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Progressive collapse analysis of RC building frames with different seismic design levels

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PROGRESSIVE COLLAPSE ANALYSIS OF RC BUILDING FRAMES WITH DIFFERENT SEISMIC DESIGN LEVELS

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ABSTRACT

Progressive collapse prevention of buildings has recently become the focus of many researchers, design engineers, and officials all over the world particularly after the failure of the twin World Trade Center towers, New York City, USA in September 2001 and the increasing terrorist acts against governmental buildings. The progressive collapse is defined in the commentary of the American Society of Civil Engineers Standard 7-02 *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-02) as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". To date, there is no design code for blast resistant building design and progressive collapse prevention, but only design guideline sexist which are prepared by different bodies like Departments of defense (DoD) in USA as well as other countries and the General Service Administration (GSA) and the Federal Emergency Management Agency (FEMA).

While design for progressive collapse prevention is possible at design stage, it becomes more challenging for already existing buildings. On the other hand, almost all recently designed and constructed buildings are designed for seismic resistance according to the seismic zone they are located in according to existing codes. Seismic design provisions allows for higher resistance to lateral reversible loads, and more ductility of structural frames and systems. Determining how much such provision adds to the building resistance to progressive collapse help when upgrading existing building for progressive collapse or when designing new ones. Furthermore, utilizing this seismic resistance and added ductility saves when designing for progressive collapse. This research focuses on identifying the effect of seismic design level on resisting progressive collapse.

In this research the progressive collapse of three-story, multi-bay reinforced concrete structure is conducted. The procedure was conducted according to GSA and DoD guidelines. At first the building was designed according to Egyptian Code of Practice for design and construction of concrete structures (ECP203-2007). Three different levels of seismic design are considered by assuming that the building can be located in seismic zones 1, 3, and 5 according to the Egyptian Code for Calculating Loads and Forces on Structures and Buildings (201-2003). GSA guidelines and procedures are followed for progressive collapse analysis. All the three stages; gravity load, seismic load, and progressive collapse load analyses; are performed using commercially general purpose software computer program, SAP2000, that is available in the Construction lab, AUC. . Nonlinear static analysis was carried out where plastic hinges were allowed to form at designated locations of maximum moment. A total of 39 design and analysis case were considered and the results were analyzed to evaluate the effect of the different parameters on the building performance and its resistance to progressive collapse.

The resistance to progressive collapse is measured by the number of formed plastic hinges and the resulting failed beams. The relationship between the seismic design levels, the slab thickness, number of formed plastic hinges and failed beams are presented graphically. The results showed that the vulnerability to progressive collapse becomes less as the seismic design level increased in higher seismic zone. This is mainly due to the increased member capacity, added ductility, and the seismic requirement for reinforcement details. It was also found that the slab membrane and bending actions contribute significantly in resisting progressive collapse and thus must be considered in the analysis.

ACKNOWLEDGMENTS

From my heart, I would like to thank Prof. Dr. Mohamed Abdel- Mooty for his supervision and support. I owe the completion of this thesis to his support.

Also, I would like to thank everyone who helped me.

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Dedication

When I was at a primary school, a teacher said that there are only two persons who are really wished to be better than them, your father and your teacher. I couldn't realize the exact meaning at that time. When my God gave me my son, Abdel_Rahman, I realized the exact meaning of that statement, so I do dedicate this thesis to my son wishing that he, IN SHA ALLA, will be better than me and will complete what I have started. Also I can't forget my mother's help; she taught me how to be patient and be kindly. Finally I beg our God for mercy of my father's soul who was very patient with me, and taught me how to overcome a difficulty that maybe facing me.

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LIST OF ABBREVIATIONS

2_D	Two_ dimensional
3_D	Three_ dimensional
AISC	The American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ECP201_2003	Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings
ECP203_2007	Egyptian Code of Practice for Design and Construction of Concrete Structures
ESLMA	Equivalent Static Load Method of Analysis
FEMA	Federal Emergency Management Agency
FRF	Force Reduction Factor
IBC	International Building Code
MDOF	Multiple Degrees of Freedom
MRF	Moment Resisting Frame
PGA	Peak Ground Acceleration
R	Response Modification (force reduction) Factor
SDOF	Single Degree of Freedom
SRSS	Square Root of the Sum of the Square
UBC	Uniform Building Code
USA	United State of America

LIST OF SYMBOLS

μ	Poisson's ratio / percentage of longitudinal reinforcement in the cross section
μ_{max}	maximum percentage of tension reinforcement in reinforced concrete section
μ_{min}	minimum percentage of tension reinforcement in reinforced concrete section
a	shorter effective span for the slab / actual length of the equivalent stress block
a_g	Designed Ground Acceleration
b	concrete cross section breadth
c	concrete cover of the section
d	effective cross section depth
DL	Dead Load
E	modulus of elasticity
f_{cu}	characteristic Concrete Compressive strength
FC	Flooring cover
f_y	yield strength or proof strength for reinforcement
G	Shear Modulus
GPa	Giga Pascal
kN/m^3	Kilo Newton per cubic meter
LL	Live load
mm	Millimeter
MPa	Mega Pascal

M_{ul}	Ultimate limit moment capacity of the section
M_{umax}	Ultimate moment capacity of the section
ow	Own weight of the member
RC	Reinforced concrete
S	Soil factor
S_e (T)	Elastic Horizontal Response Spectrum
T	Vibration Period
t	concrete cross section total depth
T_B	Limit depending on the soil type
T_C	Limit depending on the soil type
T_D	Limit depending on the soil type
t_{min}	Minimum thickness of the slab
γ	nominal weight per unit volume
γ_c	material strength reduction factor for concrete
γ_s	material strength reduction factor for steel
η	Design Damping factor depending on the type of structure

CHAPTER 1

INTRODUCTION

1.1 Preface

Since the Ronan Point building, London, collapsed in 16 May 1968, the progressive collapse issue earned the attention of the structural engineers all over the world

Here is a list of some buildings that had collapsed in progressive collapse style

No.	Building Name	Location	Date
1	“St Mark’s Campanile	Venice, Italy	July 14, 1902
2	University of Aberdeen Zoology	Aberdeen, Scotland	November 1, 1966
3	Ronan Point apartment	West Ham, London	May 16, 1968
4	Skyline Towers Building	Fairfax County, Verginia	March 2, 1973
5	Commercial office Building	Wilshire Blvd, Los Angeles	December 19, 1985
6	Hotel New World	Little India, Singapore	March 15, 1986
7	Pavie Civic Tower	Pavia, Italy	March 17, 1987
8	L’Ambiance Plaza	Bridgeport, Connecticut	April 23, 1987
9	Ancient bell tower at the medieval Church of St Maria Magdalena	Goch, Germany	1992
10	Kader Toy Factory	NakhonPathom, Thailand	May 10, 1993
11	Alfred P. Murrah Federal building	Oklahoma City, Oklahoma	April 19, 1995
12	Sampoong Department Store	Seoul, South Korea	June 29, 1995

13	World Trade Center buildings	New York City	September 11, 2001
14	Windsor Tower	Madrid, Spain	February 12, 2005”

1.2 General Introduction

Progressive collapse of existing building is initiated by the sudden failure of one or more of its major load bearing elements, typically columns or walls, followed by redistribution of the loads and failure of the next elements in the vicinity in a chain-like reaction until the failure of the whole building. Many events can initiate such type of failure including: overload due to change of use or structural modifications, deterioration and degradation of structural member, or accident like impact or explosion. Recently, blast event weather accidental or as terrorist acts has increased and gained considerable attention by the structural designers. Requirements for blast resistant design and progressive collapse prevention are now mandatory in specific buildings like embassies, airports, emergency, management centers, and some critical governmental facilities which may be a target for terrorist attacks. Many of those buildings already exist and need to be upgraded. Other buildings which are not directly targeted may exist in the vicinity of targeted building. The level of blast threat varies from building to another and the level of blast protection may also vary from one owner to another. Up to now, there are no code requirements for blast design. Only guidelines are set by different bodies that help owners, developers, and design engineers in designing building facilities for withstanding blast effect, for example; General service Administration, GSA, and Department of Defense, DoD, USA. However, regardless of the level of protection of the building, progressive collapse prevention should be achieved to allow for timely evacuation of the tenant to save lives.

The performance of buildings during progressive collapse event depends on many factors. Those factors include: the actual strength to the design strength, the level of redundancy in the structural system, the level of structural integrity of the individual members to form a whole system, and the types of structural details and the ductility existent in the system. Seismic design requirement adds strength and ductility to the system which are needed only during seismic events. This represents some additional redundancy when considering only gravity load, since seismic events are rare and may exist one or two time during the lifespan of the structure and usually not coincident with blast event. Therefore, the author believes that seismic design requirement may be utilized to improve the performance of buildings and structures against progressive collapse. However, seismic design requirements according to different local and international codes depend on the seismic zone in which the building is situated.

In this study the researcher designed a fictitious building according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007). In order to study the effect of seismic zone, the building is assumed to exist in seismic zones 1, 3, and 5. Hence the building was analyzed and designed for three seismic levels as per the previous three seismic zones according to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (ECP201-2003). Finally the building was exposed to a series of removing load bearing elements according to the scenarios of General Service Administration, GSA, in order to evaluate its performance for progressive collapse.

1.3 Research Objectives

The main objectives of this research are as the following:

- To investigate the effect of design for different seismic zones, according to (ECP201-2003), on the vulnerability of the structure for progressive collapse.
- To investigate the effect of RC slab on the vulnerability of the structure for progressive collapse.

To achieve the above objectives a 3_D model was developed for a multi-story multi-bay RC building frames using a commercial computer program, SAP2000, which is available at AUC construction lab. The following scenarios were conducted:

1. The building was analyzed using, SAP2000, for the gravity loads and was designed according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007).
2. The building was analyzed using, SAP2000, for the seismic loads for different seismic zones according to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (ECP201_2003) and was designed according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007).

3. The different designs of the building located in different seismic zones were analyzed using, SAP2000, for the progressive collapse by sudden removal of one of the main load bearing vertical elements according to the General Service administration, GSA, guidelines. The number of formulated plastic hinges was observed, also the number of failed beam members was identified and used as a performance measure of progressive collapse.

For each of the above mentioned scenario, the following assumptions were made:

1. Assume the slab thickness of the building is 10 mm, fictitious building. The slab bending stiffness in this case is ignored and only used to transmit gravity loads to beam.

2. Assume the slab thickness of the building is 160 mm, fictitious building. This represents the actual case of the slab to consider its effect in resisting progressive collapse.

3. Assume that there is no slab at all; i.e. the building consisting of 3_D RC frames only. The slab gravity loads are applied directly to the beam according to Egyptian code of practice.

Thus the following total of 39 scenarios was considered:

1. A 3-D model for gravity load for 10 mm slab thickness.

2. A 3-D model for gravity load for 160 mm slab thickness.

3. A 3-D model for gravity load for no slab.

4. A 3-D model for 10 mm slab thickness for seismic zone 1.

5. A 3-D model for 160 mm slab thickness for seismic zone 1.

6. A 3-D model for no slab for seismic zone 1.

7. A 3-D model for 10 mm slab thickness for seismic zone 3.

8. A 3-D model for 160 mm slab thickness for seismic zone 3.

9. A 3-D model for no slab for seismic zone 3.

10. A 3-D model for 10 mm slab thickness for seismic zone 5.

11. A 3-D model for 160 mm slab thickness for seismic zone 5.

12. A 3-D model for no slab for seismic zone 5.

13. A 3-D model for 10 mm slab thickness for seismic zone 1, removing a load bearing element at the corner.

14. A 3-D model for 10 mm slab thickness for seismic zone 1, removing a load bearing element at the middle of a longer side.

15. A 3-D model for 10 mm slab thickness for seismic zone 1, removing a load bearing element near the middle of the shorter side.

16. A 3-D model for 160 mm slab thickness for seismic zone 1, removing a load bearing element at the corner.

17. A 3-D model for 160 mm slab thickness for seismic zone 1, removing a load bearing element at the middle of a longer side.

18. A 3-D model for 160 mm slab thickness for seismic zone 1, removing a load bearing element near the middle of a shorter side.

