An-Najah National University

Faculty of Graduate Studies

EFFECT OF THE BUILDING ASPECT RATIO ON RESPONSE MODIFICATION FACTOR FOR REINFORCED CONCRETE MOMENT RESISTING FRAMES

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Dedication

الى من قاد قلوب البشرية وعقولهم الى مرفأ الامان,معلمنا الاول محمد صلى الله عليه وسلم

الى والدتي ووالدي الغاليين

الى من كان ظلي حين يلفحني التعب زوجي العزيز

الى بذرة الفؤاد وأمل الغد, ابنتي الحبيبة رينا

الى اخوتي واختي مصدر فخري وابنائهم وبناتهم

الى والد ووالدة زوجي واهله الاعزاء الذين هم بمثابة اهلي

الى من ربطني بهم عطر الصداقة وورد المحبة

الى كل يد وقلب سار معي درب الانجاز لاكون

الى كل هؤلاء اهدي هذه الرسالة.....راجيا من الله تعالى ان تكون نافذة علم, وبطاقة معرفة,,,, وان ينفعنا وينفع بنا

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اشكر الله رب العالمين الذي خلق وهدى وسدد الخطى ,فخرج هذا العمل بعونه وتوفيقه نحمده حمدا كثيرا في المبتدى والمنتهى

وبعد..

انطلاقا من قوله تعالى " ومن شكر فانما يشكر لنفسه " (النمل 40)

ومن قوله صلى الله عليه وسلم " من لم يشكر الناس لم يشكر الله عز وجل"

فانني اتقدم بالشكر الجزيل والعرفان بالجميل لكل من مد يد العون والمساعدة , وفي مقدمتهم الدكتور الفاضل منذر دويكات والدكتور الفاضل محمد سماعنة اللذان تشرفت باشرافهما على هذا البحث, وكانت لملاحظاتهما القيمة وتوجيهاتهما السديده وأخلاقهما الطيبة ومعاملتهما الكريمة اثر كبير في وصول هذا البحث الى هذه الصورة فلهما عظيم شكري وتقديري وجزاهما الله عني خير الجزاء.

أبي الحبيب والدتي الغالية أخوتي واختي --- لا يمكن أن أنسى دعمكم لي وما قدمتموه من أجلي فلكم مني كل الحب، ومهما قلت في حقكم من كلمات الشكر فإنني لن أمنحكم ما تستحقونه.

واتقدم بكل الشكر الى زوجي الغالي الذي كان ظلي وأماني وسندي في كل لحظة من لحظات التعب والدراسة وكان مصدر الامل والتفاؤل .

Declaration

I, the undersigned, declare that I submitted the thesis entitled:

EFFECT OF THE BUILDING ASPECT RATIO ON RESPONSE MODIFICATION FACTOR FOR REINFORCED CONCRETE MOMENT RESISTING FRAMES

I declare that the work provided in this thesis, unless otherwise referenced, is the researcher's own work, and has not been submitted elsewhere for any other degree or qualification.

Student's Name: Dana Issam Abdul Kareem Nassar

Signature: Dana Issam

Date: 2-7-2023

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EFFECT OF THE BUILDING ASPECT RATIO ON RESPONSE MODIFICATION FACTOR FOR REINFORCED CONCRETE MOMENT RESISTING FRAMES

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Abstract

The response modification factor (R) is one of the essential factors in seismic design and is used to define the nonlinear behavior of buildings during an earthquake. International codes such as IBC 2016 provide a constant value for “R” for a particular structural system. However, the value of "R" may change based on parameters including zone factor, soil site class, aspect ratio of the frame, column orientation, and type of slabs. This study investigates the effect of building aspect ratio on the value of "R". In addition, it checks if the values of the R factor recommended by the IBC-2016 code are conservative. A single cycle for calculating the R factor is applied by pushover analysis (nonlinear response analysis) to achieve this goal. It was applied to many 3-D intermediate moment-resisting frame systems with different aspect ratios as the key parameter and different seismic zone factors and sites as secondary parameters to determine the behavior of the frame with increasing lateral force until collapse. The researcher selected 30 cases with different aspect ratios, zone factors, site classes, and building plans. Then, the ETABS2016 program is used to design and analyze (elastic analysis) the 3-D frames as per previous code requirements. Finite element software (ABAQUS) performs the nonlinear analysis via incremental Elasto-Plastic analysis. The reinforced concrete elements are presented in the program as line elements. Afterwards, the Xtract program is used to evaluate the inelastic properties of the sections fed to ABAQUS for the frame analysis. The obtained results of "R" are compared to the values suggested by international codes (IBC 2016). The results demonstrate an influence of the aspect ratio, zone factor, and site class on the R factor. An equation was derived to calculate the R factor through the aspect ratio of the building and the seismicity of the region. Consequently, the computed values for the cases depending on the pushover curve are larger than or equal to those recommended by the IBC2016 code. So, the international code is conservative, in term of the total base shear, force because it considers the period of the building in the calculation of the R factor. However, further ductility investigation is needed as the required R values are higher than code ones.

Keywords: Nonlinear static analysis; ABAQUS program; response modification factor

Chapter one

Introduction and Theoretical Background

1.1 Motivation

Research on natural phenomena has a long tradition. Earthquakes are a fundamental geological natural phenomena. Their occurrence in the future depends on the prediction science. Earthquake is a method of relieving the stress on the earth's surface. In fact, it occurs due to the tectonic plate movement that makes up the Earth's crust. These movements cause a sudden amount of energy release that produces seismic waves, which cause the Earth's surface to vibrate. Earthquake consequences can be seen as human death, injuries, property collapses, and other damages.

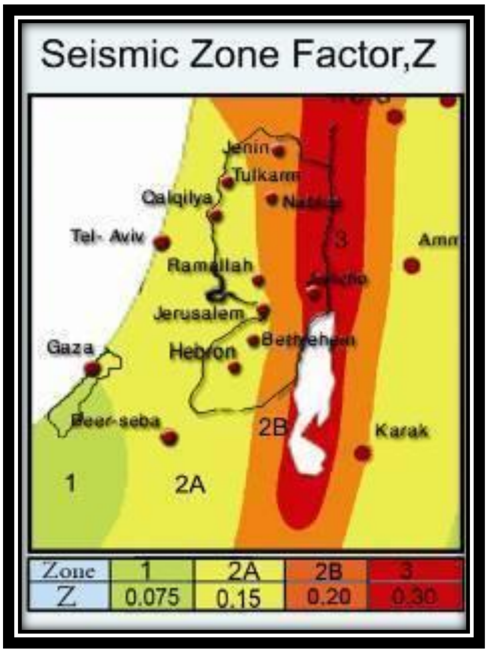
In common practice, some buildings are designed to resist gravity loads only and cannot resist lateral loads such as earthquakes. Thus, the design of buildings must be sufficient to resist the earthquake.

Recent theoretical developments have revealed that engineers use reinforced concrete frame systems in designing buildings in Palestine. The design principles applied to these frames are generally force- or displacement-based. The displacement-based design is more realistic and economical than the others [11,15].

The seismic design of buildings is based on peak ground acceleration maps (PGA maps), such as the seismic zone map in Palestine illustrated in Figure 1.1. Considering the region's historical seismicity, this map is essential in designing the facilities to resist expected earthquakes.

Figure 1.1

Seismic zone map



(ASCE -16)

In the beginning, the seismic design of buildings was based on the elastic force of the buildings. This made the dimension of the members huge and led to an increase in prices. The subsequent response modification factor "R" is found. This is one of the most critical factors in seismic design, especially in the equivalent static force method. This factor is used to reduce the linear seismic force and reduce the acceleration of the building when exposed to earthquakes by integrating the plastic energy dissipation capacity of the structure into the design process. A higher "R" value indicates a greater ability of the structure to dissipate energy in the plastic phase. Moreover, using "R" reduces the design sections of the structural elements.

Response modification factors depend on many parameters, such as the building's ductility, overstrength, redundancy, and damping. A direct relationship exists between "R" and these parameters .[15]

IBC code (IBC-2016) recommended using R values depending on the type of building system and seismic design category. It is affected by the height and period of the buildings, irregularity in plans, irregularity in the elevations, seismic location of the building, and the site class of the soil. [5]

To obtain an accurate value of the response modification factor, the aspect ratio of the building and the seismicity of the region must be considered. This will help to verify the precise values for the building by comparing them to those specified in the IBC 2016 code. [1]

1.2 Problem statement

The international code recommends using the symbol 'R' factor to represent the building's ductility, overstrength, and behavior. This symbol has different values depending on the type of structural system. It represents the behavior of the building under seismic conditions. The value of the R factor has various values from one code to another. For example, the R factor for intermediate reinforced concrete moment-resisting frames is 5 in IBC-2016 and 6 in the Pakistan code [IBC-2016, NESPAK-2006]. These variations depend on the seismicity of the region.

The following factors will be investigated for their impact on the seismic design and behavior of buildings:

1. Aspect ratio: The percentage of the height of buildings to their width.
2. Site class: Change the type of soil.
3. Zone factor: The nature of the seismicity region.

Based on the short review above, this study has become necessary because of the developments in structures worldwide. In addition, international codes such as (IBC2016) do not provide a guideline on the impact of this development on the value of "R," which is particularly important in seismic design. Thus, this study comes as a first step to verify the values of the "R" given by the IBC 2016 code under the influence of some parameters with a focus on the aspect ratio of the buildings. By the same token, this study focuses mainly on this topic because of the limited number of published papers on the effect of aspect ratio on the value of the response modification factor.

1.3 Objectives

The main objective of this study is to apply a single calculation cycle for the R factor by pushover analysis to verify the R-value suggested in codes (IBC2016) and other codes if they are applicable for different vertical aspect ratios of the buildings, different seismic region and change the type of soil and seismic design category. In addition, an assessment of the behaviour of structures with previous parameters under earthquake loads establishes a relationship between the aspect ratio, the region's seismicity, and the R factor.

To achieve the main goals, the following sub-objectives are needed:

1. Perform a pushover analysis for many cases to study the behaviour of the structures under earthquake, considering the change of vertical aspect ratio for the buildings and seismicity of the region.
2. Calculate the accurate R-value for these cases as recommended in the ASCE2016 code, considering the parameters' effects, and compare it with the values specified in international codes such as IBC2016.
3. Study the effect of different zone factors type of soils (site class) on R factor.

By applying these sub-objectives, the main goals of this study will be achieved, validating the R factor suggested by the IBC-2016 code.

1.4 Structure of thesis

This thesis contains the following chapters:

- Introduction and theoretical background: include a simple introduction to the study and the problem statement will be clarified, what are the main objectives and emphasize the importance of this study in design practices. Clarification of response modification factor and its components and the effect of this factor in seismic design, what it represents and how it can be obtained, the method of pushover analysis will be explained, the relationship between "R" and push curve and how "R "can be calculated through the push curve. The literature review contains reading and collecting data from previous studies most relevant to this study. Also, what is related to the topic will be summarized briefly.
- Modeling: Case studies chosen will be designed as recommended by the IBC2012 code and analyzed using the commercial software program (ABAQUS) to obtain

push-over curves for these cases in order to calculate the value of "R" for RC-IMRF. Also, the input file method to represent the reinforced concrete element as a line element will be clarified in detail. The designing and analyzing of cases will be applied one time as a single cycle.

- Results: The results obtained will be clarified and compared with international codes. The effect of the aspect ratio, variation of the seismic zone, and variation of a soil type. The seismic design category on the results will be discussed and verified whether the results are safe or not.
- Conclusion and recommendations: Summarize the results for values for R and some recommendations to use in the future.

Chapter Two

Response modification factor (R)and pushover analysis

2.1 Response modification factor

2.1.1 Overview

This section presents a review of recent literature on the thesis's topic. The traditional seismic design in the buildings relies on elastic force by equivalent lateral force procedure (ELF). In this procedure, the elastic base shear is computed for designing the building based on the estimated period of the building, site-specific ground acceleration and response spectrum curve, site class of the site and type of building system used to resist the lateral forces, and the distribution of lateral forces over the height of the building. Some authors have also suggested that static analysis of the building for these forces provides the design forces, including shears and overturning moments for the various stories, with some codes permitting reductions in the statically computed overturning moments .[9]

Prior research suggests that the static force procedure becomes useless since it gives very large sections and uneconomic members. Hence, dynamic analysis is required for buildings with nonlinear behaviour under seismic load. Therefore, design codes such as (IBC 2016) and others allow the reduction of the elastic design force to a realistic inelastic force by using the response modification factor.

Figure (A.1) in Appendix A shows the elastic and inelastic behavior of the structures. When the building behaves elastically up to the failure point, the induced elastic force design (V_e) is very large, which is permitted for regular buildings with short periods. The dynamic analysis procedure is required for other structures (Chopra 2020). In the nonlinear behavior, the structure continues elastically up to the yielding point (V_y), and at this point, the building behaviour transforms from elastic to inelastic.

2.1.2 Components of response modification factor

The response modification factor is an indicator of the non-linear behavior of the building under the influence of seismic forces and the ability of the building to absorb and dissipate the energy through inelastic behavior and utilizing the extra strength of the structure as a reserved capacity. This factor depends on three parameters: ductility, over-strength, redundancy, and damping .[5,6]

$$R = R_\mu \times R_s \times R_r \times R_\xi \qquad (2.1a)$$

Where:

R: response modification factor

R_μ: ductility reduction factor

R_s: overstrength factor

R_r: redundancy factor

R_ξ: damping factor

- Ductility reduction factor (R_μ):

This factor is used to reduce the elastic force demand (V_e) of the structure to inelastic force demand (V_y), and it can be calculated by Eq (2.1b):

$$R_\mu = \frac{V_e}{V_y} \qquad (2.1b)$$

According to ATC (2019), global ductility (μ) is defined as the ratio between the maximum drift capacity and the yield displacement of the roof building as expressed in Eq. (2.2):

$$\mu = \frac{\Delta_m}{\Delta_y} \qquad (2.2)$$

where:

μ: global ductility

Δ_m: maximum displacement corresponding to the maximum base shear

Δ_y: yield displacement corresponding to yield base shear.

Newmark and Hall (1982) found a relation between global ductility (μ) and ductility reduction factor (R_μ) depending on the period R_ μ_T relationship as shown in Eq (2.3):

Short period T < 0.2 second $R_\mu = 1$

intermediate period $0.2 \leq T \leq 0.5$ second $R_\mu = \sqrt{\mu - 1}$ (2.3)

Long period T > 0.5 second $R_{\mu} = \mu$

Miranda and Bertero (1994) found a relationship between global ductility (μ) and ductility factor (R_{μ}) and the time period (T) to calculate the ductility factor R_ μ_T, and this relation depends on the site class. Hence, it is found for rock, alluvium, and soft soil, as shown in equation (2.4):

$$R_{\mu} = \frac{\mu-1}{\emptyset} + 1 \quad (2.4)$$

The factor (ϕ) represents the ductility according to the site class of the soil. This factor can be calculated based on the following equations:

For rock soil:

$$\emptyset = 1 + \frac{1}{10T-\mu T} - \frac{1}{2T e^{-1.5\,(\ln(T)-0.6)^2}}$$

For alluvium soil:

$$\emptyset = 1 + \frac{1}{12T-\mu T} - \frac{2}{5T e^{-2\,(\ln(T)-0.2)^2}}$$

For soft soil:

$$\emptyset = 1 + \frac{T_g}{3T} - \frac{3T_g}{4T e^{-3\,(\ln(T/T_g)-0.25)^2}}$$

Where:

T_g: predominant period for the ground motion.

- Over strength reduction factor (R_s):

The strength factor is essential to prevent the collapse of the buildings. The over-strength factor (R_s) can be defined as the ratio between the actual and design base shear:

$$R_s = \frac{V_y}{V_s} \quad (2.5)$$

Where:

R_s: Overstrength factor

V_y: the yielding base shear.

V_s: the designing base shear when the first plastic hinge occurs.

- Redundancy factor:

The redundancy moment resisting frame system must consist of multiple lines of vertical seismic framing. These frames are designed to transfer seismic-induced inertial lateral loads to the foundations. The lateral load is shared by different frames depending on each frame's strength and stiffness characteristics. According to ATC-19, this factor should be assumed as 1.

- Damping factor:

This factor explains the effects of additional viscous damping. It is mainly applied to structures with additional power dissipation devices. If such devices are not used, this factor's value is assumed to be 1 (ATC-19).

So, the response modification factor is presented in equation (2.6):

$$R = \frac{V_e}{V_y} \times \frac{V_y}{V_s} \times 1 \times 1 = \frac{V_e}{V_s} \tag{2.6}$$

2.2 Nonlinear analysis

2.2.1 Overview

Many types of procedure are used to analyze structures, such as linear static procedure (LSP), linear dynamic procedure (LDP), nonlinear static procedure (NSP), and nonlinear dynamic procedure (NDP), and the chosen analysis procedure depend on the level of designing the structures. Modern analysis methods consider the nonlinear behavior of structures. The linear static and dynamic procedures (LSP, LDP) are helpful when the structures behave elastically and do not reach the inelastic range. Conversely, when the structures act inelastically, accurate results can be obtained using the nonlinear static or nonlinear dynamic procedure (NSP, NDP). Nonlinear dynamic analysis requires more data, effort, and time.

Nonlinear static analysis (pushover analysis) and nonlinear dynamic analysis (response history) are essential for the performance-based seismic design of the structures. In pushover analysis, some components are controlled by lateral deformation, whereas others are controlled by lateral force. The demand parameters calculated from these analyses are drift, acceleration of each story, and a capacity curve for the structure. These demands used to predict the following parameters depend on codes such as (ASCE/SEI 7, PEER2010, LATBSDC 2011, FEMA P-58, and FEMA P-695): [3,8,7]

a) Evaluation of the matches of acceptance criteria in the codes.
b) Predict the behavior of the building under earthquake and safety.
c) Failure risk evaluation.

The nonlinear analysis has an incremental load or deformation on the structure. Each incremental step produces a shear force and top displacement. The cumulative of these incremental shear forces and top displacement control the behavior of the building by forming a force-deformation curve .[4]

2.2.2 Pushover analysis

2.2.2.1 Overview

Pushover analysis is an efficient alternative to dynamic analysis because it is a simple method that can provide essential elastic and inelastic data for the structure. Pushover analysis provides many services, such as NIST (2017)

- Assessment of the distribution drifts for stories along the height of the building.
- Determine a real evaluation for demand forces in the brittle members, such as the axial force on column and moment on beams, which may affect the structure stability.
- Determine the strength and stiffness irregularities in plan or elevation that cause changes in the dynamic response characteristics in the inelastic range.
- Verifying and adequacy of the seismic load path.
- Determine the critical region (plastic hinges) where the inelastic deformation occurred and when the first yield point occurred because the elements are weak.
- Determine the deformation demands for ductile members.

2.2.2.2 Limitation of pushover analysis

Pushover analysis has assumptions and limitations that must be determined for accurate results. These limitations relate to the top displacement controlling the structure's capacity curve and the selection of lateral load. These limitations are illustrated in the following:

- Target displacement: the global displacement expected in a seismic design at the centre of roof mass for a multi-degree of freedom as a single degree. The deformation of the structure must be in one mode shape to get accurate behavior of the structure.
- Lateral load: The lateral load must be distributed along the height of the building, which is equivalent to the seismic force that the structure is exposed to.

2.2.2.3 Performance of the structure

Nonlinear static analysis is used in this study because it is the easiest and simplest procedure. This analysis accurately approximates the structure's performance under earthquake load [NIST,2017]. The lateral load is a monotonic load representing a load distributed on the height of the building. It is increased step by step until the member reaches the yield, then the stiffness of the member is reduced, and the first plastic hinge has occurred, then the member redistributes the load itself, the process is repeated, and the load is continued increasing until the structure become unstable and collapse. The result of pushover analysis is a capacity curve (pushover curve), and this can be represented as a force-displacement curve, which depends on the structure's stiffness, strength, ductility of the members in structures, and the formation of the plastic hinge. The plastic hinge properties can be defined as the moment-curvature of the section to control the pushover curve (NIST, 2017)

The gravity load affects the pushover curve. Hence, to avoid the risk of collapse or failure, it must be considered in the nonlinear model. The points in the capacity curve represent a damaged condition in the buildings, and the capacity curve determines the performance level of the structure. Appendix A (A.2) illustrates the performance level and point conditions based on FEMA-356. The capacity curve has three performance levels:

- Immediate occupancy: The strength of the building is not significantly affected, but minor damage may occur to the structure. It is still safe but needs rehabilitation before usage.
- Life safety: At this level, significant damage occurs in the buildings. Some effects still exist from the impact of seismic force, but the building is probably safe during earthquakes.

- Collapse prevention: A partial or total collapse occurred in the building under the earthquake at this level.

In the pushover analysis, many properties must be known, such as the element's material properties, hinge properties, hinge location, stress-strain response, and moment-curvature response. All these must be defined to evaluate the performance of the structure. The performance of the building must be selected before designing, and the ASCE/SEI7-16 sets an acceptance criterion for the building components' performance in the pushover analysis. These criteria are used to evaluate the member's performance in the building. Figure (A.3) in Appendix A displays the acceptance criteria based on ASCE/SEI7-16.

The moment-curvature relationship for the members in the structure must be determined to select the hinge property that creates the points (A, B, C, D, E) in the above figure. The moment-curvature relationship for the element evaluates the value of these points. This relationship depends on the element's geometry, material properties, longitudinal reinforcement distributed, shear reinforcement distributed, and the element's load. The points in the deformation-force curve are displayed as follows:

- Point A: The base point must start from (0,0).
- Point B: The yield point. At this point, the structure transfers from elastic behavior to inelastic. The structure does not have any deformation.
- Point C: Ultimate point. At this point, the structure has the maximum strength to resist the earthquake, and cracks occur in the structure.
- Point D: This point represents residual strength in the building, and the slope between points C, D, D, and E is positive.
- Point E: This point represents accumulative failure. If the collapsibility is not preferable, maximum deformation should be set at point D.

