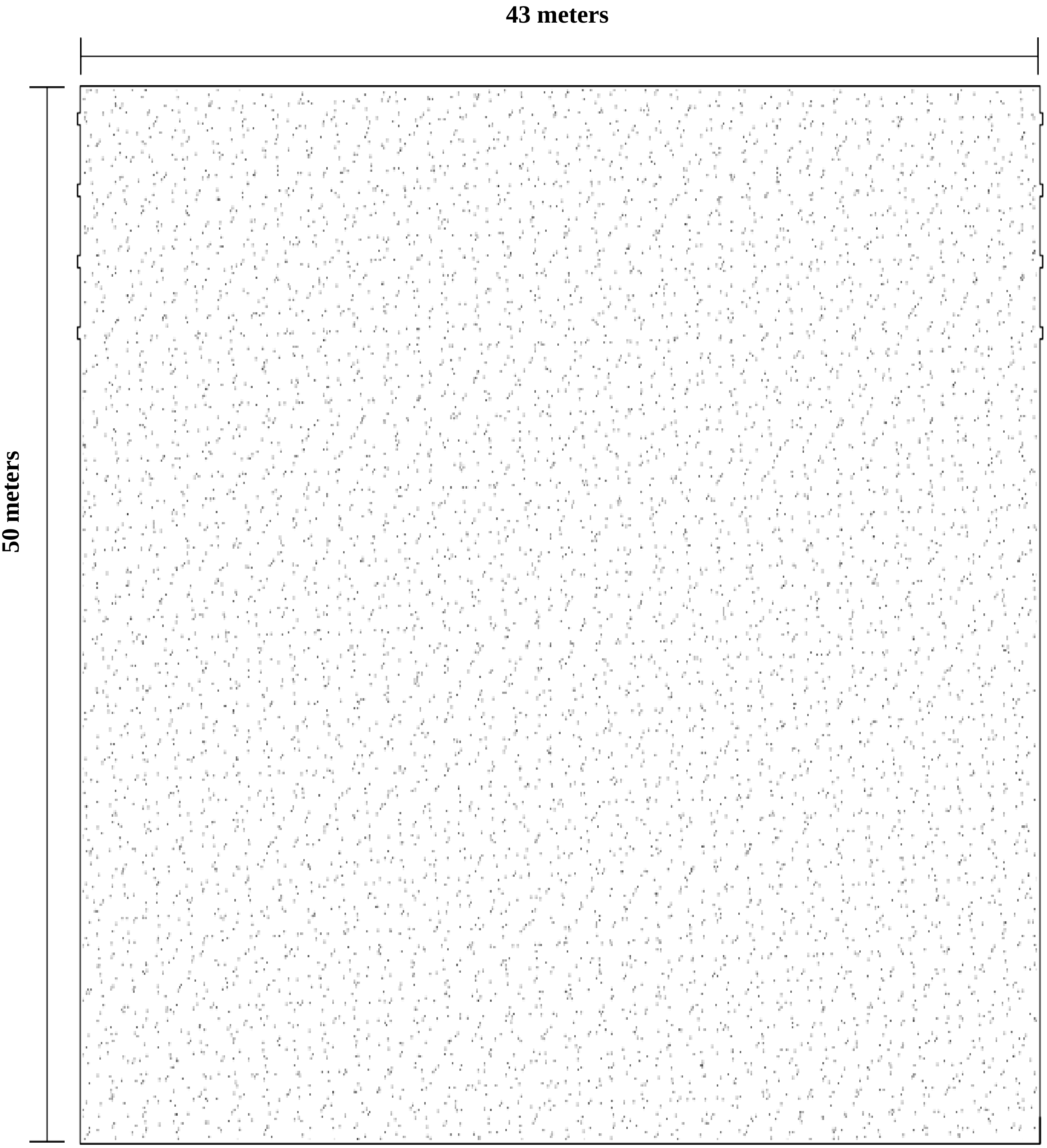

Figure 92: Figure: Ground floor, podium floors and 1st office floor plans dimension

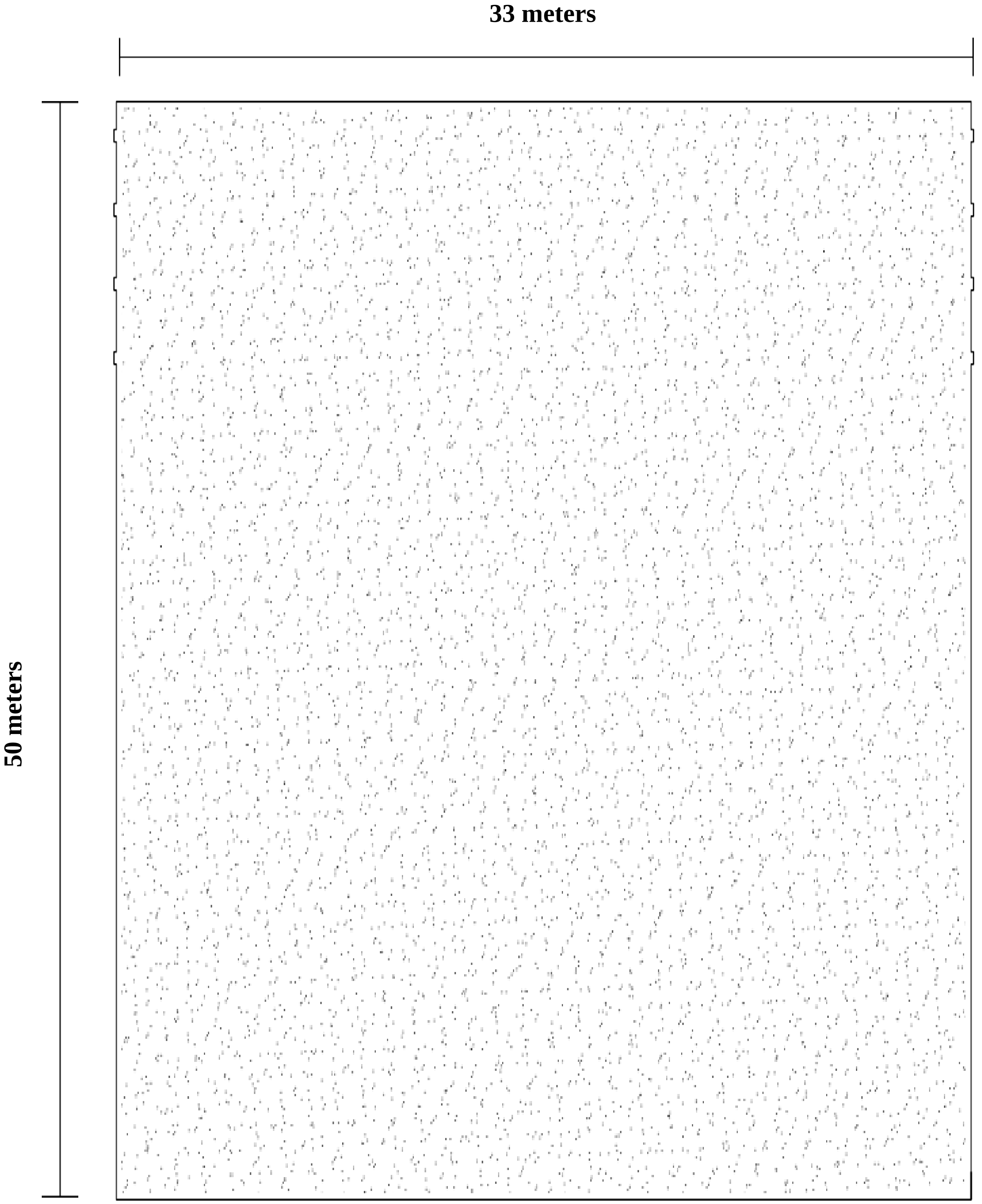

Figure 93: Typical floors plan dimension

Appendix B: Codes referencing

λ	is the correction factor, the value of which is equal to: $\lambda = 0{,}85$ if $T_1 \leq 2\ T_C$ and the building has more than two storeys, or $\lambda = 1{,}0$ otherwise.

Figure 94: Correction factor magnitude

$$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2{,}5}{q} - \frac{2}{3}\right)\right] \quad (3.13)$$

$$T_B \leq T \leq T_C : \ S_d(T) = a_g \cdot S \cdot \frac{2{,}5}{q} \quad (3.14)$$

$$T_C \leq T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2{,}5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \geq \beta \cdot a_g \end{cases} \quad (3.15)$$

$$T_D \leq T : \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2{,}5}{q} \cdot \left[\frac{T_C T_D}{T^2}\right] \\ \geq \beta \cdot a_g \end{cases} \quad (3.16)$$

Figure 95: Horizontal components of the seismic action the design spectrum, Sd(T)

β	is the lower bound factor for the horizontal design spectrum. NOTE The value to be ascribed to β for use in a country can be found in its National Annex. The recommended value for β is 0,2.

Figure 96: β Value consideration in Eurocode

5.3.3 Behaviour factor

(1) A behaviour factor q of up to 1,5 may be used in deriving the seismic actions, regardless of the structural system and the regularity in elevation.

Figure 97: Behavior Factor Value from Eurocode

R4.3—Design loads

R4.3.1 The provisions in Chapter 5 are based on ASCE/SEI 7. The design loads include, but are not limited to, dead loads, live loads, snow loads, wind loads, earthquake effects, prestressing effects, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and predicted unequal settlement of supports. Other project-specific loads may be specified by the licensed design professional.

R5.2—General

R5.2.1 Provisions in the Code are associated with dead, live, wind, and earthquake loads such as those recommended in ASCE/SEI 7. The commentary to Appendix C of ASCE/SEI 7 provides service-level wind loads W_a for serviceability checks; however, these loads are not appropriate for strength design.

Figure 98: ACI 318-19 code referring to the ASCE 7-16 code for wind analysis

26.7.2 Surface Roughness Categories. A ground surface roughness within each 45° sector shall be determined for a distance upwind of the site, as defined in Section 26.7.3, from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Section 26.7.3.

Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous, closely spaced obstructions that have the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions that have heights generally less than 30 ft (9.1 m). This category includes flat, open country and grasslands.

Surface Roughness D: Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.

Figure 99: Surface roughness category based on ASCE 7-16 code

26.7.3 Exposure Categories.

Exposure B: For buildings or other structures with a mean roof height less than or equal to 30 ft (9.1 m), Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance greater than 1,500 ft (457 m). For buildings or other structures with a mean roof height greater than 30 ft (9.1 m), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance greater than 2,600 ft (792 m) or 20 times the height of the building or structure, whichever is greater.

Exposure C: Exposure C shall apply for all cases where Exposure B or D does not apply.

Exposure D: Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance greater than 5,000 ft (1,524 m) or 20 times the building or structure height, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 ft (183 m) or 20 times the building or structure height, whichever is greater, from an Exposure D condition as defined in the previous sentence.

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

Figure 100: Exposure category based on ASCE 7-16 code

26.8.2 Topographic Factor. The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1 K_2 K_3)^2 \qquad (26.8\text{-}1)$$

where K_1, K_2, and K_3 are given in Fig. 26.8-1.

If site conditions and locations of buildings and other structures do not meet all the conditions specified in Section 26.8.1, then $K_{zt} = 1.0$.

Figure 101: Choosing the topographical factor

26.11.1 Gust-Effect Factor. The gust-effect factor for a rigid building or other structure is permitted to be taken as 0.85.

Figure 102: Guest effect factor from ASCE 7-16 code

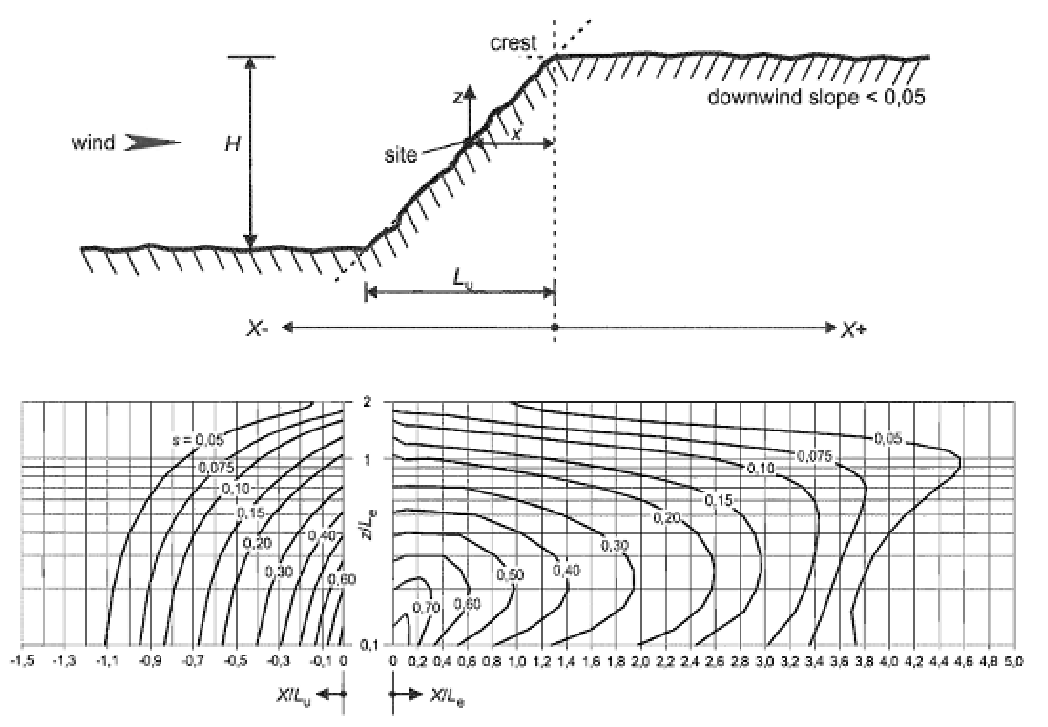

Figure A.2 — Factor *s* for cliffs and escarpments

Figure 103: Factors s for cliffs and escarpments (A.2)

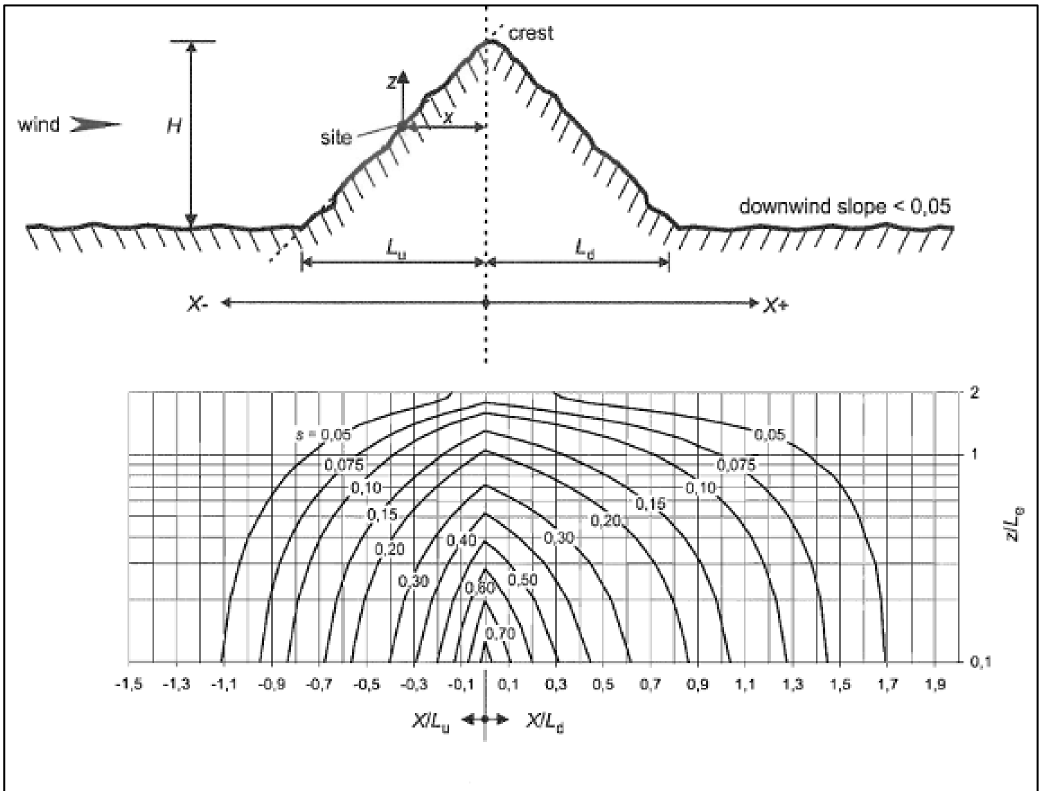

Figure A.3 — Factor s for hills and ridges

Figure 104: Factors s for cliffs and escarpments (A.3)

It is defined by:

$c_o = 1$	for	$\Phi < 0{,}05$	(A.1)
$c_o = 1 + 2 \cdot s \cdot \Phi$	for	$0{,}05 < \Phi < 0{,}3$	(A.2)
$c_o = 1 + 0{,}6 \cdot s$	for	$\Phi > 0{,}3$	(A.3)

Figure 105: Orography factor determination

k_I is the turbulence factor. The value of k_I may be given in the National Annex. The recommended value for k_I is 1,0.

Figure 106: Turbulence factor value recommendation

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 \qquad (4.10)$$

NOTE 2 The values for ρ may be given in the National Annex. The recommended value is 1,25 kg/m^3.

Figure 107: Code recommendation for air density

From the ACI 318-19 code, chapter 18: Earthquake Resistant Structures, page 285, code number 18.1.

> The provisions of Chapter 18 relate detailing requirements to type of structural framing and SDC. Seismic design categories are adopted directly from ASCE/SEI 7, and relate to considerations of seismic hazard level, soil type, occupancy, and use. Before the 2008 Code, low, intermediate, and high seismic risk designations were used to delineate detailing requirements. For a qualitative comparison of seismic design categories and seismic risk designations, refer to Table R5.2.2. The assignment of a structure to a SDC is regulated by the general building code (refer to 4.4.6.1).

Figure 108: ACI 318-19 adopting the seismic design categories from ASCE 7-16

From the ACI 318-19 code, chapter 5: Loads, page 61, number 5.2.2.

> **R5.2.2** Seismic Design Categories (SDCs) in this Code are adopted directly from ASCE/SEI 7. Similar designations are used by the International Building Code (2018 IBC) and the National Fire Protection Association (NFPA 5000 2012).

Figure 109: ACI 318-19 adopting the seismic design categories from ASCE 7-16 (Another referencing)

From ASCE 7-16 code, page 101, section 12.8 Equivalent lateral force (ELF) procedure:

12.8 EQUIVALENT LATERAL FORCE (ELF) PROCEDURE

12.8.1 Seismic Base Shear. The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \quad (12.8\text{-}1)$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1, and
W = the effective seismic weight per Section 12.7.2.

Figure 110: Seismic base shear equation from ASCE 7-16 code

From ASCE 7-16 code, page 204, Chapter 20, section 20.3.

Table 78: Site classes from ASCE 7-16 code

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50 blows/ft	>2,000 lb/ft^2
D. Stiff soil	600 to 1,200 ft/s	15 to 50 blows/ft	1,000 to 2,000 lb/ft^2
E. Soft clay soil	<600 ft/s	<15 blows/ft	<1,000 lb/ft^2
	Any profile with more than 10 ft of soil that has the following characteristics: — Plasticity index $PI > 20$, — Moisture content $w \geq 40\%$, — Undrained shear strength $\bar{s}_u < 500$ lb/ft^2		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

Note: For SI: 1 ft = 0.3048 m; 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.

From ASCE 7-16 code, page 84, chapter 11, section 11.4-1

Table 79: Short period site coefficient from ASCE 7-16 code

Table 11.4-1 Short-Period Site Coefficient, F_a

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period					
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

From ASCE 7-16 code, page 84, chapter 11, section 11.4-2.

Table 80: Long period site coefficient from ASCE 7-16 code

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.
[a]Also, see requirements for site-specific ground motions in Section 11.4.8.

From ASCE 7-16 code, page 84, chapter 11, section 11.4.5.

11.4.5 Design Spectral Acceleration Parameters. Design earthquake spectral response acceleration parameters at short periods, S_{DS}, and at 1-s periods, S_{D1}, shall be determined from Eqs. (11.4-3) and (11.4-4), respectively. Where the alternate simplified design procedure of Section 12.14 is used, the value of S_{DS} shall be determined in accordance with Section 12.14.8.1, and the value for S_{D1} need not be determined.

$$S_{DS} = \frac{2}{3} S_{MS} \qquad (11.4\text{-}3)$$

$$S_{D1} = \frac{2}{3} S_{M1} \qquad (11.4\text{-}4)$$

Figure 111: Design spectral acceleration parameters from ASCE 7-16 code

From ASCE 7-16 code, page 4, chapter 1, Table 1.5-1

Table 81: Risk category of buildings and other structures for flood, wind, snow, earthquake, and ice loads from ASCE 7-16 code

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released[a]	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

From ASCE 7-16 code, page 5, chapter 1, Table 1.5-2.

Table 82: Importance factors by risk category of buildings and other structures for snow, ice, and earthquake loads from ASCE 7-16 code

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

From ASCE 7-16 code, page 85, chapter 1, Table 11.6-1.

Table 83: Seismic design category based on short-period response acceleration parameter from ASCE 7-16 code

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS}$	D	D

From ASCE 7-16 code, page 85, chapter 1, Table 11.6-2.