19. A 3-D model for no slab for seismic zone 1, removing a load bearing element at the corner.

20. A 3-D model for no slab for seismic zone 1, removing a load bearing element at the middle of a longer side.

21. A 3-D model for no slab for seismic zone 1, removing a load bearing element near the middle of a shorter side.

22. A 3-D model for 10 mm slab thickness for seismic zone 3, removing a load bearing element at the corner.

23. A 3-D model for 10 mm slab thickness for seismic zone 3, removing a load bearing element at the middle of a longer side.
24. A 3-D model for 10 mm slab thickness for seismic zone 3, removing a load bearing element near the middle of a shorter side.
25. A 3-D model for 160 mm slab thickness for seismic zone 3, removing a load bearing element at the corner.
26. A 3-D model for 160 mm slab thickness for seismic zone 3, removing a load bearing element at the middle of a longer side.
27. A 3-D model for 160 mm slab thickness for seismic zone 3, removing a load bearing element near the middle of a shorter side.
28. A 3-D model for no slab for seismic zone 3, removing a load bearing element at the corner.
29. A 3-D model for no slab for seismic zone 3, removing a load bearing element at the middle of a longer side.
30. A 3-D model no slab for seismic zone 3, removing a load bearing element near the middle of a shorter side.
31. A 3-D model for 10 mm slab thickness for seismic zone 5, removing a load bearing element at the corner.
32. A 3-D model for 10 mm slab thickness for seismic zone 5, removing a load bearing element at the middle of the longer side.
33. A 3-D model for 10 mm slab thickness for seismic zone 5, removing a load bearing element near the middle of a shorter side.
34. A 3-D model for 160 mm slab thickness for seismic zone 5, removing a load bearing element at the corner.

35. A 3-D model for 160 mm slab thickness for seismic zone 5, removing a load bearing element at the middle of a longer side.

36. A 3-D model for 160 mm slab thickness for seismic zone 5, removing a load bearing element near the middle of a shorter side.

37. A 3-D model for no slab for seismic zone 5, removing a load bearing element at the corner.

38. A 3-D model for no slab for seismic zone 5, removing a load bearing element at the middle of a longer side.

39. A 3-D model for no slab for seismic zone 5, removing a load bearing element near the middle of a shorter side.

The removal of a load bearing elements and the analyses were conducted according to GSA requirements. The analyses were performed using nonlinear static analysis method. The GSA guidelines allow using either static linear analysis method or static nonlinear analysis method.

1.4 Organization of the Thesis

Due to the nature of this research, the analyses, results, and design for each of the above mentioned scenarios were included in its own chapter. Hence this thesis consists of six chapters including this chapter, chapter one, that introduces the thesis, gives some examples for some famous progressive collapse failures, states the research objectives and methodology, and finally provides the outlines of the thesis.

Chapter two presents some literatures that directly related to the progressive collapse issue and some topics that related to the background of the topic.

Chapter three gives the description the study building, and builds the basic model using the computer program, SAP2000, available in the construction lab at AUC. Also presented in this chapter is the analysis for the scenario of gravity loads according to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (201_2003), analysis results, and design of sections for all the building elements according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007). This includes three scenarios, (scenario 1 through 3).

Chapter four presents the analysis for the scenario of seismic loads according to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (201_2003), analysis results, and design of the beam element sections for all the building beam elements according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007). This includes nine scenarios, (scenario 4 through 12).

In Chapter five, the progressive collapse analyses according to General Service Administration, GSA, guidelines were presented. The guidelines applied for each scenario that has been mentioned before, provides the analysis results, and design of the beam element sections for all the building beam elements according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007). This includes twenty seven scenarios, (scenario 13 through 39).

Finally, chapter six provides the researcher's comments, conclusions, and suggestions for future work in this topic.

CHAPTER 2

LITERATURE REVIEW

Shankar , R. Nair made a comparison between five codes and standards that deal with progressive collapse phenomena. He started with exploring the three well known structures that suffer from progressive collapse. He began with Ronan Point apartment tower located in Newham, east London, then Murrah Federal Office Building located in Oklahoma City, USA, and finally the most famous event in the last century, the World Trade Center 1 and 2 of New York. The codes and standards that he investigated are: ASCE 7-02, ACI 318-02, GSA PBS Facilities Standards 2000, GSA PBS Facilities Standards 2003, and GSA Progressive Collapse Guidelines 2003. He investigated the previous codes and standards considering the points of: redundancy or alternate load paths, local resistance, interconnection or continuity.
Finally he emphasized on the three methods for designing a structure to reduce the vulnerability to disproportionate collapse: redundancy or alternate load paths, local resistance, and interconnection or continuity. And he stated that "the emphasis on redundancy over all alternatives in some recent codes and standards and user agency requirements may not lead to buildings that are less susceptible to disproportionate collapse as a result of deliberate attack".

Ashraf Habuibullah and Stephen Pyle (1998) presented detailed steps that required using the capability of the widely used and well known structural analysis program in structural analysis field, SAP2000 ®, to perform a pushover analysis for a simple three-dimensional building. They showed the simplicity of performance of pushover nonlinear analysis using SAP2000. Also they stated the steps in details, so it was a very useful paper for this research, particular in the static nonlinear analysis stage. They also illustrated the output steps in graphs which are clear and easily for following. They also pointed out that a research can define more than one pushover load case in the same analysis case. They clarify that the pushover analysis can be either force controlled or displacement controlled. I expect that paper is very useful for any researcher who searches in nonlinear static analysis.

ChanhTrung Huynh, Jongyul Park, Jinkoo Kim, and Hyunhoon (2007) carried out experiments "to investigate the progressive collapse resisting capacity of RC beam-column sub assemblages designed with and without seismic load". They carried out their experiment on five and eight-story RC frames. The variables that they considered were: concrete strength, amount of reinforcement, and detailing of re-bars. They designed five and eight-story RC frames for gravity loads and seismic loads. They follow the ACI detailing Manual (ACI 2004).They assumed that the five story structure had some deterioration. This is achieved through reducing the concrete strength via increasing the water/ cement ratio. They carried out the test and concluded that the dominant factors are: the compressive strength of the concrete, the seismic design and detailing, and standard hooks of lower re-bars. Those factors have a significant effect on enhancing the vulnerability of RC frames to progressive collapse.

Ali Kazemi and MehrdadSasani (2010) studied the "Effects of Beam Growth and Axial Force in Progressive Collapse Analysis of RC Structures". In their introduction, again they pointed out to the Ronan Point apartment building (1968) located in England, Murrah Federal building (1995) located in Oklahoma City, USA, and the most famous one, the twin World Trade Center towers (2001), New York city, USA. They stated that "Progressive collapse is defined as the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". In their paper they removed four adjacent columns in the ground floor and two beams in the floor above, although General Service Administration, GSA, and Department of Defense, DoD, stated another scenarios. They modeled, using the computer program SAP 2000, an actual structure consisted of eleven stories above the ground. However, they gave a full description for the structure, the mechanical properties of materials, and they conducted tests for some samples of concrete and steel. They modeled the structure in 3_D, and inserted plastic hinges to account for the nonlinearity of the structure. They defined the beam growth phenomenon as "following the formulation of cracks the mid-height of the beam section is under tensile strain. As a result, the length of the beam at its centerline increases by the sum of crack widths at the mid-height of the beam".

Mehmet Inel and HayriBaytanOzmen (2006) studied the "Effect of plastic hinge properties in nonlinear analysis of reinforced concrete buildings". In their paper, they tried to find if there are differences in the results of using the user-defined plastic hinges or the predefined default plastic hinges. They modeled two RC frames; one with four stories and the other with seven stores. Both building have the same plan dimensions, and they have the same characteristic materials. They used 2_D modeling via a general-purpose structural analysis program for static and dynamic analysis of structures. They carried out non linear static analysis, assuming strong column- weak beam response. They assigned plastic hinges at both ends for every beam element. They studied five cases for every frame. Of the many results they obtained, the most relevant to this research, is that: “it is important that the user-defined hinges model is more successful in capturing the hinging mechanism compared to the model with default hinges”.

Ted Krauthammer, Robert L. Hall, Stanley C. Woodson, James T. Baylot, John R. Hayes, and Yong Sohn (2002) introduced a theoretically and numerically procedures for studying progressive collapse phenomenon. Also they made laboratory tests to verify the procedure. Again they pointed out to Ronan Point residential apartment building, London, 1968, and the most famous one, the World Trade Center Towers, New York city, USA, 2001. They differentiate between progressive collapse and global collapse. Because that study was somewhat old, so they stated “However, to date, no adequate tools exist that can perform a progressive collapse analysis with acceptable reliability”. It’s supposed that this statement is still right, somewhat, till now because of the complexity of the phenomenon either the material that the structure is composed from or the factors affecting the analysis such as causes of initiation damage, characteristic of the building, i.e. width height ratio, regularity, etc. They pointed out to the great efforts of both GSA and DoD to introduce progressive collapse consideration that must be taken into consideration in design process.

Hyun-Su Kim, Jinkoo Kim, and Da-WoonAn (2009) study’s evaluated the damage level. They used the integrated system for progressive collapse analysis. To achieve this goal they used the existing nonlinear analysis program code OpenSees. They analyzed a two-dimensional structure when a column was removed. Again they pointed out to the Ronan Point apartment building (1968) located in England, Alfred P. Murrah Federal in Oklahoma City (1995), USA, and the most famous one, the twin World Trade Center towers (2001), New York city, USA. They pointed out that both GSA and DoD guidelines’ allow three analysis procedures for progressive collapse analysis. These procedures are: linear static analysis, nonlinear static analysis, and nonlinear dynamic analysis. They criticized both of the guidelines with-respect-to the dynamic amplification factor of two. It will be shown later in this study that the slab effect has an important factor in determining the number of plastic hinges. Also they carried out the analysis using two-dimensional structure neglecting the effect of the third dimension. Finally they concluded that "the analysis results also showed that the collapse mechanism for progressive collapse depends greatly on the modeling technique for failed members".

Dhileep. M, Trivedi. A, and Bose. P. Researcher studied "the behavior of high frequency modal responses in nonlinear static pushover analysis of structures". To get the exact solution of the equation of motion for a multi-degree of freedom system they used non-linear dynamic analysis. To overcome the difficulties of solution by non-linear dynamic analysis, a non-linear static pushover analysis was proposed (Chopra and Goel, 2001; Kalkan and Kunnath, 2004; Barros and Almeida, 2005).

CHAPTER 3

BUILDING DESCRIBTION, GRAVITY LOADS, ANALYSIS AND DESIGN

3.1 INTRODUCTION

In this study, a model of a typical regular reinforced concrete structure is considered for progressive collapse analysis. The researcher began by building the basic model using a commercially available computer program, SAP2000. The loads that are applied to the structure are assumed according to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (201_2003). Of course to begin analyzing a model using SAP2000, the member cross sections have to be fed to the program. The researcher made an assumption for element cross sections of the beams and columns from his previous experience in the field of design of reinforced concrete structures. With respect to the case of slab thickness of 160 mm, the slab section was assumed to satisfy the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007) requirements. After analysis was performed, for each case of the following cases, via SAP2000, the design was made according to ECP203-2007 requirements as mentioned earlier in Chapter 1, there are three scenarios for gravity load cases, and those are:

I. The slab thickness is assumed as 10mm, (factitious building).

II. The slab thickness is assumed as 160mm.

III. There is no slab, (perimeter frame building).

3.2 Description of the Building

The building consists of three reinforced concrete, RC, floors. The1st floor height is 5.0m followed by two floors with height equal to 3.50m for each, with a total height of 12.00 m. The dimensions in the plan are 18.0 m x 24.0 m. The RC structure consists of four bays in the x-direction and three bays in the y-direction, the bay lengths are six meters in both directions. Figure 3.1shows the three - dimensional view of the building. The columns are assumed to be fixed to the foundations.