2.3 Literature review

2.3.1 Previous studies

Response modification factor affected by some parameters such as:

1. Peak ground acceleration (PGA).
2. Horizontal or vertical structure irregularity.
3. Type of slab.
4. Orientations of the columns
5. The shape of the building.
6. Numbers of stories.

These parameters are necessary to determine the response modification factor of the building, and they should be considered in the guidelines of international design codes for seismic-resistant construction.

The following are some of the results from previous studies related to this topic with different parameters:

In 2015, Alashker et al. studied the effect of configuration on the behaviour of buildings with earthquakes by pushover analysis. Because this analysis is close to the realistic behavior of the buildings and its simplified method, their conclusion explained that good behavior could be obtained when the demand curve crosses with the elastic zone in the pushover curve. Poor behaviour can be obtained when the demand curve crosses with the inelastic zone in the pushover curve, and when the aspect ratio of the plan increases, the number of hinges increases, and the story drift increases. They explained that the plastic hinges are formatted when the yielding has occurred .[22]

This has also been explored in prior studies by Kamath et al. (2016). They analysed the performance of steel diagrid structures. The circular plane of the models was chosen, and two parameters were approved in this study. The first is the aspect ratio (the height of the structure to the base width), and its value was chosen from 2.67 to 4.26 since the base width of the structure was constant for all cases, and the height was varied. The second perimeter was the angle of the external brace of the structure. The nonlinear analysis was performed for the cases with defined plastic hinges based on the moment-curvature relationship recommended by FEMA 356, the nonlinear static performance (Pushover curve) compared with the structure's seismic response (base shear with displacement). In the results, the pushover curve for the structure was influenced by the aspect ratio and the angle of the brace. When the angle was increased, the base shear force increased, and the roof displacement decreased when the aspect ratio decreased. [21]

Several authors (Nishanth et al., 2017) have considered five cases that differ in the number of stories, four in zone factor and two in building systems. The first was the ordinary moment resisting the frame, and the second was the particular moment resisting the frame. SAP2000 software is used for designing and analyzing the cases. Pushover analysis is used to study structure behaviour and calculate the R factor. In the results, they concluded that when there is an increase in the zone factor, the response modification factor decreases, and when the period increases, the R factor increases. These effects are the same for different types of building frames. [16]

In 2017, Patel and Vyas evaluated the building resistance in modern earthquakes. They calculated the response modification factor based on ATC-19, and two parameters were chosen to evaluate the structure. The first was the system of design (special moment resisting frame and ordinary moment resisting frame), and the second parameter was the zone factor. IS-1893-2002 specified the value of R =5 for (SMRF) and 3 for (OMRF). They performed the pushover analysis for four models and calculated the R-value. They concluded that the R-value decreases when the zone factor increases. [17]

Some authors (Soliya et al., 2018) have driven the further development of the topic. They studied the effect of aspect ratio on evaluating the response modification factor. They evaluated the R factor based on pushover analysis for the special moment resisting frame (SMRF). Horizontal and vertical aspect ratios of the building were the key factors. Then, the result of the R factor is compared with the IS1893 code value. They studied nine cases, 3 variations in the length of structures (12m, 24m, and 36m) and 3 variations in the height (4story,8story and 12story). These cases have the same characteristics for seismic load, and the R-value is recommended as 5 in the IS code. SAP2000 was used to design and analyze the structures. They got the pushover curves for all cases, and the R-value has been calculated based on the curves. In the result, they found that the R-value increases when the number of bays increases. When the number of stories increases, the R factor increases, the range of R values between (5.43 and 8.01) and by comparison with IS code, all values were higher than IS code. [20]

In 2018, Zaid investigated the effects of disoriented columns on the values of the R factor for frame buildings in Palestine. Nonlinear pushover analysis evaluated R values and compared them with the IBC 2012 Code. Two parameters were adopted: the number of stories and the orientation of columns. Six cases were studied with different orientations and geometry of the columns (square column with the main direction in the axis, square column with the main direction in the axis, rectangle geometry with the main direction in the X axis, and rectangle with the main direction in the y-axis. Also, there are 3 cases with different numbers of stories (4 stories, 8 stories, and 12 stories). The IBC-2012 code recommends the value of the R factor for the intermediate moment resisting frame to be 5. The results were found to be too close to the suggested values in the IBC 2012 Code and other codes such as Japanese, Euro, American (UBC), and Egypt, and he found that the IBC code was conservative in some cases and nonconservative in others. Zaid found that the R factor increased with an increase in the height of the story and with various orientations of the column. The maximum value of R is obtained when the orientation of the column is in a strong direction, and the minimum value is when the column is in a weak direction. [15]

Some scholars evaluated the reinforced concrete frame under lateral load by nonlinear analysis. This study was done by simulating three cases on the SAP2000 software program. The user defined the plastic hinge in the first model, and the second model defined the final fibre hinge as an auto hinge. The results of these cases were compared with experimental results and analytical results calculated by the VecTor2 program. In the results, the pushover curve for concentrated hinges that defined based on FEMA-356 overestimated the structures' behaviour, but when fibre hinges were used, the behavior of the building was decreased remarkably. The plastic hinge results were too close to the experimental results tested by (Duong 2006). These results are too close to numerical results calculated by the VecTor2 program, which means simulating the structure based on criteria recommended by FEMA-356 provides the realistic behaviour of the building by pushover analysis.[24]

As has been previously reported in the literature, Abdi (2019) studied the development of modern earthquake-resistant buildings by evaluating building response characteristics that mainly affect the response modification factor, which is formulated based on three aspects (strength, ductility, and redundancy factors). The height of structures affects the

response modification factor. He aimed to summarize that overstrength, ductility, and R have relevant information from different experimental and analytical studies.[12]
A large number of existing studies in the broader literature have examined the effect of lintel beam. Shendkar et al. (2019) studied the effect of lintel beam on the R factor for reinforced concrete buildings designed in India and seismically evaluated different infill configurations considering the openings in infill to simulate a realistic model. Four stories were chosen in this study: the first was a reinforced concrete with an infill frame without a lintel beam, the second was a bare reinforced concrete frame without a lintel beam, and the third was reinforced concrete full infill with a lintel beam. The final case was a bare reinforced concrete frame with a lintel beam. Nonlinear pushover analysis was used to evaluate R values and compared with recommended values by IS 1893 part_1(2016) and BIS code. In the results, the R-value for the full infill frame with lintel beam was higher than other frames because the stiffness increased, and the computed R values for all cases were less than the suggested values in IS and BIS codes.[14]

For example, research has provided evidence for parameter effects. Mohamed et al. (2019) studied the effects of parameters such as the number of bays, number of stories, load patterns and fundamental periods on the seismic response modification factor for reinforced concrete frame by pushover analysis. SAP 2000 was used to analyze. Reinforced concrete members were designed according to the Egyptian Code of Practice ECP_203 and ECP_201. In the results, the values in ECP_Code were conservative, but the R-factor result values were higher than those given by the code. [13]

A considerable body of literature exists on the effects of aspect ratio on the response reduction factor on reinforced concrete with semi-interlock and unreinforced masonry walls. Shendkar (2020) explained that the response reduction factor is a factor that reduces the elastic force to inelastic design force. It is the same as the behavior factor. Pushover analysis was used to get the behaviour of buildings, which was used to get the initial stiffness, yielding point, maximum displacement, maximum base shear, modified stiffness, the failure state, and the response modification factor computed by the pushover curve. The results of their study found that the ductility in bare frames was higher than in infill frames, and the base shear in semi-interlock masonry walls in frames was higher than in unreinforced masonry walls and bare frames. Also, the strength factor for infilled frames was higher than the bare frame. Moreover, the response reduction factor decreased with increased aspect ratio and number of stories because the structure's flexibility increased. [23]

Abdel Hamid’s (2020) work can provide a more comprehensive description. They studied the effects of an increasing number of stories on the response modification factor. ABAQUS software was used to apply two methods of analysis. The first one was pushover analysis. The second one was time history analysis. They said the R factor reflects the capability of the building to behave nonlinearly to resist the lateral load without collapsing. They performed three cases using the retaining wall at the first two stories, and the soil pressure represented to them. The first case used 5 stories, the second 10, and the last 15 stories. They obtained the force-displacement curve and used a solid element for the concrete element and a line element for reinforced steel.

In the results, the R factor was calculated based on ATC-19 and compared with the value suggested by ECP and international code. They concluded that the R-value calculated for two stories that were rounded by shear wall did not match with the ECP code since the stiffness was increased, but these values matched with the Euro code because the Euro code suggests a value for buildings with irregularity in elevation. Also, the results of R values based on pushover analysis were too close to values calculated based on time history analysis. Furthermore, they deduced that the increase in the number of stories leads to increased R factor, increased displacement, and decreased base shear, subsequently increased ductility. [19]

The literature review (Palanisamy et al., 2020) studied the effect of the long period of vibration on the ductility factor. They adopted two parameters in this study: the first one was four structures with different periods. The second one was different zone factors used for a seismic design for ordinary and special moment resisting frames. SAP2000 software was used to design and analyze the structures. Then, the R factor was computed by nonlinear static analysis for the 2D reinforced concrete frame and the plastic hinge defined according to FEMA guidelines. The relationship between these parameters was found, and R values were compared with those recommended in codes (ASCE7, EC8, and IS1893). In the results, they concluded that the increase in zone factor decreased base shear, so the ductility factor and response modification factor decreased, and an inverse relationship was found between the time period and ductility factor. However, a positive relationship was found between the time period response modification factor. [18]

Nasr et al. (2020) studied the effect of irregularity in the plan on the response modification factor for an ordinary moment resisting frame. SAP2000 software was used to design and analyze the structures. Three parameters were adopted: the first was story number (5 stories and 10 stories), and the second was the zone factor (0.15g,0.2g, and 0.25g). The last one was the plan irregularity (rectangle plane, L shape plan, and T shape plan). The pushover analysis has been applied for the cases based on the ECP201 code. The results concluded that the response modification factor decreases when the zone factor increases. When the number of stories increased, the response modification factor increased, the R factor was less than the values recommended in the ECP201 code, and they recommended decreasing the irregularity in the architectural plan. [25]

In 2021, Nasr et al. (2021) studied the effects of shear walls with openings in multi-story buildings on response modification factor, SAP2000. The ETABS software used to model the cases, the structures designed by Egyptian code (ECP2012). The cases were two stories with one bay for a 2-D frame and 8-story with dual system building. The zone factor was 0.15g and 0.25g, with different ratios for openings in the wall. The pushover analysis was used in this study. In the results, they concluded that the response reduction factor decreases with the increase of zone factor and increases with an increased number of stories, and the openings affect the maximum base shear and maximum displacement, which causes a decrease in the response reduction factor.[26]

Previous work completed by Butt studied the effects of multi-span reinforced concrete bridges on response modification factor, three existing simply supported RC bridges with bent modeled and analyzed by pushover analysis, AASHTO-LRFD used to design the bridges in Pakistan, response modification factor calculated for longitudinal and transverse direction for the bridges, the AASHTO code recommends the R factor to be between 4.5 to 5. In the results, they concluded that R values were not the same for longitudinal and transverse directions for the same bridge, but the suggestion values in the AASHTO code are conservative for the directions in bridges. [27]

In the previous studies, there are some gaps, and here the researcher explains how to overcome them:

1. Number of cases: The number of cases in the previous studies was not enough to judge the results, so in this study, the number of cases has been increased and chosen to be comprehensive to many aspect ratios and zone factors.
2. 2-D frame performed to model the structures: In this study, the frames were modeled as 3-D frames on the software programs to be more realistic and to obtain accurate results.

Furthermore, this study will focus on the aspect ratio of the buildings (the ratio of the height of the building to the width) as the main parameter because of leakage of the published papers related to this topic, and the secondary parameter is zone factor and the site class of the soil.

2.3.2 Codes recommendation for response modification factor

The design code recommends using the response modification factor to reduce the seismic design force and simplify the design process. This factor is significant in designing the dimension of members in the structures. The response modification factor consists of several factors multiplied by each other.

2.3.2.1 American code

Different steps have evaluated equivalent elastic lateral seismic force in UBC-97 over time:

- The lateral seismic force is a percentage of the building weight, as in equation (2.7),

$$V = C \times W \tag{2.7}$$

Where:

C: The lateral force coefficient is assumed to be a percent of the gravity load.

W: The weight of the building.

In 1925, the Santa Barbara earthquake in California directed the governments to apply for some roles about the C coefficient to equal 20% of gravity.

- The minimum base shear force value can be calculated by equation (2.8):

$$V = ZCKW \tag{2.8}$$

(UBC,97)

Where:

Z: Zone factor: equal 1 for the high seismicity region (Zone 1), 0.5 for the medium seismicity region (Zone 2), and 0.25 for the low seismicity region (Zone 3).

C: Time period factor for the building.

K: factor expressed on building system type.

The previous stages do not depend on the response modification factor

- in the next stage. Calculation lateral base shear mainly stands on site-specific ground motion maps, time period of the building, soil site factor, importance factor and response modification factor. Equation (2.9) shows the relation between all these parameters.

$$V = \frac{C_v \times I \times W}{R \times T} \qquad (2.9).\ (UBC,97)$$

Where:

C_v: velocity seismic coefficient

I: the seismic importance factor.

R: response modification factor.

T: Time period of the building.

In 1970, the frame system types were assembled according to the K value. K is a horizontal force factor (predecessor R and Rw factor), and this factor represents the type of buildings categorized into four groups.

In 1988, the UBC code used the Rw factor, which was recommended for a twenty-nine-frame system to accomplish higher lateral force-resisting system ductility. Also, six groups of seismic zones are considered based on the seismicity of regions 0,1,2-a,2-b,3, and 4.

In 1997 UBC code, design base shear was determined by ultimate design strength. The soil profile classes (S_1 to S_4) were replaced by (S_A through S_F). Also, R_w factor was removed and replaced by a new R factor. Table 2.1 below shows the R factor values for the moment resisting frame recommended by the UBC-97 code.

Table 2.1

R factor for moment resisting frame by UBC97 code

Basic structural system	Lateral-force-resisting system description	R
Moment resisting frame system	1. special moment resisting frame (SMRF)	
	a. steel	8.5
	b. concrete	8.5
	2. Masonry moment resisting wall frame (MMRWF)	6.5
	3. Concrete intermediate moment resisting frame (IMRF)	5.5
	4. Ordinary moment resisting frame (OMRF)	
	a. steel	4.5
	b. concrete	3.5
	5. Special truss moment resisting of steel	6.5

(UBC97 code)

In 2018, the International Building Code (IBC) was developed, and the seismic design category (SDC) was founded, which depends on location, building usage, soil type, and site class. The SDC is essential to design the reinforcement details for the members in the seismic design building.

The values of the R factor according to the UBC97 code for intermediate reinforced concrete moment resisting frame (IMRF) are recommended to be 5.5. However, the IBC-2018 code recommends this value to be 5.

2.3.2.2 Europe code (EC8)

To reduce the linear spectra demand to be a design base shear force with nonlinear behaviour, Europe code (EC8) relies on the behavior factor (q) instead of the response modification factor (R), where: q is a factor that depends on time period, ductility, type of frame, stiffness, and strength, the equation (1.10) express the calculation of response reduction factor.

$$q = q_0 \times K_D \times K_R \times K_W \qquad (2.10)$$

(EC-8 code)

Where:

q_0Behavior factor, and this depends on the type of building system, shown in (Table 2.2)

K_D: ductility factor, shown in (Table 2.3).

K_R: factor expresses the regularity in elevation in the buildings, and this factor for regular structure equals 1 and for irregular structure equals 0.8.

K_W: factor expresses the prevailing failure mode; for MRF, this factor equals 1.

Table 2.2

The behavior factor, according to EC8

Structural type		**q_0**
Frame system		5
Dual system	Frame equivalent	5
	Wall equivalent, with coupled walls	5
	Wall equivalent, with uncoupled walls	4.5
Wall system	With coupled walls	5
	With uncoupled walls	4
Core system		3.5
Inverted pendulum system		2

(EC8 code)

Table 2.3

The ductility class factor according to EC8.

Ductility class	K_D
DC"H"	1
DC"M"	0.75
DC" L"	0.5

(EC8 code)

2.3.2.3 Egyptian CODE(ECP 2012)

R-value in Egyptian Code relies on the type of ductility, R values ranging from 5 to 7. The ductility in this code is assigned as sufficient or non-sufficient. To get the value of the response modification factor, follow Table 2.4.

Table 2.4

Response modification factor according to Egyptian code for moment resisting frame.

Type of ductility	R-factor
Sufficient	5
Non-sufficient	7

(Egyptian code)

2.3.2.4 2 Canadian code (NBCC,2005)

The minimum lateral force can be calculated by Equation (2.11):

$$\frac{S(2)M_v I_e W}{R_d R_0} \leq V = \frac{S\,(T_a)M_v I_e W}{R_d R_0} \leq \frac{2\times S(0.2)I_e W}{3\times R_d R_0} \qquad (2.11)$$

Where:

T_a: the fundamental lateral period in the direction under consideration.

$S(T_a)$: Design spectral acceleration.

M_V: factor to account for higher mode effect on base shear.

W: weight of the building.

I_e: earthquake importance factor

R_d: ductility-related force modification factor reflecting the capability of a structure to dissipate energy through inelastic behavior.

R_0: over-strength-related force modification factor accounting for the dependable portion of reserve strength in a structure designed.

R_d can be evaluated depending on the level of ductility of the structure. R_0 depends on the type of building system.
For concrete frame (IMRF), the Rd factor is recommended to equal 2.5, and R0 is recommended to equal 1.5, so the R equals 3.75.

2.3.3 Summary

As seen above, codes use different symbols and values for response modification factors based on design conditions. The response reduction factor depends on the structure's reinforcement details, which are expressed through the structure's ductility. Also, the response modification factor depends on the soil condition, building system type, and region's seismicity.

The aspect ratio factor influences the response modification factor, which is related to the behavior of the building, including the stiffness of the building, lateral load distribution, and the ability to drift.

Also, another factor can affect the response modification factor, which is the region's seismicity, which means the region with a higher level of seismic activity may require a higher value of R factor to resist the seismic forces.

When considering these factors, it is vital to analyze and evaluate the response modification factor considering the design conditions, building system, seismic activity, and other parameters. International codes give guidelines and recommendations that should be considered in the design process for selecting R values based on the system type and seismic design category.

Chapter Three

Case of Study and Verifications

3.1 Overview

Research on 3-D has a long tradition. A 3-D finite element model is constructed using commercial software (ABAQUS) to understand the behaviour of structures with different aspect ratios, seismicity, and soil types. The selected structure is symmetric, with uniform loads for each story. An analytical analysis is performed for a simple frame made from steel and unconfined reinforced concrete to verify the result from ABAQUS.

Two types of members can be used for modeling columns and beams of the structures in ABAQUS to perform the nonlinear static analysis: solid element and line element.

Solid element: It is a 3-dimensional element that consists of one material without including gaps or special properties such as connectors or hinges. Considering the following points (Abaqus Manual User, 2014): [28]

1. Many shapes for the solid element can be used as tetrahedral, brick, triangular, and circular.
2. A layer composite must be used if the composite element is needed.
3. The material can be defined as an element with elastic and plastic properties.
4. One of its features is that each element represents reality in the assembly, and all elements bind together by connections to become one part.

This method is more accurate and better represents reality with multi-axial response in addition to fracture and cracking.

Line element is a 1-dimensional element used to model beams, columns and bracing. It represents a complex element but with kinematic assumption as the plane sections before the bending moment remain plane after bending. It needs elastic and plastic properties for material and can be obtained by defining the moment-curvature relationship for the elements.

3.2 Moment-curvature relationship

The moment-curvature relationship for a practical beam for any element section goes through three stages: the first one is cracking, the second one is yielding on tension steel, and the last one reaches ultimate strain in the concrete. [10]

To calculate the curvatures and moments for reinforced concrete or steel sections with bending moment and axial force, many requirements must be considered, such as strain compatibility and equilibrium of force in addition to these assumptions:

1. Plane sections before the bending moment remain plane after bending.
2. The stress-strain curves for concrete and steel are known.

To calculate the values of moment and curvature at yielding, tension steel reaches the yield strain before the concrete reaches the peak strain where the stress in concrete is less than cylinder strength. The stress-strain relationship for a doubly reinforced concrete beam section is shown in Figure (A.4) in Appendix A.

$$\mathrm{k} = \sqrt{(\rho + \acute{\rho})^2 \times \mathrm{n}^2 + 2 \times \left(\rho + \frac{\acute{\rho} \times \acute{\mathrm{d}}}{\mathrm{d}}\right) \times \mathrm{n}} - (\rho + \acute{\rho}) \times \mathrm{n} \qquad (2.1)$$

$$M_y = A_s \times f_y \times jd \qquad (2.2)$$

$$\varphi_y = \frac{\frac{f_y}{E_s}}{d \times (1-k)} \qquad (2.3)$$

where:

k: factor to calculate the distance between the centroid of stresses and extreme compression stress for the section.

$\acute{A}s$: area of compression steel

As: area of tension steel

b: width of section

d: effective depth of tension steel

$\acute{d}$: the distance from extreme compression fibre to the centroid of compression steel

Ec :modulus of elasticity for concrete

Es :modulus of elasticity for steel

f_y :yield strength of steel

jd :the distance between the centroid of concrete and steel compressive force and the centroid of tension force.

$n = \frac{Es}{Ec}$

ρ $\frac{As}{b \times d}$the tension-steel ratio

$\acute{\rho} = \frac{\acute{A}s}{b \times d}$The compression steel ratio

φ_y: the yielding curvature.

The ultimate moment and curvature values occurred when the concrete compression concrete reaches ultimate strain. They can be computed for unconfined doubly reinforced concrete beams using equations (3.4 to 3.7).

$$a = \frac{A_s \times f_y - f_y \times \acute{A}_s}{0.85 \times b \times \acute{f}_c} \quad (3.4)$$

$$Mu = 0.85 \times f_c \times a \times b \times \left(d - \frac{a}{2}\right)' + A_s \times f_y \times' (d - \acute{d}) \quad (3.5)$$

$$\varphi_u = \frac{\varepsilon_c}{c} \quad (3.6)$$

$$\acute{\varepsilon}_s = \varepsilon_c \times \frac{c - \acute{d}}{c} \quad (3.7)$$

where:

Mu: ultimate moment for the section.