Table 84: Seismic design category based on 1-s period response acceleration parameter from ASCE 7-16 code

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	D

From ASCE 7-16 code, page 90, chapter 12, Table 12.2-1.

Table 85: Design coefficients and factors for seismic force-resisting systems value from ASCE 7-16 code

90

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category B	C	D^e	E^e	F^f
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
6. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[g]	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
7. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
8. Intermediate precast shear walls[g]	14.2	5	2½	4½	NL	NL	40[i]	40[i]	40[i]
9. Ordinary precast shear walls[g]	14.2	4	2½	4	NL	NP	NP	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	160	160	100
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
13. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
14. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
16. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP

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From ASCE 7-16 code, page 102, chapter 12, Table 12.8-2.

Table 86: Values of approximate period parameters Ct and x parameter from ASCE 7-16 code

Table 12.8-2 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	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

[a]Metric equivalents are shown in parentheses.

From ASCE 7-16 code, page 102, chapter 12, Table 12.8-1.

Table 87: Coefficient for upper limit on calculated period from ASCE 7-16 code

Table 12.8-1 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤0.1	1.7

From ASCE 7-16 code, page 101, chapter 12, section 12.8.

12.8 EQUIVALENT LATERAL FORCE (ELF) PROCEDURE

12.8.1 Seismic Base Shear. The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad (12.8\text{-}1)$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1, and
W = the effective seismic weight per Section 12.7.2.

12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, C_s, shall be determined in accordance with Eq. (12.8-2).

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad (12.8\text{-}2)$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range as determined from Section 11.4.5 or 11.4.8;
R = the response modification factor in Table 12.2-1; and
I_e = the Importance Factor determined in accordance with Section 11.5.1.

The value of C_s computed in accordance with Eq. (12.8-2) need not exceed the following:

for $T \le T_L$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \qquad (12.8\text{-}3)$$

for $T > T_L$

$$C_s = \frac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)} \qquad (12.8\text{-}4)$$

C_s shall not be less than

$$C_s = 0.044 S_{DS} I_e \ge 0.01 \qquad (12.8\text{-}5)$$

Figure 112: Calculation of seismic response coefficient from ASCE 7-16 code

From ASCE 7-16 code, page 102, chapter 12, section 12.8.3.

12.8.3 Vertical Distribution of Seismic Forces. The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \qquad (12.8\text{-}11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8\text{-}12)$$

where

C_{vx} = vertical distribution factor;
V = total design lateral force or shear at the base of the structure [kip (kN)];
w_i and w_x = portion of the total effective seismic weight of the structure (W) located or assigned to level i or x;
h_i and h_x = height [ft (m)] from the base to level i or x; and
k = an exponent related to the structure period as follows:

- for structures that have a period of 0.5 s or less, $k = 1$;
- for structures that have a period of 2.5 s or more, $k = 2$; and
- for structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

Figure 113: Vertical distribution of seismic forces from ASCE 7-16 code

From the ACI318-19 code, section 4.3 at page 51, and in section 5.2 at page 61:

R4.3—Design loads

R4.3.1 The provisions in Chapter 5 are based on ASCE/SEI 7. The design loads include, but are not limited to, dead loads, live loads, snow loads, wind loads, earthquake effects, prestressing effects, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and predicted unequal settlement of supports. Other project-specific loads may be specified by the licensed design professional.

R5.2—General

R5.2.1 Provisions in the Code are associated with dead, live, wind, and earthquake loads such as those recommended in ASCE/SEI 7. The commentary to Appendix C of ASCE/SEI 7 provides service-level wind loads W_a for serviceability checks; however, these loads are not appropriate for strength design.

Figure 114: ACI 318-19 code referring to the ASCE 7-16 for wind loads calculations

Table 88: Live loads from the ACI 318-19 / ASCE 7-16

Occupancy or Use	Live Load	
	Uniform psf (kN/m^2)	Concentrated lb (kN)
Residential dwellings, apartments, hotels		
Private rooms and corridors serving them	40 (1.92)	
Public rooms and corridors serving them	100 (4.79)	
Hospitals		
Patient rooms	40 (1.92)	1,000 (4.45)
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Office buildings		
Lobbies and first floor corridors	100 (4.79)	2,000 (8.90)
Offices	50 (2.40)	2,000 (8.90)
Corridors above first floor	80 (3.83)	2,000 (8.90)
Recreational uses		
Bowling alleys, poolrooms, and similar	75 (3.59)	
uses	100 (4.79)	
Dance halls and ballrooms, gymnasiums	60 (2.87)	
Stadiums and arenas with fixed seats		
Stores		
Retail		
First floor	100 (4.79)	1,000 (4.45)
Upper floors	75 (3.59)	1,000 (4.45)
Wholesale, all floors	125 (6.00)	1,000 (4.45)
Storage warehouses		
Light	125 (6.00)	
Heavy	250 (11.97)	
Manufacturing		
Light	125 (6.00)	2,000 (8.90)
Heavy	250 (11.97)	3,000 (13.40)
Schools		
Classrooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
First floor corridors	100 (4.79)	1,000 (4.45)

4.3—Design loads

4.3.1 Loads and load combinations considered in design shall be in accordance with Chapter 5.

R4.3—Design loads

R4.3.1 The provisions in Chapter 5 are based on ASCE/SEI 7. The design loads include, but are not limited to, dead loads, live loads, snow loads, wind loads, earthquake effects, prestressing effects, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and predicted unequal settlement of supports. Other project-specific loads may be specified by the licensed design professional.

Figure 115: Design loads consideration from ASCE code

Table 89: Area loads & line loads from the Eurocode

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to 2,0	2,0 to 3,0
- Stairs	2,0 to 4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

From EN 1991-1-1:2002 (E)- page 22

Table 90: Category of use, from the Eurocode

Table 6.1 - Categories of use

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3**: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores
[1] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3		
NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex. NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2 NOTE 3 See 6.3.2 for storage or industrial activity		

From the Eurocode: (1991-1-1:2002)- Page 27

6.3.3 Garages and vehicle traffic areas (excluding bridges)

6.3.3.1 Categories

(1)P Traffic and parking areas in buildings shall be divided into two categories according to their accessibility for vehicles as shown in Table 6.7.

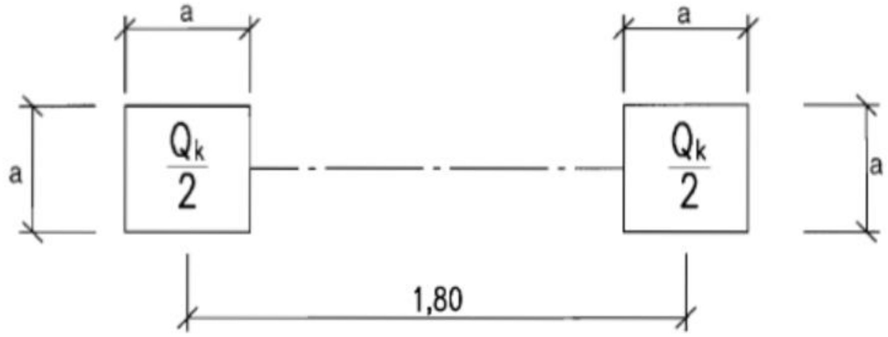
Table 6.7 - Traffic and parking areas in buildings

Categories of traffic areas	Specific Use	Examples
F	Traffic and parking areas for light vehicles (≤ 30 kN gross vehicle weight and ≤ 8 seats not including driver)	garages; parking areas, parking halls
G	Traffic and parking areas for medium vehicles (>30 kN, ≤ 160 kN gross vehicle weight, on 2 axles)	access routes; delivery zones; zones accessible to fire engines (≤ 160 kN gross vehicle weight)
NOTE 1 Access to areas designed to category F should be limited by physical means built into the structure.		
NOTE 2 Areas designed to categories F and G should be posted with the appropriate warning signs.		

6.3.3.2 Values of actions

(1) The load model which should be used is a single axle with a load Q_k with dimensions according to Figure 6.2 and a uniformly distributed load q_k. The characteristic values for q_k and Q_k are given in Table 6.8.

NOTE q_k is intended for determination of general effects and Q_k for local effects. The National annex may define different conditions of use of this Table.



NOTE For category F (see Table 6.8) the width of the square surface is 100 mm and for category G (see Table 6.8) the width of a square surface is 200 mm.

Figure 116: Garages & vehicles traffic area (excluding bridges), from Eurocode

Table 91: Imposed loads on garages & vehicle traffic areas from the Eurocode

Table 6.8 - Imposed loads on garages and vehicle traffic areas

Categories of traffic areas	q_k [kN/m^2]	Q_k [kN]
Category F Gross vehicle weight: $\leq$ 30 kN	q_k	Q_k
Category G 30 kN < gross vehicle weight $\leq$ 160 kN	5,0	Q_k
NOTE 1 For category F. q_k may be selected within the range 1,5 to 2,5 kN/m^2 and Q_k may be selected within the range 10 to 20 kN. NOTE 2 For category G, Q_k may be selected within the range 40 to 90 kN. NOTE 3 Where a range of values are given in Notes 1 & 2, the value may be set by the National annex. The recommended values are underlined.		

c) Storey drifts shall be limited, to limit P-Δ effects in the columns (see **4.4.2.2(2)-(4)**).

d) A substantial percentage of the top reinforcement of beams at their end cross-sections shall continue along the entire length of the beam (see **5.4.3.1.2(5)P**, **5.5.3.1.3(5)P**) to account for the uncertainty in the location of the inflection point.

(2) Second-order effects (P-Δ effects) need not be taken into account if the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0{,}10 \qquad (4.28)$$

where

θ is the interstorey drift sensitivity coefficient;

P_{tot} is the total gravity load at and above the storey considered in the seismic design situation;

Figure 117: P-Δ effects based on Eurocode

(6) At each level and for each direction of analysis x and y, the structural eccentricity e_o and the torsional radius r shall be in accordance with the two conditions below, which are expressed for the direction of analysis y:

$$e_{ox} \le 0{,}30 \cdot r_x \tag{4.1a}$$

$$r_x \ge l_s \tag{4.1b}$$

where

e_{ox} is the distance between the centre of stiffness and the centre of mass, measured along the x direction, which is normal to the direction of analysis considered;

r_x is the square root of the ratio of the torsional stiffness to the lateral stiffness in the y direction ("torsional radius"); and

l_s is the radius of gyration of the floor mass in plan (square root of the ratio of (a) the polar moment of inertia of the floor mass in plan with respect to the centre of mass of the floor to (b) the floor mass).

Figure 118: Torsional irregularity requirement from the Eurocode

From Eurocode 8, section 4.3.3.3.1 (5).

(3) The requirements specified in paragraph **(2)**P may be deemed to be satisfied if either of the following can be demonstrated:

– the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;

– all modes with effective modal masses greater than 5% of the total mass are taken into account.

&

(5) If the requirements specified in **(3)** cannot be satisfied (e.g. in buildings with a significant contribution from torsional modes), the minimum number k of modes to be taken into account in a spatial analysis should satisfy both the two following conditions:

$$k \ge 3 \cdot \sqrt{n} \tag{4.13}$$

and

$$T_k \le 0{,}20\ \mathrm{s} \tag{4.14}$$

Figure 119: Minimum number of modes based on Eurocode 8

From the Eurocode 2, Table3.1, page 29.

Table 92: Strength classes for concrete form the Eurocode 2

Table 3.1 Strength and deformation characteristics for concrete

Strength classes for concrete															Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	2.8
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0,30\times f_{ck}^{(2/3)} \leq$C50/60 $f_{ctm}=2,12\cdot\ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk;0,05} = 0,7\times f_{ctm}$ 5% fractile
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk;0,95} = 1,3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 ε_{c1} (‰) $= 0,7\, f_{cm}^{0,31} \leq 2,8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1}(‰)$=2,8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2}(‰)$=2,0+0,085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2}(‰)$=2,6+35[(90-f_{ck})/100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n=1,4+23,4[(90- f_{ck})/100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{c3}(‰)$=1,75+0,55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3}(‰)$=2,6+35[(90-f_{ck})/100]^4$

From Eurocode 8, section 4.2.4, Table 4.2.4, page 52.

Table 93: Mass source from Eurocode 8

4.2.4 Combination coefficients for variable actions

(1)P The combination coefficients ψ_{2i} (for the quasi-permanent value of variable action q_i) for the design of buildings (see **3.2.4**) shall be those given in EN 1990:2002, Annex A1.

(2)P The combination coefficients ψ_{Ei} introduced in **3.2.4(2)**P for the calculation of the effects of the seismic actions shall be computed from the following expression:

$$\psi_{Ei} = \varphi \cdot \psi_{2i} \tag{4.2}$$

NOTE The values to be ascribed to φ for use in a country may be found in its National Annex. The recommended values for φ are listed in Table 4.2.

Table 4.2: Values of φ for calculating ψ_{Ei}

Type of variable action	Storey	φ
Categories A-C*	Roof	1,0
	Storeys with correlated occupancies	0,8
	Independently occupied storeys	0,5
Categories D-F* and Archives		1,0

* Categories as defined in EN 1991-1-1:2002.

From Eurocode 0, Appendix 1, Table A1.1, page 52.

Table 94: ψ Factor recommended values for buildings

Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0

NOTE The ψ values may be set by the National annex.
* For countries not mentioned below, see relevant local conditions.

From Eurocode 8, Section 4.3.1, note No.7, page 54.

(7) Unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete and masonry elements may be taken to be equal to one-half of the corresponding stiffness of the uncracked elements.

Figure 120: Cracked section modifiers based on Eurocode 8

From Eurocode 8, section 4.3.3.3.1 (5).

(3) The requirements specified in paragraph **(2)**P may be deemed to be satisfied if either of the following can be demonstrated:

– the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;

– all modes with effective modal masses greater than 5% of the total mass are taken into account.

&

(5) If the requirements specified in **(3)** cannot be satisfied (e.g. in buildings with a significant contribution from torsional modes), the minimum number k of modes to be taken into account in a spatial analysis should satisfy both the two following conditions:

$$k \geq 3 \cdot \sqrt{n} \tag{4.13}$$

and

$$T_{\mathrm{k}} \leq 0{,}20\ \mathrm{s} \tag{4.14}$$

Figure 121: Minimum number of modes based on Eurocode 8

Appendix C: Previous studies referencing

Table 95: Torsional irregularity definition for different codes (İlerisoy 2019)

Table 1 Definitions of torsional irregularity in different earthquake codes.

	Definitions
TURKEY	The case where Torsional Irregularity Factor η_{bi}, which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum storey drift at any storey to the average storey drift at the same storey in the same direction, is greater than 1.2. Storey drifts shall be calculated by considering the effects of ± 5 % additional eccentricities.
CHINA	Under the action of specified horizontal force, the maximum elastic horizontal displacement or (storey drift) of a storey is larger than 1.2 times the elastic horizontal displacement (or storey drift) at both ends of this storey.
IRAN	In each story, the maximum drift, including accidental torsion, at one end of the structure shall not exceed 20 % of the average of the story drifts of the two ends of the structure. In each story, the distance between the centres of mass and stiffness in each orthogonal direction shall not exceed 20 % of the building dimension in that direction.
NEW ZEALAND	Mass to centre of rigidity offset > 0.5 width (severe) Mass to centre of rigidity offset > 0.3 width (significant) Mass to centre of ≤ 0.3 width or effective torsional resistance available from elements orientated perpendicularly (insignificant).
MEXICO	At any story, the torsional eccentricity *es* shall not exceed 10 per cent of the in-plan dimension parallel to the eccentricity.
INDIA	Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure.
EUROCODE-8	At each level and for each direction of analysis x and y, the structural eccentricity eo and the torsional radius r shall be in accordance with the two conditions below, which are expressed for the direction of analysis y: $eox \leq 0.30.\ rx;\ rx \geq ls$ where; eox is the distance between the centre of stiffness and the centre of mass, measured along the x direction, which is normal to the direction of analysis considered; rx is the square root of the ratio of the torsional stiffness to the lateral stiffness in the y-direction; ls is the radius of gyration of the floor mass in plan
ASCE/SEI 7-10	*Torsional irregularity* is defined to exist where the maximum story drift, computed including accidental torsion with $Ax = 1.0$, at one end of the structure transverse to an axis, is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid. *Extreme torsional irregularity* is defined to exist where the maximum story drift, computed including accidental torsion with $Ax = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.

Table 96: Different storey drifts limitations based on different building codes (Smith 2011)

Standard/ Reference	Effect	Type	Inter-storey Drift Ratio Limit	Top Deflection Limit
Chinese Standard				
JGJ3-2002 Technical specification for concrete structures of tall building	Wind (H>250m)	Concrete/ Steel/ Composite	1/500	No guidance
	Seismic (50yrs) (H>250m)	Concrete/ Steel/Composite	1/500	No guidance
JGJ 99-98 Technical specification for steel structure of tall building	Wind	Steel Structure	1/400	No guidance
	Seismic (50yrs)	Steel Structure	1/250	No guidance
DG/TJ08-015-2004 Code for design of steel – concrete hybrid structures for high – rise buildings (Shanghai)	Wind (H>250m)	Composite	1/500	No guidance
	Seismic (50yrs) (H>250m)	Composite	1/500	No guidance
Hong Kong Code				
Code of practice on Wind Effects in Hong Kong 2004	Wind	RC/Steel	No guidance	No guidance
Code of Practice for Structural use of Concrete 2004	Wind	RC	No guidance	1/500
Code of Practice for Structural Use of Steel 2005	Wind	Steel	1/400	1/500
Eurocode				
Eurocode 3 ENV 1993-1-1:2005:	Wind	Steel	No guidance	No guidance
Eurocode 8 EN 1998-1-2004	Seismic (approx 95 year)	Steel / Concrete (limits depend on finishes)	1/200 – 1/100	No guidance
British Standards				
BS 5950 – structural steel in buildings	Wind	Steel	1/300	No guidance
BS 8110 – structural use of concrete (limit applies unless partition, claddings have been specifically detailed ..)	Wind	Concrete	1/500	No guidance
American Standards				
ASCE 7-05 – Minimum design loads for buildings and other structures	Wind		No guidance	No guidance
	Seismic (2/3 of 2475 event)	Steel / Concrete	1/100 – 1/200	No guidance

Appendix D: Some provisions result from software analysis

The following are the tables obtained from the ETABS analysis for the compared parameters.