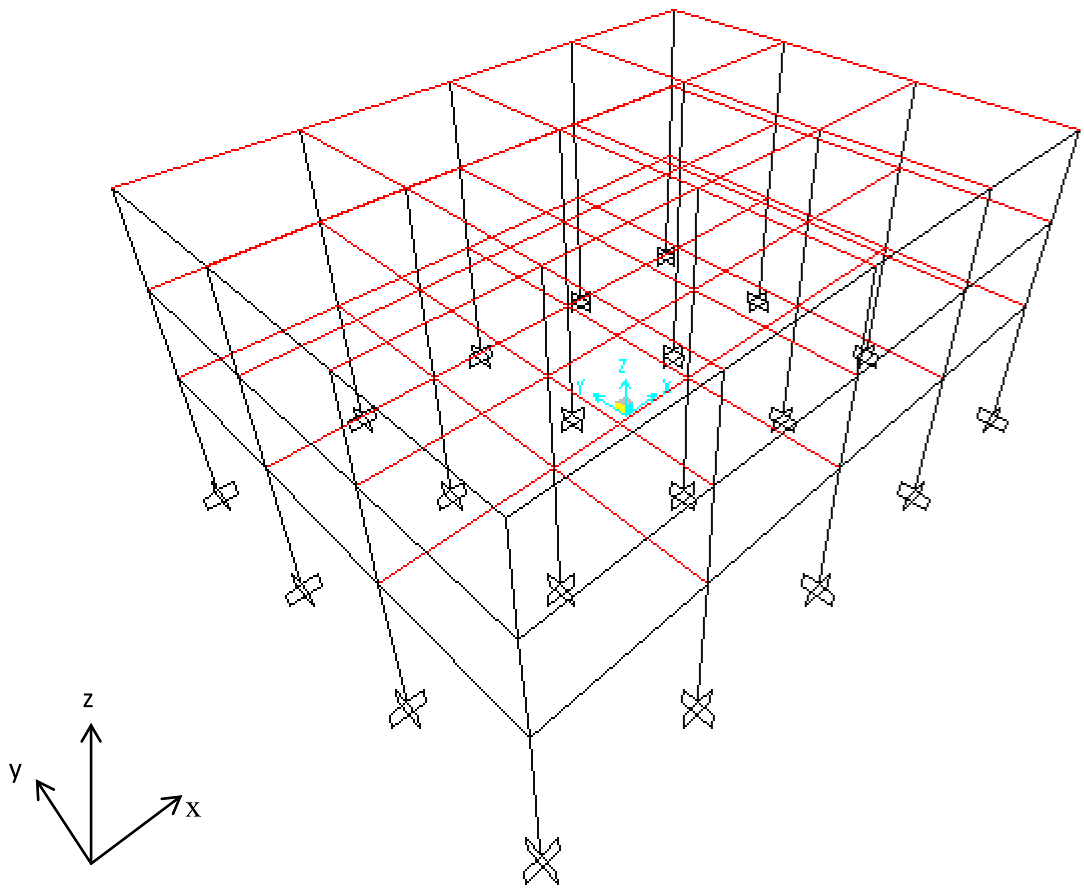


Fig 3.1 A 3_D view of the building

3.2.1 Nomenclature

Because the building has 153frame elements; 60 column frame elements and 93 beam frame elements, therefore it is better to maintain the system of numbering of the elements according to the SAP program model.

The following figures illustrate the numbering of the building members, according to SAP2000 program.