φ_u: ultimate curvature for the section.

$\acute{\varepsilon}_s$: the strain for longitudinal steel.

ε_c: the strain for concrete.

a: the distance between the center of compression stress and the extreme compression for the section.

c: the distance from the centroid of compression and tension stress to extreme compression stress.

For the compression steel to yield, the following equation (Eq 2.8) shall be satisfied:

$$\varepsilon_c \times \left[1 - \beta 1 \times \acute{d} \times \frac{0.85 \times f_c^{\prime} \times b}{A_s \times f_y - \acute{A}_s \times f_y}\right] \geq \frac{f_y}{E_s} \tag{3.8}$$

If Eq. (2.8) is not satisfied, compressive steel does not yield, and ‘a’ can be calculated by Eq. (2.9) and ultimate moment M_u by Eq. (2.10)

$$\frac{1}{2} \times \left(\frac{a}{d}\right)^2 + \frac{a}{d} \times \left[\frac{\acute{\rho} \times \varepsilon_c \times E_s - \rho \times f_y}{1.7 \times f_c^{\prime}}\right] - \frac{\acute{\rho} \times \varepsilon_c \times E_s \times \beta 1 \times d}{1.7 \times f_c^{\prime} \times d} = 0 \tag{3.9}$$

$$M_u = 0.85 \times f_c^{\prime} \times a \times b \times \left(d - \frac{a}{2}\right) + \acute{A}_s \times E_s \times \varepsilon_c \times \frac{a - \beta 1 \times \acute{d}}{a} \times (d - \acute{d}) \tag{3.10}$$

where:

$\beta 1$: factor depends on the compression force of the concrete ($f_c^{\prime}$).

To determine the values of moment and curvature for the steel section, the same reinforced concrete assumptions and requirements should be followed, and the equations below are derived to calculate them.

$$M_y = S_x \times f_y \tag{3.11}$$

$$M_p = Z_x \times f_y \tag{3.12}$$

$$\varphi_y = \frac{\varepsilon_y}{c} \tag{3.13}$$

$$\varphi_u = \frac{\varepsilon_u}{c} \tag{3.14}$$

where:

Sx: elastic of modulus for the section

Zx: plastic of modulus for the section

C: depth of neutral axis

f_y: the yielding strength of steel material.

3.3 Model Validation

This section compares the manual calculation of the moment-curvature curve and the results from the ABAQUS software program to ensure matching the results. Each section of members in the frames consists of steel and concrete materials. Steel sections are defined as line elements with their properties. The concrete sections are defined as solid and elastic-plastic properties for the concrete material.

In order to properly evaluate the accuracy of this approach, material assumptions and details of manual calculations were performed. Then, a comparison was made between the results obtained from the analytical solution and those obtained from ABAQUS software.

The verification process's main goal is to ensure that the assumptions and the inputs inserted into the ABAQUS software program ban are based on the multi-stories cases.

3.3.1 Steel simple frame

The section (HE280A) with material (A36) was used for all members in the simple 2-D frame. The stress-strain for the material was assumed to be elastic and perfectly plastic. The properties of the material and the dimensions used in this case are given in Appendix A in Table (A.1) and Figure (A.5).

Analytical calculation:

Moment-curvature calculations:

$$\mathrm{My} = \mathrm{fy} \times \mathrm{Sx} = 1.013 \times 10^{-3} \times 10^{9} \times 250 \times 10^{-6} = 253.25\mathrm{KN.m}$$

$$\mathrm{Mp} = \mathrm{fy} \times \mathrm{Zx} = 1.112 \times 10^{-3} \times 10^{9} \times 250 \times 10^{-6} = 278\mathrm{KN.m}$$

$$\mathrm{Ky} = \frac{\varepsilon \mathrm{y}}{\mathrm{y}} = \frac{0.00124}{135} = 0.00000918\ \mathrm{rad/mm}$$

$$\mathrm{Ku} = \frac{\varepsilon \mathrm{u}}{\mathrm{y}} = \frac{0.02}{135} = 0.000148\ \mathrm{rad/mm}$$

Load-deflection calculations

$$\mathrm{Py} = \frac{2 \times \mathrm{My}}{\mathrm{L}} = \frac{2 \times 253.25}{3.5} = 144.7\mathrm{KN}$$

$$Pp = \frac{2 \times Mp}{L} = \frac{2 \times 278}{3.5} = 158.8KN$$

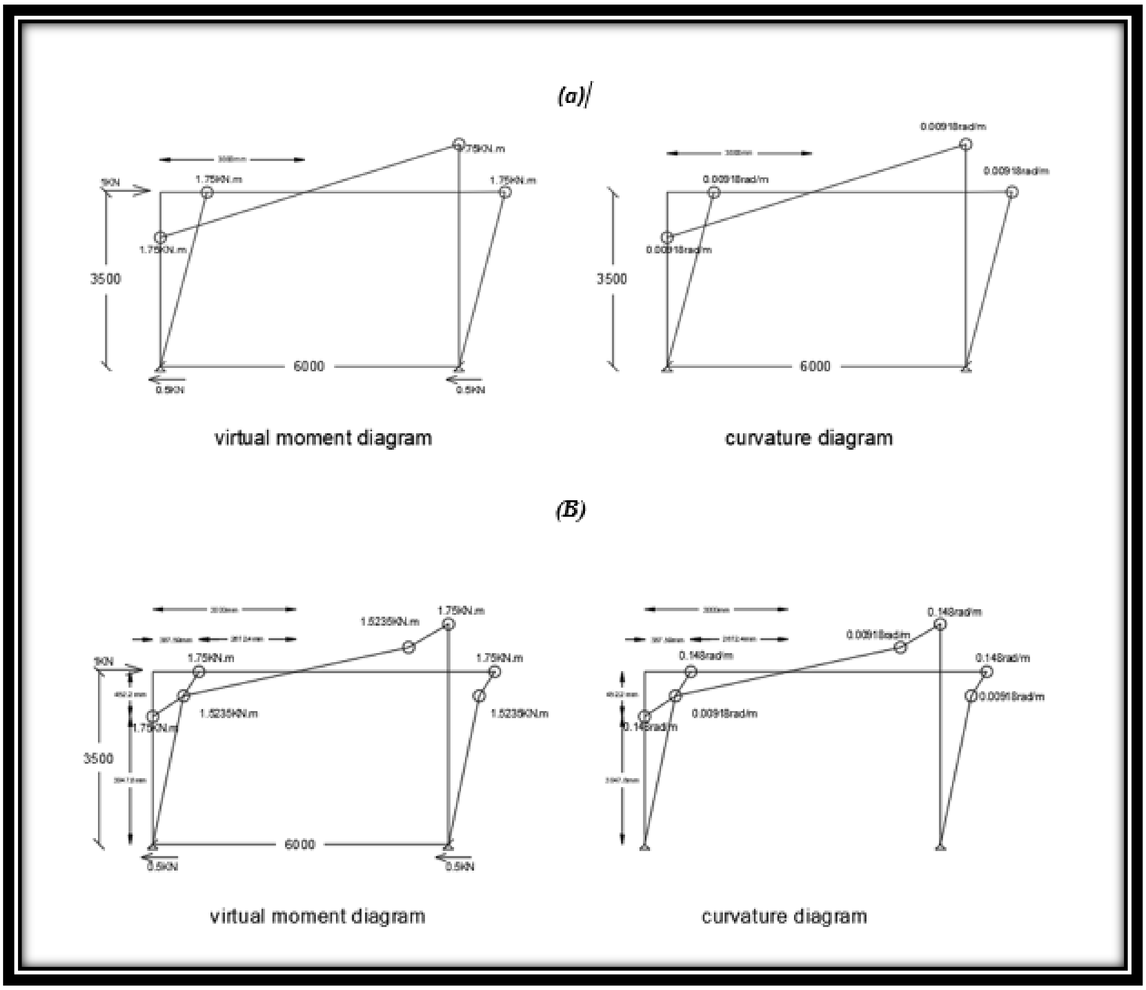
virtual work method was used to calculate the deflection:

$$\Delta = \int K \times M1 \, dx$$

For trapezoidal integration=$\frac{L}{6} \times (2 \times u1 \times v1 + 2 \times u2 \times v2 + u1 \times v2 + u2 \times v1)$

Figure 3.1

The virtual work digram and curvitaure diagram for 2-D frame (a)at yielding (b) at ultimate



At yielding:

$$= \left(\frac{3.5}{6} \times (2 \times 0.00918 \times 1.75) \times 2\right) + \left(\frac{3}{6} \times (2 \times 0.00918 \times 1.75) \times 2\right) = 0.0702m$$

$$= 70.2mm$$

At ultimate:

$$\Delta u_{\text{for column}} = (3.0478/6 \times ((2 \times 0.00918 \times 1.5239) \times 2) + (0.452/6 \times [(2 \times 0.00918 \times 1.5239) + (2 \times 0.148 \times 1.75) + (0.0124 \times 1.75) + (0.148 \times 1.5239)] \times 2) = 0.1205\text{m}$$

$$\Delta u_{\text{for beam}=}(2.6124/6 \times ((2 \times 0.00918 \times 1.5239) \times 2) + (0.38759/6 \times [(2 \times 0.00918 \times 1.5239) + (2 \times 0.148 \times 1.75) + (0.00918 \times 1.75) + (0.148 \times 1.5239)] \times 2) = 0.1033$$

$$\Delta u = \Delta_{\text{u column}} + \Delta_{\text{u beam}} = 0.1205 + 0.103 = 0.2238\text{m} = 223.8\text{mm}$$

3.3.2 Reinforced Concrete Simple Frame

The simple concrete frame was represented as a solid element by the ABAQUS program. The load assumed on 2-D frame 1-bay is superimposed dead load (40kN/m) and live load (20kN/m). The bay length is (5m)and the frame height is 3.3m. The used beam is a rectangle beam section (300*400mm) with reinforcement (3φ18 top and bottom), and the column is a square section (400*400mm) with reinforcement (6φ18). The section properties for the simple concrete frame members are illustrated in table (A.2.a) and figure (A.6) Appendix A.

Materials:

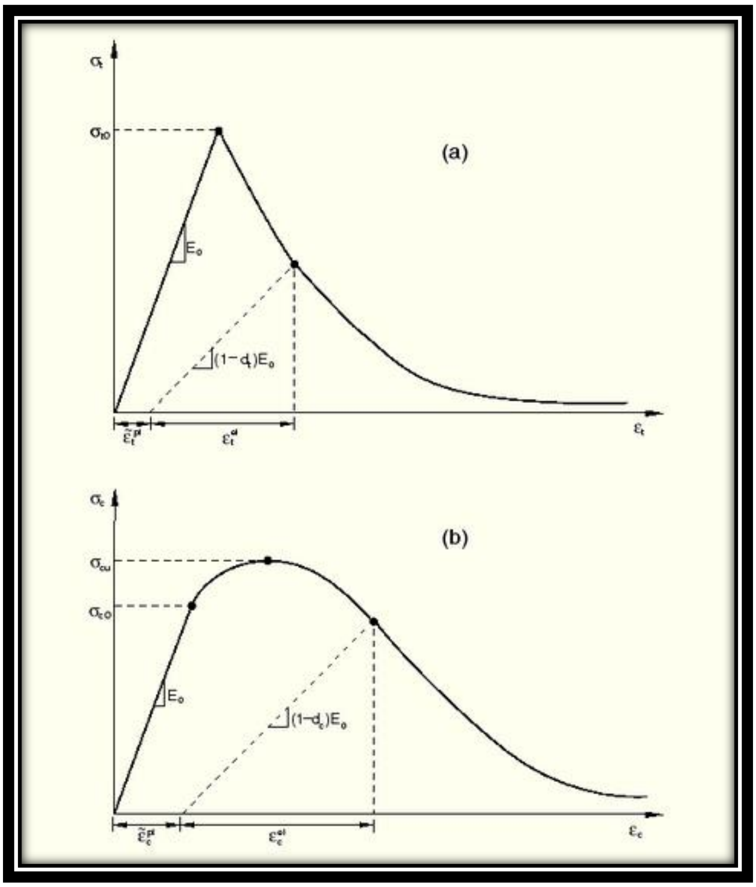
- Concrete:

Concrete is a non-homogenous material that goes through different tension and compression loading stages. Due to these loads, two main failures will appear in strength and stiffness: tensile cracking and compressive crushing. Therefore, to consider the nonlinear behavior in the model by the ABAQUS program, the concrete damage plasticity (CDP) method will be used.

Failure development is controlled by defining the tension plastic strain (ε_{pt})and compression plastic strain (ε_{pc}). To define the mechanical behavior of the material, assume the tensile and compression of the plasticity damage property as illustrated in Figure (3.2).

Figure 3.2

The uniaxial stress-strain loading of concrete in(a) tension and (b)compression.



(Abaqus analysis users guide,2014)

The damage variables (d_t and d_c) must be defined. The variable dt expresses the damage of material in tension, and dc expresses the damage of material in compression. These variables vary from 0 to 1, whereas the value of 0 represents when the material is undamaged, and 1 is when the material is damaged. To represent this in the Abaqus program, stress-plastic strain must be manually calculated, and the results must be used as data inputs for materials in Abaqus. To calculate the compression and tension stress, follow the equation (2.15), (2.16):

$$\sigma_t = (1 - d_t) \times E_0 \times (\varepsilon_t - \varepsilon_t^{pl}) \tag{3.15}$$

$$\sigma_c = (1 - d_c) \times E_0 \times (\varepsilon_c - \varepsilon_c^{pl}) \tag{3.16}$$

The plastic strain in tension and compression can be calculated as given in equations (2.17) and (2.18), respectively:

$$\varepsilon_t^{pl} = \varepsilon_t^{ck} - \frac{d_t}{1-d_t} \times \frac{\sigma_t}{E_0} \tag{3.17}$$

$$\varepsilon_c^{pl} = \varepsilon_c^{in} - \frac{d_c}{1-d_c} \times \frac{\sigma_c}{E_0} \tag{3.18}$$

where σ_t: the tension, stress, σ_c: the compression stress, d_t: the tension damage parameter. It is calculated by $d_t = \sigma_t - \sigma_{t\,yield}$, d_c: the compression damage variable and it is calculated by $d_c = \sigma_c - \sigma_{c\,yield}$, ε_t: the tension strain, ε_t^{pl}: the plastic tension strain, ε_c: the compression strain, ε_t^{pl}: the plastic compression strain, ε_t^{ck}: the cracking tension strain, ε_c^{in}: inelastic tension strain.

- **Steel:**

The steel behaviour is assumed to be elastic and perfectly plastic. It has elastically behavior until the yield point occurs, then it becomes constant as in Figure (A.7) illustrated in Appendix A.

The materials used in the reinforced concrete frame were assumed as Mandar with grade (B300) and strength (fc' = 24MPa) reported from (by Lubliner et al. (1989) and Lee and Fenves (1998)). [29,30]

Figure (A.8) in Appendix A shows the plastic properties of the concrete material:

a) The uniaxial compression stress -inelastic strain curve for concrete.
b) The tension stress-cracking strain.
c) Damage compression parameter -plastic strain.
d) Damage tension parameter versus cracking strain curve for concrete.

An elastic, perfectly plastic stress-strain curve was assumed for steel material with yield strength fy=417.495Mpa.

Table (A.3) in Appendix A shows the parameters for concrete, and Table (A.4)in Appendix A shows the parameters for steel to define the materials in ABAQUS.

Analytical calculations:

Moment-curvature calculations:

The first yield hinge was assumed in the beam.

At crack:

$A = bh + (n-1)(As + \acute{A}s)$=$250 \times 400 + (9.85 - 1)(763.02 + 401.92)$ =133514.2581 mm^2

$\bar{y} = 200\text{mm}$

$$I = \left(\frac{b \times h^3}{12}\right) + \left(A \times \left(\bar{y} - \frac{h}{2}\right)^2\right) + \left((n-1) \times As \times (d - \bar{y})^2\right) + \left((n-1) \times \acute{A}s \times (\bar{y} - d)^2\right)$$

$$I = \left(\frac{300 \times 400^3}{12}\right) + \left(133514.2581 \times \left(200 - \frac{400}{2}\right)^2\right) + \left((9.85 - 1) \times 763.02 \times (340 - 207.0957)^2\right) + \left((9.85 - 1) \times 763.02 \times (200 - 60)^2\right) = 1775685355\text{mm}^2$$

$fr = 0.62 \times \lambda \times \sqrt{\acute{f}c}$ $= 0.62 \times 1 \times \sqrt{24.324}$ =3.057Mpa

$Mcr = \frac{fr \times I}{y\ bottom}$ $= \frac{3.057 \times 1775685355}{200}$ =27.14 KN.m

$$\varphi cr = \frac{fr/Ec}{y_{bottom}} = \frac{3.056/20800}{200} = 7.34 \times 10^{-7}\text{rad/mm}$$

At yielding:

$$k = \sqrt{(\rho + \acute{\rho})^2 \times n^2 + 2 * \left(\rho + \frac{\acute{\rho} \times \acute{d}}{d}\right) \times n} - (\rho + \acute{\rho}) \times n$$

$= \sqrt{(0.00748 + 0.00748)^2 \times 9.855^2 + 2 * (0.00748 + 0.00748 \times 60/340) \times 9.855}$ $-(0.00748 + 0.00748) \times 9.855$

=0.294

$kd = 0.294 \times 340 = 100.08\text{mm}$

$$\varepsilon s = \varepsilon cmax. \times \frac{d \times (1 - k)}{kd} = 0.00205 \times \frac{340 \times (1 - 0.294)}{100.08} = 0.0049 > \varepsilon y = 0.00203$$

$$\varepsilon c = \varepsilon s_y \times \frac{kd}{d - kd} = 0.00203 \times \frac{100.08}{340 - 100.08} = 0.00084$$

$fc = \varepsilon c \times Ec = 0.00084 \times 20800 = 17.67 = 0.7 \times \acute{f}c$

So the strain is elastic.

$$\acute{\varepsilon}s = \varepsilon c \times \frac{kd - \acute{d}}{kd} = 0.00084 \times \frac{100.08 - 60}{100.08} = 0.00034$$

$$\acute{f}s = \acute{\varepsilon}s \times Es = 0.00034 \times 205000 = 69.76 \text{ Mpa}$$

$$Cc = 0.5 \times fc \times b \times k \times d = 0.5 \times 17.67 \times 250 \times 0.294 \times 340 = 265.3KN$$

$$Cs = \acute{A}s \times \acute{f}s = 763.02 \times 69.76 = 53.23KN$$

$$Cc + Cs = 318.55KN$$

$$\bar{y} = \frac{(Cs \times \acute{d}) + (Cc \times kd)}{Cc + Cs} = \frac{(53.23 \times 60) + (256.3 \times 100.08)}{318.55} = 93.39mm$$

$$Jd = d - \bar{y} = 340 - 93.39 = 246.6 \text{ mm}$$

$$My = As \times fy \times Jd = 763.02 \times 417.49 \times 246.6 = 78.558KN.m$$

$$\varphi_y = \frac{\varepsilon y}{d - kd} = \frac{0.00203}{340 - 100.08} = 8.488 \times 10^{-6} \text{rad/mm}$$

At ultimate:

The compression steel is not yielding, so by trial and error try $fs' = 39Mpa$.

$$a = \frac{(fy \times As) - (fs' \times \acute{A}S)}{0.85 \times \acute{f}c \times b} = \frac{(417.49 \times 763.02) - (39 \times 763.02)}{0.85 \times 24.32 \times 250} = 46.56mm$$

$$C = \frac{a}{\beta} = \frac{46.56}{0.85} = 54.785mm$$

$$\acute{\varepsilon}s = \varepsilon c \times \frac{c - \acute{d}}{c} = 0.00205 \times \frac{54.785 - 60}{54.785} = 0.000195$$

$$fs' = \varepsilon s' \times Es = 0.000195 \times 205000 = 39.99 \text{Mpa}$$

$$Mu = 0.85 \times \acute{f}c \times b \times a \times \left(d - \frac{a}{2}\right) + \acute{A}s \times fs' \times \left(d - \acute{d}\right)$$

$$= 0.85 \times 24.32 \times 250 \times 9.855 \times \left(340 - \frac{46.56}{2}\right) + 763.02 \times 39.99 \times (340 - 60) = 100.011KN.m$$

$$\varphi u = \frac{\varepsilon cu}{c} = \frac{0.00205}{54.785} = 3.74 \times 10^{-5} \text{rad/mm}$$

Load-deflection calculations:

At cracking:

$$Pcr = \frac{2 \times Mcr}{L} = \frac{2 \times 27.14}{3.5} = 15.5\ KN$$

$$\Delta cr = (0.5 \times 7.34 \times 10^{-4} \times 3.5 \times 1.165 \times 2) + (0.5 \times 7.34 \times 10^{-4} \times 2 \times 1.165 \times 2)$$

=4.7mm

At yielding:

$$\mathrm{Py} = \frac{2 \times \mathrm{My}}{\mathrm{L}} = \frac{2 \times 78.558}{3.5} = 44.89\ \mathrm{KN}$$

$$\Delta \mathrm{y} = (0.5 \times 8.48 \times 10^{-3} \times 3.5 \times 1.165 \times 2) + (0.5 \times 8.48 \times 10^{-3} \times 2 \times 1.165 \times 2)$$

=54.39mm

At ultimate:

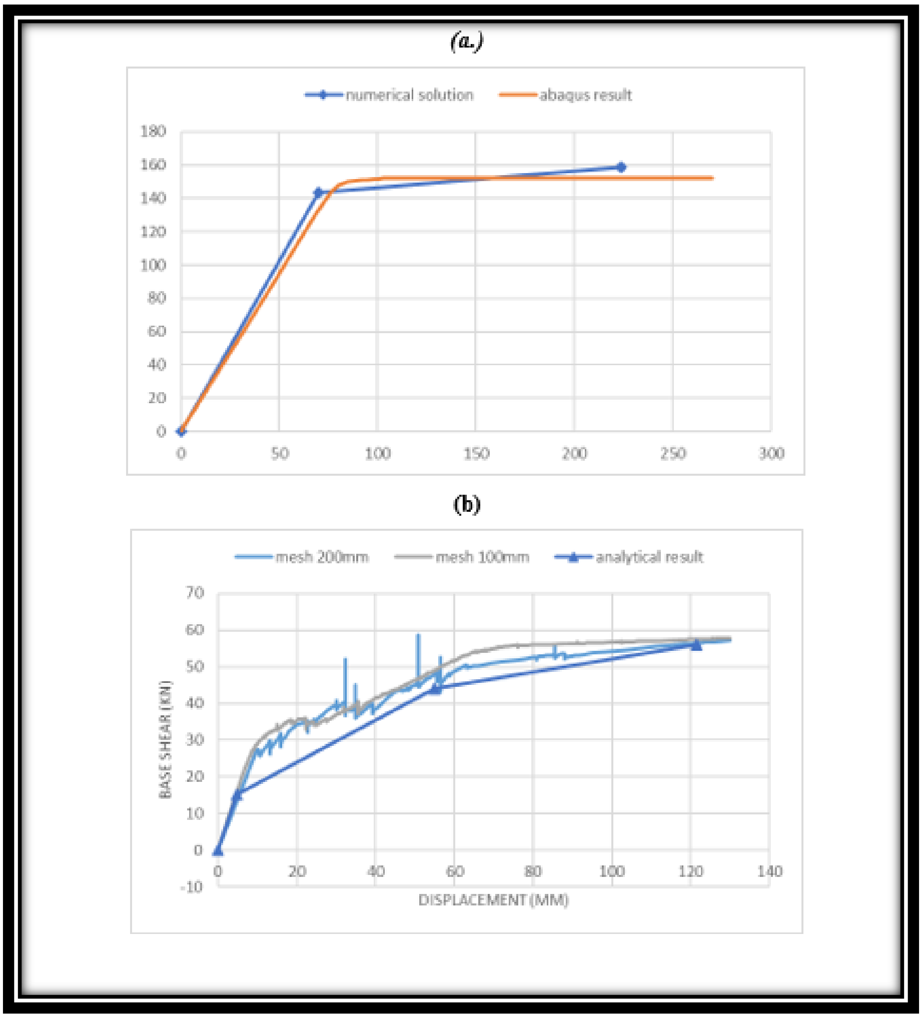
$$\mathrm{Pu} = \frac{2 \times \mathrm{Mu}}{\mathrm{L}} = \frac{2 \times 100.01}{3.5} = 57.149\ \mathrm{KN}$$

$$\Delta u = \Delta u\ column + \Delta u\ beam$$

$$\begin{aligned} &= \big((0.5 \times 8.48 \times 10^{-3} \times 2.74 \times 0.896) \\ &+ (0.5 \times (8.48 \times 10^{-3} + 3.74 * 10^{-2}) \times 1.614 \times 0.75)\big) \times 2 \\ &+ \big((0.5 \times 8.48 \times 10^{-3} \times 1.57 \times 0.896) \\ &+ (0.5 \times (8.48 \times 10^{-3} + 3.74 \times 10^{-2}) \times 1.614 \times 0.429)\big) \times 2 \\ &= 123.81\ mm \end{aligned}$$

__Figure 3.3__

the results of load-deflection curve from analytical solution and from ABAQUS program for



(a)2-D steel frame (b)R.C simple frame with different mesh size

The comparison results indicate that the ABAQUS program is one of the best commercial programs that can be relied on to simulate the behavior of structures under the influence of lateral loads such as earthquakes and to obtain closer push-over curve results to reality.

Based on this study and the comparison between mesh size 100 and 200, mesh size 100 was selected to avoid errors in the results.

3.3.3 3D model validation

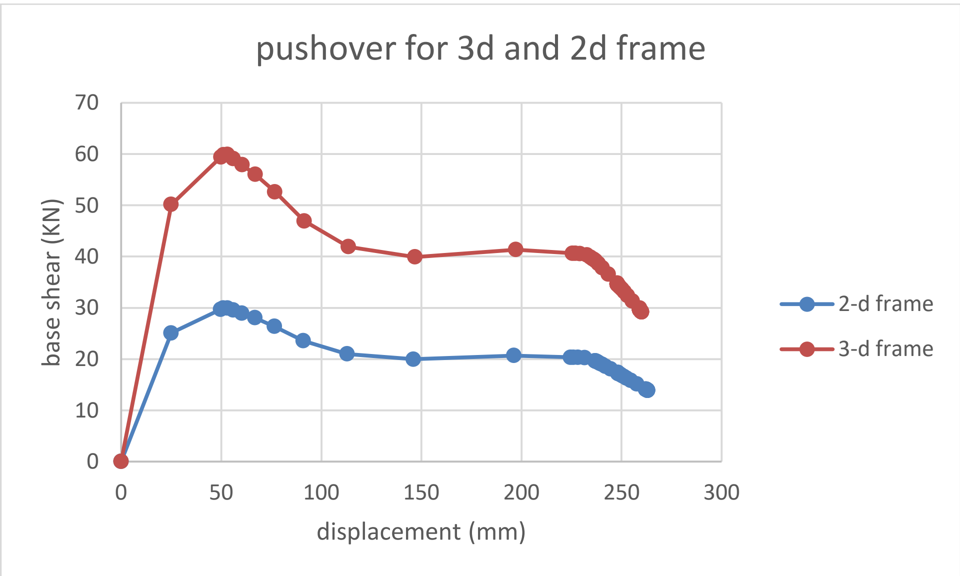
In order to choose between 3D and 2D in this study, the same calculations for the R-value must be done in both models. Pushover analysis was applied using ABAQUS software for the 3D-1bay model, and the R-value was computed. Subsequently, 2D-1bay went through the same pushover analysis procedure to compute the R-value, and then R values for the 3-D frame and 2-D frame were compared. Consequently, if there is a difference between them, 3D will be used since it is more realistic, but if it is not, each model can be used.

ETABS 2018 software was used to design a 1bay frame with a width of 5m and height of 3.5m and has distribution load as superimposed load =25KN/m and live load =10KN/m, and the type of this frame is intermediate reinforced concrete (R=5). A rectangular section with dimensions (300mm* 400mm) was used for the beam. The reinforcement 2φ16 bars at the top and 3φ16 bars at the bottom. The columns have a square section with dimensions of (500mm*500mm). The reinforcement in the column 12φ20 bars.

The details of material sections are illustrated in table (A.2.b) Appendix A. Figure (3.4) shows the pushover analysis results for the 2D and 3D frames. The R factor is calculated depending on these curves.

Figure 3.4

Pushover analysis for 3-D and 2-D frame



R computation:

Depending on ASCE 2016, the elastic shear force is computed as Eqs. (3.19 to 3.21) with R=1.

$$Ve = Cs * W \tag{3.19}$$

$$Cs = minimum \begin{cases} \frac{SD_S * I}{R} \\ \frac{SD_1 * I}{T * R} \end{cases} \tag{3.20}$$

but not less than $0.044 * SD_S * I$

$$W = dead\ load + super\ imposed\ dead\ load + 0.25 * live\ load \tag{3.21}$$

Where:

Ve : elastic base shear force

Cs: seismic response coefficient

W: the total weight of the frame

SD_S, SD_1: design spectral acceleration parameters.

T: the period of structure.

Table 3.1

Summary of calculations of 2-D frame and 3-D frame

Type of frame	W	Cs	Ve	Vy	R
2-D frame	233.75kN	0.0675	78.89kN	25kN	3.15
3-D frame	483.125kN	0.0675	163.05kN	50kN	3.25

From the result calculations, the elastic base shear and the yielding base shear for a 3-D frame are equal to double the value of a 2-D frame, and this is due to the degree of freedom for a 3-D frame being more than the degree of freedom for 2-D, and this indicates that pushover analysis for 3-D frames is more realistic. Moreover, the R-value is too close between 2-d and 3-d frames. So, a more accurate 3-D frame was used in this study.

3.4 Parametric study

3.4.1 Overview

ASCE 16 code limits the height of a building based on the seismic design category for each type of structure system. In this study, the intermediate moment resisting frame system is selected. For this system, seismic design categories B and C are only allowed with no limit on the building height. Other seismic design categories are prohibited.

The key parameter in this study is the aspect ratio of the building. Symmetric structures include a square shape (20m*20m) and a rectangular shape (40m*10m) with a fixed area (400 m^2) and a different number of stories (4 stories, 6 stories, 8 stories, 10 stories and 12 stories). The aspect ratio (height to width of the building) values are (0.66,0.99,1.32,1.65 and 1.98) and (0.33, 0.495, 0.66, 0.825 and 0.99) for these buildings. The second parameter is the seismicity with different site conditions. Three seismic zone parameters are (Z=0.15, Z=0.13 and Z=0.1). The last parameter is a type of soil (B and C). The architectural plans of the cases are illustrated in figures (B. 1 and B.2) in Appendix B.

Details of structures used in the parametric study are assumed in the Table (3.2):

Table 3.2

Summary of cases used in the study

Case	No.of story	dimension	Z	Aspect ratio	Site class	SDC
M.0.66-Z0.15.C-1	4	20m*20m	0.15	0.66	B	C
M.0.99-Z0.15.C-1	6	20m*20m	0.15	0.99	B	C
M.1.32-Z0.15.C-1	8	20m*20m	0.15	1.32	B	C
M.1.65-Z0.15.C-1	10	20m*20m	0.15	1.65	B	C
M.1.98-Z0.15.C-1	12	20m*20m	0.15	1.98	B	C
M.0.66-Z0. 132.B-1	4	20m*20m	0.132	0.66	B	B
M.0.99-Z0. 132.B-1	6	20m*20m	0.132	0.99	B	B
M.1.32-Z0. 132.B-1	8	20m*20m	0.132	1.32	B	B
M.1.65-Z0. 132.B-1	10	20m*20m	0.132	1.65	B	B
M.1.98-Z0. 132.B-1	12	20m*20m	0.132	1.98	B	B
M.0.66-Z0. 1025.C-1	4	20m*20m	0.1025	0.66	C	C
M.0.99-Z0. 1025.C-1	6	20m*20m	0.1025	0.99	C	C
M.1.32-Z0. 1025.C-1	8	20m*20m	0.1025	1.32	C	C
M.1.65 -Z0. 1025.C-1	10	20m*20m	0.1025	1.65	C	C
M.1.98-Z0. 1025-C-1	12	20m*20m	0.1025	1.98	C	C
M.0.33-Z0. 15-C-2	4	40m*10m	0.15	0.33	B	C
M.0.495-Z0. 15-C-2	6	40m*10m	0.15	0.495	B	C
M.0.66-Z0. 15-C-2	8	40m*10m	0.15	0.66	B	C
M.0.825-Z0.15-C-2	10	40m*10m	0.15	0.825	B	C
M.0.99-Z0. 15-C-2	12	40m*10m	0.15	0.99	B	C
M.0.33-Z0. 132-B-2	4	40m*10m	0.132	0.33	B	B
M.0.495-Z0.132-B-2	6	40m*10m	0.132	0.495	B	B
M.0.66-Z0. 132-B-2	8	40m*10m	0.132	0.66	B	B
M.0.825-Z0. 132-B-2	10	40m*10m	0.132	0.825	B	B
M.0.99-Z0. 132-B-2	12	40m*10m	0.132	0.99	B	B
M.0.33-Z0. 1025-C-2	4	40m*10m	0.1025	0.33	C	C
M.0.495-Z0.1025-C-2	6	40m*10m	0.1025	0.495	C	C
M.0.66-Z0. 1025-C-2	8	40m*10m	0.1025	0.66	C	C
M.0.825-Z0. 1025-C-2	10	40m*10m	0.1025	0.825	C	C
M.0.99-Z0. 1025-B-2	12	40m*10m	0.1025	0.99	C	C

3.4.2 Design of cases

The loads in the structures transmitted from the slabs to the beams, then to the columns and finally to the foundations. The slabs in the cases are assumed to be two-way solid slabs with a depth of 20 mm, the beams are distributed in both directions (x and y directions), and the dimensions of the beams are assumed to be the same in both directions and that due to the symmetry and regularity in the floor plans, the columns have a various dimension from one story to another.

Property or stiffness modification factors are assumed for the beams 0.35 and columns 0.7 in the design process. These factors are responsible for the behavior and relative stiffness of the structural elements.

The cases were designed to resist the earthquake, so two cases were submitted with different types of soil: rock soil, very dense soil and soft rock—the following details of seismic load in cases a, b and c.

Case a (Z=0.15, Rock soil B):

- S1= 0.28125
- Ss= 0.5625
- Fa= 0.9
- Fv= 0.8
- SDs= 0.3375
- SD1=0.15
- SDC(SDs)= C
- SDC(SD1) =C
- SDC =C

Case b (Z=0.13, Rock soil B):

- S1= 0.2475
- Ss= 0.495
- Fa= 0.9
- Fv= 0.8
- SDs= 0.297
- SD1=0.132
- SDC(SDs)= B

- SDC(SD1) =B
- SDC =B

Case c (Z=0.1, Very dense soil and soft rock C):

- S1= 0.192
- Ss= 0.384
- Fa= 1.3
- Fv= 1.5
- SDs= 0.192
- SD1=0.333
- SDC(SDs)= C
- SDC(SD1) =C
- SDC =C

Load combination for parametric study:

All structures in the cases have a seismic load and gravity load to design. The following displays the service and strength load combination in cases a, b and c.[1,2]

Service combinations:

1. 1.4D.L
2. 1.2D. L+1.6L.L
3. D.L+0.7EQ
4. D.L+0. 75L.L+0.525EQ
5. 0.6D. L+0.7EQ

strength combinations:

6. 1.4D.L
7. 1.2D. L+1.6L.L
8. 1.2D. L+EQ+L.L
9. 0.9D. L+EQ
10. D.L+0.7EQ
11. D.L+0. 75L.L+0.525EQ
12. 0.6D. L+0.7EQ

The building is composed of a resisting reinforced concrete frame, the span between columns is 5m, and the height of the story is 3.3 m. The columns are assumed to have pin supports at the base.

Superimposed dead and live loads are assumed to be 8 KN/m^2 and 4 KN/m^2, respectively. The seismic load was carried out based on the ASCE-16 code. Computer software (ETABS-18) is used to design the structures.

All the details of designing the beams and columns for each case, the seismic load and design combination loads are presented in Appendix B.

Material property

The reinforced concrete material was selected with $\acute{f}c = 28MPa$ for beams and $\acute{f}c = 32MPa$for column ,density 2.5 gm/cm^3, poisons ratio 0.2, modulus of elasticity Ec=24870.06MPa For beam concrete and Ec=26587.215MPa For column concrete, steel type ASTM A615 grade 60 (fy=413.7Mpa) used for reinforcement with modulus of elasticity Es=200000Mpa, poison ratio 0.3 and the density is 7.67gm/cm^3.

Elastic analysis

To verify and check the elastic response for 3-D models for all cases that have many checks as the following:

- Compatibility check: It Is important to ensure that all the members in the model connect well with each other. This is done by showing the deformation of the modal building and starting the animation. Figure (C.1) in Appendix C show the result of the compatibility check for one case. The compatibility check is achieved. Other cases are the same as this case.

Equilibrium check: It can be confirmed by calculating the gravity load and comparing it with the Etabs program's base reaction. Table (C.1) in Appendix C illustrates the manual calculation for gravity loads, and table (C.2) in Appendix C has the manual calculation and Etabs results for one case.

The following equation is used to calculate the error percentage between manual calculation and Etabs results for one case, and the other cases have the same process.

Dead load:

The error between manual calculation and ETABS results:

$$\frac{manual\ dead\ load - ETABS\ dead\ load}{manual\ dead\ load} \times 100\% = \frac{8840.89 - 8705.43}{8840.89} \times 100\%$$

$= 1.53\% < 5$ ➔ the error is acceptable

Superimposed dead load :

$$\frac{manual\ calculation - ETABS\ result}{manual\ calculation} \times 100\% = \frac{12800 - 12800}{12800} \times 100\%$$

$= 0\% < 5$ ➔ the error is acceptable

live load :

$$\frac{manual\ calculation - ETABS\ result}{manual\ calculation} \times 100\% = \frac{6400 - 6400}{6400} \times 100\%$$

$= 0\% < 5$ ➔ the error is acceptable

An equilibrium check is achieved.

- A stress-strain check can be done by manually calculating the moment and deformation and comparing it with the ETABS program. The minimum and maximum moment are calculated manually as equations (3.22) and equation (3.23), then display the moment in the ETABS program and check if it exceeds the minimum and maximum value.

For case M0.66-Z0.15-C:

$$As_{minimum} = 0.0033 * h * b$$
(3.22)

$$\emptyset \times Mn = 0.9 \times As_{minimum} \times fy \times (h - As_{minimum} \times \frac{fy}{(1.7 \times fc \times b)})$$
(3.23)

Where:

$\emptyset$: reduction factor and this for moment=0.9

Mn: nominal moment (KN.m)

$As_{minimum}$: the minimum area of steel in 1 meter of slab

fy: the yield strength of the steel

h : The depth of the slab

fc: the compression strength of concrete

b: the width of the slab, and in this equation, b=1m.

So, to calculate the moment:

$As_{minimum} = 0.0033 \times 150 \times 1000 = 499.5mm$, 4φ14mm/m

$$As_{used} = 4 * \frac{14^2 \times 3.14}{4} = 615.44mm$$

$$\emptyset \times Mn = 0.9 \times 615.44 \times 413.7 \times \left(150 - 615.44 \times \frac{413.7}{1.7 \times 28 \times 1000}\right) \times 10^{-6}$$
$$= 33.14kN.m$$

Figure (C.2) in Appendix C shows the moment value between the minimum and maximum values of Mn for one case (case M0.66-Z0.15-C). So, the check is achieved. This check was achieved for all cases as the same as case M0.66-Z0.15-C.

- Elastic period: It can be approved by calculating the structure period by the Rayleigh method, which depends on calculating the mass and stiffness for each story and comparing it with the ETABS program.

$$T_n = 2 \times \pi \times \sqrt[2]{\frac{\sum_{i=1} W_i \times \Delta i^2}{g \times \sum_{i=1} f_i \times \Delta i}}$$

(3.24)

Where:

Wi: the weight for each story at story level i (dead load+ live load+ superimposed dead load), the live load multiplied by coefficient 0.25.

Δi(m): The maximum displacement for each story when subjected to a lateral force.

fi (kN/m^2): the lateral force on each story.

g: the gravitational acceleration.

Table (C.3) in Appendix C shows the calculation for a component of the Rayleigh equation for one case.

$$Tn = 2 \times \pi \times \sqrt[2]{\frac{10790.55726}{9.81 \times 10517.392}} = 2.03 second$$

The comparison time period :

$$\frac{manual\ calculation - ETABS\ result}{manual\ calculation} \times 100\% = \frac{2.03 - 2.02}{2.03} \times 100\%$$

$= 0.5\%$ ➔ the error is acceptable

3.4.3 Modeling

3.4.3.1 Overview

A nonlinear static analysis (pushover analysis) for models in the case study was performed using ABAQUS software by following these steps:

1. Define the parts: Beams and columns were independently defined with 5m and 3.3m lengths, respectively.
2. Meshing: The line element was meshed to be suitable with the length of the element
3. Interaction: The pin rigid body interaction was defined to connect the reference point for each story with their nodes, and the lateral load was defined on the reference point.
4. Load condition: Gravity load was defined as line load, and lateral load was defined as point load.

The following strategies can be used to define plastic and elastic properties for the reinforced concrete line elements sections.

1. Fortran subroutine.
2. Script interface.
3. The Input file: means of communication between the preprocessor, usually ABAQUS/CAE, and the analysis product, ABAQUS/Standard. It contains a complete description of the numerical model. The input file is a text file that has an intuitive, keyword-based format, so it is easy to modify using a text editor if necessary. (Abaqus standard-2016).

The direct way is the input file. Before running the job, elastic and plastic properties values, which include moment-curvature relationships, Axial stress-plastic strain relationships, and distribution of steel in the section plastic hinge properties, need to be defined, and to obtain these properties accurately, Xtract software program was used for beams and columns.

3.4.3.2 Xtract sectional analysis

Xtract is a proactive software program and a simple appliance used to analyse the moment-curvature, axial force-moment interaction and capacity orbit (moment-moment interaction) for the concrete, steel and composite section. Xtract has a library that offers a standard shape by template, allowing the creation of a nonlinear analysis for any section by some minutes. Also, the user can draw any section by Xtract without the standard section and get the analysis.

To calculate the properties of any reinforced concrete section, many properties for the material and section require input, such as the shape of the section, longitudinal and shear steel distribution in the section, cover in the section, the stress-strain response for confined concrete, stress-strain response curve for unconfined concrete material, the response of stress-strain of steel material response, mesh of the section.

Xtract the discrete section to a finer mesh to get an accurate result. The results may be inaccurate if the mesh is coarse. In the analysis of the cross-section, the axial force for the element has axial force as a column and the moment in the x-axis and moment in the y-axis. Furthermore, the eccentricity of loads can be inserted in the section's information. Then, run the model, and the results can be obtained.

When the properties are gotten by XTRACT software, the axial load was defined for columns as equation (3.25):

$$P=0.25\times(fc)^{\prime}\times A \tag{3.25}$$

Where:

P: The axial load on the column section

$(fc)^{\prime}$: the compressive strength of the section

A: the gross area of the section. The axial load in beams is small, as seen in the analysis and design of the buildings, so the axial load is neglected in the Xtract program.

3.4.3.3 Input file

The section of the element was defined in the input file as a nonlinear generalized section with linear and nonlinear behaviour. The linear properties are related to linear material behavior, including the modulus of elasticity , shear modulus , poisons ratio and the density of the material. The geometric properties of the beam general section were defined. These properties are important to calculate the section's axial force, bending moment and torsion.