Table 97: Vertical distribution of storey force for both codes

Floor	Forces from the ACI Code Model (kN)	Forces from the Eurocode Model (kN)	Forces Difference (kN)
Ground Floor	1.51	36.85	-35.34
1st Floor	3.47	52.93	-49.46
2nd Floor	6.56	72.77	-66.21
3rd Floor	10.62	92.62	-82
4th Floor	15.66	112.47	-96.81
5th Floor	21.67	132.31	-110.64
6th Floor	28.66	152.16	-123.5
7th Floor	36.62	172.01	-135.39
8th Floor	52.99	223.11	-170.12
9th Floor	60.02	224.84	-164.82
10th Floor	74.01	249.68	-175.67
11th Floor	89.47	274.51	-185.04
12th Floor	106.10	298.53	-192.43
13th Floor	124.13	322.53	-198.4
14th Floor	143.88	347.23	-203.35
15th Floor	165.07	371.93	-206.86
16th Floor	187.74	396.64	-208.9
17th Floor	211.85	421.35	-209.5
18th Floor	237.41	446.04	-208.63
19th Floor	264.45	470.76	-206.31
20th Floor	292.93	495.46	-202.53
21st Floor	322.86	520.15	-197.29
22nd Floor	340.60	523.82	-183.22
23rd Floor	356.47	524.42	-167.95
24th Floor	388.05	547.15	-159.1
25th Floor	421.00	569.91	-148.91
26th Floor	455.27	592.65	-137.38
27th Floor	490.88	615.38	-124.5
28th Floor	527.85	638.14	-110.29

29th Floor	566.13	660.87	-94.74
30th Floor	601.37	678.84	-77.47
31st Floor	642.04	701.41	-59.37
32nd Floor	684.07	724.02	-39.95
33rd Floor	727.42	746.60	-19.18
34th Floor	772.07	769.17	2.9
35th Floor	818.10	791.78	26.32
36th Floor	865.44	814.36	51.08
37th Floor	914.09	836.92	77.17
38th Floor	946.68	843.97	102.71
39th Floor	973.98	846.05	127.93
40th Floor	1024.46	867.68	156.78
41st Floor	1076.28	889.37	186.91
42nd Floor	1129.35	911.04	218.31
43rd Floor	1183.58	932.61	250.97
44th Floor	1238.62	953.82	284.8
45th Floor	1295.22	975.27	319.95
46th Floor	1353.31	996.89	356.42
47th Floor	1412.76	1018.56	394.2
48th Floor	1473.45	1040.21	433.24
49th Floor	1535.41	1062.42	472.99
50th Floor	1369.49	928.58	440.91

Table 98: P-Delta due to EQx in the ACI 318-19 model

Storey	Height	Force	Shear (Vx)	Δx	θx	Consideration of P-Delta
	m	kN	kN	m		
50	3.6	28882.98	2910.871	0.002863	0.0020	No P-delta
49	3.6	57846.55	5445.234	0.002981	0.0022	No P-delta
48	3.6	86810.12	7153.165	0.003081	0.0026	No P-delta
47	3.6	115773.7	8170.191	0.003183	0.0031	No P-delta
46	3.6	144736.3	8720.637	0.00328	0.0038	No P-delta
45	3.6	173699.9	9064.924	0.003372	0.0045	No P-delta
44	3.6	202681.3	9406.721	0.003453	0.0052	No P-delta
43	3.6	231664.3	9823.078	0.003538	0.0058	No P-delta
42	3.6	260648.2	10278.93	0.003621	0.0064	No P-delta
41	3.6	289632.2	10710.54	0.003698	0.0069	No P-delta
40	3.6	318615.2	11090.32	0.00377	0.0075	No P-delta
39	3.6	347599.1	11432.12	0.003828	0.0081	No P-delta
38	3.6	377709.2	11768.27	0.003855	0.0086	No P-delta
37	3.6	407818.2	12101.01	0.003911	0.0092	No P-delta
36	3.6	437928.3	12398.07	0.003966	0.0097	No P-delta
35	3.6	468038.3	12644.5	0.004018	0.0103	No P-delta
34	3.6	498147.4	12860.27	0.004066	0.0109	No P-delta
33	3.6	528257.4	13084.89	0.004112	0.0115	No P-delta
32	3.6	558367.5	13339.53	0.004154	0.0121	No P-delta
31	3.6	588476.6	13606.9	0.004193	0.0126	No P-delta
30	3.6	618586.6	13852.05	0.004227	0.0131	No P-delta
29	3.6	649505.5	14063.81	0.004255	0.0136	No P-delta
28	3.6	680425.3	14262.62	0.004278	0.0142	No P-delta
27	3.6	711344.2	14478.3	0.004296	0.0147	No P-delta
26	3.6	742264	14715	0.004308	0.0151	No P-delta

25	3.6	773183.9	14951.69	0.004314	0.0155	No P-delta
24	3.6	804102.7	15176.64	0.004312	0.0159	No P-delta
23	3.6	835022.6	15406.19	0.004307	0.0162	No P-delta
22	3.6	868330.1	15670.99	0.004247	0.0163	No P-delta
21	3.6	901636.7	15958.14	0.004242	0.0166	No P-delta
20	3.6	934944.2	16216.85	0.00423	0.0169	No P-delta
19	3.6	968251.8	16439.55	0.004214	0.0172	No P-delta
18	3.6	1001558	16670.11	0.004194	0.0175	No P-delta
17	3.6	1034866	16955.8	0.004169	0.0177	No P-delta
16	3.6	1068173	17288.35	0.004136	0.0177	No P-delta
15	3.6	1101480	17619.44	0.004091	0.0178	No P-delta
14	3.6	1134788	17937.38	0.004037	0.0177	No P-delta
13	3.6	1168095	18283.84	0.003972	0.0176	No P-delta
12	3.6	1201555	18664.47	0.003889	0.0174	No P-delta
11	3.6	1235016	19000.61	0.003797	0.0171	No P-delta
10	3.6	1268476	19246.76	0.003686	0.0169	No P-delta
9	3.6	1301936	19491.04	0.003509	0.0163	No P-delta
8	3	1338897	19894.13	0.003699	0.0207	No P-delta
7	3	1370883	20361.31	0.003529	0.0198	No P-delta
6	3	1402868	20881.8	0.003243	0.0182	No P-delta
5	3	1434854	21580.38	0.00293	0.0162	No P-delta
4	3	1466839	22465.2	0.002642	0.0144	No P-delta
3	3	1498825	23298.63	0.002392	0.0128	No P-delta
2	3	1530810	24217.84	0.00222	0.0117	No P-delta
1	3	1562796	25405.4	0.002101	0.0108	No P-delta
Base	5	1601402	26242.24	0.001238	0.0038	No P-delta

Table 99: P-Delta due to EQy in the ACI 318-19 model

Storey	Height	Force	Shear (Vy)	Δy	θy	Consideration of P-Delta
	m	kN	kN	m		
50	3.6	28882.98	2472.617	0.006356	0.0052	No P-delta
49	3.6	57846.55	4821.215	0.00642	0.0053	No P-delta
48	3.6	86810.12	6628.497	0.006466	0.0059	No P-delta
47	3.6	115773.7	7909.6	0.00651	0.0066	No P-delta
46	3.6	144736.3	8709.298	0.006551	0.0076	No P-delta
45	3.6	173699.9	8805.818	0.006596	0.0090	No P-delta
44	3.6	202681.3	8840.812	0.006643	0.0106	No P-delta
43	3.6	231664.3	8871.814	0.006689	0.0121	No P-delta
42	3.6	260648.2	8961.789	0.006734	0.0136	No P-delta
41	3.6	289632.2	9032.439	0.006776	0.0151	No P-delta
40	3.6	318615.2	9104.613	0.006814	0.0166	No P-delta
39	3.6	347599.1	9112.786	0.006845	0.0181	No P-delta
38	3.6	377709.2	9199.403	0.006868	0.0196	No P-delta
37	3.6	407818.2	9270.437	0.006888	0.0210	No P-delta
36	3.6	437928.3	9558.652	0.006905	0.0220	No P-delta
35	3.6	468038.3	9872.667	0.006916	0.0228	No P-delta
34	3.6	498147.4	10188.71	0.006919	0.0235	No P-delta
33	3.6	528257.4	10481.62	0.006913	0.0242	No P-delta
32	3.6	558367.5	10727.51	0.006899	0.0249	No P-delta
31	3.6	588476.6	10910.51	0.006876	0.0258	No P-delta
30	3.6	618586.6	11029.49	0.006842	0.0266	No P-delta
29	3.6	649505.5	11101.49	0.006798	0.0276	No P-delta
28	3.6	680425.3	11153.97	0.006742	0.0286	No P-delta
27	3.6	711344.2	11215.1	0.006675	0.0294	No P-delta
26	3.6	742264	11303.02	0.006595	0.0301	No P-delta

25	3.6	773183.9	11421.47	0.006501	0.0306	No P-delta
24	3.6	804102.7	11564.47	0.006391	0.0309	No P-delta
23	3.6	835022.6	11725.23	0.006256	0.0309	No P-delta
22	3.6	868330.1	11916.12	0.006112	0.0309	No P-delta
21	3.6	901636.7	12151.19	0.005985	0.0308	No P-delta
20	3.6	934944.2	12416.6	0.00585	0.0306	No P-delta
19	3.6	968251.8	12689.26	0.005701	0.0302	No P-delta
18	3.6	1001558	12944.92	0.00554	0.0298	No P-delta
17	3.6	1034866	13173.94	0.005365	0.0293	No P-delta
16	3.6	1068173	13390.75	0.005176	0.0287	No P-delta
15	3.6	1101480	13626.87	0.004972	0.0279	No P-delta
14	3.6	1134788	13910.84	0.004752	0.0269	No P-delta
13	3.6	1168095	14254.47	0.004518	0.0257	No P-delta
12	3.6	1201555	14663.6	0.00427	0.0243	No P-delta
11	3.6	1235016	15161.01	0.004012	0.0227	No P-delta
10	3.6	1268476	15789.99	0.003738	0.0209	No P-delta
9	3.6	1301936	16579.52	0.00341	0.0186	No P-delta
8	3	1338897	17634.29	0.003115	0.0197	No P-delta
7	3	1370883	18637.33	0.002961	0.0181	No P-delta
6	3	1402868	19654.77	0.002674	0.0159	No P-delta
5	3	1434854	20696.89	0.002374	0.0137	No P-delta
4	3	1466839	21747.29	0.002064	0.0116	No P-delta
3	3	1498825	22685.18	0.001743	0.0096	No P-delta
2	3	1530810	23478.55	0.001409	0.0077	No P-delta
1	3	1562796	24253.93	0.001044	0.0056	No P-delta
Base	5	1601402	24765.18	0.00046	0.0015	No P-delta

Table 100: P-Delta due to EQx in the Eurocode model

Floor	Height	Force	Shear (Vx)	Δx	θx	Consideration of P-Delta
	m	kN	kN	m		
50	3.6	28911.2	1013.795	0.003148	0.025	No P-delta
49	3.6	57874.78	2139.967	0.003222	0.024	No P-delta
48	3.6	86838.35	3239.359	0.003307	0.025	No P-delta
47	3.6	115801.9	4286.933	0.003414	0.026	No P-delta
46	3.6	144764.5	5296.989	0.003531	0.027	No P-delta
45	3.6	173728.1	6272.704	0.003655	0.028	No P-delta
44	3.6	202709.5	7215.655	0.00378	0.029	No P-delta
43	3.6	231692.5	8126.922	0.003917	0.031	No P-delta
42	3.6	260676.5	9007.17	0.004054	0.033	No P-delta
41	3.6	289660.4	9857.841	0.004189	0.034	No P-delta
40	3.6	318643.4	10680.4	0.004318	0.036	No P-delta
39	3.6	347627.4	11475.74	0.004437	0.037	No P-delta
38	3.6	377737.4	12262.63	0.004517	0.039	No P-delta
37	3.6	407846.5	13037.33	0.004626	0.040	No P-delta
36	3.6	437956.5	13786.45	0.004735	0.042	No P-delta
35	3.6	468066.6	14511.09	0.00484	0.043	No P-delta
34	3.6	498175.7	15211.81	0.004943	0.045	No P-delta
33	3.6	528285.7	15888.58	0.005041	0.047	No P-delta
32	3.6	558395.7	16541.47	0.005134	0.048	No P-delta
31	3.6	588504.8	17171.22	0.00522	0.050	No P-delta
30	3.6	618614.9	17779.16	0.005298	0.051	No P-delta
29	3.6	649533.7	18370.16	0.005368	0.053	No P-delta
28	3.6	680453.6	18939.73	0.005429	0.054	No P-delta
27	3.6	711372.4	19487.35	0.00548	0.056	No P-delta
26	3.6	742292.3	20013.56	0.00552	0.057	No P-delta

25	3.6	773212.1	20519.85	0.005548	0.058	No P-delta
24	3.6	804131	21007.19	0.00556	0.059	No P-delta
23	3.6	835050.8	21474.9	0.005564	0.060	No P-delta
22	3.6	868358.4	21941.56	0.00547	0.060	No P-delta
21	3.6	901664.9	22405.21	0.00547	0.061	No P-delta
20	3.6	934972.5	22848.74	0.005456	0.062	No P-delta
19	3.6	968280	23273.02	0.005434	0.063	No P-delta
18	3.6	1001587	23676.76	0.005405	0.064	No P-delta
17	3.6	1034894	24058.04	0.005365	0.064	No P-delta
16	3.6	1068202	24416.62	0.005314	0.065	No P-delta
15	3.6	1101508	24753.86	0.005247	0.065	No P-delta
14	3.6	1134816	25070.1	0.005165	0.065	No P-delta
13	3.6	1168123	25363.62	0.005068	0.065	No P-delta
12	3.6	1201583	25633.85	0.00494	0.064	No P-delta
11	3.6	1235044	25881.25	0.004806	0.064	No P-delta
10	3.6	1268504	26104.69	0.004649	0.063	No P-delta
9	3.6	1301964	26302.58	0.004401	0.061	No P-delta
8	3	1338925	26495.78	0.004127	0.070	No P-delta
7	3	1370911	26644.28	0.003962	0.068	No P-delta
6	3	1402896	26771.2	0.003758	0.066	No P-delta
5	3	1434882	26875.94	0.00351	0.062	No P-delta
4	3	1466868	26961.2	0.003207	0.058	No P-delta
3	3	1498853	27024.51	0.00284	0.053	No P-delta
2	3	1530839	27068.05	0.002395	0.045	No P-delta
1	3	1562824	27108.6	0.001842	0.035	No P-delta
Base	5	1601430	27131.48	0.00084	0.010	No P-delta

Table 101: P-Delta due to EQy in the Eurocode model

Floor	Height	Force	Shear (Vy)	Δy	θy	Consideration of P-Delta
	m	kN	kN	m		
50	3.6	28911.2	1309.099	0.006226	0.0382	No P-delta
49	3.6	57874.78	2748.507	0.006307	0.0369	No P-delta
48	3.6	86838.35	4100.729	0.006372	0.0375	No P-delta
47	3.6	115801.9	5371.265	0.006425	0.0385	No P-delta
46	3.6	144764.5	6567.863	0.006481	0.0397	No P-delta
45	3.6	173728.1	7701.882	0.006539	0.0410	No P-delta
44	3.6	202709.5	8783.271	0.006597	0.0423	No P-delta
43	3.6	231692.5	9815.016	0.006655	0.0436	No P-delta
42	3.6	260676.5	10798.17	0.006713	0.0450	No P-delta
41	3.6	289660.4	11734.81	0.006769	0.0464	No P-delta
40	3.6	318643.4	12627.65	0.006821	0.0478	No P-delta
39	3.6	347627.4	13480.37	0.006865	0.0492	No P-delta
38	3.6	377737.4	14317.33	0.006898	0.0506	No P-delta
37	3.6	407846.5	15138.25	0.006934	0.0519	No P-delta
36	3.6	437956.5	15929.31	0.006968	0.0532	No P-delta
35	3.6	468066.6	16689.19	0.006996	0.0545	No P-delta
34	3.6	498175.7	17416.97	0.007018	0.0558	No P-delta
33	3.6	528285.7	18113.24	0.007032	0.0570	No P-delta
32	3.6	558395.7	18780.12	0.007038	0.0581	No P-delta
31	3.6	588504.8	19420.84	0.007036	0.0592	No P-delta
30	3.6	618614.9	20038.24	0.007024	0.0602	No P-delta
29	3.6	649533.7	20636.86	0.007002	0.0612	No P-delta
28	3.6	680453.6	21210.86	0.006969	0.0621	No P-delta
27	3.6	711372.4	21758.91	0.006925	0.0629	No P-delta
26	3.6	742292.3	22281.37	0.006868	0.0636	No P-delta

25	3.6	773212.1	22780.18	0.006798	0.0641	No P-delta
24	3.6	804131	23257.97	0.006714	0.0645	No P-delta
23	3.6	835050.8	23715.91	0.0066	0.0646	No P-delta
22	3.6	868358.4	24172.13	0.006467	0.0645	No P-delta
21	3.6	901664.9	24622.11	0.006354	0.0646	No P-delta
20	3.6	934972.5	25046.78	0.006233	0.0646	No P-delta
19	3.6	968280	25447.45	0.006097	0.0644	No P-delta
18	3.6	1001587	25826.96	0.005948	0.0641	No P-delta
17	3.6	1034894	26186.69	0.005784	0.0635	No P-delta
16	3.6	1068202	26524.52	0.005604	0.0627	No P-delta
15	3.6	1101508	26837.19	0.005408	0.0617	No P-delta
14	3.6	1134816	27123.84	0.005194	0.0604	No P-delta
13	3.6	1168123	27386.85	0.004962	0.0588	No P-delta
12	3.6	1201583	27630.05	0.004711	0.0569	No P-delta
11	3.6	1235044	27853.11	0.004446	0.0548	No P-delta
10	3.6	1268504	28051.99	0.00417	0.0524	No P-delta
9	3.6	1301964	28225.87	0.003806	0.0488	No P-delta
8	3	1338925	28396.3	0.003472	0.0546	No P-delta
7	3	1370911	28528.69	0.003342	0.0535	No P-delta
6	3	1402896	28641.82	0.003035	0.0496	No P-delta
5	3	1434882	28736.41	0.002723	0.0453	No P-delta
4	3	1466868	28818.41	0.002397	0.0407	No P-delta
3	3	1498853	28883.22	0.002056	0.0356	No P-delta
2	3	1530839	28928.42	0.001694	0.0299	No P-delta
1	3	1562824	28981.44	0.001293	0.0232	No P-delta
Base	5	1601430	28999.31	0.000608	0.0067	No P-delta

Appendix E: Column reinforcement ratio comparison

Due to the huge amount of data obtained from the model, sample of columns reinforcement will be shown. The samples taken are for every four floors in the building.

Table 102: Columns reinforcement ratio comparison for 4th podium floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Code for Columns Design
C1- 0.80 m x1.50 m	0.662	0.636	0.026	Eurocode
C2-1.20 m x 1.30 m	0.695	0.595	0.1	Eurocode
C3-1.70 m x 1.50 m	0.687	0.651	0.036	Eurocode
C3A-1.80 m x 1.60 m	0.794	0.752	0.042	Eurocode
C4-1.10 m x1.50 m	0.760	0.639	0.121	Eurocode
C4A-1.20 m x 1.50 m	0.796	0.962	0.166	ACI Code
C5-0.40 m x 0.90 m	0.403	0.807	0.404	ACI Code

- Based on the table above the conservative code for the concrete columns design in the 4th Podium floor is the Eurocode.

Table 103: Column reinforcement ratio comparison for 8th podium floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1- 0.80 m x1.50 m	0.598	0.556	0.042	Eurocode
C2-1.20 m x 1.30 m	0.642	0.55	0.092	Eurocode
C3-1.70 m x 1.50 m	0.653	0.636	0.017	Eurocode
C3A-1.80 m x 1.60 m	0.725	0.678	0.047	Eurocode
C4-1.10 m x1.50 m	0.699	0.589	0.11	Eurocode
C4A-1.20 m x 1.50 m	0.743	0.843	0.1	ACI Code
C5-0.40 m x 0.90 m	0.455	0.75	0.295	ACI Code

- Based on the table above the conservative code for the concrete columns design in the 8th Podium floor is the Eurocode.

Table 104: Column reinforcement ratio comparison for 4th office floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1- 0.80 m x1.50 m	0.525	0.476	0.049	Eurocode
C2-1.20 m x 1.30 m	0.581	0.50	0.081	Eurocode
C3-1.70 m x 1.50 m	0.604	0.658	0.054	ACI Code
C3A-1.80 m x 1.60 m	0.648	0.596	0.052	Eurocode
C4-1.10 m x1.50 m	0.628	0.533	0.095	Eurocode
C4A-1.20 m x 1.50 m	0.671	0.741	0.07	ACI Code

- Based on the table above the conservative code for the concrete columns design in the 4th office floor is the Eurocode.

Table 105: Column reinforcement ratio comparison for 8th office floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1- 0.80 m x1.30 m	0.535	0.479	0.056	Eurocode
C2-1.20 m x 1.30 m	0.502	0.434	0.068	Eurocode
C2A-1.20 m x 1.35 m	0.503	0.437	0.066	Eurocode
C3-1.70 m x 1.50 m	0.637	0.606	0.031	Eurocode
C4-1.10 m x1.50 m	0.554	0.472	0.082	Eurocode
C4A-1.20 m x 1.50 m	0.579	0.625	0.046	ACI Code

- Based on the table above the conservative code for the concrete columns design in the 8th office floor is the Eurocode.

Table 106: Column reinforcement ratio comparison for 12th office floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1- 0.80 m x1.30 m	0.453	0.396	0.057	Eurocode
C2-1.20 m x 1.30 m	0.438	0.380	0.058	Eurocode
C2A-1.20 m x 1.35 m	0.439	0.381	0.058	Eurocode
C3-1.70 m x 1.50 m	0.550	0.551	0.001	ACI Code
C4-1.10 m x1.50 m	0.479	0.411	0.068	Eurocode
C4A-1.20 m x 1.50 m	0.487	0.512	0.025	ACI Code

- Based on the table above the conservative code for the concrete columns design in the 12th office floor is the Eurocode.

Table 107: Column reinforcement ratio comparison for 16th office floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1- 0.60 m x1.00 m	0.774	0.67	0.104	Eurocode
C2-1.00 m x 1.00 m	0.762	0.671	0.091	Eurocode
C3-1.50 m x 1.20 m	0.813	0.823	0.01	ACI Code
C4-0.80 m x 1.10 m	0.965	0.992	0.027	ACI Code

- Based on the table above the conservative code for the concrete columns design in the 16th office floor is the Eurocode.

Table 108: Column reinforcement ratio comparison for 20th office floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1- 0.60 m x1.00 m	0.645	0.566	0.079	Eurocode
C2-1.00 m x 1.00 m	0.641	0.563	0.078	Eurocode
C3-1.50 m x 1.20 m	0.684	0.764	0.08	ACI Code
C4-0.80 m x 1.10 m	0.775	0.789	0.014	ACI Code

- Based on the table above the conservative code for the concrete columns design in the 20^{th} office floor is the Eurocode.

Table 109: Column reinforcement ratio comparison for 24th residential floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1-0.60 m x 1.0 m	0.525	0.461	0.064	Eurocode
C2-1.0 m x 1.0 m	0.52	0.457	0.063	Eurocode
C3-1.50 m x 1.20 m	0.556	0.633	0.077	ACI Code
C4-0.80 m x 1.10 m	0.622	0.596	0.026	Eurocode

- Based on the table above the conservative code for the concrete columns design in the 24^{th} Residential floor is the Eurocode.

Table 110: Column reinforcement ratio comparison for 28th residential floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1-0.60 m x 1.0 m	0.418	0.374	0.044	Eurocode
C2-1.0 m x 1.0 m	0.403	0.353	0.05	Eurocode
C3-1.50 m x 1.20 m	0.427	0.502	0.075	ACI Code
C4-0.80 m x 1.10 m	0.488	0.432	0.056	Eurocode

- Based on the table above the conservative code for the concrete columns design in the 28^{th} Residential floor is the Eurocode.