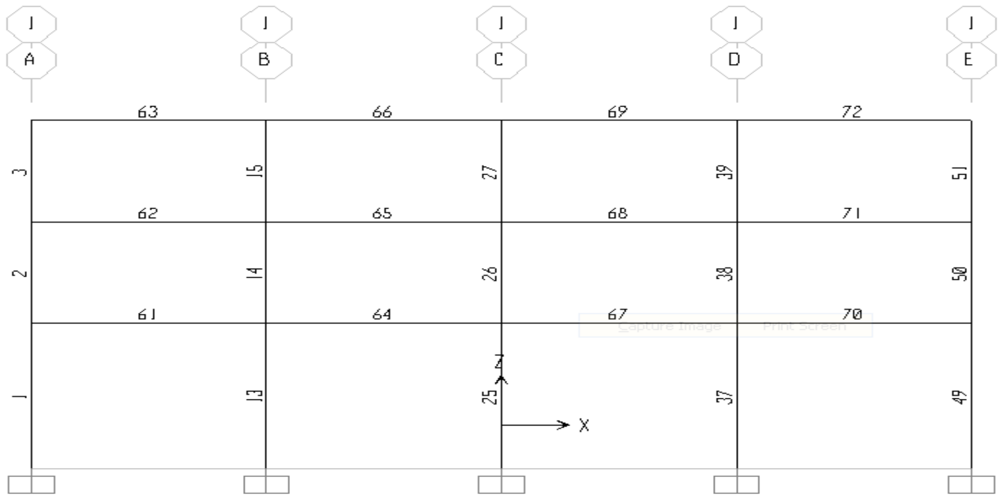


Fig 3.2 Numbering of column and beam frame elements on axis 1

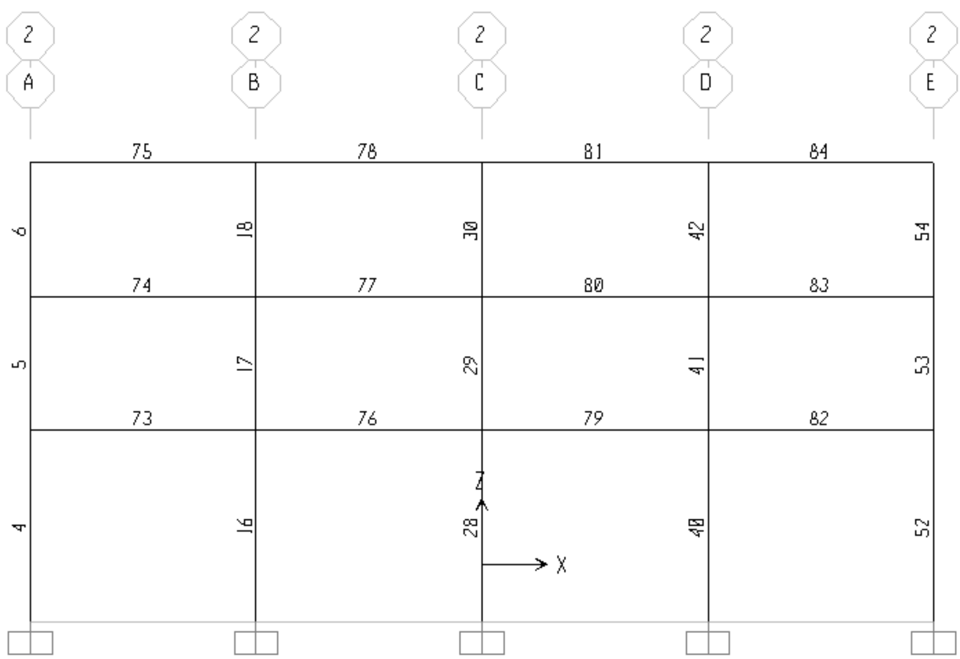


Fig 3.3 Numbering of column and beam frame elements on axis 2

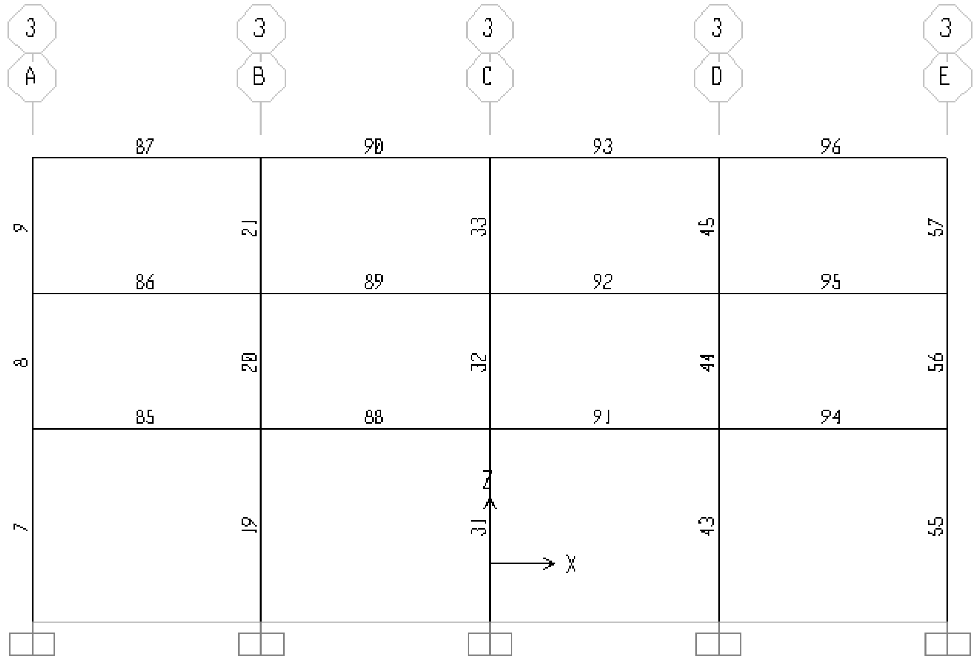


Fig 3.4Numbering of column and beam frame elements on axis 3

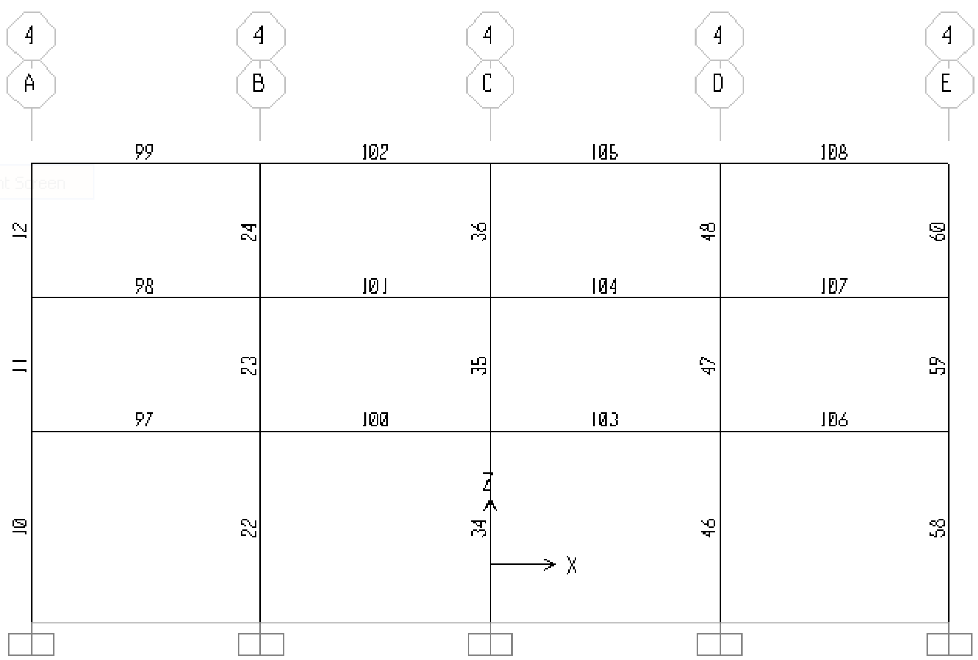


Fig 3.5 Numbering of column and beam frame elements on axis 4

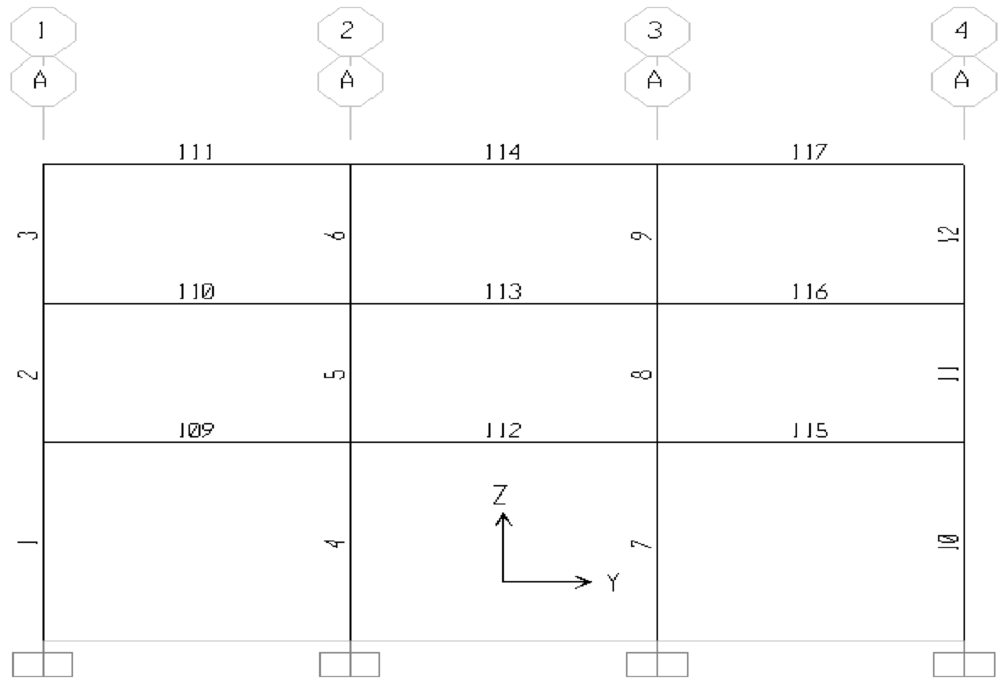


Fig 3.6 Numbering of column and beam frame elements on axis A

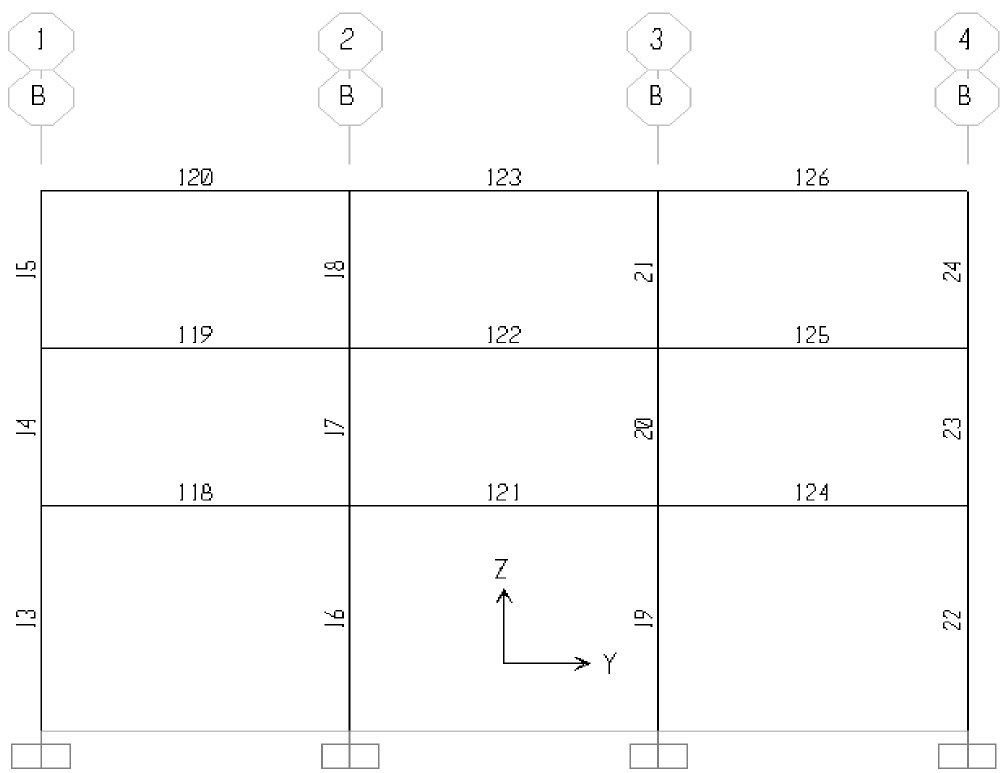


Fig 3.7 Numbering of column and beam frame elements on axis B

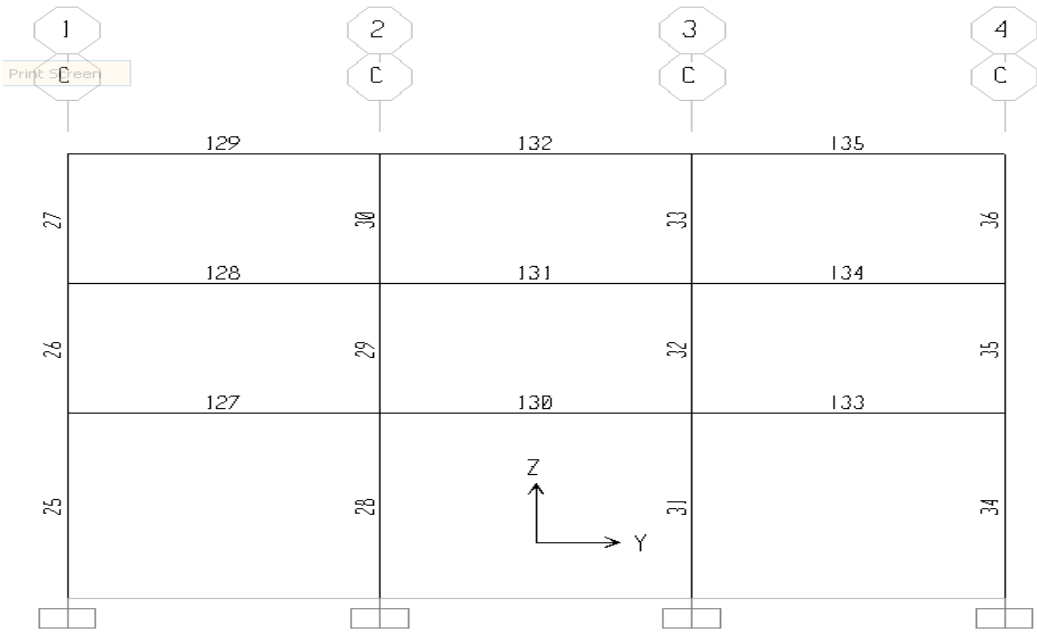


Fig 3.8 Numbering of column and beam frame elements on axis C

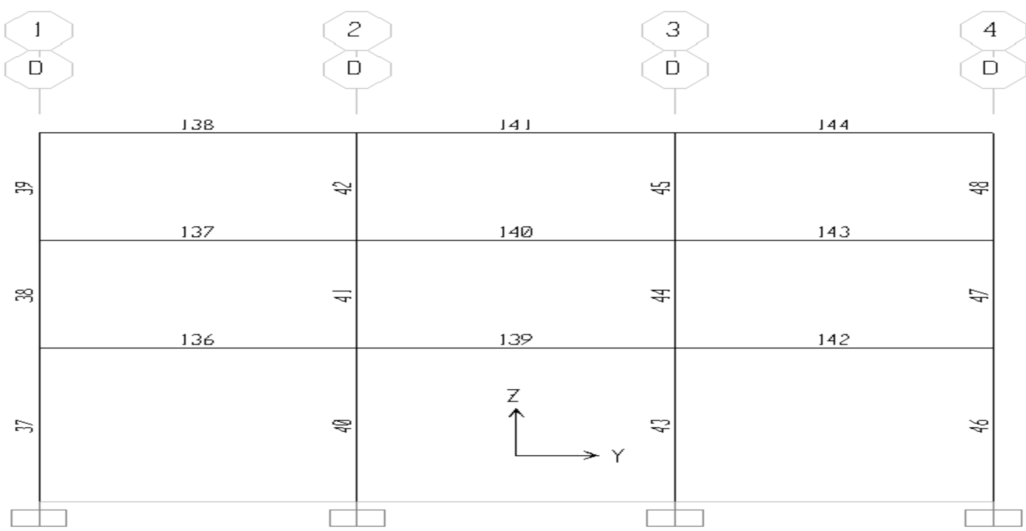


Fig 3.9 Numbering of column and beam frame elements on axis D

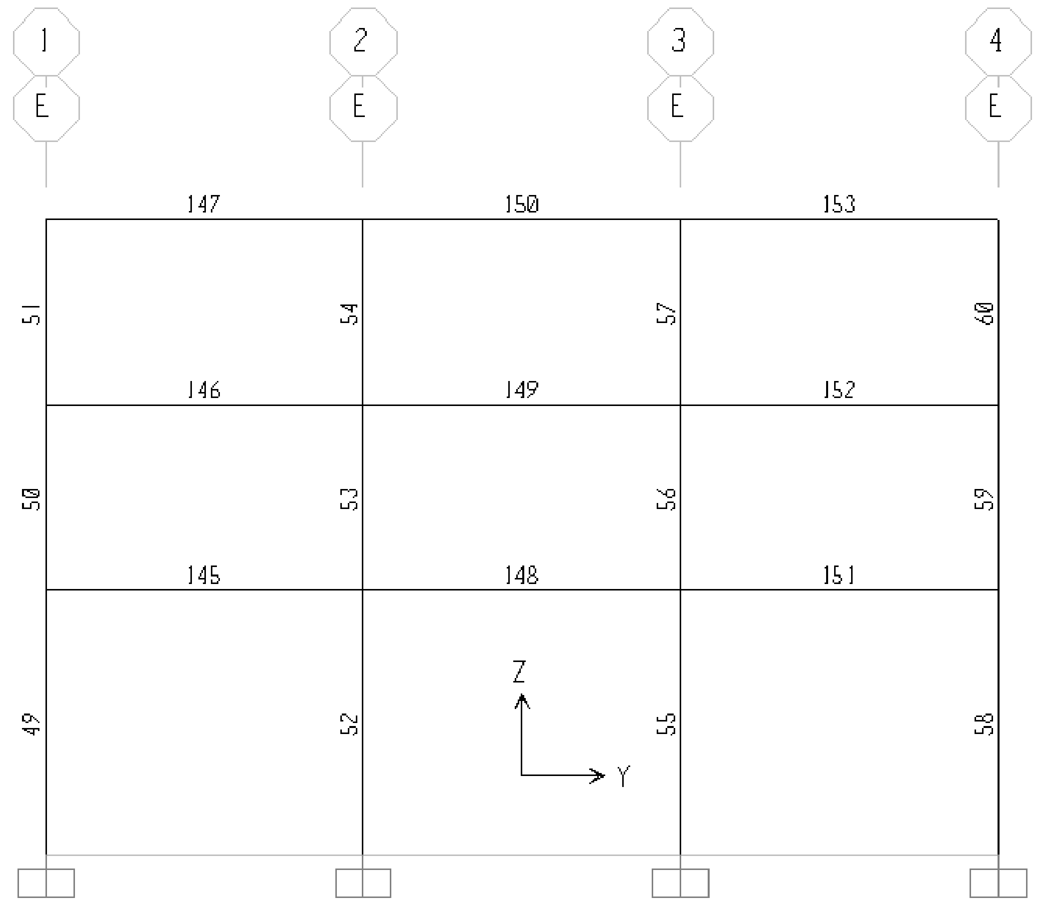


Fig. 3.10 Numbering of column and beam frame elements on axis E

3.3 Material Properties

The building was assumed to be constructed from reinforced concrete frame, so the following are the properties of the materials

3.3.1 Concrete Properties

The reinforced concrete has the following properties

- nominal weight per unit volume, γ =25 kN/m^3
- nominal mass per unit volume = 2500 kN/$^{m2\ s2}$
- Specified Concrete Compressive strength, f_{cu}= 30MPa
- isotropic material data

- modulus of elasticity, E =24GPa
- Poisson's ratio, μ = 0.2
- Shear Modulus, G = 10.41GPa

3.3.2 Steel Properties

The steel bar reinforcement has the following properties

- nominal weight per unit volume, γ =78 kN/m^3
- Tensile yield strength, fy = 360 MPa

3.4 Model Dimensions

To be able to conduct analysis via SAP2000, the frame member sections must be assumed. For the slab dimension, the researcher will follow the ECP203_2007 requirements, but with respect to beam frame member sections and the column frame member sections, they are assumed according to common practice.

3.4.1 Slab Dimension

The building has bays of dimensions 6.0 m by 6.0 m and it is assumed as a solid slab system, therefore the slab floor system is a two-way solid slab, and according to ECP203-2007, the minimum slab thickness is as following:

- For simply supported two-way solid slab, $t_{min.} = \frac{a}{35} = \frac{6.0}{35} = 0.17$ m
- For a slab continuous from one, $t_{min.} = \frac{a}{40} = \frac{6.0}{40} = 0.15$ m
- For a slab continuous from both sides, $t_{min.} = \frac{a}{45} = \frac{6.0}{45} = 0.13$ m

Where *a* is the shorter effective span for the slab.

As the floor system does not have a simply supported bay, and it has four bays that continuous from one side, the corner bays. Hence the minimum slab thickness was assumed to be equal as 160 mm, for construction purpose.

3.4.2Beam Dimensions

Although the outer beams carries half of the load carried by the inner beams, but there is an external wall on them, hence all the beam member sections were assumed as250 mm by 600 mm.

3.4.3Column Dimensions

As the gravity loads increase almost linearly as downwards, so the column cross section dimensions were assumed as following

- For ALL of the ^{1st} floor columns, the cross section dimensions are 300 x 500 mm.
- For ALL of the 2^{nd} floor columns, the cross section dimensions are 300 x 400 mm.
- For ALL of the 3^{rd} floor columns, the cross section dimensions are 300 x 300 mm.

3.5 Gravity Loads

The gravity loads consists mainly of two parts:

1. Dead loads that include the self- weight of the members, floor covering, ceiling, and wall loads. These are usually denoted as *g*
2. Live loads that include the occupancy's weighs. These are usually denoted as *p*.