The section of the element was defined in the input file as a nonlinear generalized section with linear and nonlinear behavior. The linear properties are related to linear material behavior, which includes the modulus of elasticity (E), shear modulus (G), poisons ratio (ν) and the density of the material. The geometric properties of the beam general section were defined as the Area of section (A), a moment of inertia (I11, I12, I22), the torsional constant (J), the sectorial moment of section (Γ0) and warping constant of the section (Γw), Γ0 and Γw is zero for closed sections.

These properties are important to calculate the section's axial force, bending moment and torsion. Besides these properties, a nonlinear response for the section was defined in the input file as moment-curvature relationship for dimensions x and y (m1 and m2), Axial stress-strain, and torsion-twist property for each 3-D element.

Moment-curvature can be defended in the input file on the x-axis- and y-axis. The moment-curvature parameter includes the distribution of steel and the material properties, so it is necessary to get the pushover analysis results. This parameter may assume an elastic or inelastic nonlinear response, and the values must start from (0,0) and be increased and positive. Figure (A.9) in Appendix A illustrates the difference between the assumptions of the relationships.

The input file uses symbols such as (*Axial, *M1, *M2, *TORQUE) to define axial stress-strain, moment-curvature, and torsional-twist relationships, respectively. These symbols accurately represent the section's behaviour and obtain realistic and accurate results.

3.4.4 Pushover analysis

Pushover analysis indicates the plastic behaviour of the beam, which is used to include the nonlinear response of the beam, considering the beam assumptions (plane sections before bending moment remain plane after bending) . Beam behaviour contains two types: ductile behaviour and brittle behaviour. The ductile beam goes through several stages:

(a) The crack stage occurs when the concrete tension zone cracks.

(b) The yielding stage occurs when the tensile steel in the beam yields.

(c) The ultimate stage occurs when the beam or building reaches the maximum strength to resist the load.

(d) Failure stage: After the building reaches maximum strength, failure occurs. In this type, the building can resist after reaching maximum strength, and it fails gradually with some cracks in the building members until it fails. The maximum deformation is not equal to the end deformation that the building fails on. Figure (A.10. a) in Appendix A illustrates the ductile behavior of the building.

In beams with brittle failure, the ultimate stage comes before the yielding stage. When the failure occurs in the brittle behavior of the beams, the building fails directly after reaching the maximum strength, and the maximum deformation of the building equals the end deformation that the building fails. Figure (A.10. b) in Appendix A illustrates the brittle behavior of the building.

The ABAQUS program performed a pushover analysis. All members were represented as 3d line elements with nonlinear generalised geometry sections and defined the elastic properties and inelastic behavior of the sections in the input file. In addition to axial and torsional behaviour, the m-phi curve was defined in each section direction.

The pushover curve is controlled by displacement on the joints in each story. The loads are transmitted from slab to ground, so the beams have moment, torsion and shear loads. In addition, columns have an axial, moment, torsion and shear. Because the structures were regular, there is no torsion, so the torsion is omitted by defining a constant relationship between torsion and twist for all sections and between axial and strain for beam section.

3.4.5 Defining the load conditions for pushover analysis:

The pushover curve can be affected by gravity and lateral loads. Both loads were defined as nonlinear static general by the Abaqus program. The gravity load (live load, superimposed dead load and the self-weight of the slab) was distributed on the beams as line load. The lateral load performed on each node in the stories is a displacement load that gradually increases as we go up in the floors.

Chapter four

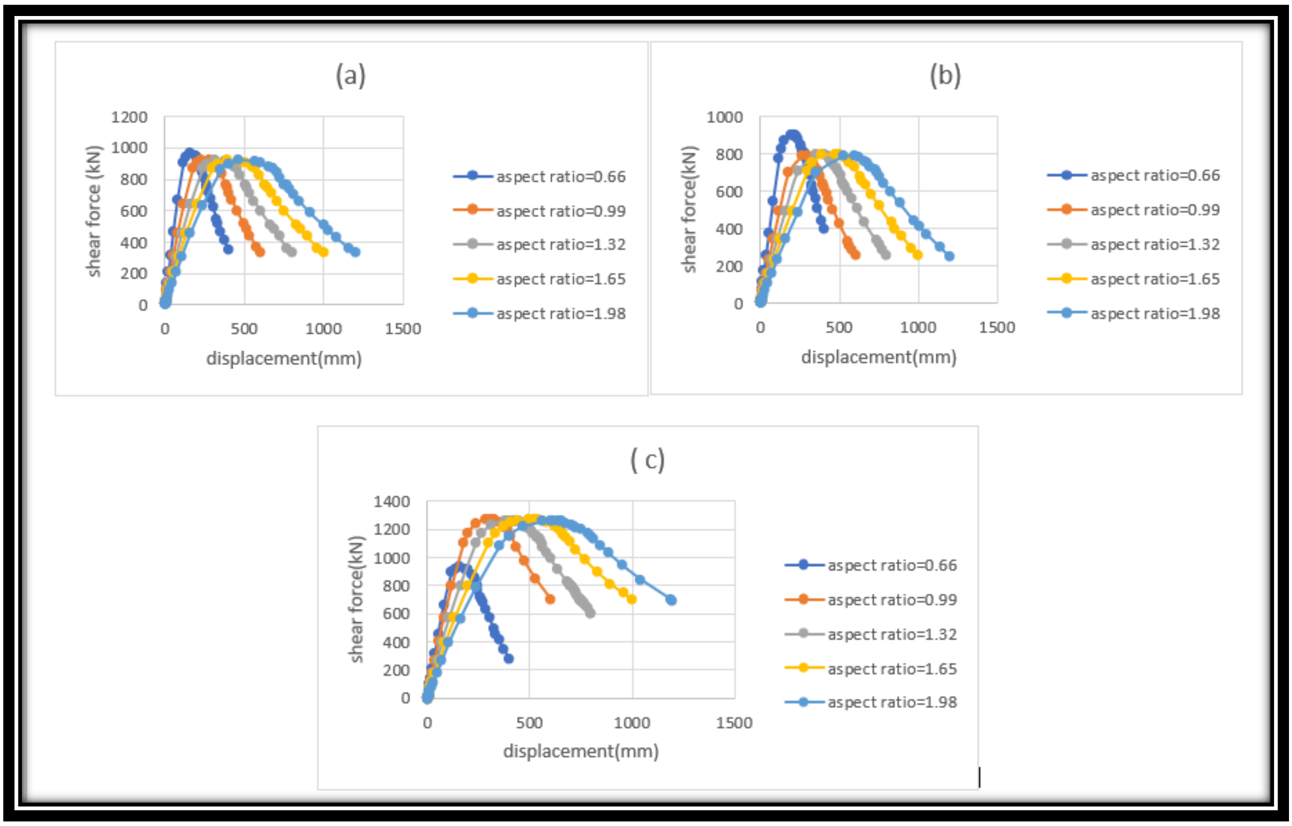
Results of pushover analysis

4.1 Pushover analysis for cases

The pushover curve was obtained from the analysis of structures. The displacement of the roof story controls the behavior of the structure. The structure resists this deformation until ultimate strength is achieved and fails. Push-over curve results can be used to calculate the response modification factor for the structure. The figures below illustrate the comparison between the behavior of different cases.

Figure 4.1

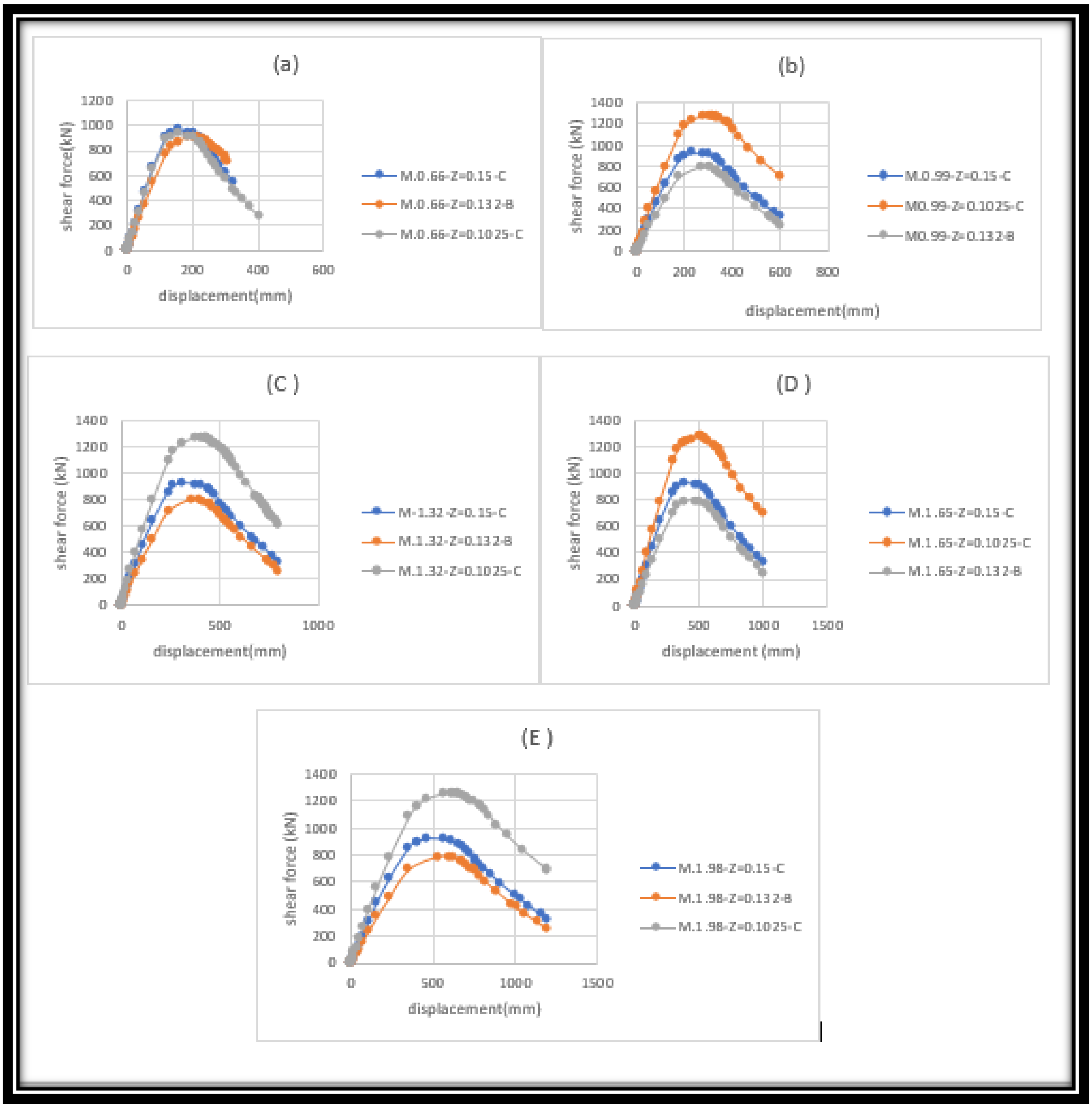
The comparison of P-Δ curve based on aspect ratio for structure for square plan case.



(a) when Z=0.15, Rock soil and seismic design category is C (b) when Z=0.132, Rock soil and seismic design category is B. (c) when Z =0.1025, very dense soil and soft rock and seismic design category is C.

Figure 4.2

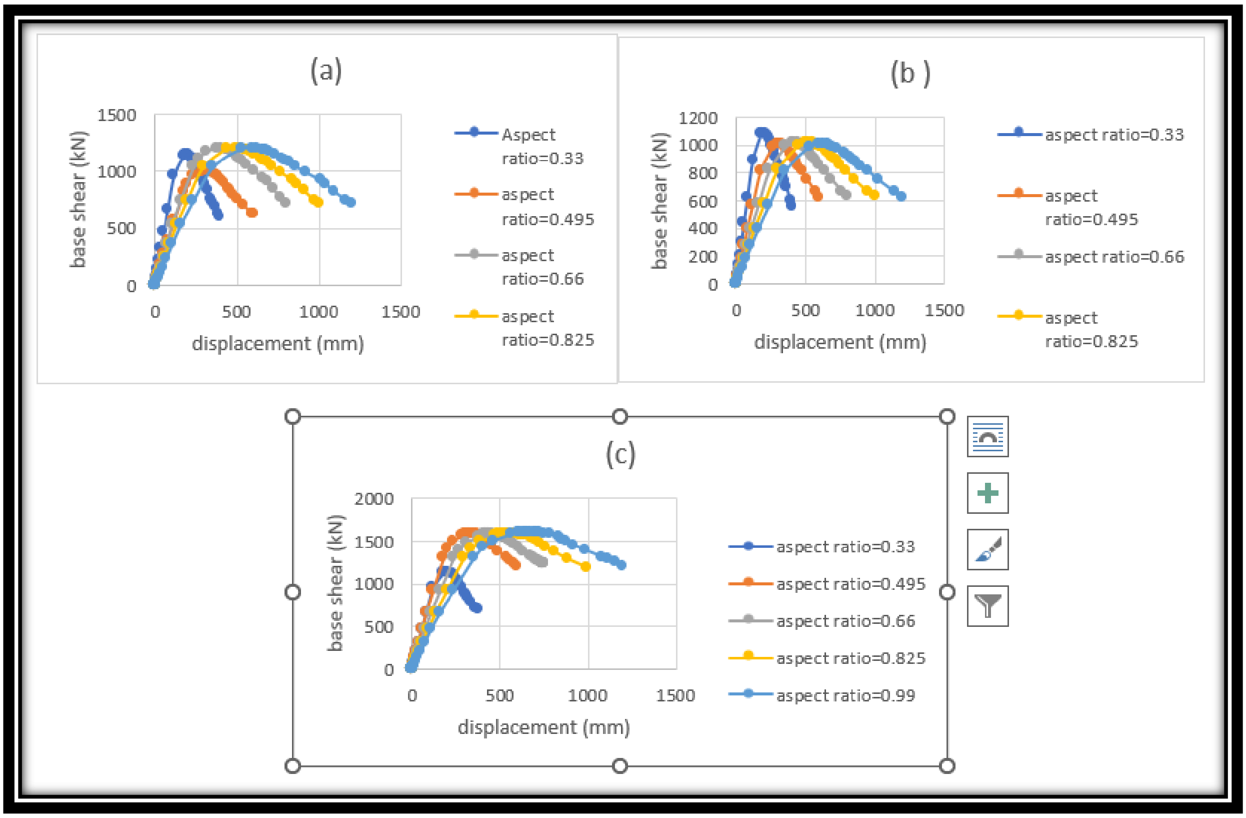
Comparison between models in square plan case for seismic design category and variation of zone factor with variation of type of soil.



(a)for the models have aspect ratio 0.66. (b) aspect ratio 0.99. (c) aspect ratio 1.32. (d) aspect 1.65.(E) aspect ratio 1.98.

Figure 4.3

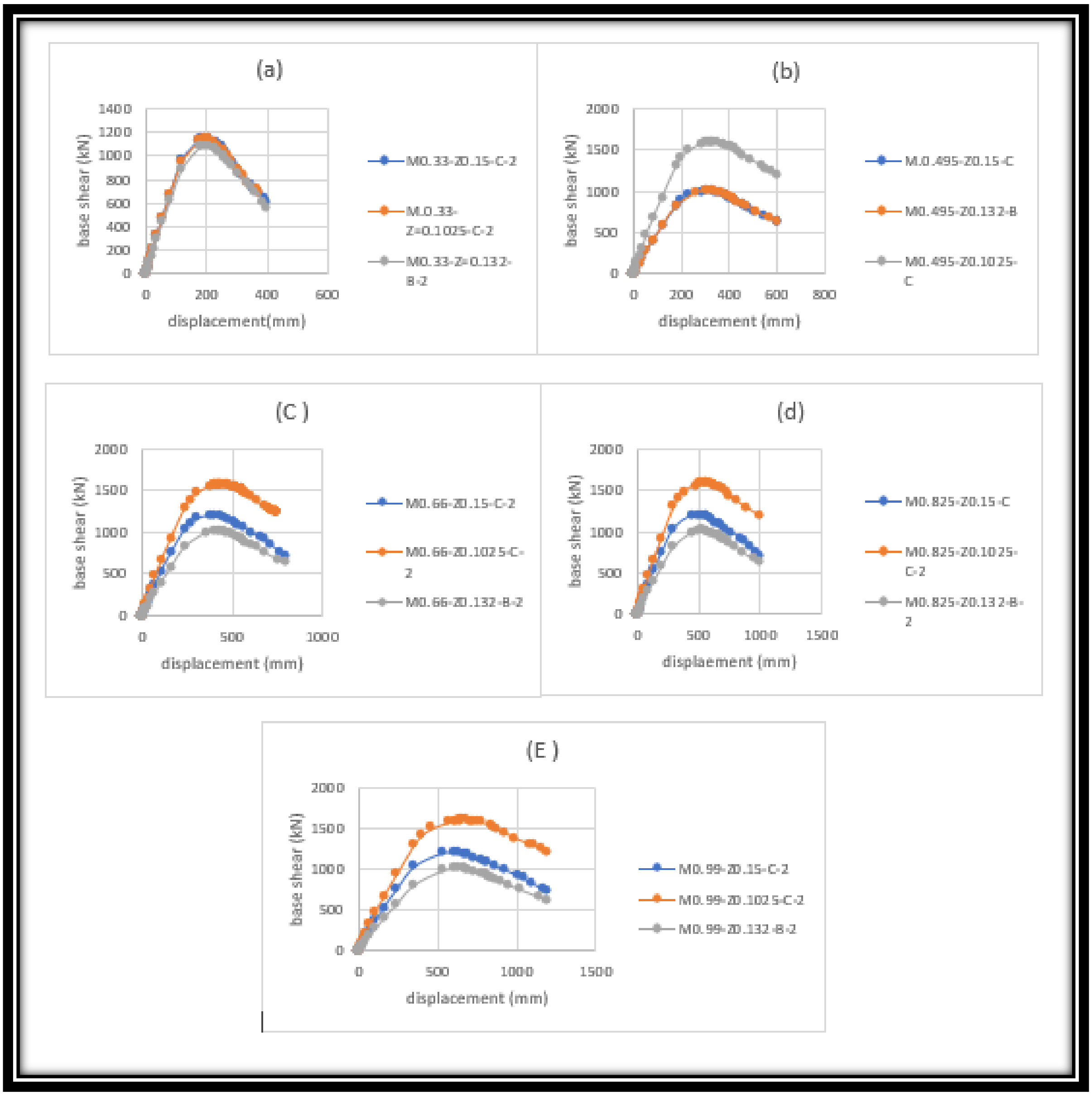
The comparison of P-Δ curve based on aspect ratio for structure for rectangle plan case.



(a) when Z=0.15, Rock soil and seismic design category is C (b) when Z=0.132, Rock soil and seismic design category is B. (c) when Z =0.1025, very dense soil and soft rock and seismic design category is C.

Figure 4.4

Comparison between models in rectangle plan case for seismic design category and variation of zone factor with variation of type of soil.



(a)for the models have aspect ratio 0.33. (b) aspect ratio 0.495. (c) aspect ratio 0.66. (d) aspect ratio 0.825.(E) aspect ratio 0.99.

4.2 Curve behavior description

Figure (4.1) and (4.3) shows the effects of aspect ratio on the behavior of the building in the same zone factor and seismic design category. In most cases, the yielding strength of the building to resist the load (V_y) obviously decreases. The yielding displacement increases when the aspect ratio increases—also, the ductility and the capacity of the building increase with increasing the aspect ratio.

Figures (4.2) and (4.4) show the effects of the zone factor on the building behaviour at the same aspect ratio. The value of (V_y) is affected by changing the seismicity and the site category. When the zone factor decreases at the same time as the seismic design category, the yielding displacement increases and the yielding force increases, then the capacity and ductility of the building increase. When the seismic design category changed from C to B, the yielding force and displacement decreased, and the building capacity decreased. This is due to changes in the design of the members of the building.

In all cases, the pushover curve has a ductile failure after reaching the maximum shear force. This indicates that the buildings in this study have ductile behavior.

4.3 Response modification factor

To know the valuation of the structures and if aspect ratio, seismicity of the region and type of soil influence the building performance. The response modification factor can be calculated depending on the pushover curve yield base shear value obtained when the first plastic hinge occurs. Elastic base shear is calculated based on ASCE-16 code recommendation as mentioned in the previous section considers R=1. [ASCE7-16, section 12.8].

Table 4.1

Summary of R computation for all cases

	aspect ratio	W(KN)	Cs	Ve(KN)	Vy(KN)	R
M.0.66-Z.0.15-C-1	0.66	24257.66	0.039	4730.24	908.90	5.20
M.0.99-Z.0.15-C-1	0.99	36528.28	0.027	4931.32	864.27	5.71
M.1.32-Z.0.15-C-1	1.32	49180.47	0.021	5188.54	862.22	6.02
M.1.65-Z.0.15-C-1	1.65	62307.03	0.017	5358.40	861.46	6.22
M.1.98-Z0.15-C-1	1.98	75763.59	0.015	5530.74	859.36	6.44
M.0.66-Z.0.132-B-1	0.66	24228.28	0.034	4106.69	773.40	5.31
M.0.99-Z.0.132-B-1	0.99	36579.84	0.024	4298.13	697.50	6.16
M.1.32-Z.0.132-B-1	1.32	49113.44	0.018	4444.77	702.51	6.33
M.1.65-Z.0.132-B-1	1.65	62240	0.015	4605.76	701.86	6.56
M.1.98-Z0.132-B-1	1.98	75232.5	0.013	4890.11	695.06	7.04
M.0.66-Z0.1025-C-1	0.66	19965.2	0.050	4991.30	888.70	5.62
M.0.99-Z0.1025-C-1	0.99	40553.44	0.035	7096.85	1099.69	6.45
M.1.32-Z0.1025-C-1	1.32	54215.63	0.027	7319.11	1095.28	6.68
M.1.65-Z0.1025-C-1	1.65	68604.84	0.022	7546.53	1096.80	6.88
M.1.98-Z0.1025-C-1	1.98	83841.34	0.019	7881.09	1082.31	7.28
M.0.33-Z.0.15-C-2	0.33	24325.71	0.039	4743.51	963.46	4.92
M.0.495-Z.0.15-C-2	0.50	36725.23	0.027	4957.91	966.73	5.13
M.0.66-Z.0.15-C-2	0.66	49703.91	0.021	5243.76	1041.40	5.04
M.0.825-Z.0.15-C-2	0.83	63027.84	0.017	5420.39	1041.14	5.21
M.0.99-Z0.15-C-2	0.99	76708.18	0.015	5599.70	1039.78	5.39
M.0.33-Z.0.132-B-2	0.33	24515.04	0.034	4155.30	895.40	4.64
M.0.495-Z.0.132-B-2	0.50	36947.98	0.024	4341.39	818.41	5.30
M.0.66-Z.0.132-B-2	0.66	49631.51	0.018	4491.65	823.75	5.45
M.0.825-Z.0.132-B-2	0.83	62955.45	0.015	4658.70	823.52	5.66
M.0.99-Z0.132-B-2	0.99	76134.6	0.013	4948.75	817.28	6.06
M.0.33-Z0.1025-C-2	0.33	20020.35	0.050	5005.09	951.70	5.26
M.0.495-Z0.1025-C-2	0.50	40983.71	0.035	7172.15	1301.27	5.51
M.0.66-Z0.1025-C-2	0.66	54800.88	0.027	7398.12	1298.85	5.70
M.0.825-Z0.1025-C-2	0.83	69403.23	0.022	7634.36	1300.00	5.87
M.0.99-Z0.1025-C-2	0.99	84920.64	0.019	7982.54	1311.25	6.09

Also, Table (4.2) gives the R factor calculated according to common codes (UBC-97, IBC-2016, Eurocode, Egyptian code).