Table 111: Column reinforcement ratio comparison for 32nd residential floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1-0.40 m x 0.90 m	0.60	0.561	0.039	Eurocode
C2-0.60 m x 0.60 m	0.788	0.695	0.093	Eurocode
C3-1.10 m x 0.70 m	0.723	0.840	0.117	ACI Code
C4-0.60 m x 0.80 m	0.679	0.616	0.063	Eurocode

- Based on the table above the conservative code for the concrete columns design in the 32nd Residential floor is the Eurocode.

Table 112: Column reinforcement ratio comparison for 36th residential floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1-0.40 m x 0.90 m	0.419	0.409	0.01	Eurocode
C2-0.60 m x 0.60 m	0.513	0.455	0.058	Eurocode
C3-1.10 m x 0.70 m	0.493	0.612	0.119	ACI Code
C4-0.60 m x 0.80 m	0.449	0.421	0.028	Eurocode

- Based on the table above the conservative code for the concrete columns design in the 36th Residential floor is the Eurocode.

Table 113: Column reinforcement ratio comparison for 40th residential floor

Column	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Columns Design
C1A-0.40 m x 0.70 m	0.402	0.303	0.099	Eurocode
C2A-0.60 m x 0.60 m	0.252	0.234	0.018	Eurocode
C3-1.10 m x 0.70 m	0.287	0.319	0.032	ACI Code
C3A-0.90 m x 0.90 m	0.289	0.3	0.011	ACI Code
C4-0.60 m x 0.80 m	0.244	0.232	0.012	Eurocode
C4A-0.50 m x 0.80 m	0.283	0.314	0.031	ACI Code

- Based on the table above the conservative code for the concrete columns design in the 40th Residential floor is the Eurocode.

Appendix F: Shear Walls reinforcement ratio comparison

Table 114: Shear walls reinforcement ratio comparison for 4th Podium Floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T32@100cm	0.645	0.502	0.143	Eurocode
P1A	T32@100cm	0.643	0.511	0.132	Eurocode
P2	T32@100cm	0.680	0.503	0.177	Eurocode
P2A	T32@100cm	0.688	0.51	0.178	Eurocode
P3	T32@100cm	0.529	0.376	0.153	Eurocode
P3A	T32@100cm	0.532	0.383	0.149	Eurocode
P4	T32@100cm	0.528	0.375	0.153	Eurocode
P4A	T32@100cm	0.545	0.39	0.155	Eurocode
P5	T32@100cm	0.671	0.496	0.175	Eurocode
P5A	T32@100cm	0.707	0.524	0.183	Eurocode
P6	T32@100cm	0.630	0.491	0.139	Eurocode
P6A	T32@100cm	0.607	0.486	0.121	Eurocode
P7	T32@80cm	0.706	0.589	0.117	Eurocode
P7A	T32@80cm	0.697	0.556	0.141	Eurocode
P8	T32@100cm	0.685	0.598	0.087	Eurocode
P8A	T32@100cm	0.672	0.59	0.082	Eurocode
P9	T32@100cm	0.707	0.623	0.084	Eurocode
P9A	T32@100cm	0.694	0.61	0.084	Eurocode
P10	T32@100cm	0.74	0.607	0.133	Eurocode
P10A	T32@100cm	0.726	0.6	0.126	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 115: Shear walls reinforcement ratio comparison for 8th podium floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T32@100cm	0.591	0.46	0.131	Eurocode
P1A	T32@100cm	0.589	0.469	0.12	Eurocode
P2	T32@100cm	0.622	0.46	0.162	Eurocode
P2A	T32@100cm	0.630	0.467	0.163	Eurocode
P3	T32@100cm	0.488	0.345	0.143	Eurocode
P3A	T32@100cm	0.491	0.351	0.14	Eurocode
P4	T32@100cm	0.487	0.344	0.143	Eurocode
P4A	T32@100cm	0.505	0.359	0.146	Eurocode
P5	T32@100cm	0.614	0.453	0.161	Eurocode
P5A	T32@100cm	0.648	0.481	0.167	Eurocode
P6	T32@100cm	0.577	0.45	0.127	Eurocode
P6A	T32@100cm	0.551	0.443	0.108	Eurocode
P7	T32@80cm	0.647	0.509	0.138	Eurocode
P7A	T32@80cm	0.638	0.48	0.158	Eurocode
P8	T32@100cm	0.657	0.558	0.099	Eurocode
P8A	T32@100cm	0.644	0.549	0.095	Eurocode
P9	T32@100cm	0.679	0.582	0.097	Eurocode
P9A	T32@100cm	0.665	0.57	0.095	Eurocode
P10	T32@100cm	0.678	0.524	0.154	Eurocode
P10A	T32@100cm	0.664	0.517	0.147	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 116: Shear walls reinforcement ratio comparison for 4th office floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T25@100cm	0.581	0.454	0.127	Eurocode
P1A	T25@100cm	0.579	0.463	0.116	Eurocode
P2	T25@100cm	0.604	0.452	0.152	Eurocode
P2A	T25@100cm	0.613	0.459	0.154	Eurocode
P3	T25@100cm	0.480	0.341	0.139	Eurocode
P3A	T25@100cm	0.484	0.348	0.136	Eurocode
P4	T25@100cm	0.479	0.34	0.139	Eurocode
P4A	T25@100cm	0.497	0.356	0.141	Eurocode
P5	T25@100cm	0.596	0.445	0.151	Eurocode
P5A	T25@100cm	0.630	0.473	0.157	Eurocode
P6	T25@100cm	0.566	0.443	0.123	Eurocode
P6A	T25@100cm	0.539	0.434	0.105	Eurocode
P7	T32@80cm	0.578	0.443	0.135	Eurocode
P7A	T32@80cm	0.570	0.418	0.152	Eurocode
P8	T25@100cm	0.643	0.547	0.096	Eurocode
P8A	T25@100cm	0.629	0.539	0.09	Eurocode
P9	T25@100cm	0.665	0.573	0.092	Eurocode
P9A	T25@100cm	0.650	0.56	0.09	Eurocode
P10	T25@100cm	0.648	0.457	0.191	Eurocode
P10A	T25@100cm	0.634	0.485	0.149	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 117: Shear walls reinforcement ratio comparison for 8th office floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T25@100cm	0.520	0.424	0.096	Eurocode
P1A	T25@100cm	0.519	0.411	0.108	Eurocode
P2	T25@100cm	0.536	0.400	0.136	Eurocode
P2A	T25@100cm	0.542	0.407	0.135	Eurocode
P3	T25@100cm	0.429	0.303	0.126	Eurocode
P3A	T25@100cm	0.433	0.309	0.124	Eurocode
P4	T25@100cm	0.428	0.302	0.126	Eurocode
P4A	T25@100cm	0.444	0.316	0.128	Eurocode
P5	T25@100cm	0.528	0.349	0.179	Eurocode
P5A	T25@100cm	0.557	0.417	0.14	Eurocode
P6	T25@100cm	0.507	0.394	0.113	Eurocode
P6A	T25@100cm	0.486	0.389	0.097	Eurocode
P7	T32@80cm	0.504	0.374	0.13	Eurocode
P7A	T32@80cm	0.496	0.352	0.144	Eurocode
P8	T25@100cm	0.561	0.462	0.099	Eurocode
P8A	T25@100cm	0.548	0.454	0.094	Eurocode
P9	T25@100cm	0.582	0.485	0.097	Eurocode
P9A	T25@100cm	0.569	0.473	0.096	Eurocode
P10	T25@100cm	0.566	0.385	0.181	Eurocode
P10A	T25@100cm	0.553	0.409	0.144	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 118: Shear walls reinforcement ratio comparison for 12th office floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T25@100cm	0.459	0.353	0.106	Eurocode
P1A	T25@100cm	0.459	0.359	0.1	Eurocode
P2	T25@100cm	0.469	0.352	0.117	Eurocode
P2A	T25@100cm	0.474	0.357	0.117	Eurocode
P3	T25@100cm	0.379	0.266	0.113	Eurocode
P3A	T25@100cm	0.382	0.271	0.111	Eurocode
P4	T25@100cm	0.378	0.265	0.113	Eurocode
P4A	T25@100cm	0.392	0.277	0.115	Eurocode
P5	T25@100cm	0.462	0.347	0.115	Eurocode
P5A	T25@100cm	0.485	0.364	0.121	Eurocode
P6	T25@100cm	0.448	0.344	0.104	Eurocode
P6A	T25@100cm	0.432	0.342	0.09	Eurocode
P7	T32@80cm	0.432	0.31	0.122	Eurocode
P7A	T32@80cm	0.425	0.291	0.134	Eurocode
P8	T25@100cm	0.48	0.38	0.1	Eurocode
P8A	T25@100cm	0.47	0.373	0.097	Eurocode
P9	T25@100cm	0.500	0.397	0.103	Eurocode
P9A	T25@100cm	0.488	0.39	0.098	Eurocode
P10	T25@100cm	0.486	0.319	0.167	Eurocode
P10A	T25@100cm	0.475	0.338	0.137	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 119: Shear walls reinforcement ratio comparison for 16th office floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T20@100cm	0.582	0.453	0.129	Eurocode
P1A	T20@100cm	0.583	0.462	0.121	Eurocode
P2	T20@100cm	0.590	0.456	0.134	Eurocode
P2A	T20@100cm	0.597	0.464	0.133	Eurocode
P3	T20@100cm	0.481	0.351	0.13	Eurocode
P3A	T20@100cm	0.485	0.355	0.13	Eurocode
P4	T20@100cm	0.480	0.35	0.13	Eurocode
P4A	T20@100cm	0.497	0.359	0.138	Eurocode
P5	T20@100cm	0.581	0.451	0.13	Eurocode
P5A	T20@100cm	0.608	0.467	0.141	Eurocode
P6	T20@100cm	0.567	0.441	0.126	Eurocode
P6A	T20@100cm	0.551	0.445	0.106	Eurocode
P7	T20@100cm	0.765	0.423	0.342	Eurocode
P7A	T20@100cm	0.753	0.521	0.232	Eurocode
P8	T20@100cm	0.756	0.591	0.165	Eurocode
P8A	T20@100cm	0.738	0.579	0.159	Eurocode
P9	T20@100cm	0.788	0.625	0.163	Eurocode
P9A	T20@100cm	0.769	0.608	0.161	Eurocode
P10	T20@100cm	0.768	0.434	0.334	Eurocode
P10A	T20@100cm	0.751	0.529	0.222	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 120: Shear walls reinforcement ratio comparison for 20th office floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T20@100cm	0.497	0.384	0.113	Eurocode
P1A	T20@100cm	0.499	0.389	0.11	Eurocode
P2	T20@100cm	0.500	0.396	0.104	Eurocode
P2A	T20@100cm	0.507	0.403	0.104	Eurocode
P3	T20@100cm	0.414	0.308	0.106	Eurocode
P3A	T20@100cm	0.417	0.311	0.106	Eurocode
P4	T20@100cm	0.413	0.307	0.106	Eurocode
P4A	T20@100cm	0.426	0.306	0.12	Eurocode
P5	T20@100cm	0.492	0.392	0.1	Eurocode
P5A	T20@100cm	0.514	0.393	0.121	Eurocode
P6	T20@100cm	0.485	0.372	0.113	Eurocode
P6A	T20@100cm	0.473	0.376	0.097	Eurocode
P7	T20@100cm	0.642	0.342	0.3	Eurocode
P7A	T20@100cm	0.631	0.421	0.21	Eurocode
P8	T20@100cm	0.628	0.462	0.166	Eurocode
P8A	T20@100cm	0.613	0.455	0.158	Eurocode
P9	T20@100cm	0.656	0.49	0.166	Eurocode
P9A	T20@100cm	0.64	0.479	0.161	Eurocode
P10	T20@100cm	0.645	0.351	0.294	Eurocode
P10A	T20@100cm	0.631	0.427	0.204	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 121: Shear walls reinforcement ratio comparison for 24th residential floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T20@100cm	0.413	0.314	0.099	Eurocode
P1A	T20@100cm	0.415	0.321	0.094	Eurocode
P2	T20@100cm	0.414	0.334	0.08	Eurocode
P2A	T20@100cm	0.419	0.341	0.078	Eurocode
P3	T20@100cm	0.348	0.265	0.083	Eurocode
P3A	T20@100cm	0.352	0.270	0.082	Eurocode
P4	T20@100cm	0.346	0.264	0.082	Eurocode
P4A	T20@100cm	0.356	0.256	0.1	Eurocode
P5	T20@100cm	0.407	0.331	0.076	Eurocode
P5A	T20@100cm	0.423	0.328	0.095	Eurocode
P6	T20@100cm	0.403	0.308	0.095	Eurocode
P6A	T20@100cm	0.394	0.313	0.081	Eurocode
P7	T20@100cm	0.526	0.272	0.254	Eurocode
P7A	T20@100cm	0.517	0.334	0.183	Eurocode
P8	T20@100cm	0.509	0.355	0.154	Eurocode
P8A	T20@100cm	0.497	0.347	0.15	Eurocode
P9	T20@100cm	0.533	0.379	0.154	Eurocode
P9A	T20@100cm	0.52	0.367	0.153	Eurocode
P10	T20@100cm	0.529	0.278	0.251	Eurocode
P10A	T20@100cm	0.518	0.338	0.18	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 122: Shear walls reinforcement ratio comparison for 28th residential floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T20@100cm	0.329	0.250	0.079	Eurocode
P1A	T20@100cm	0.331	0.255	0.076	Eurocode
P2	T20@100cm	0.335	0.274	0.061	Eurocode
P2A	T20@100cm	0.338	0.280	0.058	Eurocode
P3	T20@100cm	0.285	0.226	0.059	Eurocode
P3A	T20@100cm	0.287	0.230	0.057	Eurocode
P4	T20@100cm	0.284	0.225	0.059	Eurocode
P4A	T20@100cm	0.287	0.210	0.077	Eurocode
P5	T20@100cm	0.331	0.272	0.059	Eurocode
P5A	T20@100cm	0.337	0.268	0.069	Eurocode
P6	T20@100cm	0.321	0.246	0.075	Eurocode
P6A	T20@100cm	0.316	0.253	0.063	Eurocode
P7	T20@100cm	0.418	0.218	0.2	Eurocode
P7A	T20@100cm	0.411	0.268	0.143	Eurocode
P8	T20@100cm	0.398	0.264	0.134	Eurocode
P8A	T20@100cm	0.389	0.258	0.131	Eurocode
P9	T20@100cm	0.418	0.288	0.13	Eurocode
P9A	T20@100cm	0.407	0.28	0.127	Eurocode
P10	T20@100cm	0.42	0.226	0.194	Eurocode
P10A	T20@100cm	0.411	0.278	0.133	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 123: Shear walls reinforcement ratio comparison for 32nd residential floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T16@200cm	0.273	0.213	0.06	Eurocode
P1A	T16@200cm	0.274	0.217	0.057	Eurocode
P2	T16@150cm	0.283	0.238	0.045	Eurocode
P2A	T16@150cm	0.283	0.244	0.039	Eurocode
P3	T16@150cm	0.240	0.214	0.026	Eurocode
P3A	T16@125cm	0.238	0.215	0.023	Eurocode
P4	T16@150cm	0.240	0.213	0.027	Eurocode
P4A	T16@150cm	0.237	0.187	0.05	Eurocode
P5	T16@150cm	0.279	0.237	0.042	Eurocode
P5A	T16@150cm	0.277	0.232	0.045	Eurocode
P6	T16@200cm	0.266	0.211	0.055	Eurocode
P6A	T16@200cm	0.263	0.219	0.044	Eurocode
P7	T16@200cm	0.348	0.176	0.172	Eurocode
P7A	T16@200cm	0.342	0.25	0.092	Eurocode
P8	T16@200cm	0.322	0.215	0.107	Eurocode
P8A	T16@200cm	0.315	0.211	0.104	Eurocode
P9	T16@200cm	0.338	0.235	0.103	Eurocode
P9A	T16@200cm	0.330	0.229	0.101	Eurocode
P10	T16@200cm	0.349	0.188	0.161	Eurocode
P10A	T16@200cm	0.342	0.263	0.079	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 124: Shear walls reinforcement ratio comparison for residential floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T16@200cm	0.183	0.145	0.038	Eurocode
P1A	T16@200cm	0.184	0.148	0.036	Eurocode
P2	T16@150cm	0.200	0.170	0.03	Eurocode
P2A	T16@150cm	0.198	0.174	0.024	Eurocode
P3	T16@150cm	0.175	0.169	0.006	Eurocode
P3A	T16@125cm	0.174	0.169	0.005	Eurocode
P4	T16@150cm	0.175	0.167	0.008	Eurocode
P4A	T16@150cm	0.161	0.133	0.028	Eurocode
P5	T16@150cm	0.199	0.170	0.029	Eurocode
P5A	T16@150cm	0.189	0.164	0.025	Eurocode
P6	T16@200cm	0.179	0.145	0.034	Eurocode
P6A	T16@200cm	0.177	0.155	0.022	Eurocode
P7	T16@200cm	0.232	0.127	0.105	Eurocode
P7A	T16@200cm	0.228	0.184	0.044	Eurocode
P8	T16@200cm	0.216	0.16	0.056	Eurocode
P8A	T16@200cm	0.214	0.174	0.04	Eurocode
P9	T16@200cm	0.222	0.164	0.058	Eurocode
P9A	T16@200cm	0.216	0.158	0.058	Eurocode
P10	T16@200cm	0.233	0.131	0.102	Eurocode
P10A	T16@200cm	0.229	0.185	0.044	Eurocode

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Table 125: Shear walls reinforcement ratio comparison for 40th residential floor

Shear Wall	Reinforcement Used	Reinforcement Ratio from ACI Model	Reinforcement Ratio from Eurocode Model	Ratio Difference	The Conservative Model for Shear Walls Design
P1	T16@200cm	0.100	0.078	0.022	Eurocode
P1A	T16@200cm	0.100	0.083	0.017	Eurocode
P2	T16@150cm	0.132	0.104	0.028	Eurocode
P2A	T16@150cm	0.127	0.106	0.021	Eurocode
P3	T16@150cm	0.119	0.151	0.032	ACI Code
P3A	T16@125cm	0.118	0.143	0.025	ACI Code
P4	T16@150cm	0.120	0.151	0.031	ACI Code
P4A	T16@150cm	0.100	0.089	0.011	Eurocode
P5	T16@150cm	0.134	0.105	0.029	Eurocode
P5A	T16@150cm	0.110	0.097	0.013	Eurocode
P6	T16@200cm	0.100	0.081	0.019	Eurocode
P6A	T16@200cm	0.100	0.09	0.01	Eurocode
P7	T16@200cm	0.126	0.085	0.041	Eurocode
P7A	T16@200cm	0.123	0.16	0.037	ACI Code
P8	T16@200cm	0.122	0.253	0.131	ACI Code
P8A	T16@200cm	0.125	0.267	0.142	ACI Code
P9	T16@200cm	0.118	0.22	0.102	ACI Code
P9A	T16@200cm	0.115	0.202	0.087	ACI Code
P10	T16@200cm	0.122	0.083	0.039	Eurocode
P10A	T16@200cm	0.121	0.196	0.075	ACI Code

As shown from the previous table it can be concluded that the Eurocode require less reinforcement than ACI Code, which makes the Eurocode more conservative than ACI Code in the concrete shear walls design.

Appendix G: Slabs analysis from RAM Concept

First Floor (ACI Code Model)

The following are the outputs for the first-floor from the ACI model, which is considered as an office floor. The outputs shows that the slab is safe in all of the checking mentioned in section 4.5.3, which allow for safe reinforcement calculation and post tension rate determination.

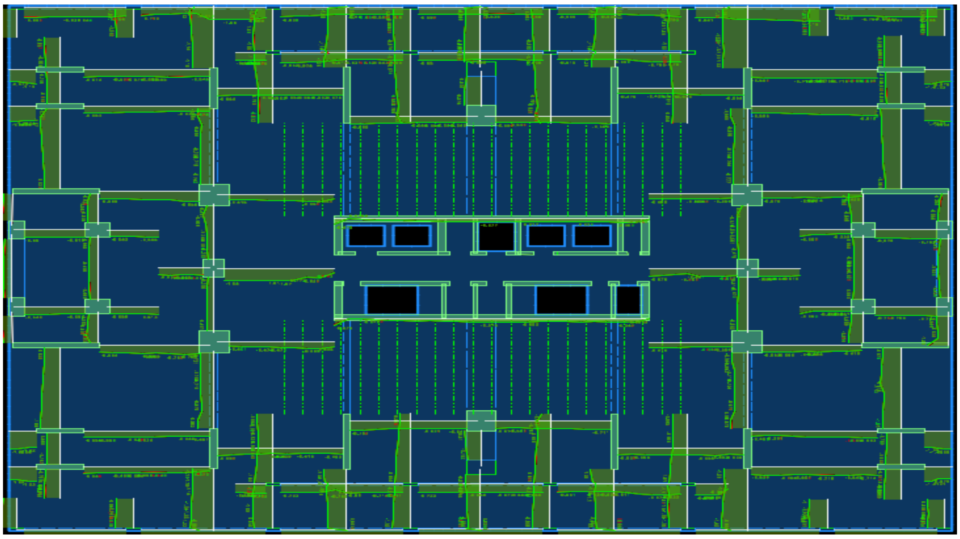


Figure 122: Precompression for the First floor (ACI code model)

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

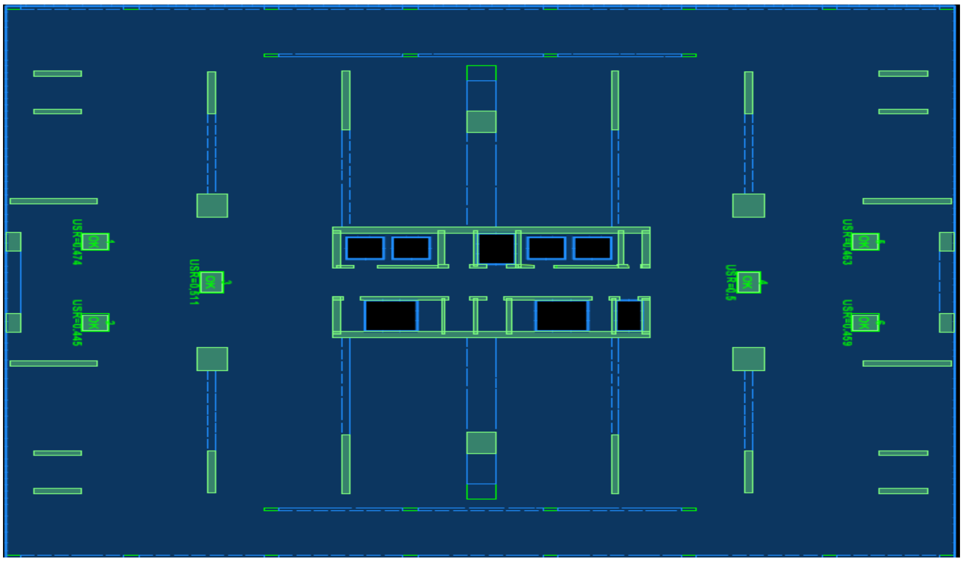


Figure 123: Punching for the First floor (ACI code model)

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long -term deflection should not exceed L/240, where L is the longest span, while for the incremental defelction the deflection should not exceed L/480. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 31.67 mm, while the limit for the incremental deflection is 15.83 mm.