3.5.1 Assumptions

The following assumptions were permitted:

- The self- weight of the member, own weight, is obtained from multiplying the nominal weight per unit volume by the volume of the element, and is considered as dead load. The SAP2000 calculates it automatically.
- The flooring cover, FC, which was assumed to be equal to 1.50 kN/m^2, and is considered as dead load.
- There are walls of thickness 12cm on the perimeters, so the own weight of it is 3 kN/ m^2.

- There is a parapet with height of one meter on the beams of the 3rd floor, so the own weight of it is 3 kN / m~.

- There are no interior walls on the floors, open area.

For every slab condition, a detailed calculation was given.

3.5.2 Case 1; Slab Thickness of 10 mm

In this case, the slab thickness was assumed to be equal to 10 mm, of course there is no in reality a reinforced concrete slab that has such this thickness, but the researcher made this assumption to study the effect of slab thickness on progressive collapse phenomenon.

3.5.2.1 Gravity Load Calculations,Slab Thickness of10 mm

As the slab thickness = 0.01 m, hence the own weight, ow., = 2.5kN /m^2, the remaining of the assumed slab thickness was fed to SAP2000 as dead loads that included the flooring cover, FC, that was assumed to be equal to be 1.5 kN /m^2.

As mentioned before, the SAP2000 has the capability of calculating the own weight of various members, hence the 0.01 m of slab thickness was calculated by SAP2000, and the remaining 0.15 m of slab thickness plus the flooring cover were fed to SAP2000. As the building was assumed as public usage building, the live load is 3 kNt /m^2 according to (ECP203-2007).

In summary the following loads were fed to the model

10 mm slab thickness was fed to SAP2000 with self- weight multiplier = 1.0.

The equivalent of 150 mm slab thickness load plus flooring cover, as dead load = 5.25 kN /m^{2}with self- weight multiplier = 1.0.

Live load = 3 kN /m^{2}withself weight multiplier = 0.0.

For edge beams, there is a wall with height of 2.9 m for 1st floor (level 5.00) and 2nd floor (level 8.50) with thickness of 0.12 m, hence the load per square meter including the plastering = 3.0 kN /m^2. As the beam depths were assumed = 0.6 m, hence the clear height of the wall = 2.9 m, therefore the wall load, as dead load, on the edge beams = 2.9 * 3 = 8.7 kN /m$^{\sim}$.

For the 3rd floor (level 12.00), there is a parapet with thickness of 0.12 m and height of 1.0 m, hence the load per meter run is 3.0 kN, this load was considered as dead load on the edge beams.

The SAP2000 has the capability of making combinations as the analysis needs. to get the ultimate straining actions according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007), the ECP203-2007 states that
U = 1.4 Dead + 1.6 Live. The researcher fed SAP2000 by this condition.

3.5.2.2 Analysis and Results, Slab Thickness of 1.0 cm

After feeding SAP2000 by the above loads and running the program, the analysis results for case of slab thickness = 10 mm were presented in appendix A

3.5.3 Case 2; Slab Thickness of 160 mm

This is the real case of the RC structure. In this case, the slab thickness was assumed to be equal to 160mm. this thickness was considered as the minimum thickness requirement (ECP203-2007) for two-way slab supported on beam in four sides.

3.5.3.1 Gravity Load Calculations, Slab Thickness of 160 mm

As the slab thickness = 0.16 m, hence the own weight, ow, = 4.0kN /m^2.

The flooring cover, FC, including plaster was assumed to be equal to be 1.50 kN /m^2.

As mentioned afore, the SAP2000 has the capability of calculating the own weight of various members, hence the 0.16 m of slab thickness was calculated by SAP2000, and the flooring cover loads was fed to SAP2000. As dead loads. As the building was assumed as public usage building, the live load is 3.0kN /m^2 according to (ECP203-2007).

Thus the following loads were fed to the model

160 mm slab thickness was fed to SAP2000 with self weight multiplier = 1.0.

Flooring cover, FC, loads = 1.5 kN /m^2 with self weight multiplier = 1.0.

Live load = 3.0 kN /m^2 with self weight multiplier = 0.0.

For edge beams, there is a wall with height of 3.5 m for 1st floor (level 5.00) and 2nd floor (level 8.50) with thickness of 0.12 m, hence the load per square meter including the plastering = 3.0 kN /m^2. As the beam depths were assumed = 0.6 m, hence the clear height of the wall = 2.9 m, therefore the wall load, as dead load, on the edge beams = 2.9 *3.0 = 8.7 kN /m˜.

For the 3rd floor (level 12.00), there is a parapet with thickness of 0.12 m and height of 1.0 m, hence the load per meter run is 3.0 kN, this load was considered as dead load on the edge beams.

The SAP2000 has the capability of making combinations as the analyst needs. to get the ultimate straining actions according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007), the ECP203-2007 states that
U = 1.4 Dead + 1.6 Live. The researcher fed SAP2000 by this condition.

3.5.3.2 Analysis and Results, Slab Thickness of 160 mm

After feeding SAP2000 by the above loads and running the program, the analysis results for case of slab thickness = 160 mm were presented in appendix A

3.5.4 Case 3; No Slabs

In this case the researcher assumed that the building is consisted of RC frames only, hence the equivalent slab loads including dead loads and live loads were calculated and applied directly to the beams.

Also as the study applied strong column- weak beam principle, and in this case the bending moment in the beam elements are the most important straining action, so the coefficient for bending moment is considered, and according to ECP203-2007 this coefficient equals 2/3.

3.5.4.1 Gravity Load Calculations, No Slabs

As the building is consisting of RC frames only, the researcher assumed that there is a fictitious slab with thickness of 0.16 m, hence the own weight, ow., = 4kN /m^2, and assuming that there was a flooring cover, FC, that was assumed to be equal to1.50 kN /m^2, hence the total dead loads that the slab supposed to carry is 5.5 kN /m^2. These total dead loads can be represented as a linear loads acting on its respective beams.

Also it was assumed that the live loads were 3.0kN /m^2according to (ECP201-2003). Hence the slab loads were calculated as following

The equivalent slab dead loads for every edge beam per meter run = 0.667 * 5.5 * 3.0 = 11kN /m.

Also for edge beams, there is a wall with height of 3.5 m for 1st floor (level 5.00) and 2nd floor (level 8.50) with thickness of 0.12 m, hence the load per square meter including the plastering = 3.0 kN /m^2. As the beam depths were assumed = 0.6 m, hence the clear height of the wall = 2.9 m, therefore the wall load, as dead load, on the edge beams = 2.9 * 3.0 = 8.7 kN /m~ ,hence the total dead load for every edge beam = 11 + 8.7 = 19.7, and the equivalent slab live load for every edge beam per meter run = 0.667 * 3 * 3 = 6 t / m ~

For every inner beam, the equivalent slab dead load = 0.667* 5.5 * 6 = 22 kN/ m~,
and the equivalent slab live load = 0.667 * 3 * 6 = 12 kN/ m~

In summary, the following dead and live loads- the self weight of the beam is calculated by SAP2000- for slabs and beams that were fed to SAP2000 are:

edge beam – 1^{st} and 2^{nd} floors- dead loads = 11 + 8.7 = 19.7 kN / m~

edge beam – 1^{st} and 2^{nd} floors- live loads = 6 kN / m~

edge beam – 3^{rd} floor- dead loads = 11+ 3 = 14 kN / m~

edge beam – 3^{rd} floor- live loads = 6 kN / m~

inner beam dead loads = 22 kN / m~

inner beam live loads = 12 kN / m~

The SAP2000 has the capability of making combinations as the analyst needs. to get the ultimate straining actions according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007), the ECP203-2007 state that U = 1.4 Dead + 1.6 Live. The researcher fed SAP2000 by this condition.

3.5.4.2 Analysis and Results, No Slabs

After feeding SAP2000 by the above loads and made run, the analysis results for case of no slab were presented in appendix A.

3.6 Design for Gravity Loads

After the analyses were made for the three cases of slab thickness via SAP2000, the design was done, using the ultimate strength design method, as following, according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007). Because of this study was not concerned with the internal straining actions in the floors, so no design was done for the floor slabs. Also it is clear that the frame is symmetric in both directions, so it is a logical procedure to takehalf of it in both directions, hence only the quarter of the building was designed.

3.6.1 Assumptions

- the characteristic strength of concrete, f_{cu} = 30MPa,
- the yield strength or proof strength for reinforcement, fy = 360 MPa .
- the percentage of longitudinal reinforcement in the cross section, μ
- the cross sectional area, mm^2, A_s ,
- $\mu_{max.}$ is the maximum percentage of tension reinforcement in reinforced concrete section,

- $\mu_{min.}$ is the minimum percentage of tension reinforcement in reinforced concrete section,
- γ_c is the material strength reduction factor for concrete,
- γ_s is the material strength reduction factor for steel.

3.6.2 Design for Sections

As the structure has similarity in both directions, therefore a quarter of the structure was designed according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007) as following, for example:

Assume f_{cu}= 30 MPa , f_y = 360 MPa,

b = 250 mm concrete cover = 50 mm

Given Mul = 105.5 kNm

Since the assumed total depth, t = 600 mm , hence the effective depth, d= 550 mm

$$R1 = \frac{Mul * 1000000}{fcu * b * d * d} = 0.0465 < R1\ max. = 0.129$$

From charts that provided by (ECP203-2007), $\omega = 0.052 < \omega\ max. = 0.18$

Hence $\mu = 0.0044 < \mu\ max. = 0.015$ &> $\mu\ min. = 0.0031$

Therefore, area of reinforcement, $A_s = \mu * b * d. = 426\ mm^2$

The remainder of the beam sections was designed with the same procedure, and for every beam element three sections were designed; one at both ends and the third one at the middle. This was done for the three cases, slab thickness of 10 mm, slab thickness of 160 mm, and no slab.

3.6.3 Case 1; Slab Thickness of 10 mm

As the beam element sections were assumed as 250 x 600 mm, therefore the reinforcement of the sections was depicted in the following figures

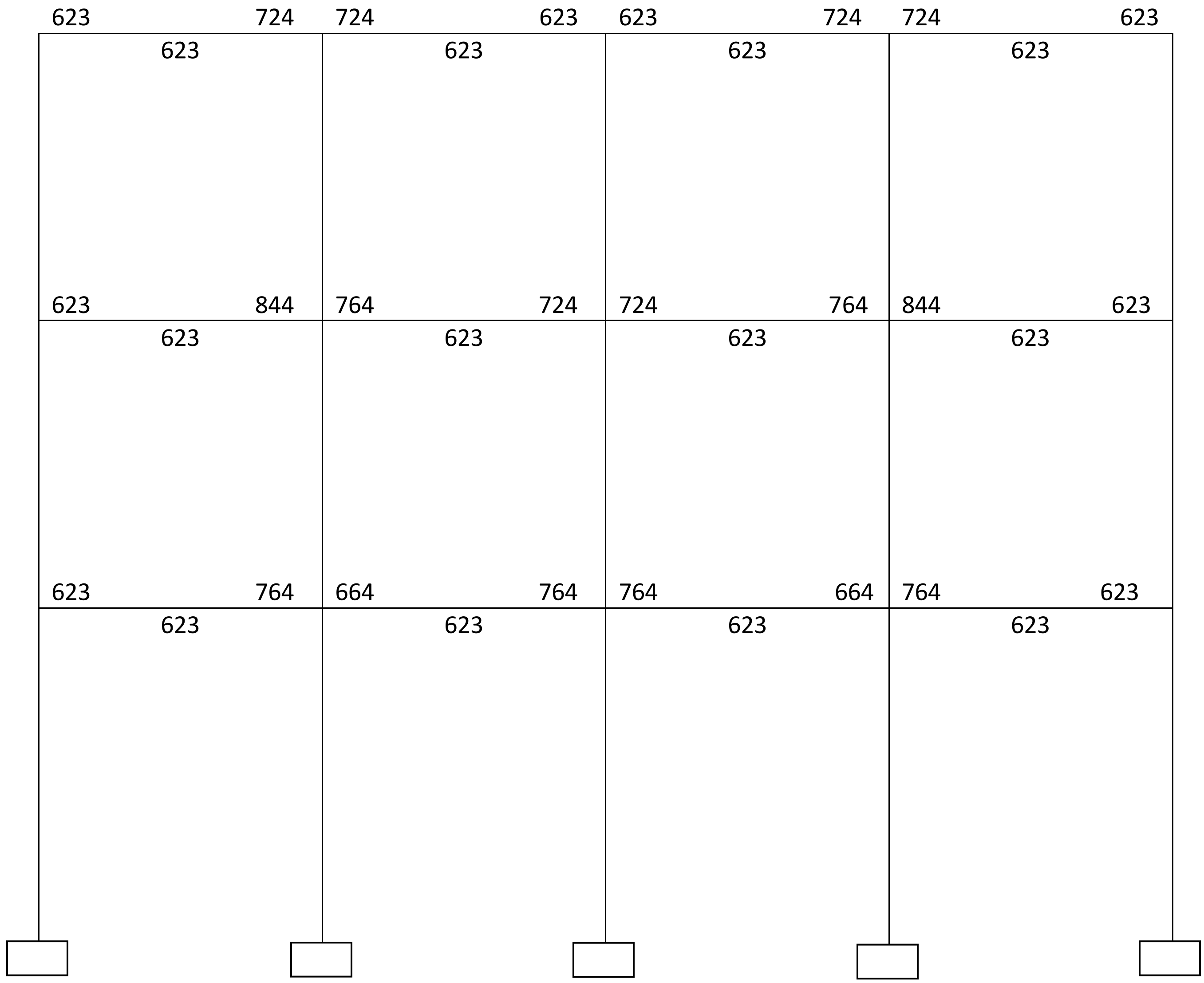


Fig. 3.11 Reinforcement (mm^2) of Beams on axes 1 and 4, Gravity Loads, t_s = 10 mm