Table 4.2

R factor according to some different codes for reinforced concrete intermediate moment resisting frame

Code	R factor for reinforced concrete (IMRF)
UBC-97	5.5
IBC-2016	5
EC-8	5
Egyptian code	5-7
NBCC-2005	3.5

Table (4.2) shows that in most cases, when the aspect ratio increases, the weight and seismicity factor of the building increases, and the elastic base shear increases; however, the yielding base shear obtained from the pushover curve of the cases decreases, and the response modification factor (R) increases.

Every code has a different value for the response modification factor, and this value is in the range between 3.5 and 7. According to ASCE -16, the intermediate moment resisting frame has a response modification factor of R 5. In the results of pushover analysis and studying the performance of buildings by nonlinear static analysis, the R-value was too close to 5, but when increasing the number of floors or the height of the building, the value of R was almost greater than the value recommended by ASCE-16 code.

4.4 Discussion of results

The following is a clarification of how the parameters in this study have an impact on the R-value:

1. Aspect ratio: From the results of the R calculation in the previous section, an increase in the building's height or aspect ratio increases the R factor, which is due to an increase in the force of the building to resist the earthquake when the building behaves elastically (elastic base shear), because when the aspect ratio increases the weight of the building increases. The seismic response coefficient decreases (Cs),

then the elastic base shear results from multiplying the weight, and the Cs factor reduces the ability of the building to resist the earthquake when the building behaves inelasticity (yielding base shear), and this is due to many reasons such as:

A. Distribution of the stiffness in the building: An increase in the aspect ratio means an increase in the height of the building, which means more flexibility on the building, leading to a higher deformation under exposure to lateral load. Therefore, the stiffness of the building decreases, and base shear also decreases.
B. P-delta effect: In the taller building, when the axial load is considered in pushover analysis, the effect of the p-delta will be obvious, which leads to a decrease in the lateral load demand, then a decrease in the base shear.

2. The time period is considered in calculations of the R factor in IBC2016. This indicates that the aspect ratio of the building is implicitly considered in these calculations.
3. Seismic zone factor: It can be seen that when the zone factor increases, the seismic response coefficient (Cs) decreases. The weight of the buildings decreases, and subsequently, the elastic base shear decreases. Also, the yielding force decreases, so the response modification factor decreases, and this indicates that the ductility and capacity of the building are decreased. That means the increase of zone factor leads to a decrease in the ductility and capacity of the building.
4. Type of soil: From the results of the calculation R factor noticed that very dense soil and soft rock (site class C) has seismic response coefficient (Cs) and weight of structure (W) values higher than rock soil (site class B) for the same seismic design category, that’s mean when the site class is weak, the elastic base shear (Ve) increases because the weakness soil greatly affected by seismicity forces and this will end up with long deformation in the building, and that’s leads that the R factor should be increased.
5. Seismic design category: noticed that when seismic design category (B) has the value of R is higher than seismic design category (C) for the same type of soil (rock soil) with changes in the zone factor.

After analyzing the results of the cases, the effects of aspect ratio and seismicity on the value of R are determined quantitatively using multiple regressions for the 24 cases, and this is represented in equation (4.1):

$$R = (1.18 \times Aspect\ ratio) - (2.98 \times SDs) + 5.62 \quad (4.1)$$

The error is the difference between R from the pushover analysis and R from the equation. The final results obtained from the relationship between the aspect ratio and error are random. This relation is shown in Figure (4.5.a).

R factor is also computed for another 6 cases to validate the regression, and the relationship between R from the equation and R from a pushover analysis is plotted in Figure (4.5.b).

Table 4.3

The regression cases calculations

R	aspect ratio	SDs	Req.	Error
5.2	0.66	0.3375	5.39305	-0.19
6.02	1.32	0.3375	6.17185	-0.15
6.22	1.65	0.3375	6.56125	-0.34
6.44	1.98	0.3375	6.95065	-0.51
5.31	0.66	0.297	5.51374	-0.20
6.16	0.99	0.297	5.90314	0.26
6.56	1.65	0.297	6.68194	-0.12
7.04	1.98	0.297	7.07134	-0.03
6.45	0.99	0.33	5.8048	0.65
6.68	1.32	0.33	6.1942	0.49
6.88	1.65	0.33	6.5836	0.30
7.28	1.98	0.33	6.973	0.31
5.13	0.50	0.3375	5.19835	-0.07
5.04	0.66	0.3375	5.39305	-0.35
5.21	0.83	0.3375	5.59365	-0.38
5.39	0.99	0.3375	5.78245	-0.39
4.64	0.33	0.297	5.12434	-0.48
5.45	0.66	0.297	5.51374	-0.06
5.66	0.83	0.297	5.70844	-0.05
6.06	0.99	0.297	5.90314	0.16
5.26	0.33	0.33	5.026	0.23
5.51	0.50	0.33	5.2207	0.29
5.70	0.66	0.33	5.4154	0.28
6.09	0.99	0.33	5.8048	0.29

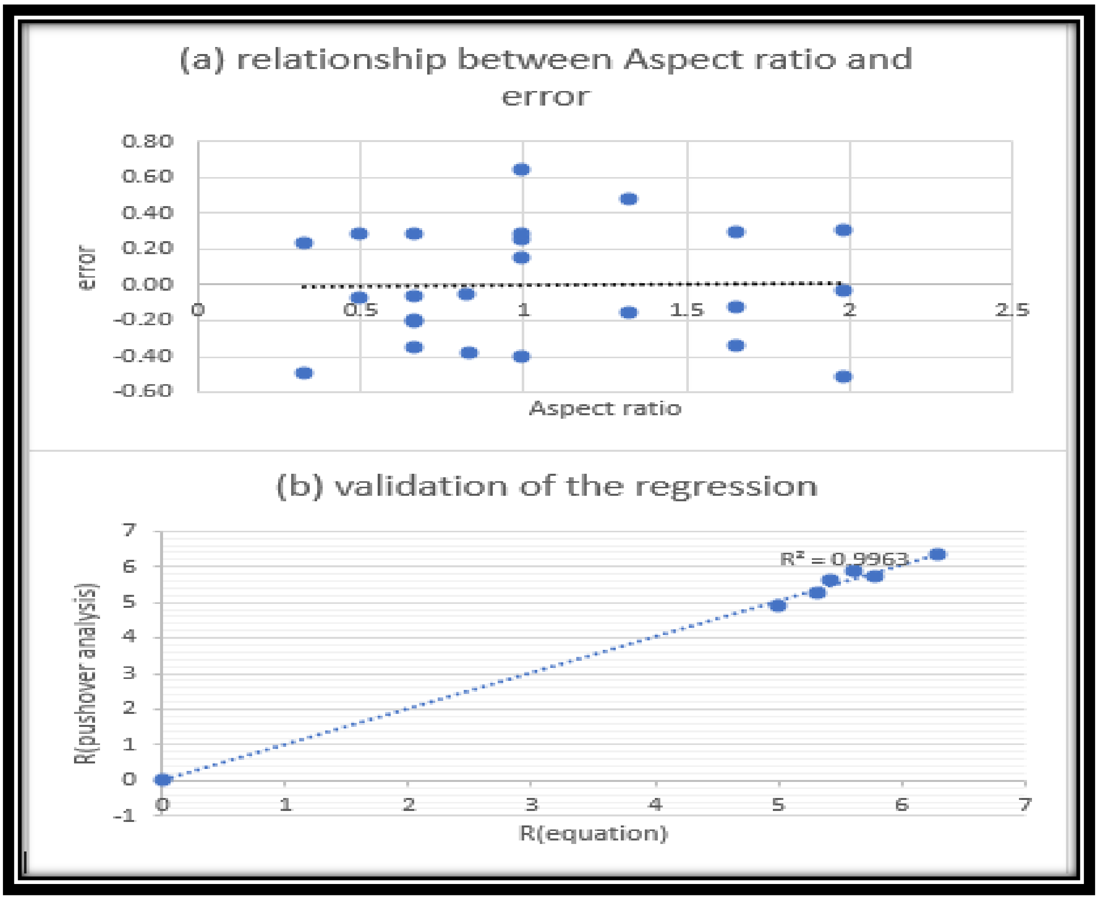
Table 4.4

the validation cases calculations

R	aspect ratio	SDs	Req	Error
0	0	0	0	0
5.62	0.66	0.33	5.42	0.20
5.71	0.99	0.3375	5.78	-0.07
6.33	1.32	0.297	6.29	0.04
4.92	0.33	0.3375	5.00	-0.08
5.30	0.50	0.297	5.32	-0.02
5.87	0.83	0.33	5.61	0.26

Figure 4.5

(a) the relationship between Aspect ratio and the error for regression of the cases (b) the validation of the regression.



The results showed that R computed from equations is too close to R computed from pushover analysis. The slope will be 1, and the error is too small. As a result, the equation is reliable for regular buildings.

Chapter five

Conclusion and recommendation

5.1 Summary

This thesis highlighted how different the values of the R coefficient of the (IMRF) are. The R values recommended in global codes, in general, and IBC code, in particular, were compared with those calculated through the pushover analysis for multiple cases.

Five cases with different building aspect ratios were adopted for regular plans to study the effect of the differences in the aspect ratio of buildings on the R-value. Also, fifteen cases were adopted to inspect the effect of differences in seismic design categories with various seismic zone factors (Z) and soil type on the R factor. The cases were designed and analyzed according to ACI code. The load-displacement curve was obtained for all cases, and the R factor was calculated depending on the pushover curves.

5.2 Conclusion

Based on the studies in this thesis and the results obtained, conclude the following:

- Based on single cycle calculation, the value of the R factor according to the IBC 2016 code is conservative in term of base shear. IBC2016 code considered the aspect ratio implicitly in the calculation of R by applying the time period of the building in the calculation of the R factor.
- According to the study, the required ductility, as presented by R, is higher than that provided by international codes. Further study is needed to investigate if the provided detailings by codes satisfy the needs.
- The regression analysis can be used to calculate the R factor by considering the aspect ratio and the seismicity of the region by using equation (4.1).

5.3 limitation of this study and future recommendation

- The plans in cases were regular, and the torsion was neglected, and this assumption simplified the analysis and the calculations.
- When pushover analysis was applied, the building was assumed as (SDOF) for the multi-story structures. This is because the lateral load performed on the building in one direction, so the building moved in the load direction.

- The axial force in the columns was assumed as equation $Pn = 0.25 \times A \times fc$, and the axial force on the beams was assumed as zero.
- In the future, many studies should be applied as:

1. This study can be repeated using another analysis method, such as nonlinear dynamic analysis (time history analysis) and comparing pushover and time history results.
2. Another type of building system can be dual or building frame systems.
3. Many other parameters can be checked if they affect the R-value, such as the type of slabs, the irregularity in the distribution of beams in slabs, the irregularity in elevation, and the irregularity in the horizontal plane in the building.

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Appendices

Appendix A

Figures in thesis

Figure A.1

Elastic and inelastic behavior of structures

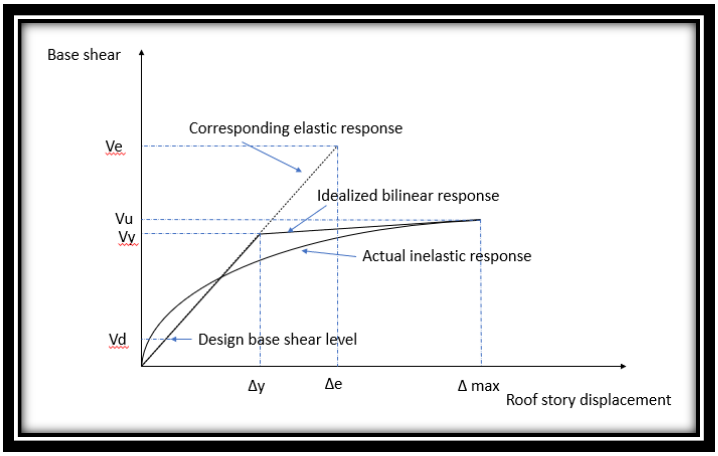


Figure A.2

The capacity curve of the structures with performance level

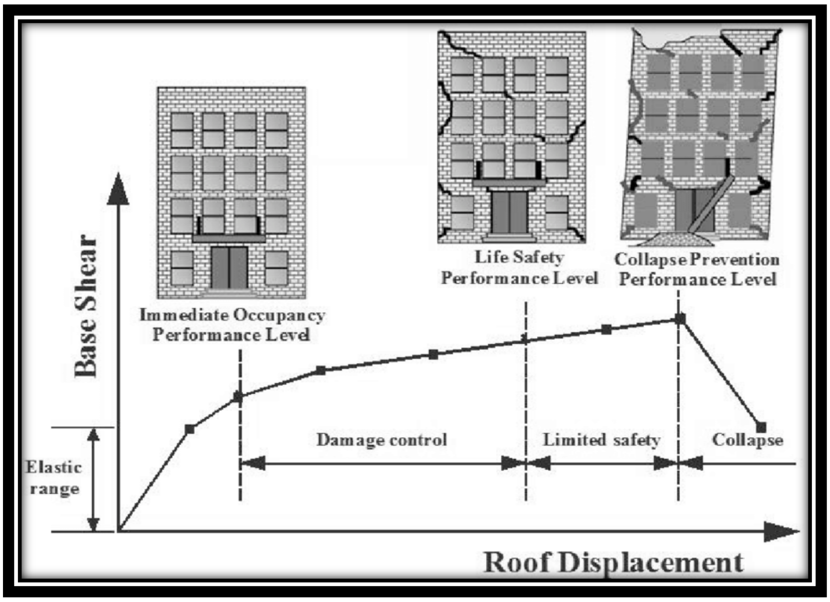
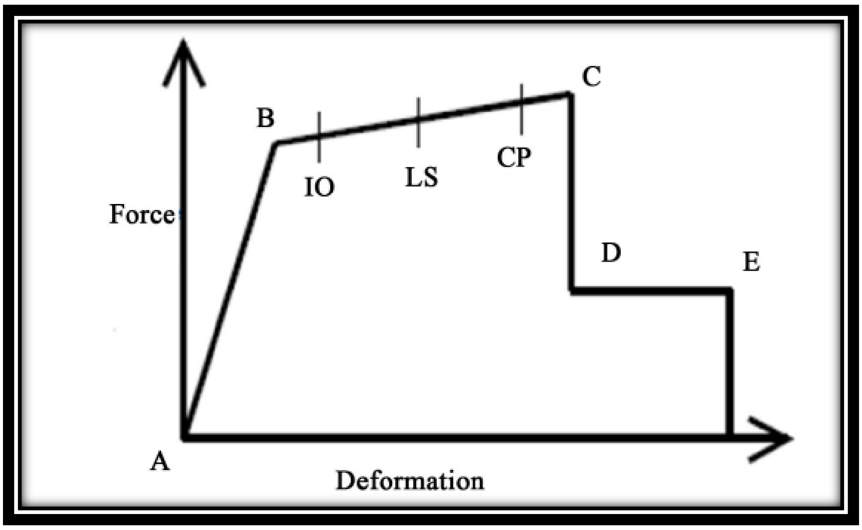


Figure A.3

Acceptance criteria in the load-deformation curve for component of the building depend on ASCE/SEI7-16, FEMA -356 and ATC-40



(ASCE/SEI-16)

Figure A.4

yielding and ultimate strain and stress cases for doubly reinforced beam

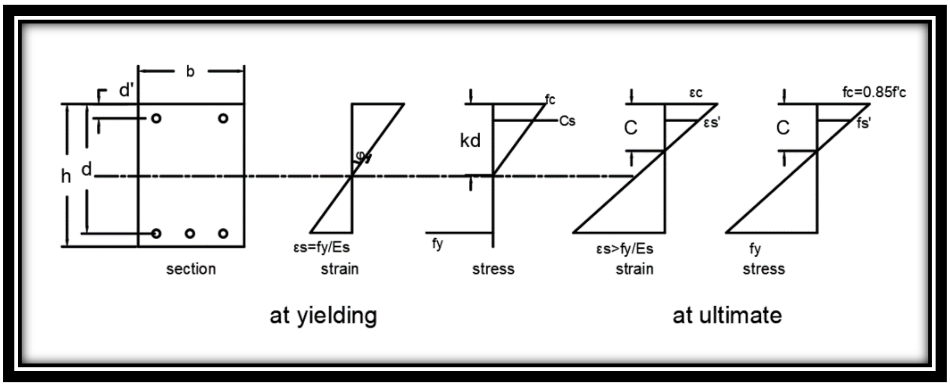
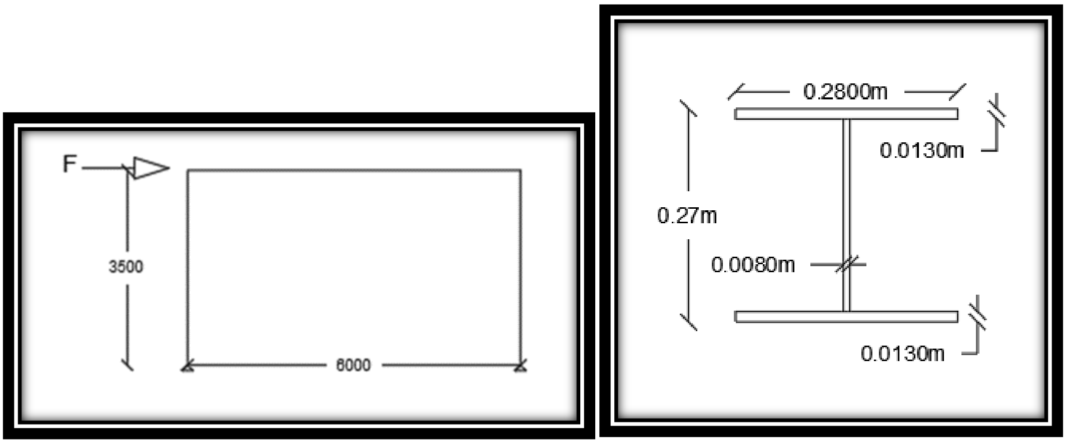


Figure A.5

dimensions of members for simple steel frame used in verifications



(a) Dimensions of simple steel frame ***(b) dimensions of (HE280A)section***

Figure A.6

the dimensions of simple 2-D R.C frame used in verifications

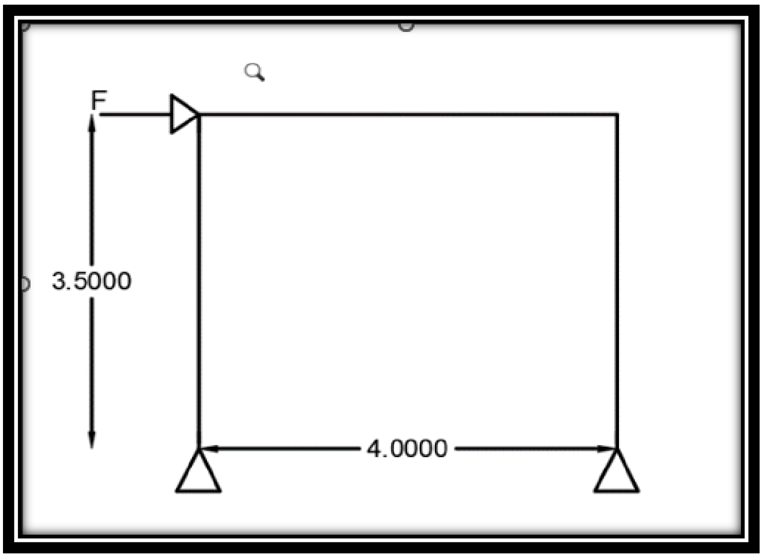


Figure A.7

uniaxial stress- strain behavior for steel used in verifications

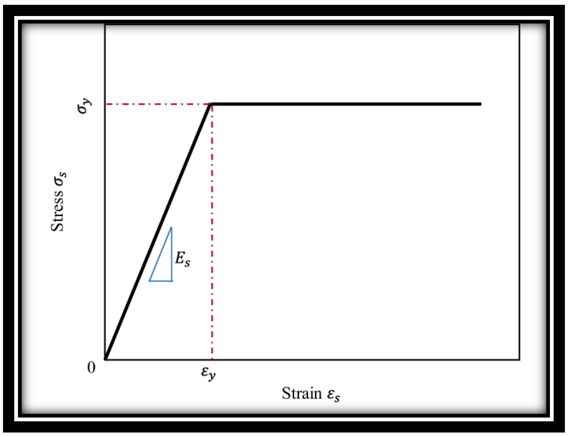
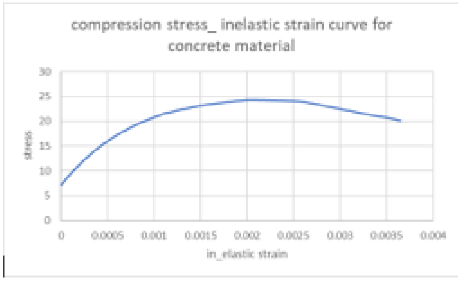
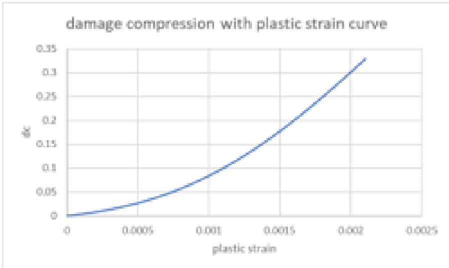