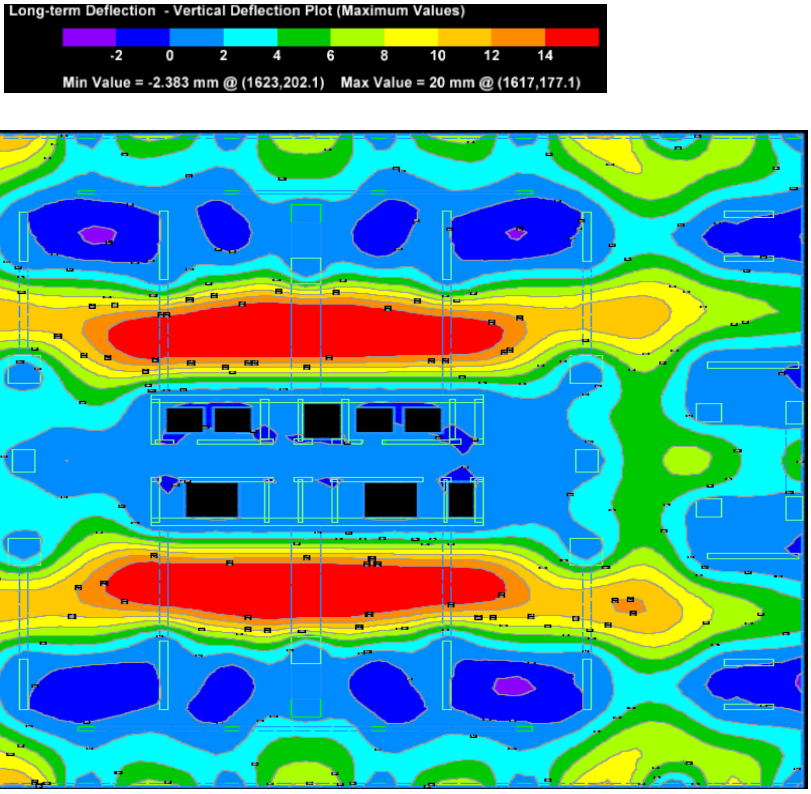


Figure 124: Long-term Deflection for the first-floor model (ACI code model)

As shown from the figure 123, the long-term deflection is 20 mm, which is less than the limit 31.67 mm, so the slab is safe in the long-term deflcetion checking.

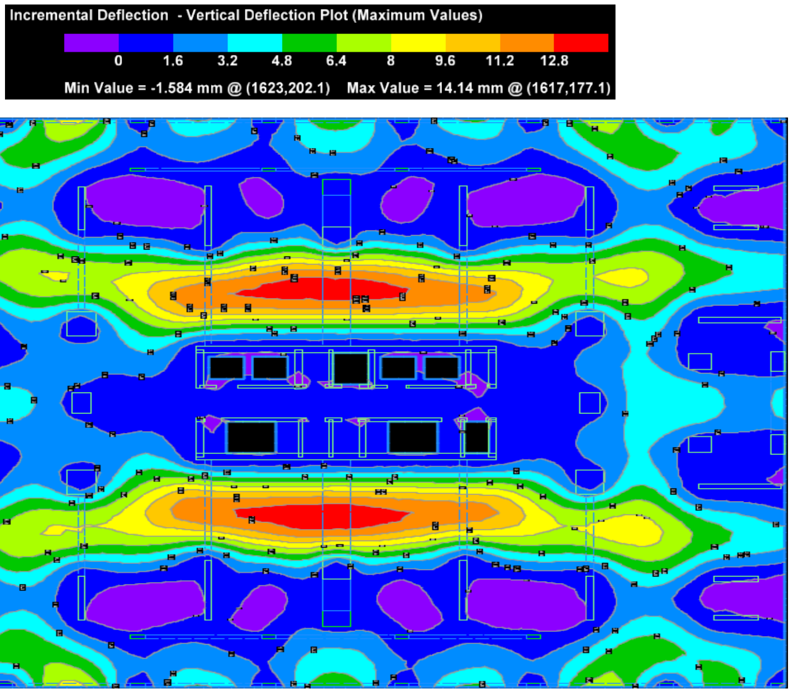


Figure 125: Incremental deflection for the first-floor model (ACI code model)

As shown from the figure 124, the incremental deflection is 14.14 mm, which is less than the limit 15.83 mm, so the slab is safe in the long-term deflcetion checking.

For the last checking which is the design status. Based on it the stresses are checked if they are within the limit or not, and based on figure 125, the design status are all passing and all the stresses are within the limit.

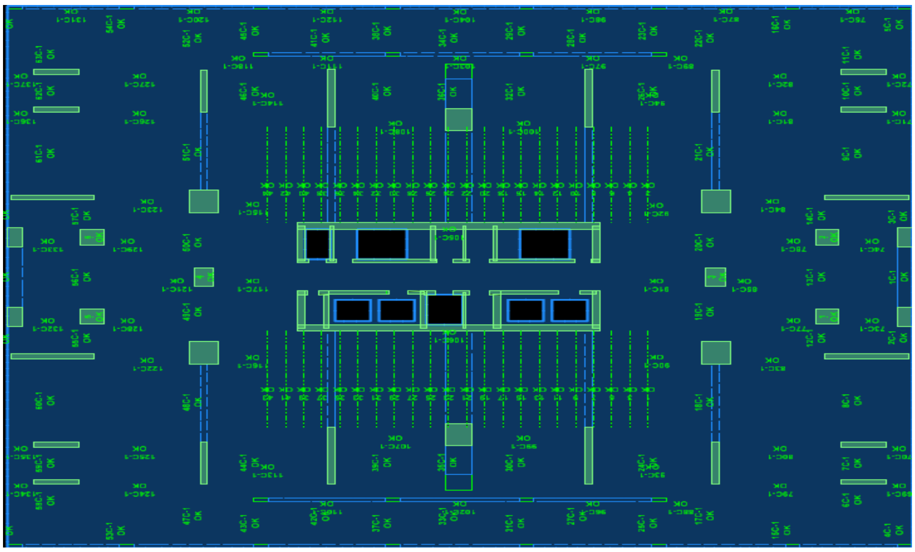


Figure 126: Design status for the first-floor model (ACI code model)

According to all the previuos checking in the post tension slab, the conclusion is that the slab is safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.

2^{nd} Floor to 7^{th} Floor (ACI Code Model)

The following are the outputs for the 2^{nd} to 7^{th} floors from the ACI model, which are considered as office floors. The outputs shows that the slab is safe in all of the checking mentioned in section 4.5.3, which allow for safe reinforcement calculation and post tension rate determination.

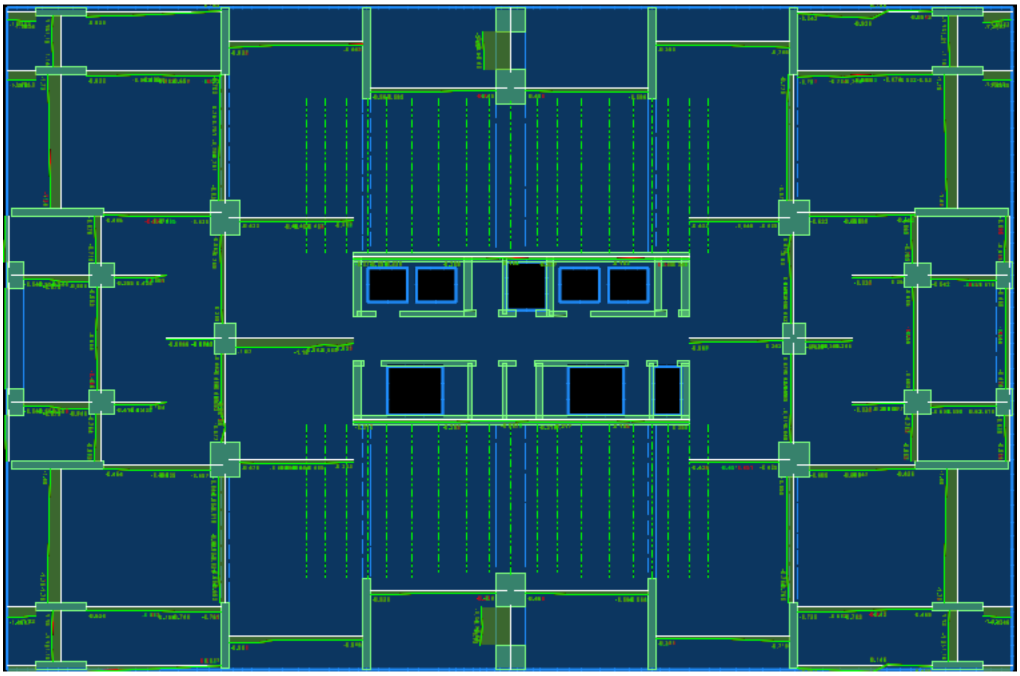


Figure 127: Precompression for 2nd to 7th floors model (ACI code model)

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

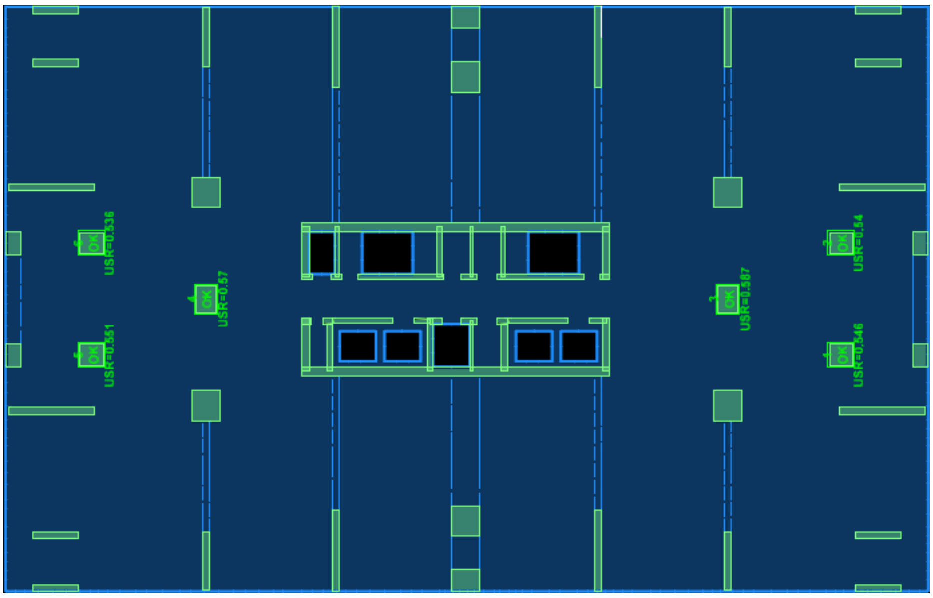


Figure 128: Punching for 2nd to 7th floors model (ACI code model)

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long -term deflection should not exceed L/240, where L is the longest span, while for the incremental defelction the deflection should not exceed L/480. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 31.67 mm, while the limit for the incremental deflection is 15.83 mm.

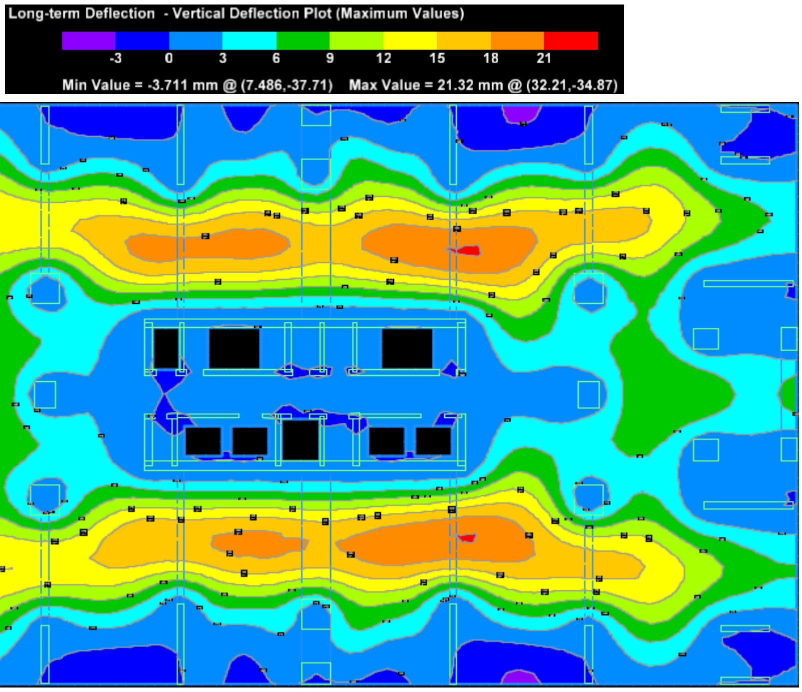


Figure 129: Long-term Deflection for 2nd to 7th floors model (ACI code model)

As shown from the figure 128, the long-term deflection is 21.32 mm, which is less than the limit 31.67 mm, so the slab is safe in the long-term deflcetion checking.

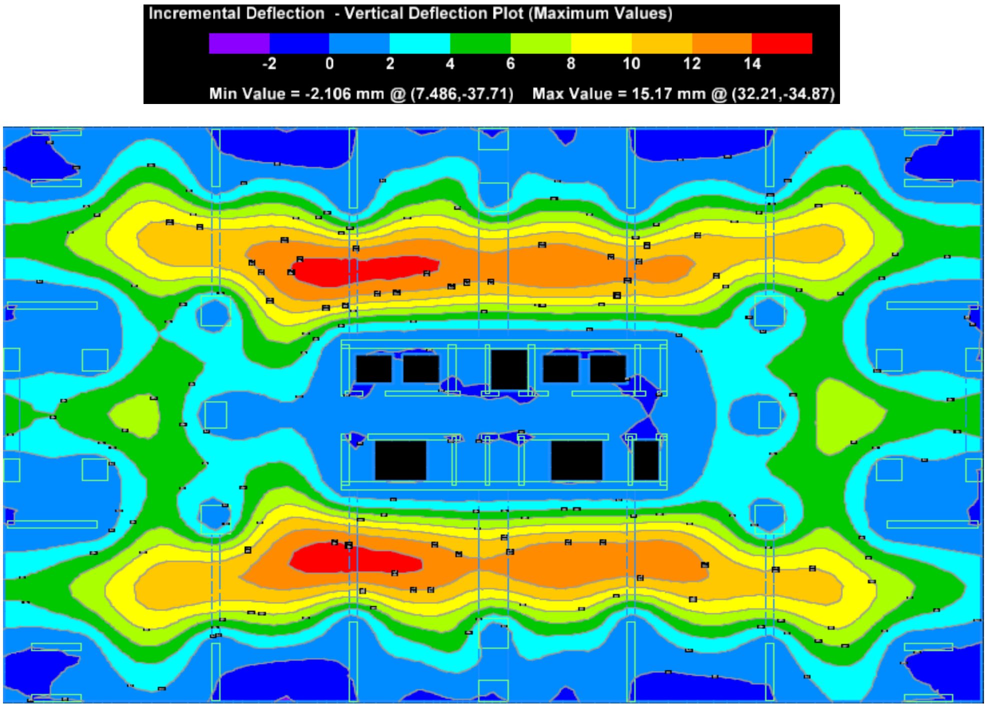


Figure 130: Incremental Deflection for the 2nd to 7th floors model (ACI code model)

As shown from the figure 129, the incremental deflection is 15.17 mm, which is less than the limit 15.83 mm, so the slab is safe in the long-term deflcetion checking.

For the last checking which is the design status. Based on it the stresses are checked if they are within the limit or not, and based on figure 130, the design status are all passing and the stresses are within the limit.

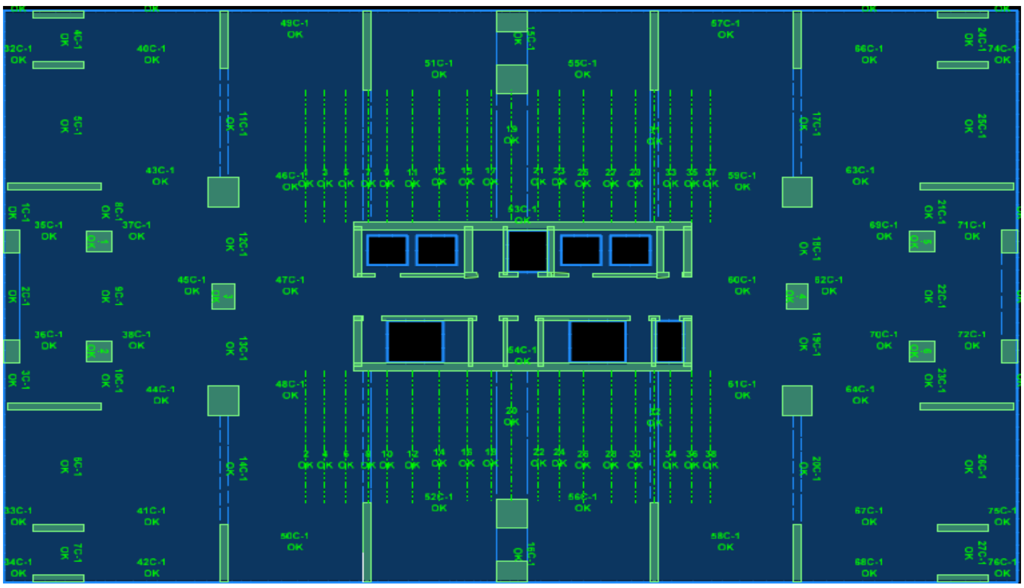


Figure 131: Design Status for the 2nd to 7th floors model (ACI code model)

According to all the previuos checking in the post tension slab, the conclusion is that the slab is safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.

8th Floor to 22nd Floor (ACI Code Model)

The following are the outputs for the 8th to 22nd floors from the ACI model, which are considered as office floors. The outputs shows that the slab is safe in all of the checking mentioned in section 4.5.3, which allow for safe reinforcement calculation and post tension rate determination.