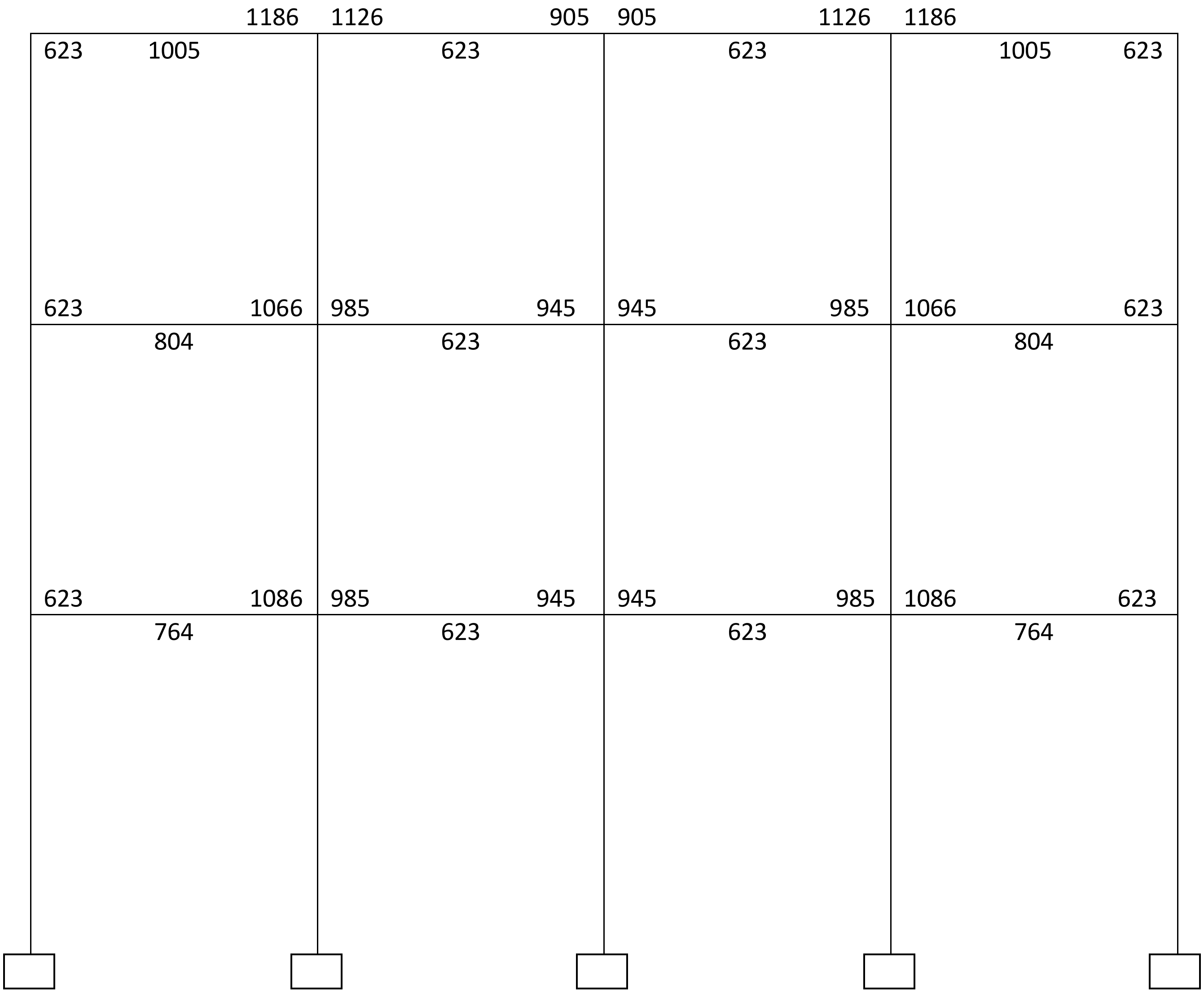


Fig. 3.12 Reinforcement (mm^2) of Beams on axes 2 and 3, Gravity Loads, t_s = 10 mm

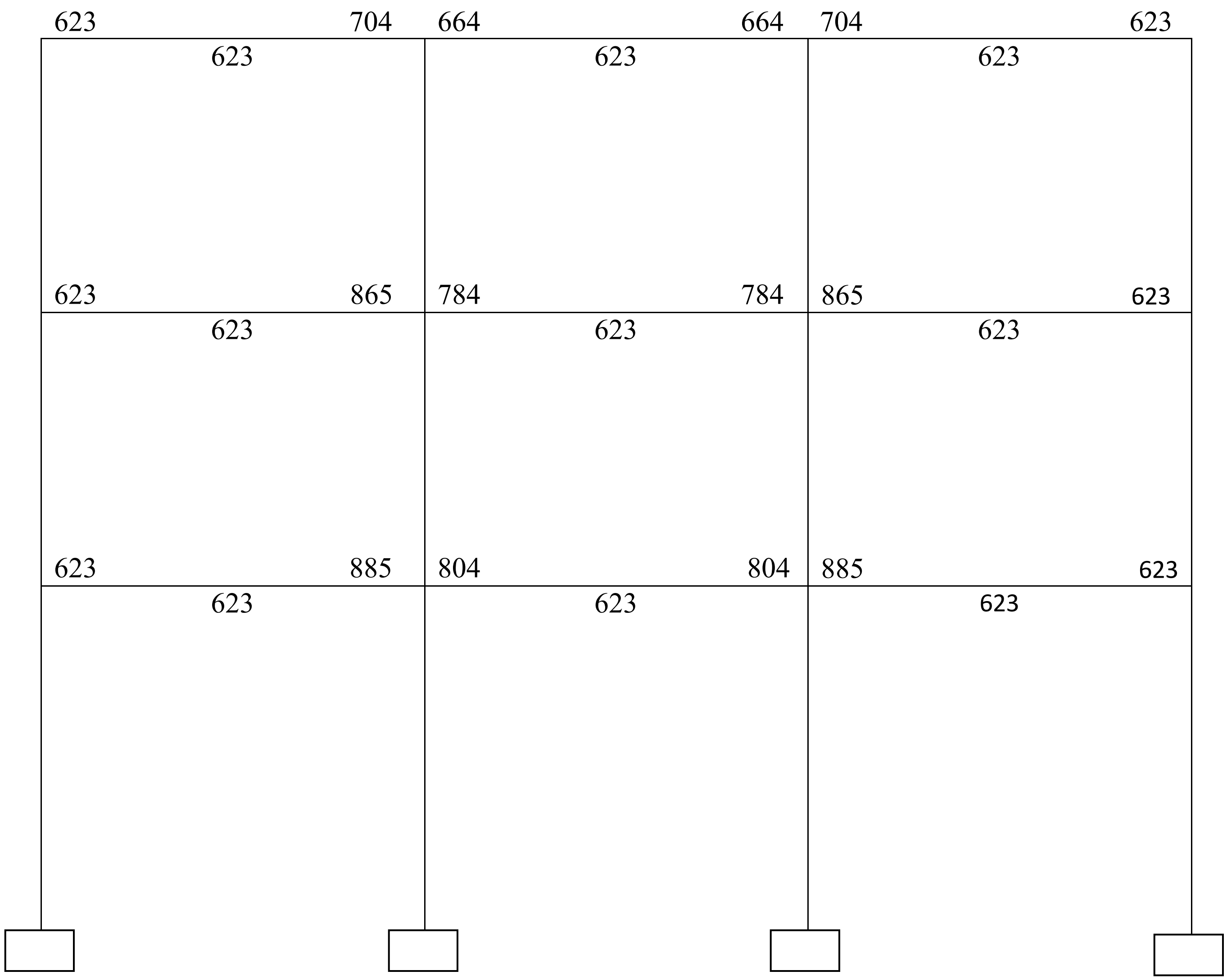


Fig. 3.13 Reinforcement (mm^2) of Beams on axes A and E, Gravity Loads, t_s= 10 mm

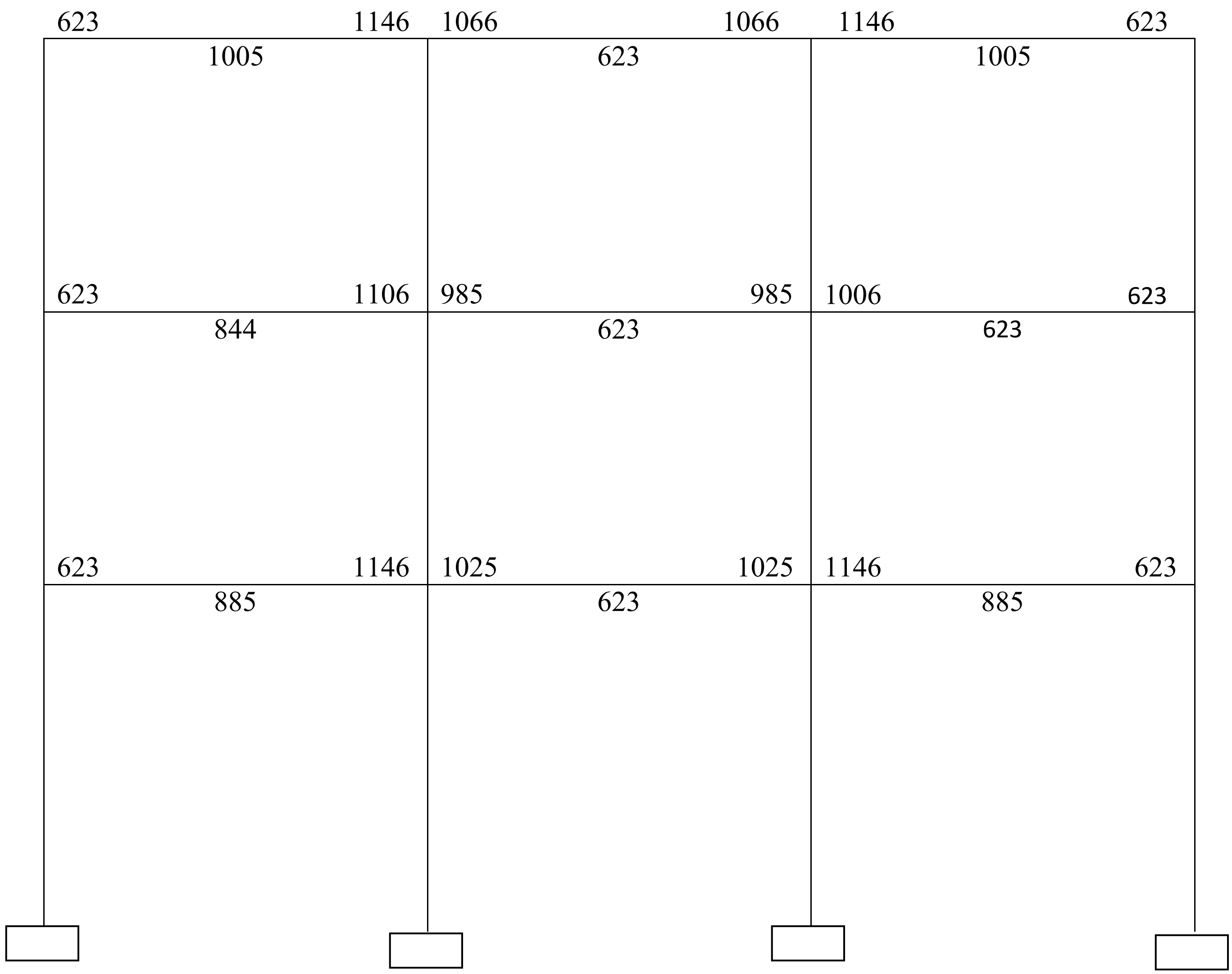


Fig. 3.14 Reinforcement (mm^2) of Beams on axes B, C, and D, Gravity Loads, t_s= 10 mm

3.6.4 Case 2; Slab Thickness of 160 mm

As the beam element loads have the same values, the straining actions and hence the section dimensions do not vary for this case, so the figures that shown above represent the same values for the design of sections.