Figure A.8

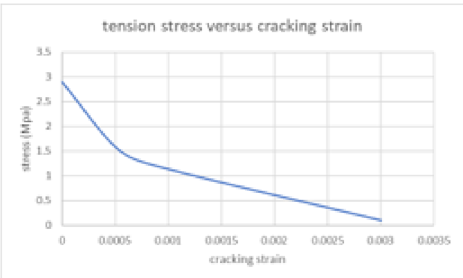
the parameters of stress strain curve for concrete material (B300) used in verification



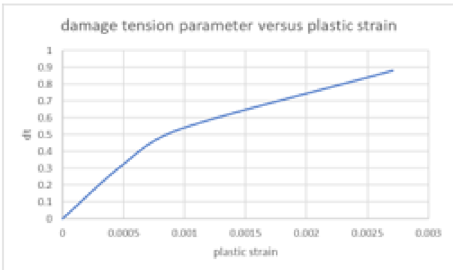
(a) Uniaxial compression stress - inelastic strain plastic curve for concrete material (B300)



(B)Damage compression parameter_ strain curve for concrete material(B300)



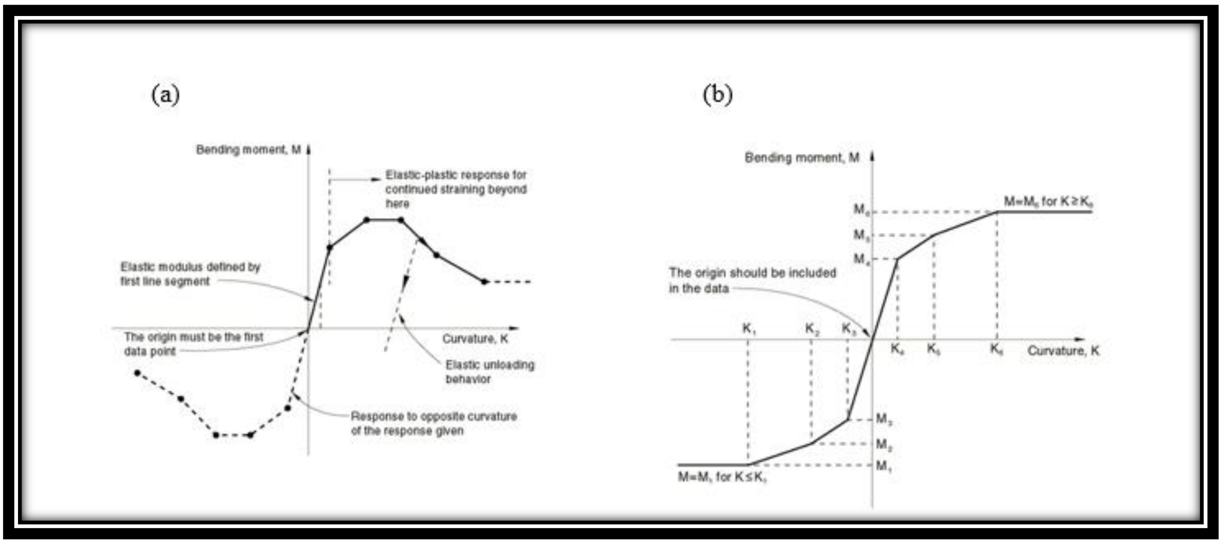
(c)tension stress-cracking strain curve strain for concrete material (B300)



(d)tension damage parameter-plastic curve for concrete material (B300)

Figure A.9

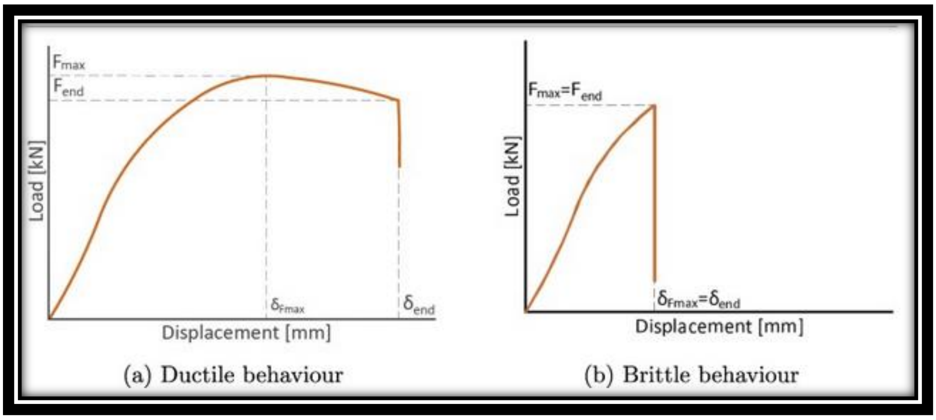
The difference between



(a)elastic nonlinear response and(b) inelastic nonlinear response for the nonlinear general beam section.

Figure A.10

The difference between brittle and ductile behavior



Tables in thesis

Table A.1

The properties of steel material and section used in steel simple frame case

Yielding strength (Fy)	248.211 Mpa
Ultimate strength(Fu)	250 Mpa
Yielding strain (εy)	0.00124
Ultimate strain (εu)	0.002
Elastic modulus (E)	200000 Mpa
Section modulus (Sx)	$1.846*10^{-4}$ m^3
Plastic modulus (Zx)	$2.097*10^{-4}m^3$
Moment of inertia (I)	$1.846*10^{-5}m^4$

Table A.2

the detailing of members in (a) simple 2-D RC frame used in verification(b) simple 3-D frame

Table (A.2.a)

	dimension	reinforcement
beam	0.3*0.4	top:3φ18 bottom:3 φ18
column	0.4*0.4	6φ18

Table (A.2.b)

	dimension	reinforcement
beam	0.3*0.4	top:2φ16 bottom:3 φ16
column	0.5*0.5	12φ20

Table A.3

the parameters of concrete material to define in ABAQUS program

Density(g/mm^3)	Ec (Mpa)	ν	Dealation angle(Ψ)	Eccentricity	fb_0/fc_0	K	Viscosity
2.5E-006	20800	0.199	36	0.1	1.16	0.667	0

Table A.4

the parameters of steel material to define in ABAQUS program

Density(g/mm^3)	Es (Mpa)	Poisons ratio (ν)
7.85E-006	205000	0.291

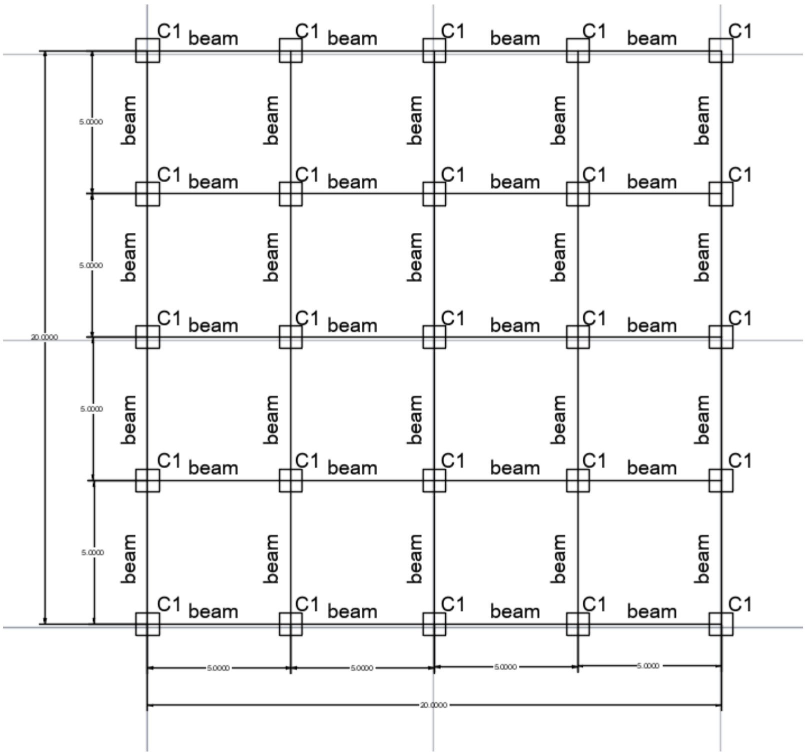
Appendix B

structural plan:

all cases have a square geometry plane (20m*20m), and the spacing between columns is 5m. the height of story is 3.3m. figure (B.1) display the column and beams distributing in plan (a)square plan (b)rectangle plan. and figure (B.2) display the variation of height for the cases.

figure B.1

a)square plane



b) rectangle plan

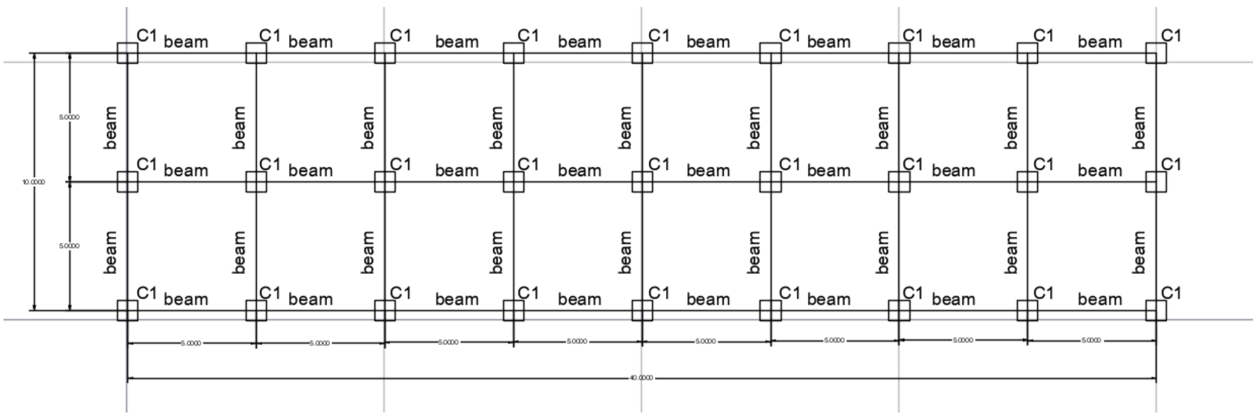
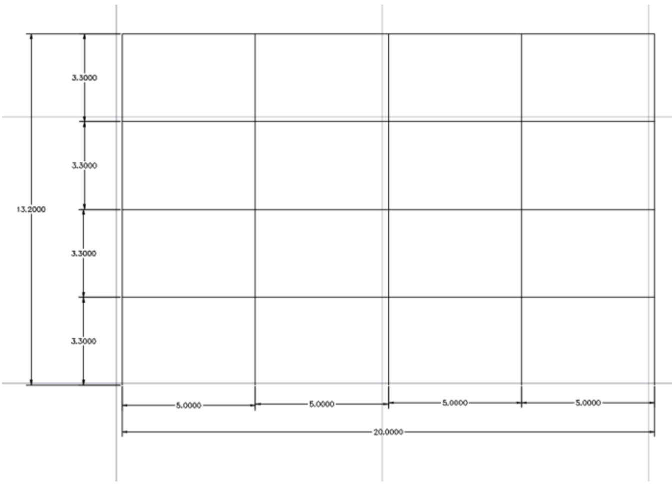
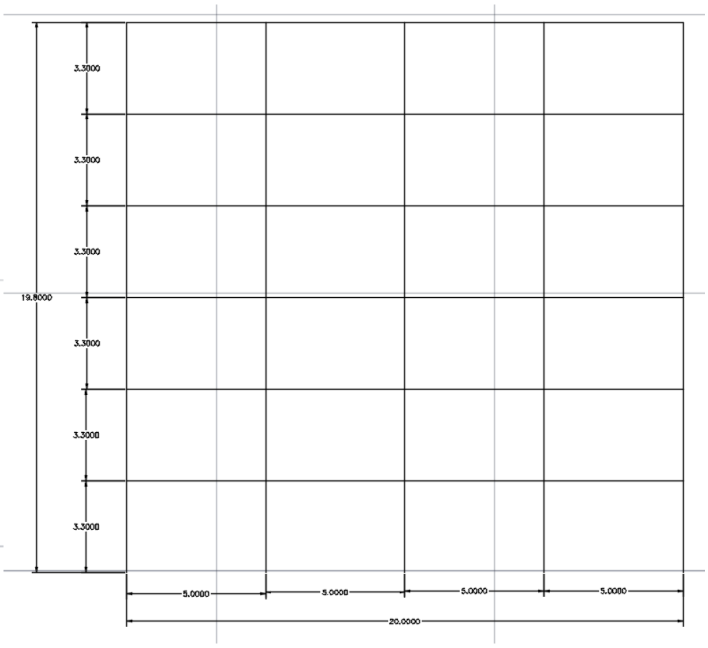


figure B.2

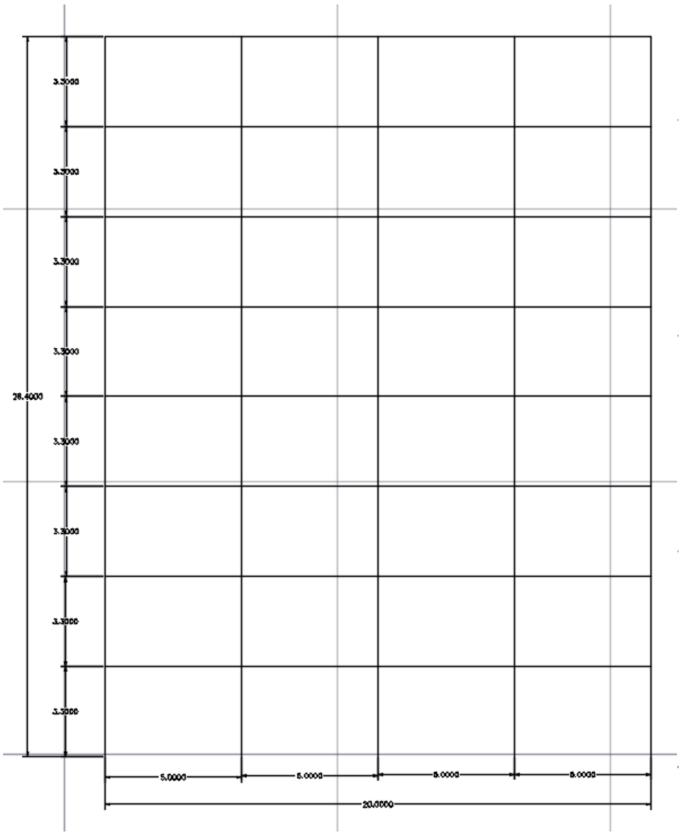
a) square plan- aspect ratio=0.66



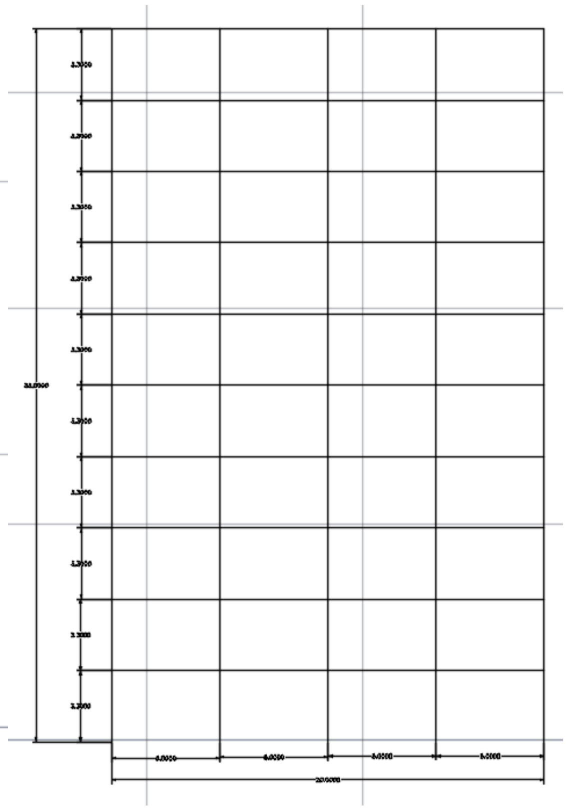
b) square plan- aspect ratio=0.99



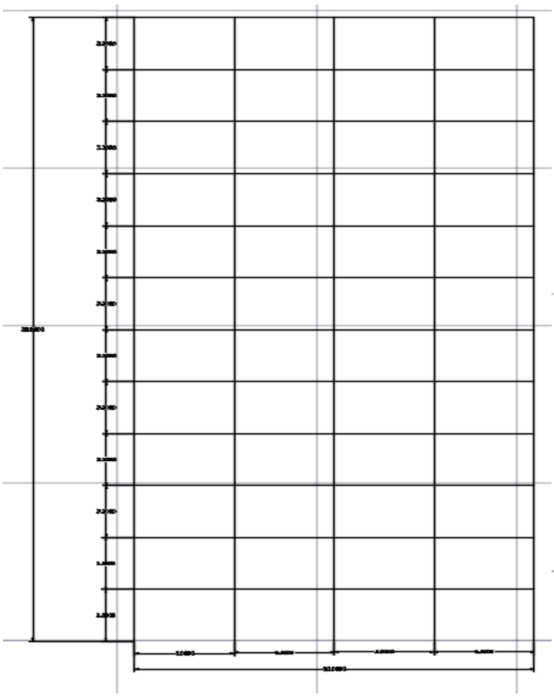
C) square plan- aspect ratio=1.32



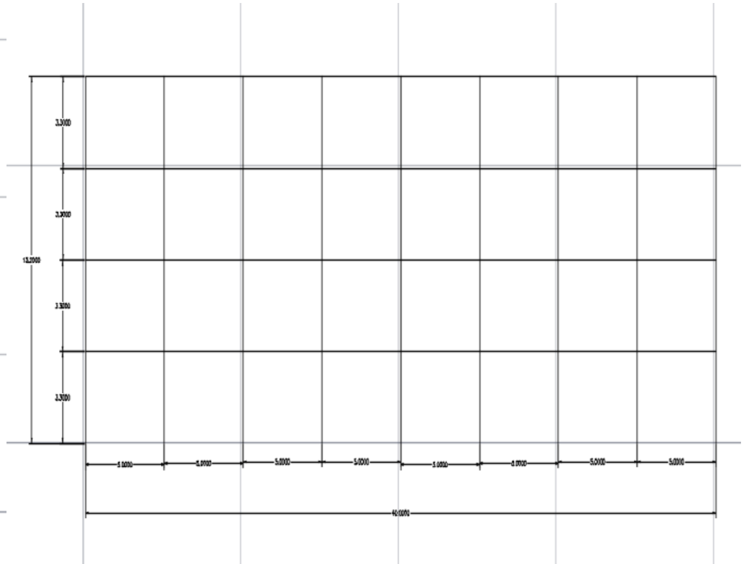
d) square plan- aspect ratio=1.65



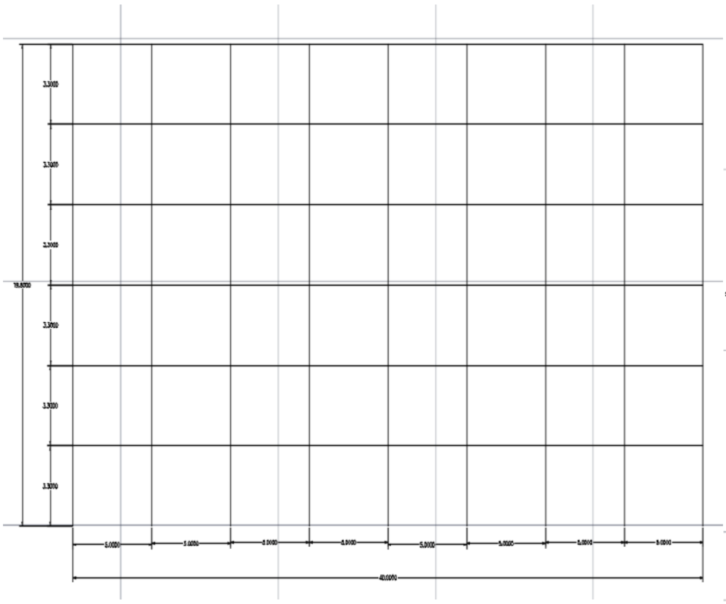
E)square plan-aspect ratio=1.98



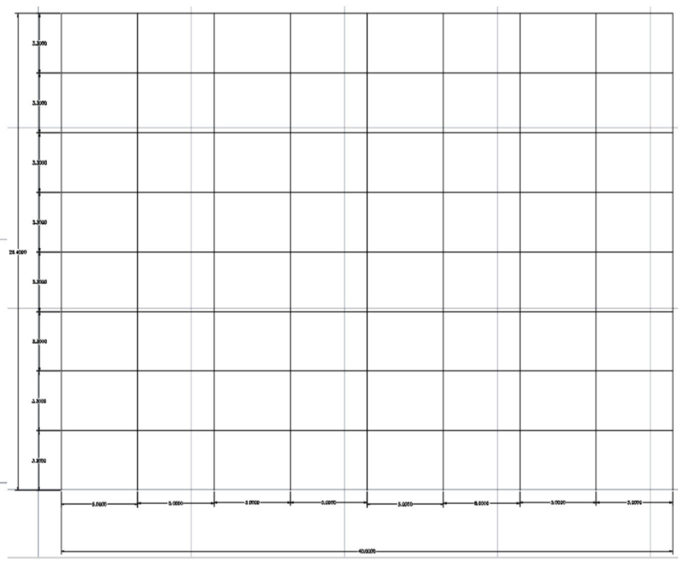
F)rectangle plan-aspect ratio=0.33



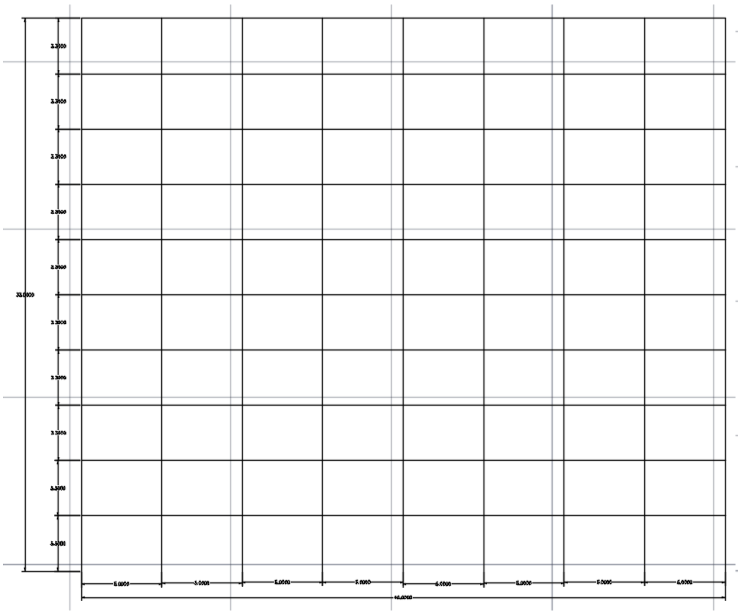
G) rectangle plan-aspect ratio=0.495



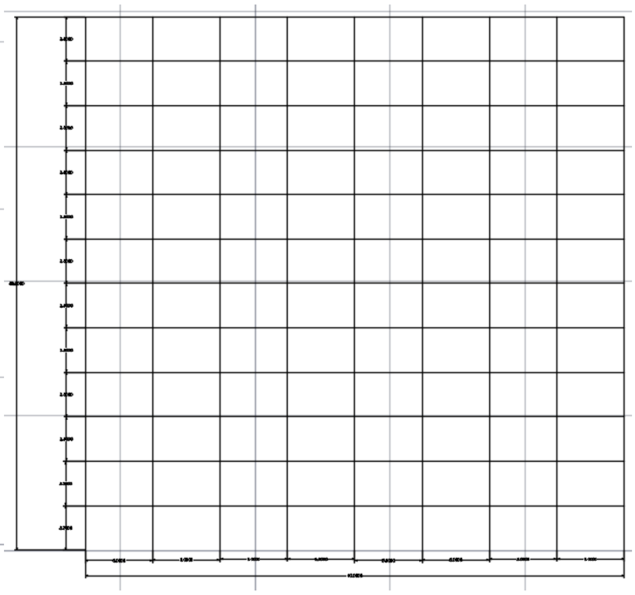
H) rectangle plan-aspect ratio=0.66



I)square plan-aspect ratio=0.825



J) rectangle plan-aspect ratio=0.99



Seismic loads for parametric study:

The cases designed to resist the earthquake, so two cases submitted in Ramallah,Qalqilya and Gaza city with different type of soil as Rock soil and very dense soil and soft rock. table (B.1) display the details of seismic load in case a, b and c.

Table B.1

display the Details of seismic loads for case a, b and c.

	case a	**case b**	**case c**
site class	(Rock soil) B	(Rock soil) B	(Dense soil) c
Z	0.15	0.132	0.1025
S1	0.28125	0.2475	0.1921875
Ss	0.5625	0.495	0.384375
Fa	0.9	0.9	1.3
Fv	0.8	0.8	1.5
SDs	0.3375	0.297	0.1921875
SD1	0.15	0.132	0.333125
SDC (Sd_s)	C	B	C
SDC (Sd_1)	C	B	C
SDC	C	B	C

Load combination for parametric study

All structures in the cases have a seismic load and gravity load to designing, in the following display the service and strength load combination in case a, b and c.

Service combinations:

13. 1.4D.L
14. 1.2D. L+1.6L.L
15. D.L+0.7EQ
16. D.L+0. 75L.L+0.525EQ
17. 0.6D. L+0.7EQ

strength combinations:

18. 1.4D.L
19. 1.2D. L+1.6L.L
20. 1.2D. L+EQ+L.L
21. 0.9D. L+EQ
22. D.L+0.7EQ
23. D.L+0. 75L.L+0.525EQ
24. 0.6D. L+0.7EQ

Designing the cases

- Designing for case a, seismic zone factor (Z=0.15), rock soil (site class B) and seismic design category (SDC C). Tables (B.2, B.3, B.4, B.5, B.6) shows the details of designing case a.

Table B.2

Design for case M0.66-Z0.15-C-1, M0.33-Z0.15-C-2.

4-story			
section	dimension	longitudinal steel	shear reinforcement
beams	300*300	3φ16 bottom	6φ8mm/m
		3φ16 top	
		Columns	
story 1	0.45*0.45	8φ18	6φ8mm/m
story 2	0.4*0.4	8φ16	6φ8mm/m
story 3+4	0.35*0.35	8φ14	6φ8mm/m

Table B.3

Design for case M0.99-Z0.15-C-1, M0.495-Z0.15-C-2.

6-story			
section	Dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom	6φ8mm/m
		3φ16 top	
		columns	
story 1	500*500	12φ16	6φ8mm/m
story 2	450*450	8φ18	6φ8mm/m
story 3	400*400	8φ16	6φ8mm/m
story 4+5+6	350*350	8φ14	6φ8mm/m

Table B.4

Design for case M1.32-Z0.15-C-1, M0.66-Z0.15-C-2.

8-story			
section	Dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom 3φ16 top	6φ8mm/m
columns			
story 1	550*550	16φ16	6φ8mm/m
story 2+3	500*500	12φ16	6φ8mm/m
story 4+5	450*450	8φ18	6φ8mm/m
story 6+7+8	350*350	8φ14	6φ8mm/m

Table B.5

Design for case M1.65-Z0.15-C-1, M0.825-Z0.15-C-2

10-story			
section	Dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom 3φ16 top	6φ8mm/m
columns			
story 1+2	600*600	12φ20	6φ8mm/m
story 3+4+5	550*550	16φ16	6φ8mm/m
story 6+7	450*450	8φ18	6φ8mm/m
story 8+9+10	350*350	8φ14	6φ8mm/m

Table B.6

Design for case M1.98-Z0.15-C-1, M0.99-Z0.15-C-2

12-story			
section	dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom	6φ8mm/m
		3φ16 top	
	columns		
story 1+2+3+4+5	650*650	16φ18	6φ8mm/m
story 6+7	500*500	12φ16	6φ8mm/m
story 8+9	450*450	8φ18	6φ8mm/m
story 10+11+12	350*350	8φ14	6φ8mm/m

- Designing for case b, seismic zone factor (Z=0.132), rock soil (site class B) and seismic design category (SDC B). Tables (B.7, B.8, B.9, B.10, B.11) shows the details of designing case b.

Table B.7

Design for case M0.66-Z0.132-B-1, M0.33-Z0.132-B-2

4-story			
section	dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom	6φ8mm/m
		3φ16 top	
	Columns		
story 1	0.5*0.5	12φ16	6φ8mm/m
story 2	0.4*0.4	8φ16	6φ8mm/m
story 3+4	0.35*0.35	8φ14	6φ8mm/m

Table B.8

Design for case M0.99-Z0.132-B-1, M0.495-Z0.132-B-2

6-story			
section	dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom	6φ8mm/m
		3φ16 top	
		columns	
story 1	550*550	12φ16	6φ8mm/m
story 2	500*500	8φ18	6φ8mm/m
story 3+4	400*400	8φ16	6φ8mm/m
story 5+6	350*350	8φ14	

Table B.9

Design for case M1.32-Z0.132-B-1, M0.66-Z0.132-B-2

8-story			
section	Dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom	6φ8mm/m
		3φ16 top	
		columns	
story 1	550*550	16φ16	6φ8mm/m
story2+3	500*500	12φ16	6φ8mm/m
story 4+5+6	450*450	8φ18	6φ8mm/m
story 7+8	350*350	8φ12	6φ8mm/m

Table B.10

Design for case M1.65-Z0.132-B-1, M0.825-Z0.132-B-2

	10-story		
section	Dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom	6φ8mm/m
		3φ16 top	
	columns		
story 1+2	600*600	12φ20	6φ8mm/m
story 3+4+5	550*550	16φ16	6φ8mm/m
story 6+7+8	450*450	8φ18	6φ8mm/m
story 9+10	350*350	8φ12	6φ8mm/m

Table B.11

Design for case M1.98-Z0.132-B-1, M0.99-Z0.132-B-2

section	Dimension	longitudinal steel	shear reinforcement
beams	300*350	5φ16 bottom	6φ8mm/m
		3φ16 top	
	columns		
story 1+2	650*650	16φ18	6φ8mm/m
story 3+4+5	600*600	16φ16	6φ8mm/m
story 6+7+8	500*500	12φ16	6φ8mm/m
story 9+10	400*400	8φ16	6φ8mm/m
story 11+12	350*350	8φ12	6φ8mm/m

- Designing for case C, seismic zone factor (Z=0.1025), very dense soil and soft rock (site class C) and seismic design category (SDC C). Tables (B.12, B.13, B.14, B.15, B.16) shows the details of designing case C.

Table B.12

Design for case M0.66-Z0.1025-C-1, M0.33-Z0.1025-C-2

	4-story		
section	dimension	longitudinal steel	shear reinforcement
beams	300*300	4φ16 bottom	6φ8mm/m
		3φ16 top	
		columns	
story 1	480*480	12φ18	6φ8mm/m
story 2	400*400	8φ18	6φ8mm/m
story 3+4	350*350	8φ14	6φ8mm/m

Table B.13

Design for case M0.99-Z0.1025-C-1, M0.495-Z0.1025-C-2

	6-story		
section	dimension	longitudinal steel	shear reinforcement
beams	300*350	4φ16 bottom	6φ8mm/m
		3φ16 top	
		columns	
story 1	600*600	12φ20	6φ8mm/m
story 2	500*500	12φ16	6φ8mm/m
story 3+4	450*450	8φ18	6φ8mm/m
story 5+6	400*400	8φ16	6φ8mm/m

Table B.14

Design for case M1.32-Z0.1025-C-1, M0.66-Z1.1025-C-2

8-story			
section	dimension	longitudinal steel	shear reinforcement
beams	350*300	5φ16 bottom	6φ8mm/m
		3φ16 top	
Columns			
story 1+2	570*570	16φ16	6φ8mm/m
story 3	480*480	12φ16	6φ8mm/m
story 4+5	450*450	8φ18	6φ8mm/m
story 6+7+8	350*350	8φ14	6φ8mm/m

Table B.15

Design for case M1.65-Z0.1025-C, M0.825-Z0.1025-C-2

10-story			
section	dimension	longitudinal steel	shear reinforcement
beams	350*300	4φ16 bottom	6φ8mm/m
		3φ16 top	
columns			
story 1	650*650	16φ18	6φ8mm/m
story 2	600*600	12φ20	6φ8mm/m
story 3+4+5+6	550*550	16φ16	6φ8mm/m
story 7+8	450*450	8φ18	6φ8mm/m
story 9+10	400*400	8φ16	6φ8mm/m

Table B.16

Design for case M1.98-Z0.1025-C-1, M0.99-Z0.1025-C-2

12-story			
section	dimension	longitudinal steel	shear reinforcement
beams	350*400	5φ16 bottom	6φ8mm/m
		3φ16 top	
columns			
story 1+2+3+4+5	700*700	24φ16	6φ8mm/m
story6+7	550*550	16φ16	6φ8mm/m
story8+9	480*480	12φ16	6φ8mm/m
story10+11+12	400*400	8φ16	6φ8mm/m

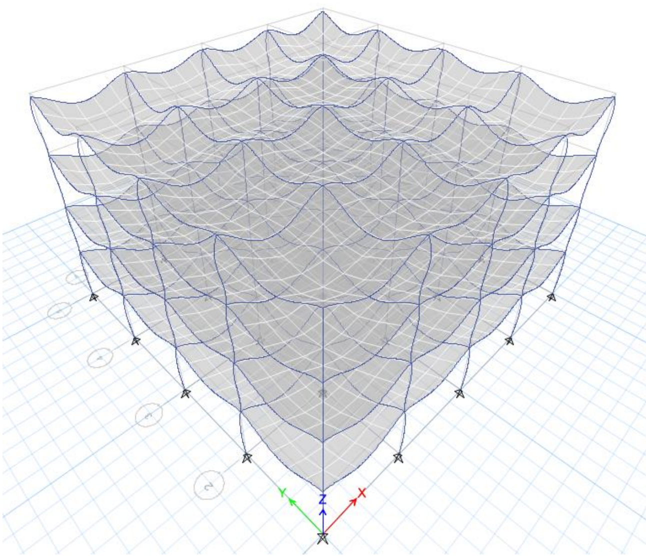
Appendix C

to verify and checks the elastic response for 3-D models for all cases, that have many checks as the following:

- compatibility check: it's important to ensure that all the member in the model connected well with each other. And this done by show the deformation of the modal building and start animation.

Figure C.1

Compatibility check for one case.



From the figure (1-c) the compatibility check is achieved. And other cases is the same of this case.

- equilibrium check: it can be confirmed by calculate the gravity load and compare it with base reaction from Etabs program. In the following tables illustrate the manual calculation for gravity loads.

Table C.1

Manual calculation for gravity loads for one case.

			dead load			
	area(m^2)	height(m)	unit weight (KN/m^3)	number	number of stories	weight (KN)
slab	400	0.15	25	1	4	6000
	0.45*0.45	3.3	25	25	1	417.656
columns	0.38*0.38	3.3	25	25	1	297.825
	0.35*0.35	3.3	25	25	2	505.3
beams	0.3*0.3	4.5	25	40	4	1620
sum						8840.89
			super imposed dead load			
load (KN/m^2)		area(m^2)	number of stories		weight	
8		400	4		12800	
			live load			
load (KN/m^2)		area(m^2)	number of stories		weight	
4		400	4		6400	

Table C.2

Comparison between manual calculation and ETABS results.

	load	manual calculation	ETABS result	error %	acceptable
M0.66-Z0.15-C	dead	8840.89	8705.4319	1.53<5	ok
	live	6400	6400	0	ok
	sd	12800	12800	0	ok

From the calculation in the previous table the check is achieved.

- Stress-strain check: it's can be done by calculate the moment and deformation manually and compare it with ETABS program. The minimum and maximum moment calculated manually as equation (C.1) and equation (C.2), then display the moment in ETABS program and check if exceed the minimum and maximum value or no.

For case M0.66-Z0.15-C:

$$As_{minimum} = 0.0033 * h * b$$
(C.1)

$$\emptyset \times Mn = 0.9 \times As_{minimum} \times fy \times (h - As_{minimum} \times \frac{fy}{(1.7 \times fc \times b)})$$

(C.2)

Where:

$\emptyset$: reduction factor and this for moment=0.9

Mn: nominal moment (KN.m)

$As_{minimum}$: the minimum area of steel in 1 meter of slab

fy: the yield strength of the steel

h : the depth of slab

fc: the compression strength of concrete

b: the width of slab and in this equation b=1m.

So, to calculate the moment:

$As_{minimum} = 0.0033 \times 150 \times 1000 = 499.5mm$, 4φ14mm/m

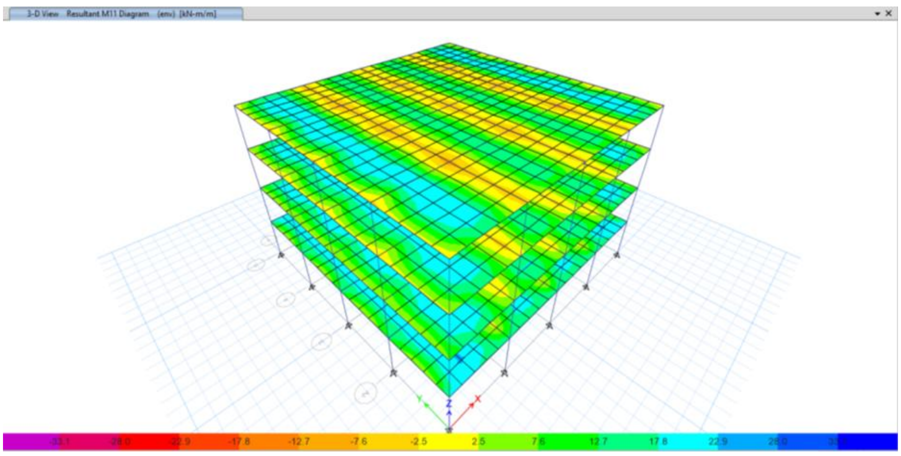
$$As_{used} = 4 * \frac{14^2 \times 3.14}{4} = 615.44mm$$

$$\emptyset \times Mn = 0.9 \times 615.44 \times 413.7 \times \left(150 - 615.44 \times \frac{413.7}{1.7 \times 28 \times 1000}\right) \times 10^{-6}$$

$$= 33.14KN.m$$

Figure C.2

Results of moment (M11) from ETABS for case M0.66-Z0.15-C.



From figure (C.2), the value of moment between the minimum and maximum values of Mn. so, the check is achieved. this check achieved for all cases as the same of case M0.66-Z0.15-C.

- Elastic period: it can be approved by calculate the period of structure by Rayleigh method that depend on calculating the mass and stiffness for each story and compare it with the ETABS program.

$$T_n = 2 \times \pi \times \sqrt[2]{\frac{\sum_{i=1} W_i \times \Delta i^2}{g \times \sum_{i=1} f_i \times \Delta i}}$$

(C.3)

Where:

Wi: the weight for each story at story level i (dead load+ live load+ superimposed dead load), the live load multiplied by coefficient 0.25.

Δi(m): The maximum displacement for each story when subjected to a lateral force.

fi (KN/m^2): the lateral force on each story.

g: the gravitational acceleration.

Table C.3

explanation on calculation for component of Rayleigh equation for one case.

story	wi(KN)	Δi(m)	wi*Δi^2	Fi	Fi*Δi
4	5766.65625	0.857841	4243.63148	4000	3431.364
3	5766.65625	0.776832	3479.992261	4000	3107.328
2	5811.825	0.620063	2234.519573	4000	2480.252
1	5931.65625	0.374612	832.4139412	4000	1498.448
sum			10790.55726		10517.392

$$Tn = 2 \times \pi \times \sqrt[2]{\frac{10790.55726}{9.81 \times 10517.392}} = 2.03 second$$

Table C.4

the summary of comparison time period with ETABS results.

	Tn manually	Etabs result	error %	acceptance
M0.66-Z0.15-C	2.03	2.02	0.5377	ok

Time period check is achieved.

جـــامعــــة النجاح الوطنية

كليـــة الدراســـــات العليـــا

تأثير نسبة ارتفاع المبنى بالنسبة لعرضه على معامل خفض القوة الزلزالية للمباني ذات الاطارات الخرسانية المسلحة

إعداد

دانه عصام عبد الكريم نصار

إشراف

د.منذر دويكات

د.محمد سماعنة

قدمت هذه الرسالة استكمالا لمتطلبات الحصول علي درجه الماجستير في هندسة الانشاءات، من كلية الدراسات العليا، في جامعة النجاح الوطنية، نابلس– فلسطين.

2023

تأثير نسبة ارتفاع المبنى بالنسبة لعرضه على معامل خفض القوة الزلزالية للمباني ذات الاطارات الخرسانية المسلحة

اعداد

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الملخص

عامل خفض القوة الزلزالية (R) هو أحد أهم العوامل في التصميم الزلزالي ويستخدم لمعرفة السلوك غير الخطي للمباني أثناء الزلزال. توفر الكودات الدولية مثل IBC 2016 قيمة ثابتة لـ "R" ومع ذلك قد تتغير قيمة "R" بناءً على معايير بما في ذلك فئات حجم الزلزال ، ونوع التربة القائم عليها البناء ، فئة التصميم الزلزالي ، ونسبة العرض إلى الارتفاع للإطار ، واتجاه العمود ونوع الاسقف المستخدمة في المبنى، إلخ.

تهدف هذه الدراسة إلى التعرف على آثار نسبة العرض إلى الارتفاع للمبنى على قيمة "R" في فلسطين ، لتحقيق هذا الهدف ، (تحليل الاستجابة الستاتيكية غير الخطية) المطبق على العديد من أنظمةالمباني ذات الإطارات الخرسانية المسلحة باختلاف نسبة ارتفاع المبنى بالنسبة لعرضه ، عوامل منطقة زلزالية مختلفة. وتغيير نوع التربة لتحديد سلوك الإطار مع زيادة التعرض للقوة الجانبية حتى الانهيار.

يستخدم برنامج ETABS2016 لتصميم الهياكل ، تم تصميم إطارات جميع المباني حسب متطلبات الكود السابقة. يستخدم برنامج العناصر المحدودة (ABAQUS) لإجراء التحليل غير الخطي عن طريق تحليل المرن Elasto-Plastic ، عناصر الخرسانة المسلحة المقدمة في البرنامج كعنصر خط. يستخدم برنامج Xtractلايجاد الخصائص غير المرنة للعناصر الخرسانية المسلحة. النتائج التي تم الحصول عليها من "R"مقارنة مع القيم المقترحة من قبل الكودات الدولية.

نتيجة لذلك ، توجد علاقات طردية بين نسبة العرض إلى الارتفاع وقيمة R ، وبين نوع التربة وقيمة R.

العلاقات العكسية بين عامل المنطقة الزلزالية (Z) وقيمة R وبين نوع فئة التصميم الزلزالي وقيمة R.

الكلمات المفتاحية: التحليل الساكن غير الخطي ، برنامج ABAQUS ، عامل خفض القوة الزلزالية.