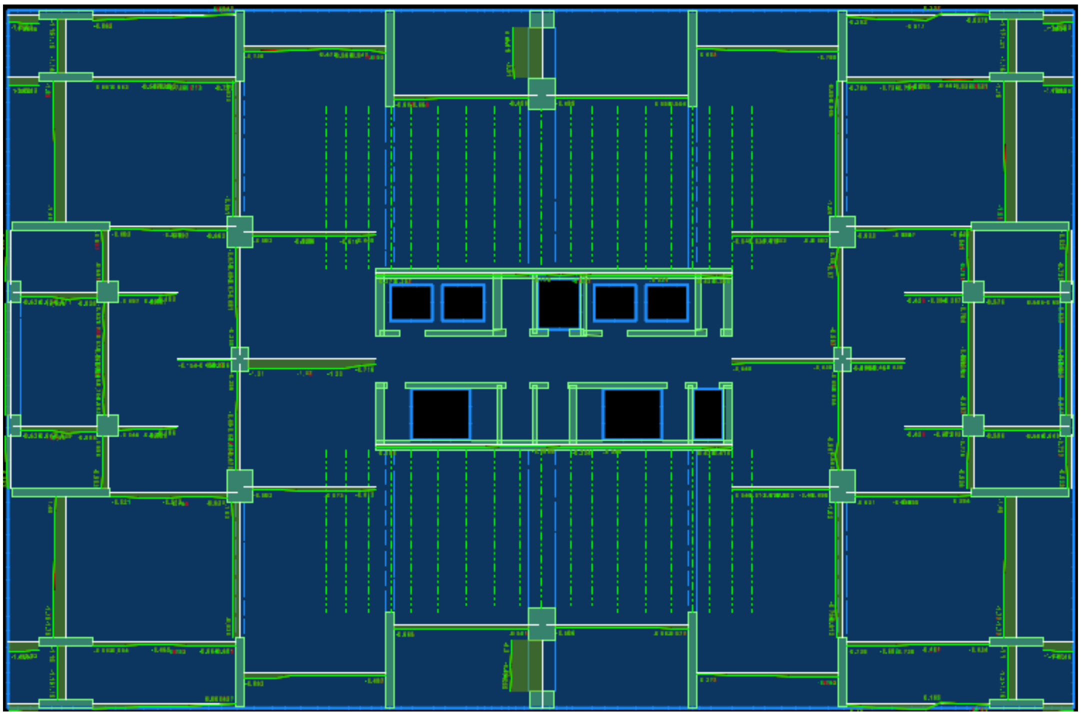


Figure 132: Precompression for 8th to 22nd floors model (ACI code model)

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

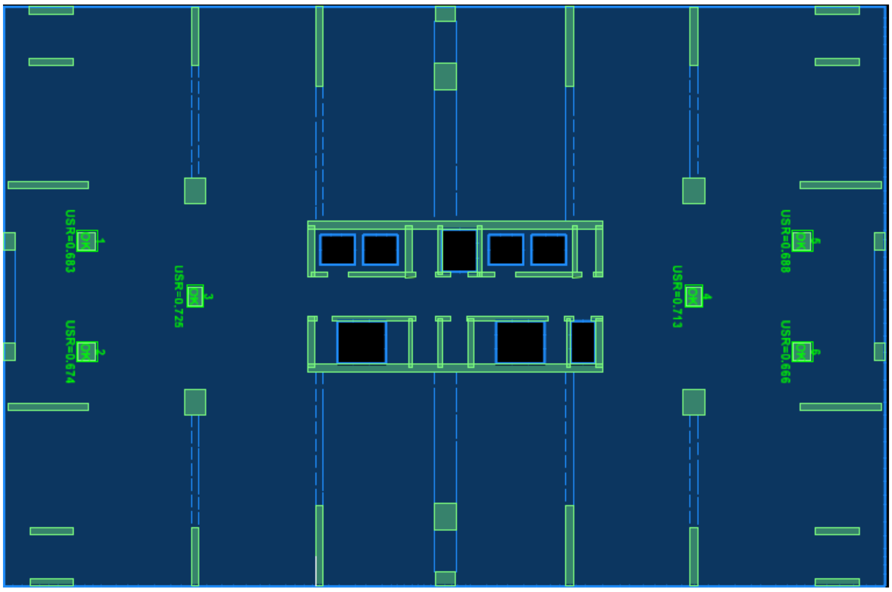


Figure 133: Punching for 8th to 22nd floors model (ACI code model)

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long-term deflection should not exceed L/240, where L is the longest span, while for the incremental defelction the deflection should not exceed L/480. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 31.67 mm, while the limit for the incremental deflection is 15.83 mm.

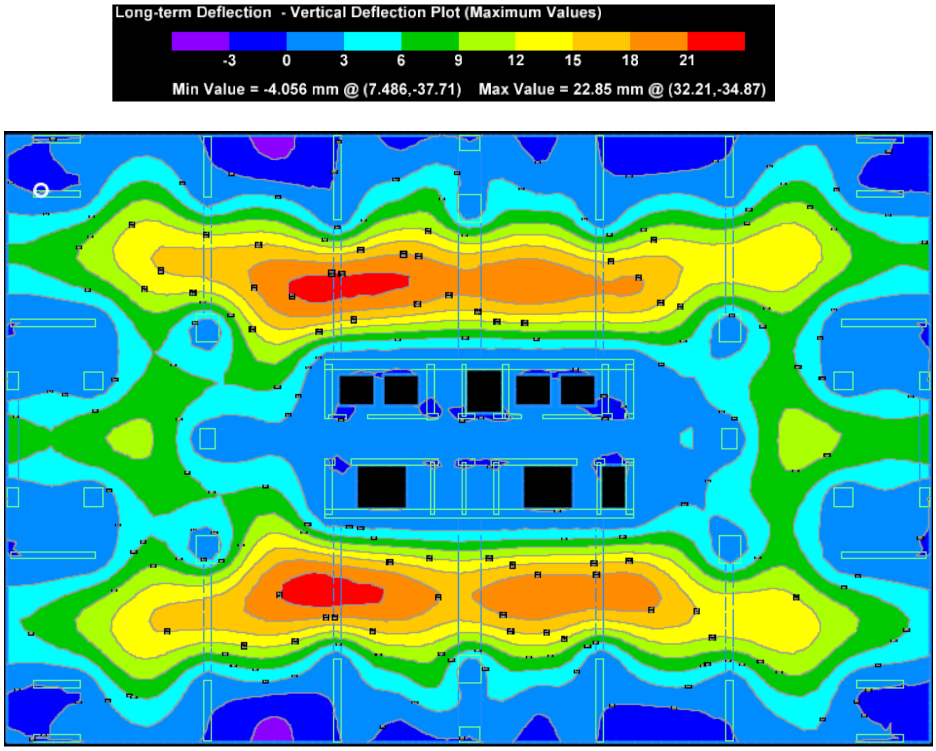


Figure 134: Long-term deflection for 8th to 22nd floors model (ACI code model)

As shown from the figure 133, the long-term deflection is 22.85 mm, which is less than the limit 31.67 mm, so the slab is safe in the long-term deflcetion checking.

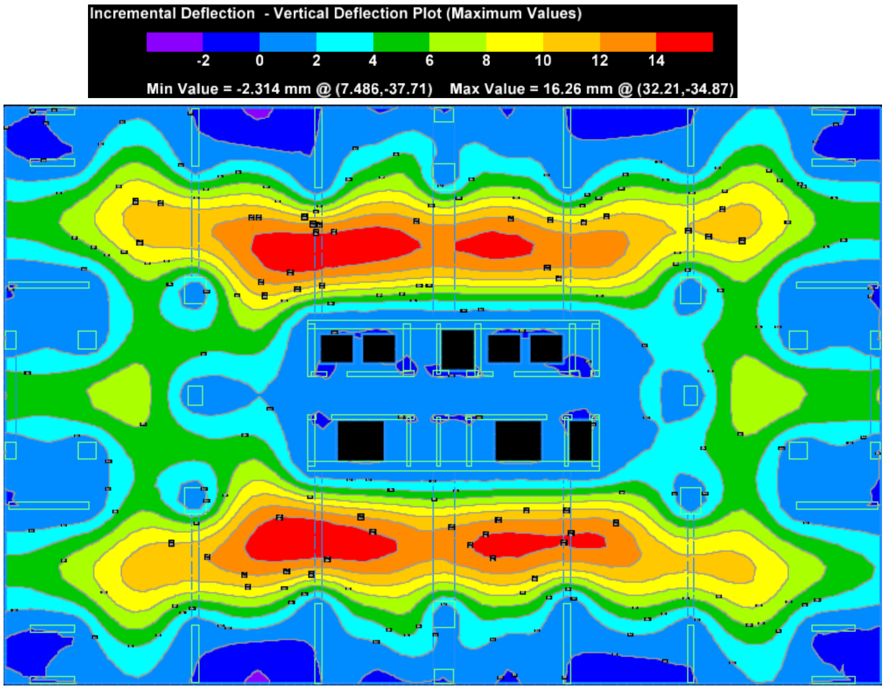


Figure 135: Incremental deflection for the 8th to 22nd floors model (ACI code model)

As shown from the figure 134, the incremental deflection is 16.26 mm, which is less than the limit 15.83 mm, so the slab is safe in the long-term deflcetion checking.

Based on it, the stresses are checked if they are within the limit or not, and based on figure 135, the design status are all passing and the stresses are wihtin the limit.

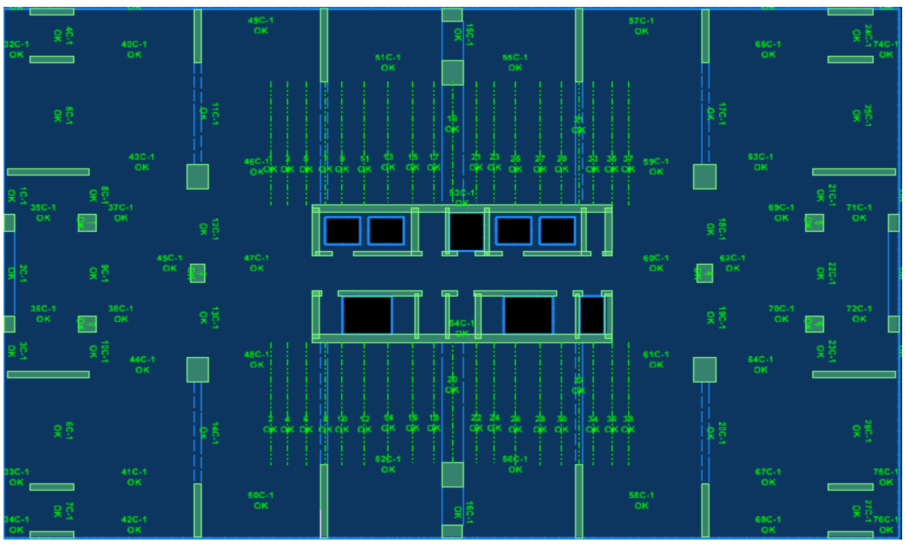


Figure 136: Design Status for the 8th to 22nd floors model (ACI code model)

According to all the previuos checking in the post tension slab, the conclusion is that the slab is safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.

23rd Floor to 42nd Floor (ACI Code Model)

The following are the outputs for the 23rd to 42nd floors from the ACI model, which is considered as residential. The outputs shows that the slab is safe in all of the checking mentioned in section 4.5.3, which allow for safe reinforcement calculation and post tension rate determination.

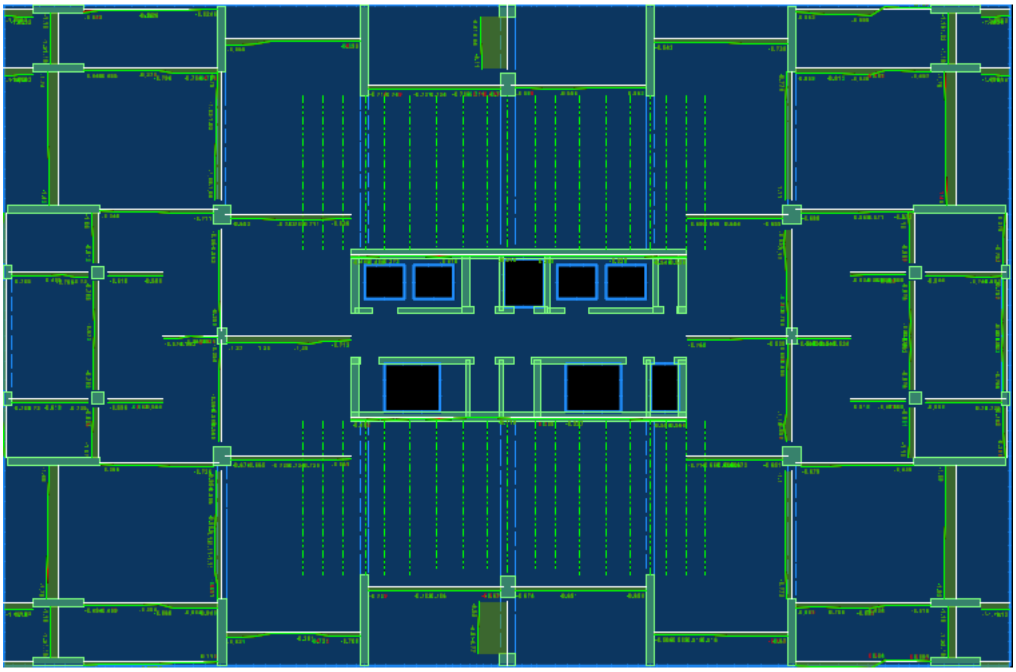


Figure 137: Precompression for 23rd to 42nd floors model (ACI code model)

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

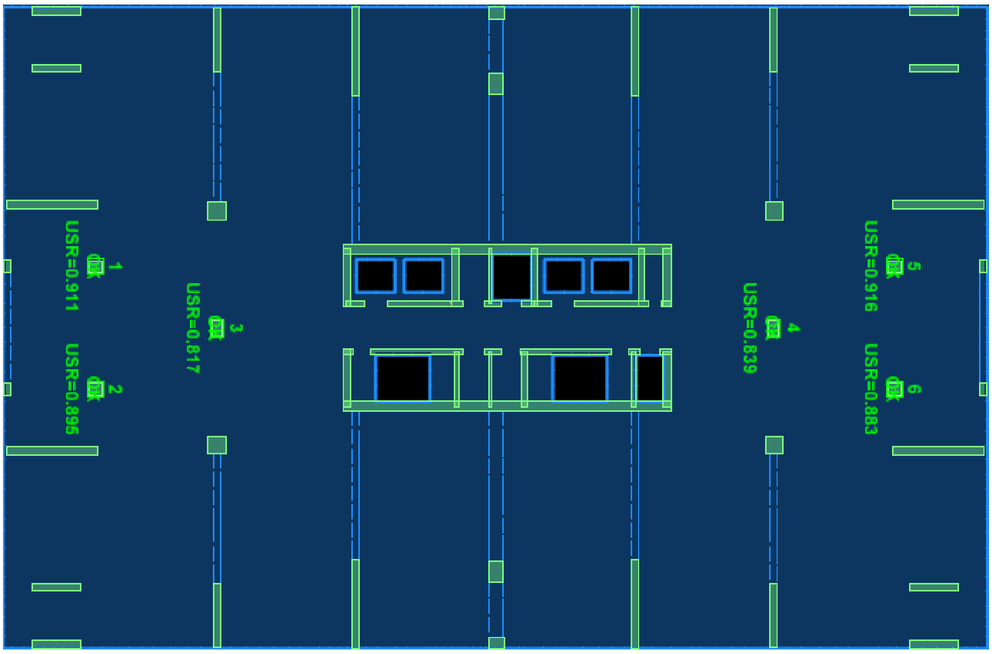


Figure 138: Punching for 23rd to 42nd floors model (ACI code model)

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long-term deflection should not exceed L/240, where L is the longest span, while for the incremental defelction the deflection should not exceed L/480. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 31.67 mm, while the limit for the incremental deflection is 15.83 mm.

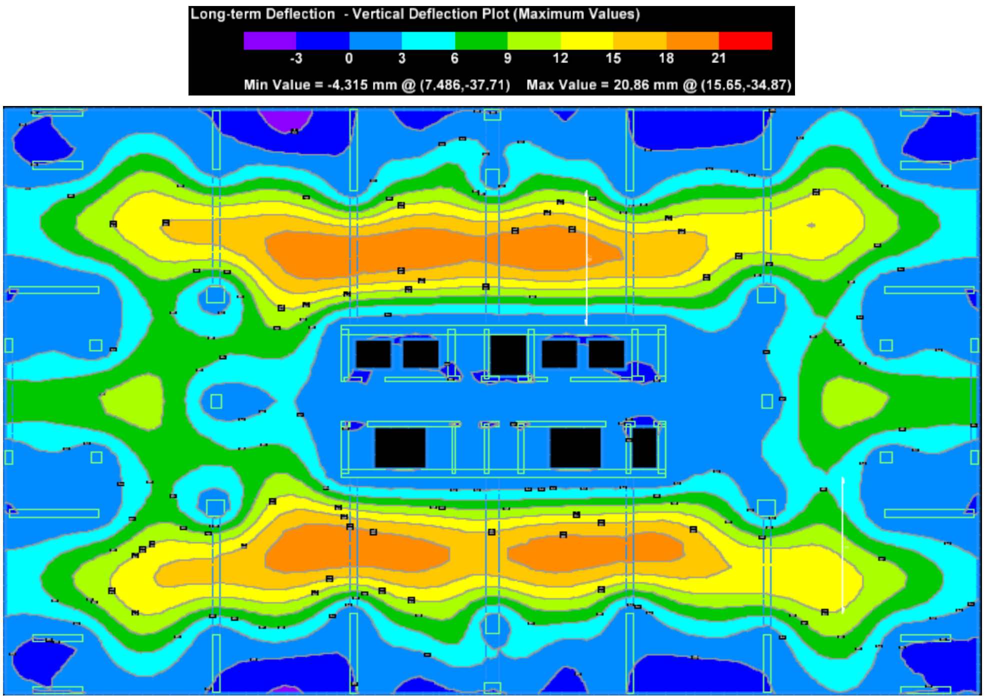


Figure 139: Long-term deflection for 23rd to 42nd floors model (ACI code model)

As shown from the figure 138, the long-term deflection is 20.86mm, which is less than the limit 31.67 mm, so the slab is safe in the long-term deflcetion checking.

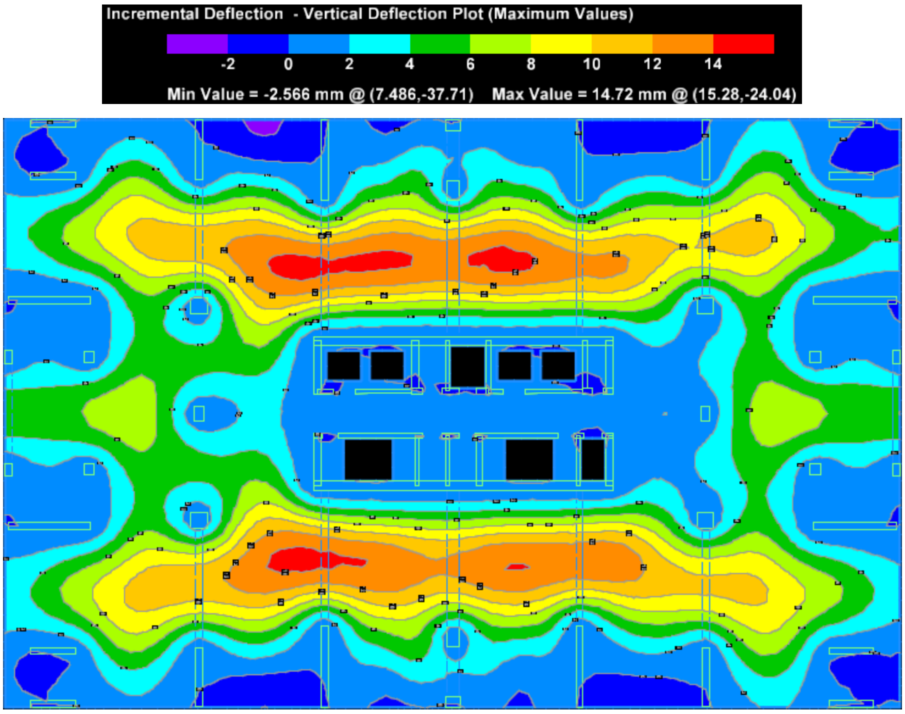


Figure 140: Incremental deflection for the 23rd to 42nd floors model (ACI code model)

As shown from the figure 139, the incremental deflection is 14.72 mm, which is less than the limit 15.83 mm, so the slab is safe in the long-term deflcetion checking.

For the last checking which is the design status. Based on it the stresses are checked if they are within the limit or not, and based on figure 140, the design status are all passing and the stresses are within the limit.

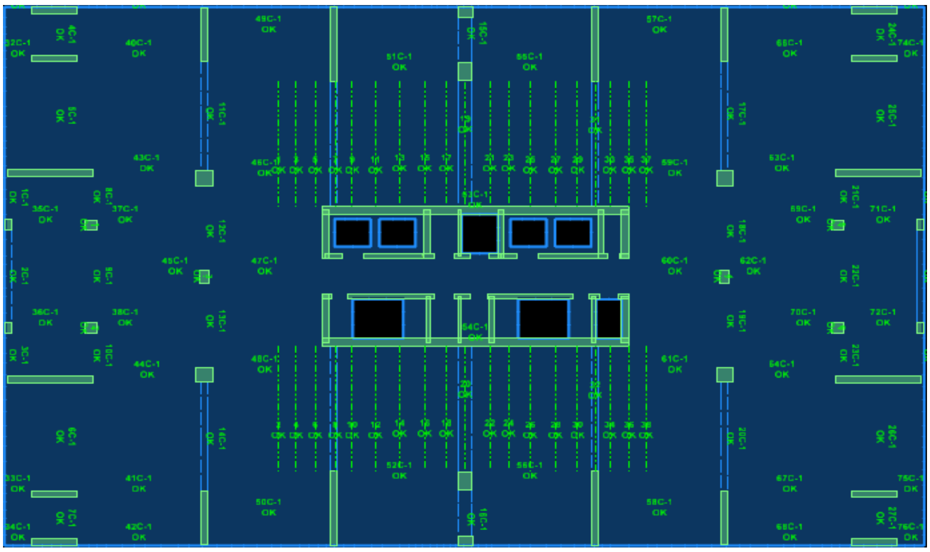


Figure 141: Design status for the 23rd to 42nd floors model (ACI code model)

According to all the previuos checking in the post tension slab, the conclusion is that the slab is safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.

First Floor (Eurocode Model)

The following are the outputs for the first-floor from the Eurocode model, which is considered as an office floor. The outputs shows that the slab is safe in all of the checking mentioned in section 4.5.3, which allow for safe reinforcement calculation and post tension rate determination.