3.6.5 Case 3; No Slab

As the beam element loads have the same values, the straining actions and hence the section dimensions do not vary for this case, so the figures that shown above represent the same values for the design of sections.

CHAPTER 4

SEISMIC ANALYSIS AND DESIGN

4.1 Introduction

On 12th October 1992 at 3:09 pm, Egypt local time, an earthquake struck Egypt. This earthquake that Egypt had not affected for about 200 years caused the following

- 545 deaths.
- 6512 injured.
- 50000 people homeless.

After that catastrophic event, the scientists and engineers gave a big attention to revise code for seismic loads, and the seismic code was modified for several times. Now the Arab Republic of Egypt is divided according to seismic activities into five zones according to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (201_2003). The building was assumed to be located in any of the three different seismic zones 1, 3, and 5.

For each of these seismic zones, the horizontal elastic and design seismic loads were calculated, and the vertical elastic and design seismic loads were calculated. After calculations were done for each of the seismic zone, the SAP2000 was fed by horizontal and vertical design seismic loads and the results were obtained. The final step is the design of the frame elements were conducted according to ECP203_2007.

4.2Assumptions

The following assumptions were made

- According to the assumed zones, the designed ground acceleration, a_g, will vary according to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (201_2003). the following table restated the values of designed ground acceleration, a_g, for the three seismic zones

Table 4.1 Designed Ground Acceleration, a_g.

Zone No.	a_g
1	0.10 g
3	0.15 g
5	0.25 g

- The soil is assumed as cohesionless (gravel, sand), so the ground type is C, hence T_B = 0.1,T_C=0 .25, T_D=1.2, and S=1.5 according to the Egyptian Code for Calculating Loads and Forces on Structures and Buildings (201_2003), where T_B , T_C, T_D, and S are values according to Response Spectrum Type (1)

- Since the structure is Reinforced Concrete Frame, Hence, Design Damping Factor, η =1, &, η_v =.7

- The building is assumed as ordinary building, hence the importance category is III, hence the importance factor, γ is 1.

Since the total height of the building measured from the top level of the foundations, H, is 12 m<60 m and it is a reinforced concrete structures, therefore C_t is 0.05.

Then, the approximate value for the Fundamental Period, according to the formula in the Egyptian Code for Calculating Loads and Forces on Structures and Buildings (201_2003) is $T = C_t\, H^{3/4}$ = 0.32 s.

Because each floor will move in both horizontal directions, north-south and east-west, at each level by the same displacement, so a diaphragm is introduced at each floor level; hence all the joints at each floor level will have the same displacement. As a result of this real condition, the straining actions at frames that lies on the axes 1 and 4, 2 and 3, A and E, and B, C, and D, are almost have the same values; hence the research designed one for each group.

Also the seismic loads can be reversible, so the reinforcement will be symmetrical.

4.3 Elastic Response Spectrum

Instead of repeating the equations and procedures in each zone, the researcher began by rewriting the seismic equations according to the Egyptian Code for Calculating Loads and Forces on Structures and Buildings (201_2003), and calculated the corresponding values for elastic and design horizontal and vertical response spectrum.

4.3.1 Elastic Horizontal Response Spectrum

According to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (201_2003), there are four different regions for calculation the Elastic Response Spectrum;

a) $0 \leq T \leq T_B$ (4.1)

b) $T_B \leq T \leq T_C$ (4.2)

c) $T_C \leq T \leq T_D$ (4.3)

d) $T_C \leq T \leq 4$ s (4.4)

Where:

S_e (T)	Elastic Horizontal Response Spectrum
T	Vibration Period,
a_g	Designed Ground Acceleration,
$T_{B,}$ T_C, and T_D	Limits depending on the soil type,
η	Design Damping factor depending on the type of structure (RC), and
S	Soil factor.

4.3.1.1 Elastic Horizontal Response Spectrum, Zone One

Since the structure was assumed that it lies in zone 1, hence the Design Ground Acceleration, a_g= 0.1g = 0.981 m/s^2. By applying the previous equations, 4.1 through 4.4, and considering the a_g= 0.981 m/s^2, the following elastic horizontal response figure is obtained and the detailed corresponding values are in the Appendix B, Table B.1

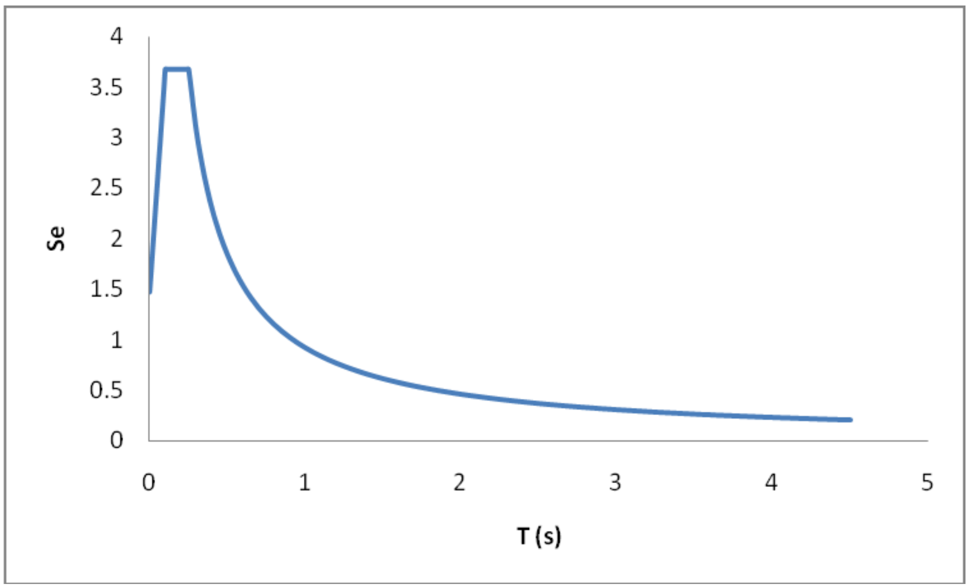


Fig 4.1 Elastic response Spectrum, zone 1

4.3.1.2 Elastic Horizontal Response Spectrum, Zone Three

Since the structure was assumed that it lies in zone 3, hence the Designed Ground Acceleration, a_g=.15g = 1.4715 m/s^2. By applying the previous equations, 4.1 through 4.4, and considering the a_g= 1.4715 m/s^2, the following elastic horizontal response figure is obtained and the detailed corresponding values are in the Appendix B, Table B.2

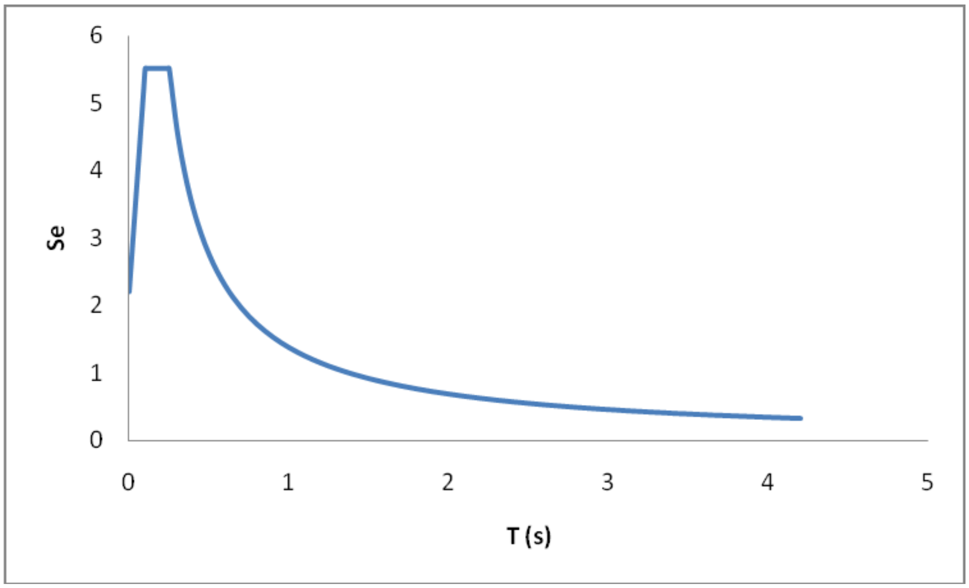


Fig 4.2 Elastic response spectrum, zone 3

4.3.1.3 Elastic Horizontal Response Spectrum, Zone Five

Since the structure was assumed that it lies in zone 5, hence the Design Ground Acceleration, a_g =.25g = 2.4525 m/s^2. By applying the previous equations, 4.1 through 4.4, and considering the a_g = 2.4525 m/s^2, the following elastic horizontal response figure is obtained and the detailed corresponding values are in the Appendix B, Table B.3

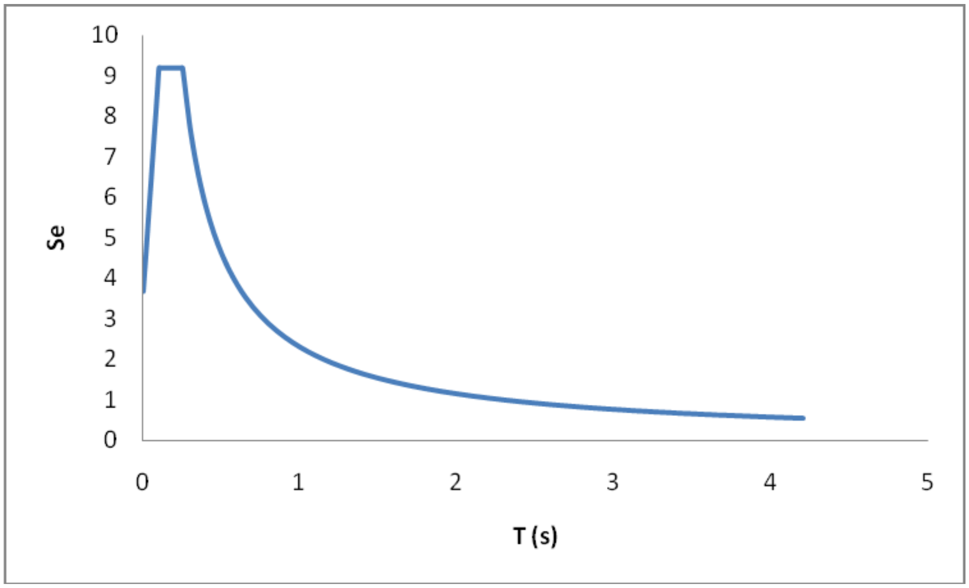


Fig 4.3 Elastic response spectrum, zone 5

4.4 Design Response Spectrum

Again, instead of repeating the equations and procedures in each zone, the researcher began by rewriting the seismic equations according to the Egyptian Code for Calculating Loads and Forces on Structures and Buildings (201_2003), and under the following assumption:

The structure is a simple space frame, the vertical load is transmitted through the frame elements, and the total lateral loads are carried by frames, hence Response Modification Factor, R. = 7

As the Response Modification (force reduction) Factor, R= 7.0, the following equations are rewritten according to the Egyptian Code for Calculating Loads and Forces on Structures and Buildings (201_2003)

a. $0 \leq T \leq T_B$ (4.5)

b. $T_B \leq T \leq T_C$ (4.6)

c. $T_C \leq T \leq T_D$ (4.7)

d. $T_C \leq T \leq 4$ s (4.8)

Where S_d (T) is the Design Horizontal Response Spectrum,

R is the Response Modification (force reduction) Factor according to the structural system of the building.