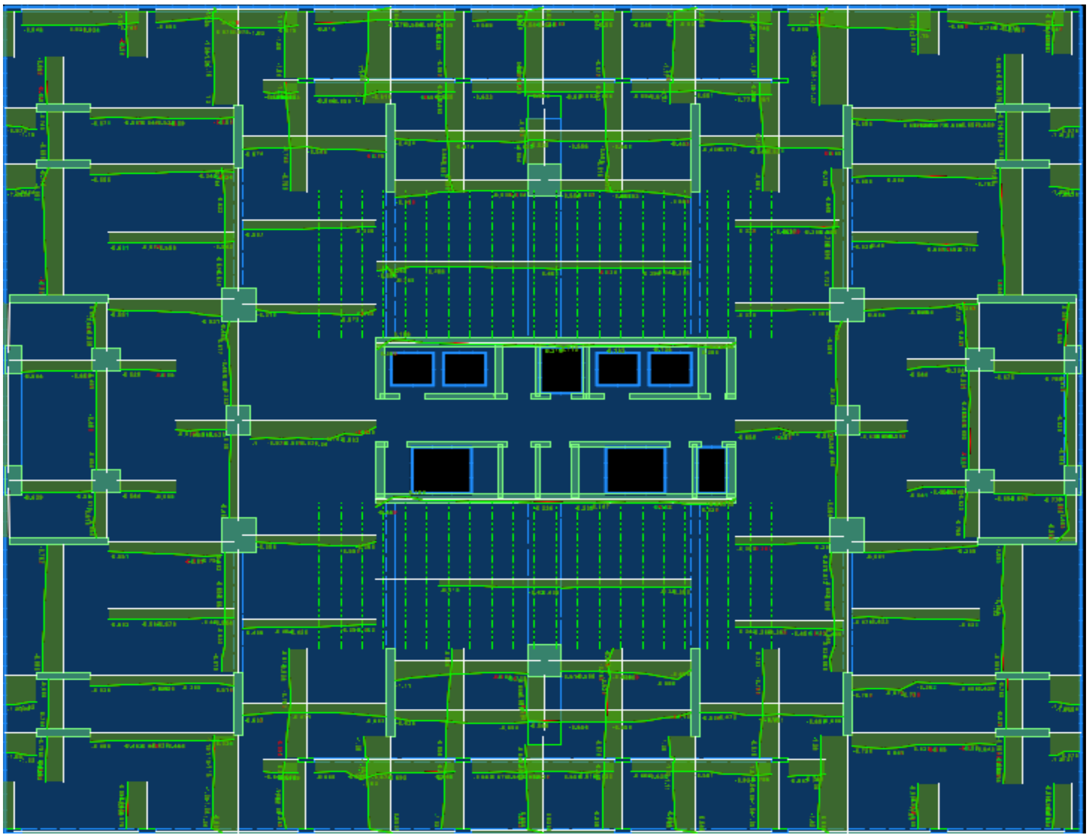


Figure 142: Precompression for the first floor (Eurocode model)

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

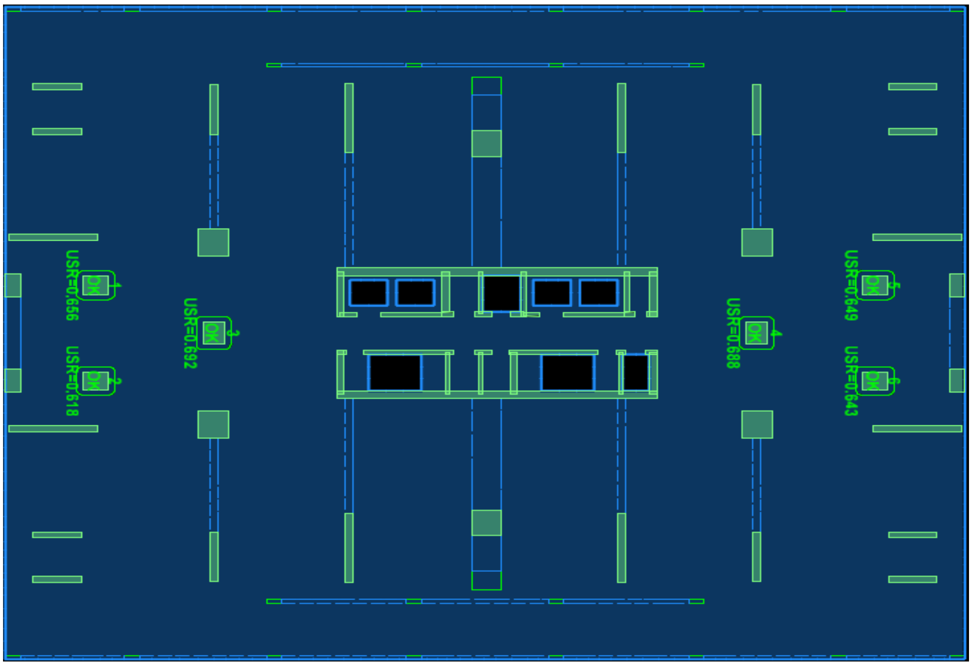


Figure 143: Punching for the First floor (Eurocode model)

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long-term deflection or the maximum deflection under quasi-permanent loads should not exceed L/250, where L is the longest span. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 30.40 mm.

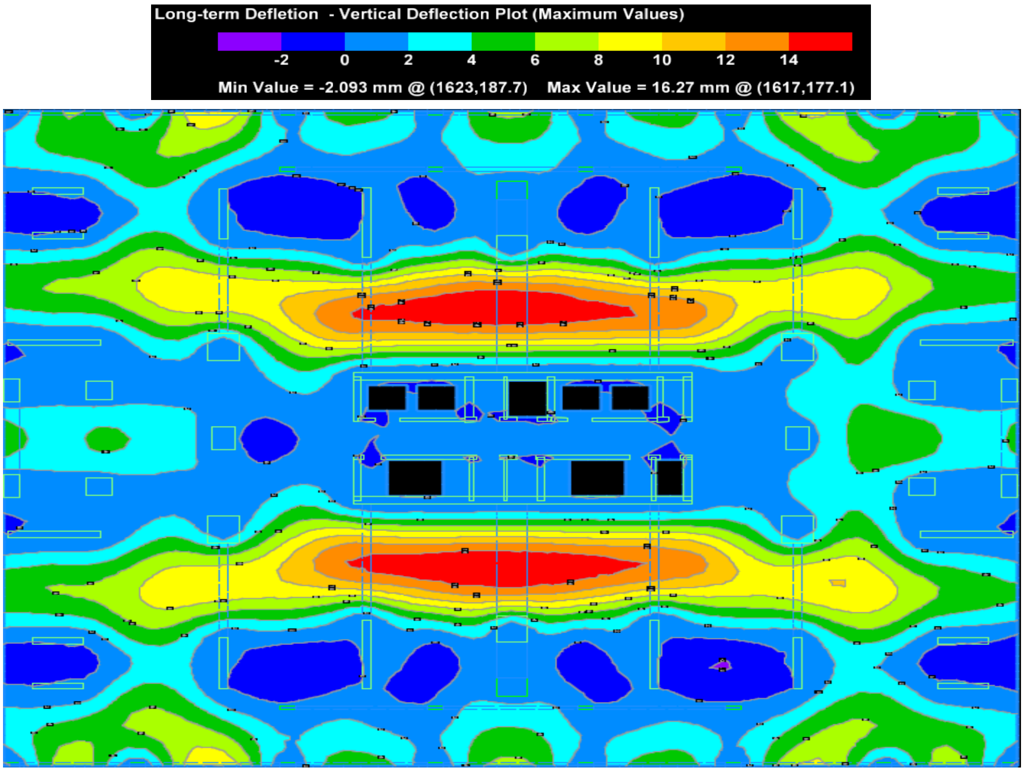


Figure 144: Long-term deflection for the first-floor model (Eurocode model)

As shown from the figure 143, the long-term deflection is 16.27 mm, which is less than the limit 30.40 mm, so the slab is safe in the long-term deflcetion checking.

For the last checking which is the design status. Based on it, the stresses are checked if they are within the limit or not, and based on figure 144, the design status are all passing and the stresses are within the limit.

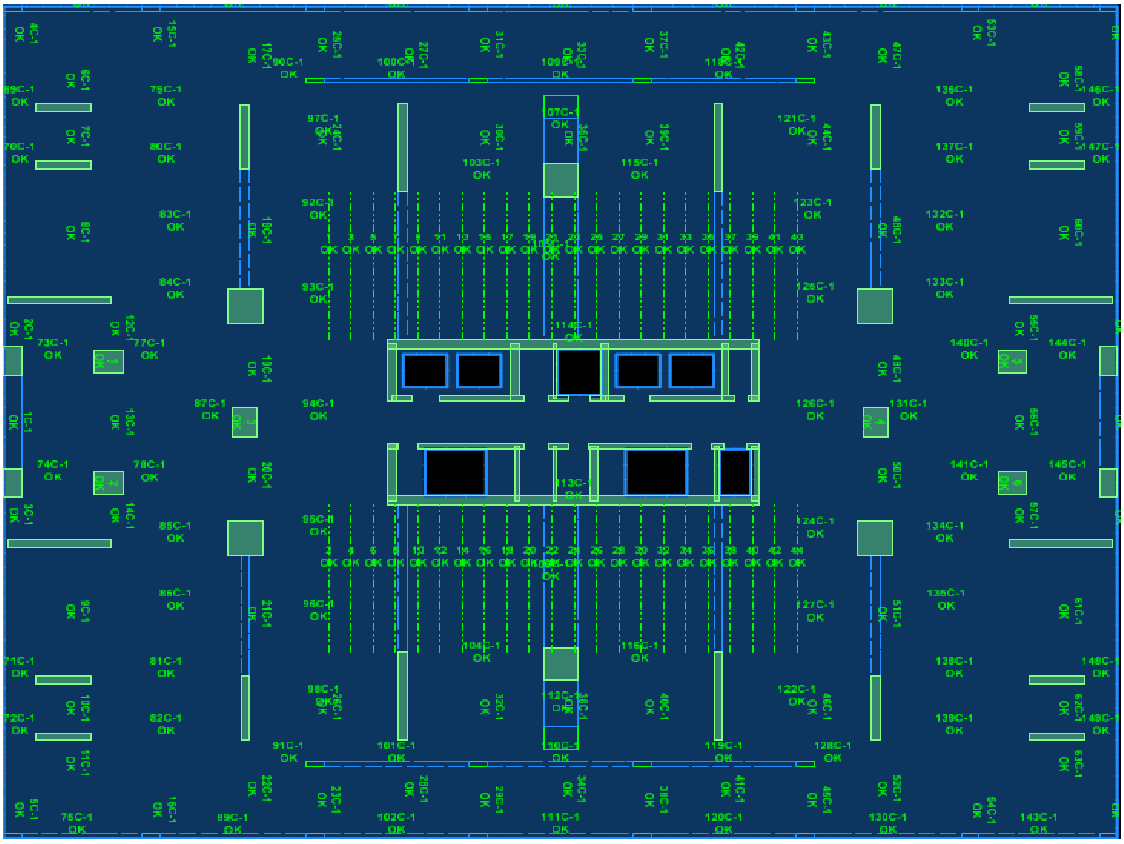


Figure 145:Design status for the first-floor model (Eurocode model)

According to all the previuos checking in the post tension slab, the conclusion is that the slab is safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.

2^{nd} Floor to 7^{th} Floor (Eurocode Model)

The following are the outputs for the floors 2^{nd} to 7^{th} from the Eurocode model, which are considered as office floors. The outputs shows that the slab is safe in all of the checking mentioned in section 4.5.3, which allow for safe reinforcement calculation and post tension rate determination.

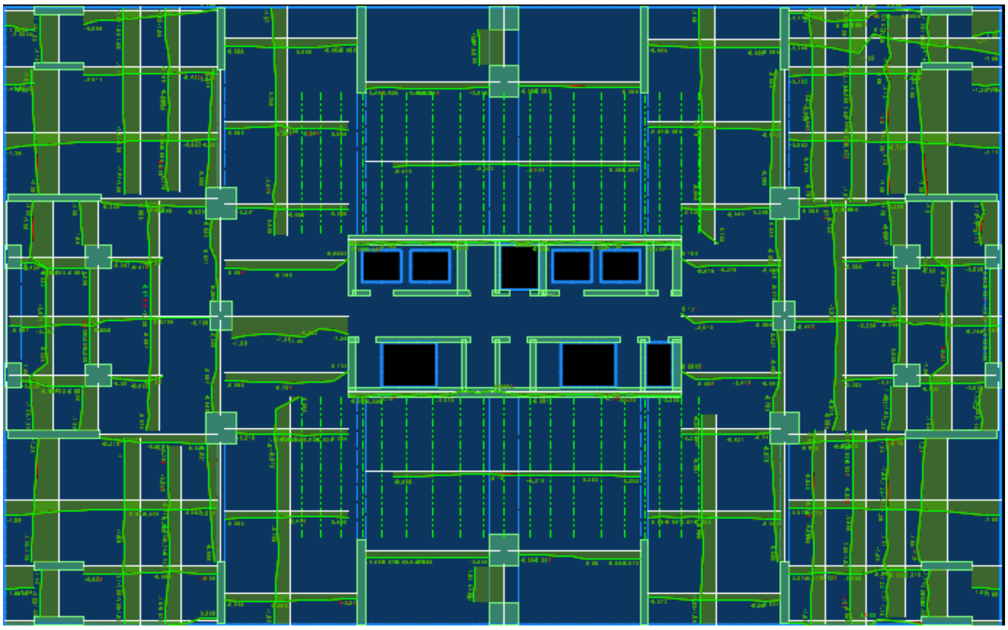


Figure 146: Precompression for 2nd to 7th floors model (Eurocode model)

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

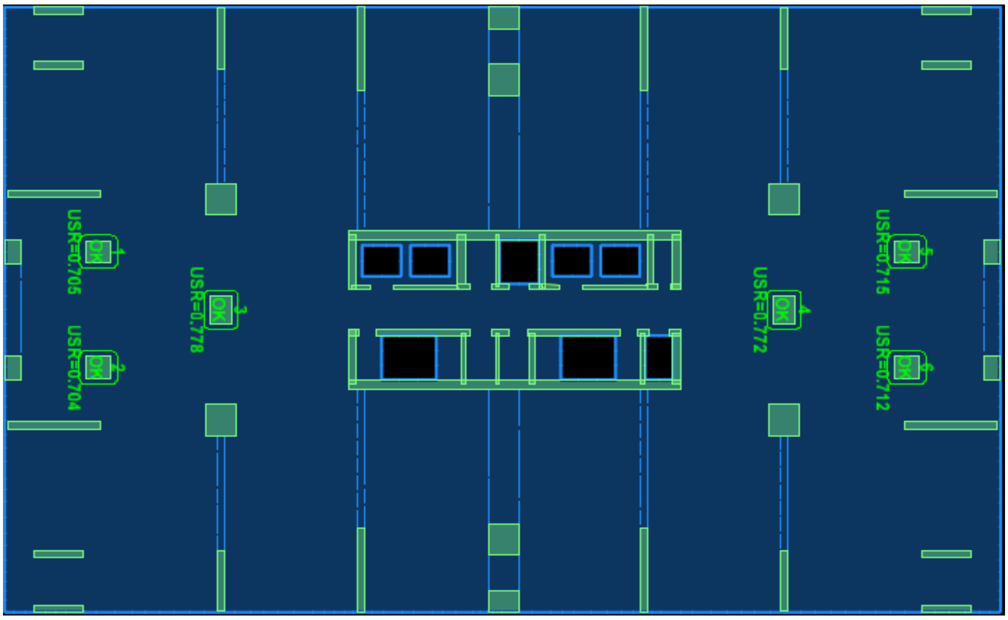


Figure 147: Punching for 2nd to 7th floors model (Eurocode model)

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long-term deflection or the maximum deflection under quasi-permanent loads should not exceed L/250, where L is the longest span. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 30.40 mm.

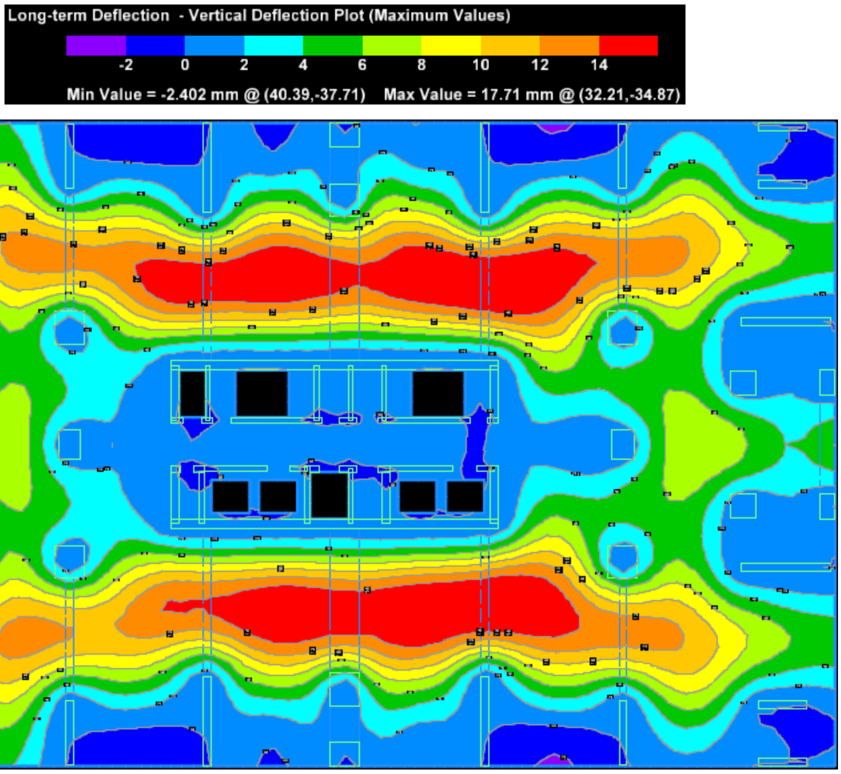


Figure 148: Long-term deflection for 2nd to 7th floors model (Eurocode model)

As shown from the figure 147, the long-term deflection is 17.71 mm, which is less than the limit 30.40 mm, so the slab is safe in the long-term deflcetion checking.

For the last checking which is the design status. Based on it, the stresses are checked if they are within the limit or not, and based on figure 148, the design status are all passing and the stresses are within the limit.

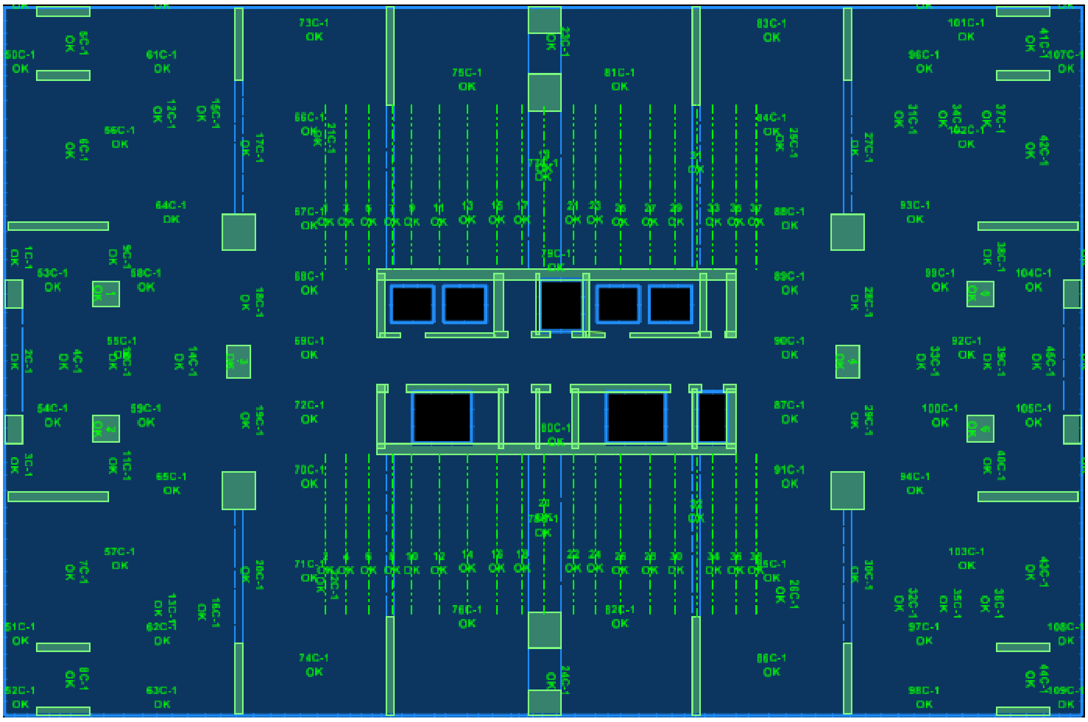


Figure 149: Design status for the 2nd to 7th floors model (Eurocode model)

According to all previuos checking in the post tension slab, the conclusion is that the slabs are safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.

8^{th} Floor to 22^{nd} Floor (Eurocode Model)

The following are the outputs for the floors 8^{th} to 22^{nd} from the Eurocode model, which are considered as office floors. The outputs shows that the slabs are safe in all of the checking mentioned in section 4.5.3, which allow for safe reinforcement calculation and post tension rate determination.

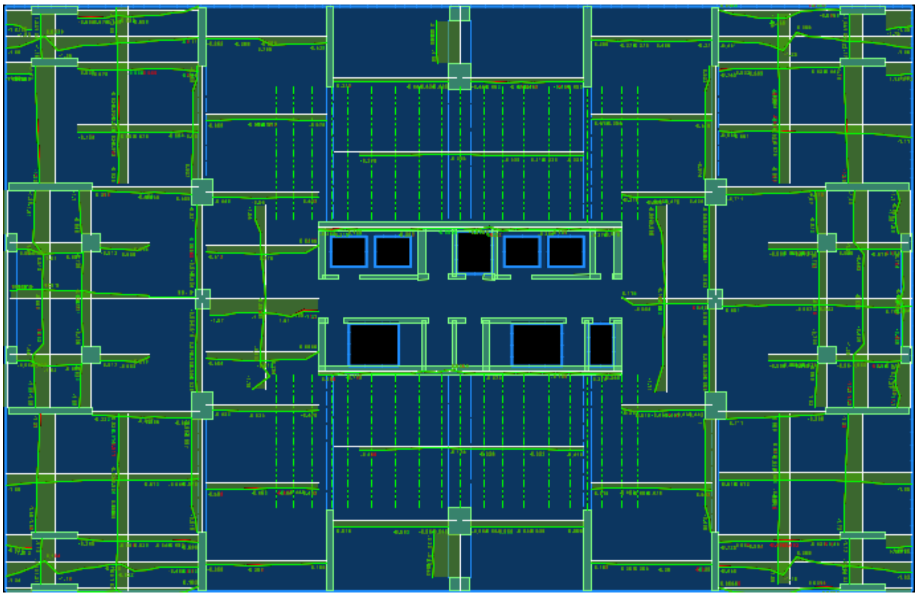


Figure 150: Precompression for 8th to 22nd floors model (Eurocode model)

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

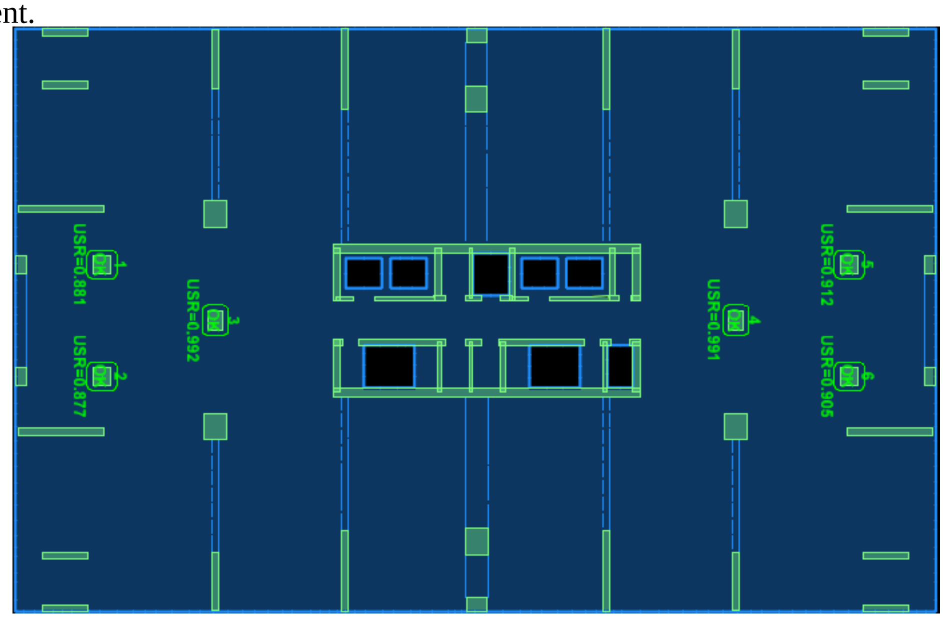


Figure 151: Punching for 8th to 22nd floors model (Eurocode model)

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long -term deflection or the maximum deflection under quasi-permanent loads should not exceed L/250, where L is the longest span. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 30.40 mm.

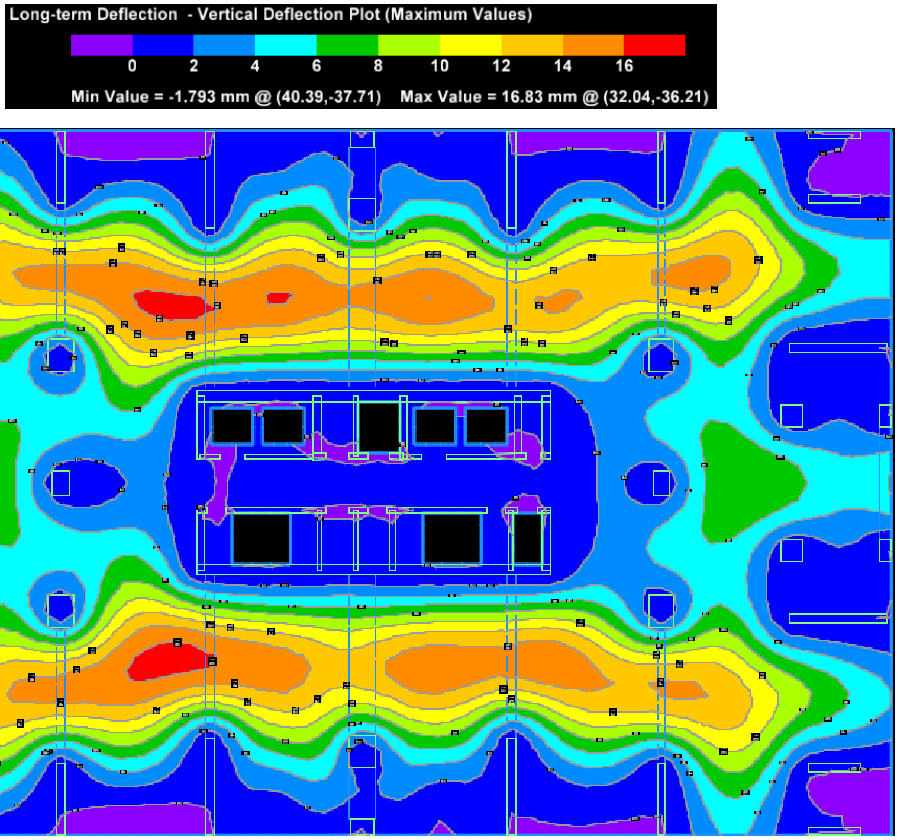


Figure 152: Long-term deflection for 8th to 22nd floors model (Eurocode model)

As shown from the figure 151, the long-term deflection is 16.83 mm, which is less than the limit 30.40 mm, so the slab is safe in the long-term deflcetion checking.

For the last checking which is the design status. Based on it, the stresses are checked if they are within the limit or not, and based on figure 152, the design status are all passing and the stresses are all within the limit.

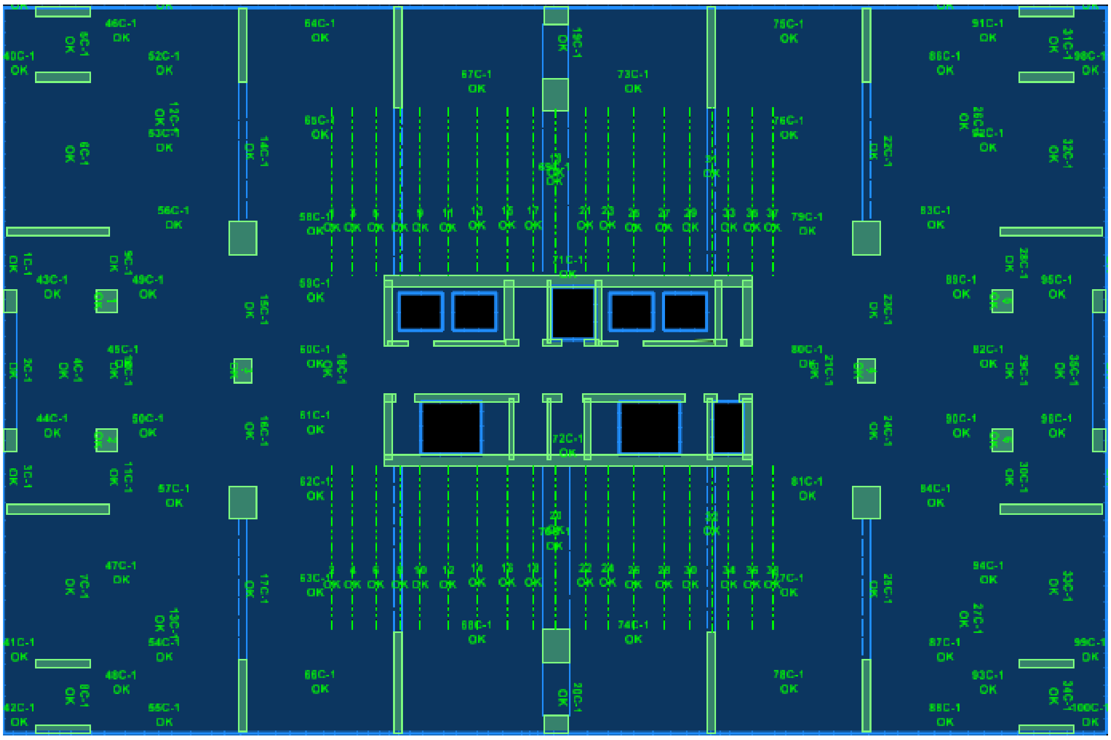


Figure 153: Design Status for the 8th to 22nd floors model (Eurocode model)

According to all the previuos checking in the post tension slab, the conclusion is that the slabs are safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.

<u>**23rd Floor to 42nd Floor (Eurocode Model)**</u>

The following are the outputs for floors 23rd to 42nd from the Eurocode model, which are considered as residential floors. The outputs shows that the slabs are safe in all of the checking mentioned in section 4.5.3, which allow for safe reinforcement calculation and post tension rate determination.

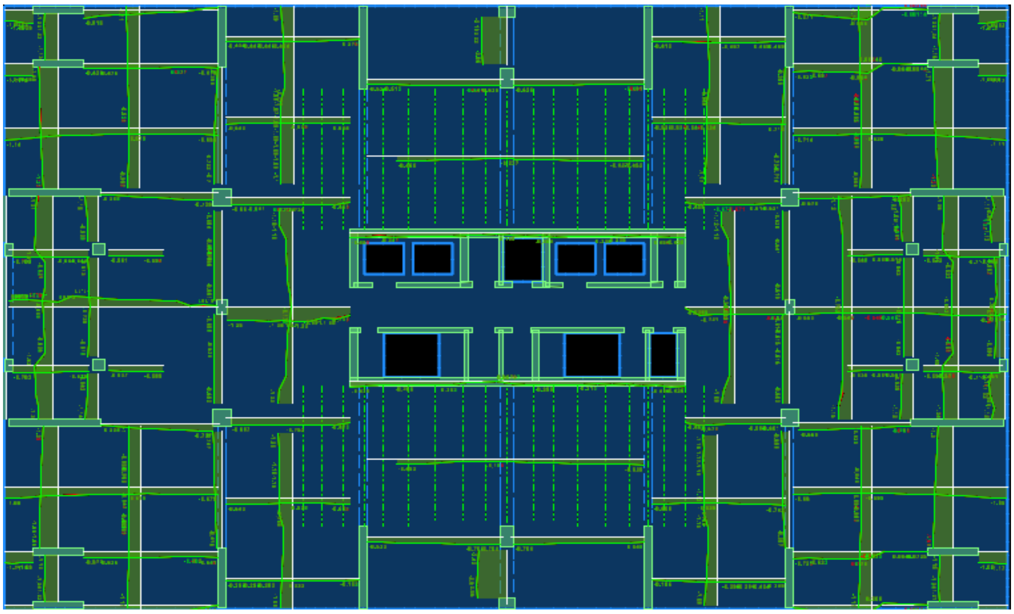


Figure 154: Precompression for 23rd to 42nd floors model (Eurocode model)

The precompression average is between 0.80 Mpa and 0.90 Mpa which follows the code requirement.

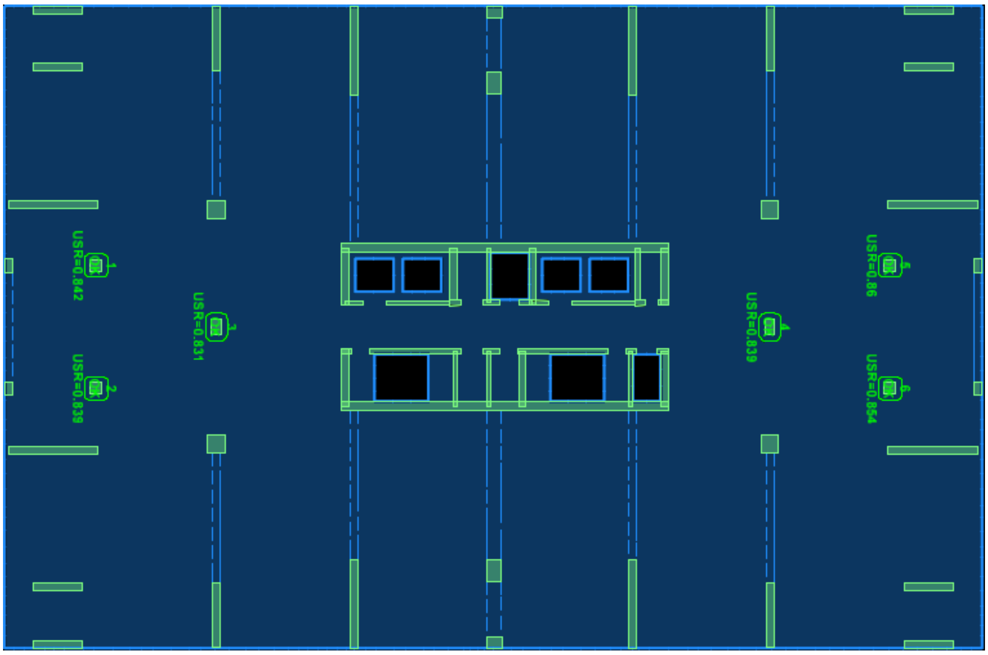


Figure 155: Punching for 23rd to 42nd floors model (Eurocode model)

- As shown in the figure above, the slabs are safe against punching.

As mentioned in the code, the long -term deflection or the maximum deflection under quasi-permanent loads should not exceed L/250, where L is the longest span. The longest span in the slab is 7.60 meters, so the limit for the long-term deflection is 30.40 mm

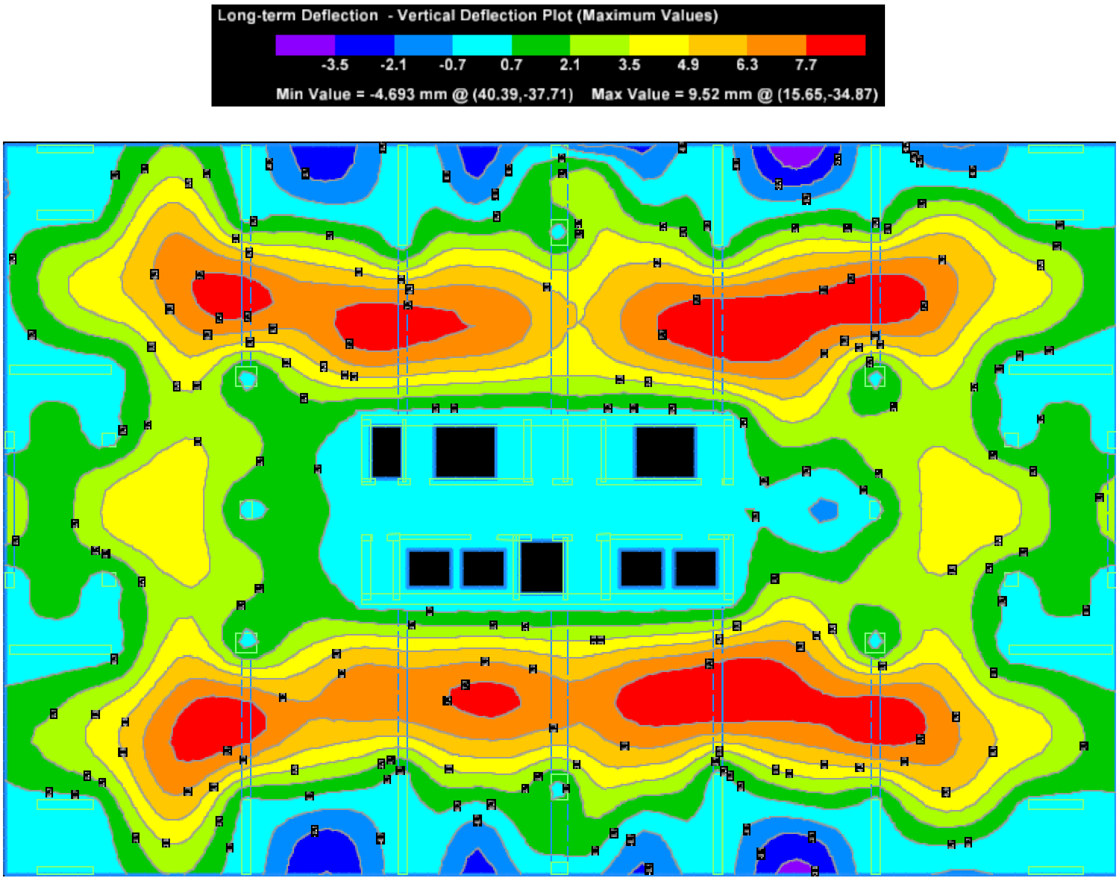


Figure 156: Long-term deflection for 23rd to 42nd floors model (Eurocode model)

As shown from the figure 155, the long-term deflection is 9.52 mm, which is less than the limit 31.67 mm, so the slab is safe in the long-term deflcetion checking.

For the last checking which is the design status. Based on it, the stresses are checked if they are within the limit or not, and based on figure 156, the design status are all passing and all the stresses are within the limit.

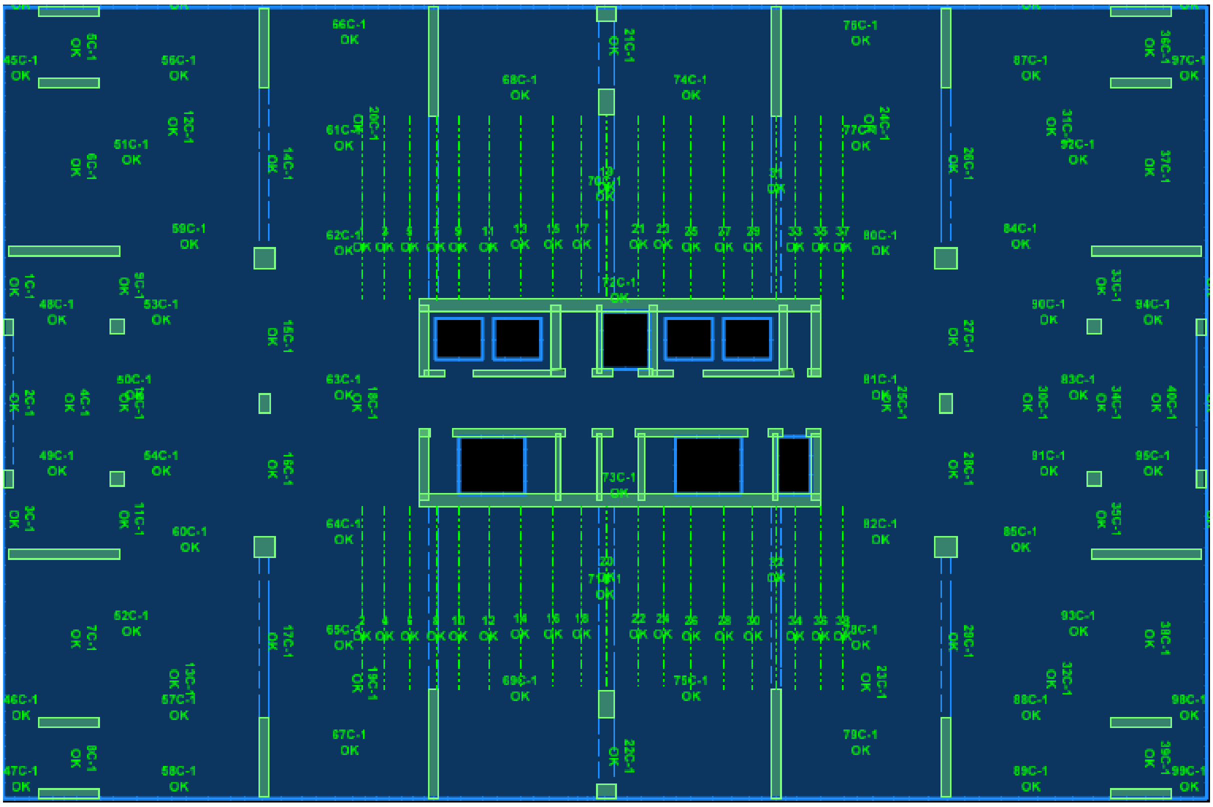


Figure 157: Design status for the 23rd to 42nd floors model (Eurocode model)

According to all the previuos checking in the post tension slab, the conclusion is that the slabs are safe, which means that the the amount of reinforcemnt can be calculated and the post tension rate can be taken directly from the model.