And the other terms had been defined in Sec. 4.3.1

Since we have T = 0.32 s, T_B =0.1, T_C = 0.25, T_D = 1.2& R= 7.0, then by applying in the above equations we got the following:

4.4.1 Design Horizontal Response Spectrum, Zone 1

Since the structure was assumed that it lies in zone 1, hence the Designed Ground Acceleration, a_g= 0.1g = 0.981 m/s^2. By applying the previous equations, 4.5 through 4.8, and considering the a_g= 0.981 m/s^2, the following designed horizontal response figure was obtained and the detailed corresponding values are in the Appendix B, Table B.4

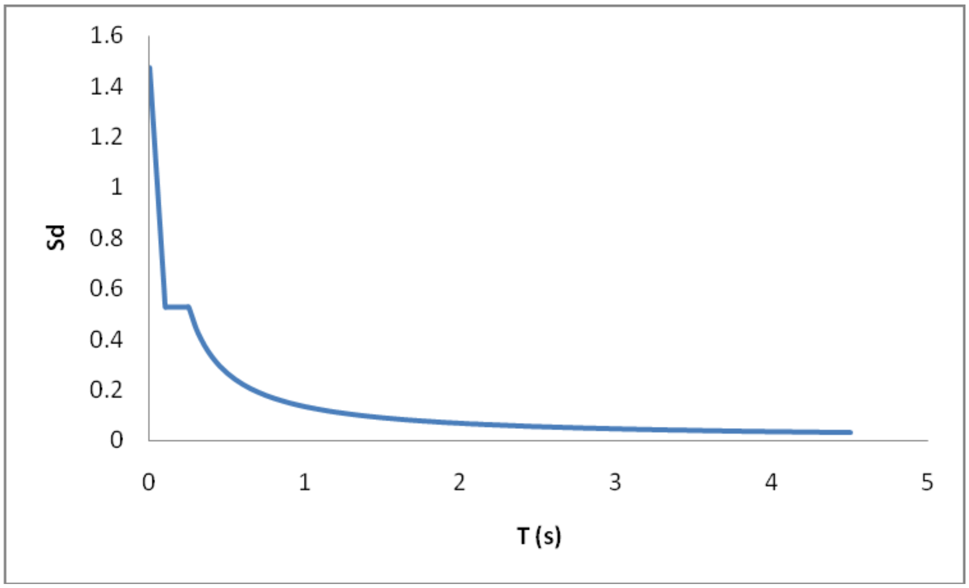


Fig 4.4 Design response spectrum, zone 1

4.4.2 Design Horizontal Response Spectrum, Zone 3

Since the structure was assumed that it lies in zone 3, hence the Designed Ground Acceleration, a_g=.15g = 1.4715 m/s^2. By applying the previous equations, 4.5 through 4.8, and considering the a_g= 1.4715 m/s^2, the following design horizontal response figure was obtained and the detailed corresponding values are in the Appendix B, Table B.5

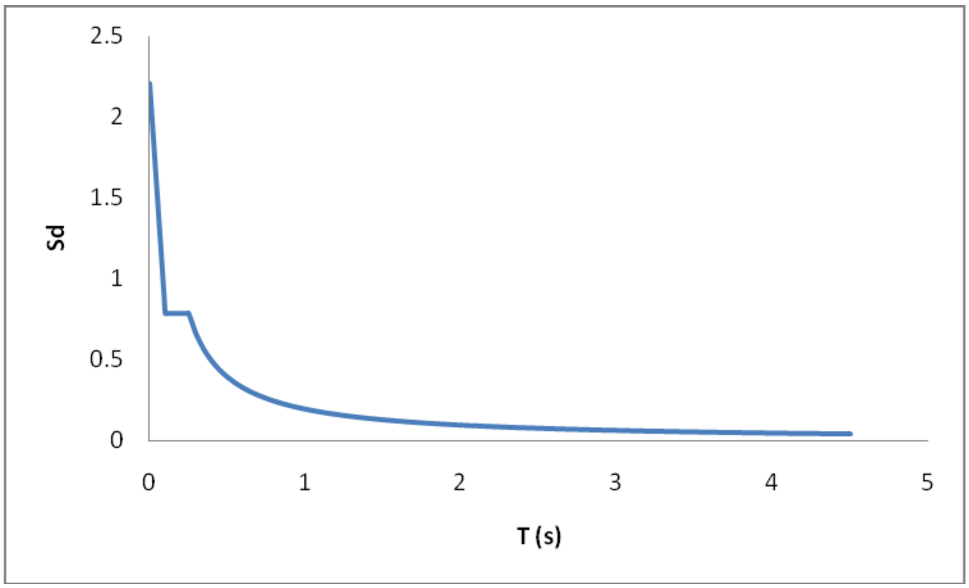


Fig 4.5 Design response spectrum, zone 3

4.4.3 Design Horizontal Response Spectrum, zone 5

Since the structure was assumed that it lies in zone 5, hence the Designed Ground Acceleration, a_g =.25g = 2.4525 m/s^2. By applying the previous equations, 4.5 through 4.8, and considering the a_g = 2.4525 m/s^2, the following designed horizontal response figure was obtained and the detailed corresponding values are in the Appendix B, Table B.6

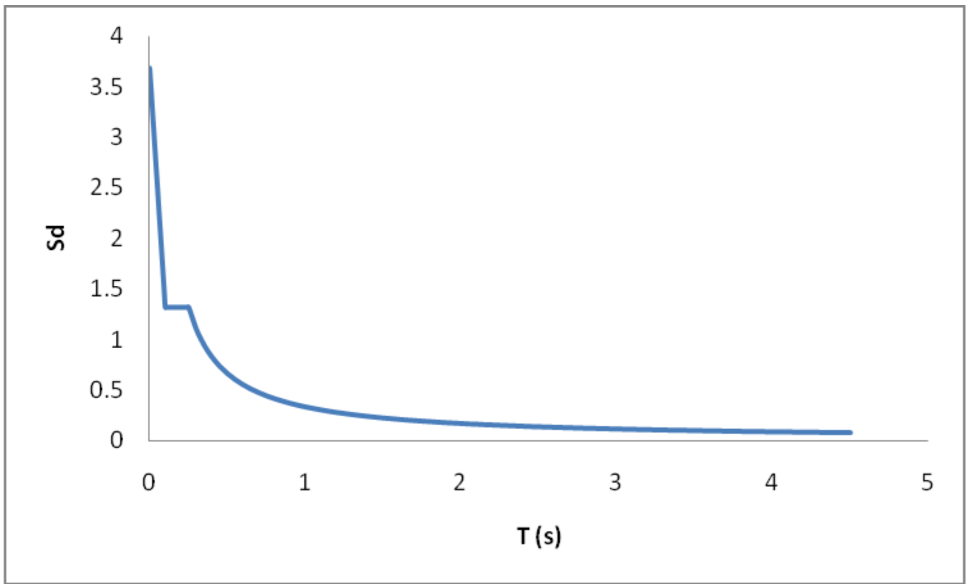


Fig 4.6 Design response Spectrum, zone 5

4.5 Design Vertical Response Spectrum

According to the Egyptian Code of Practice for Calculating Loads and Forces on Structures and Buildings (201_2003), the design vertical response spectrum shall be taken equal to the horizontal response spectrum considering R= 1.0

Therefore the previous horizontal response spectrum will be reused as design vertical response spectrum.

4.6 Seismic Analyses Results

As the structure is symmetrical in both directions, so the straining actions is typical for elements on axes A and E, for elements on axes B,C and D,for elements on axes 1 and 4,and for elements on axes 2 and 3.

4.6.1 Zone 1, Slab Thickness of 10 mm

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in appendix B, table B.7

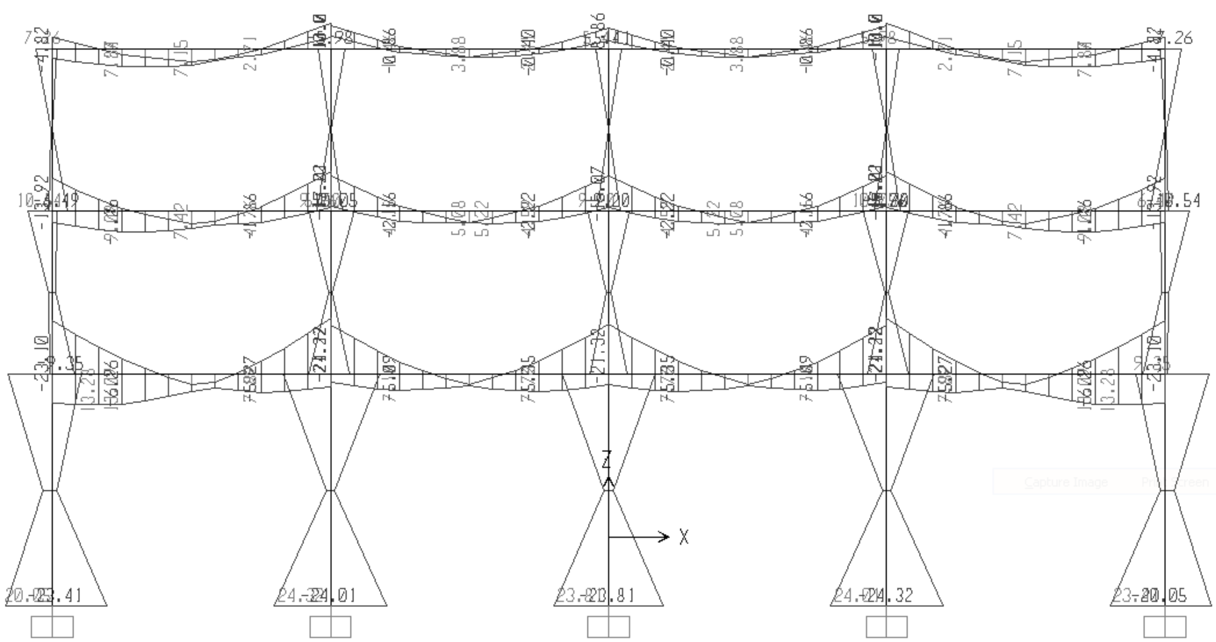


Fig 4.7 Envelope of bending moment diagram for zone 1, slab thick. = 10 mm

4.6.2 Zone 1, Slab Thickness of 160 mm

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in appendix B, table B.8

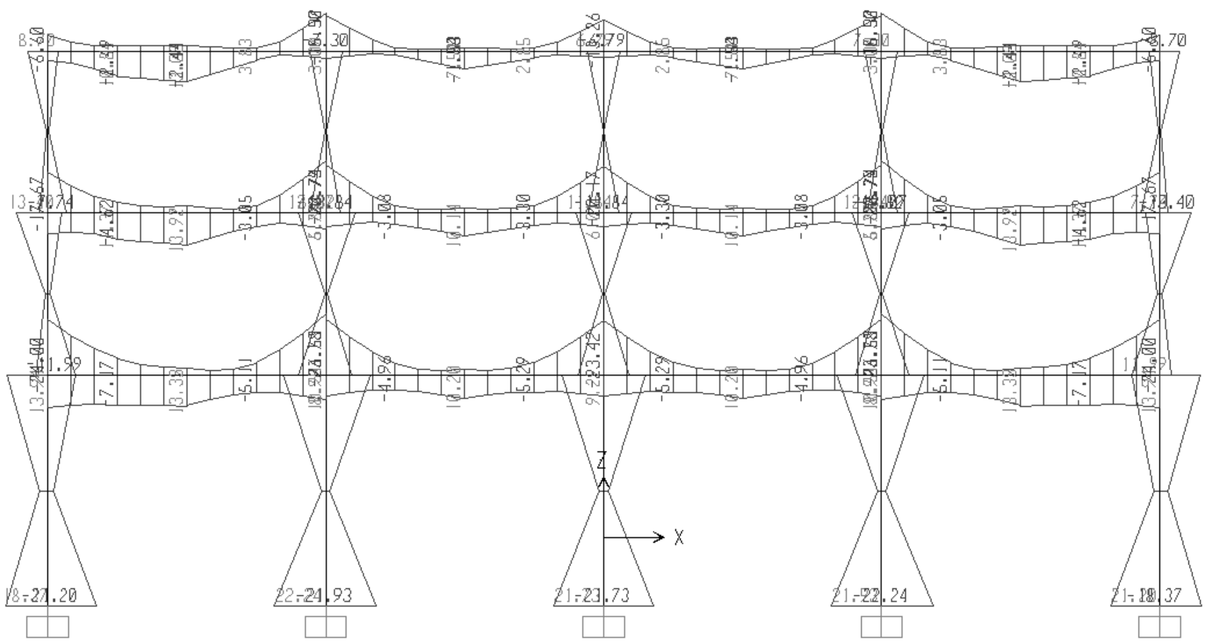


Fig 4.8 Envelope of bending moment diagram for zone 1, slab thick. = 160 mm

4.6.3 Zone 1, No Slab

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in Appendix B, Table B.9

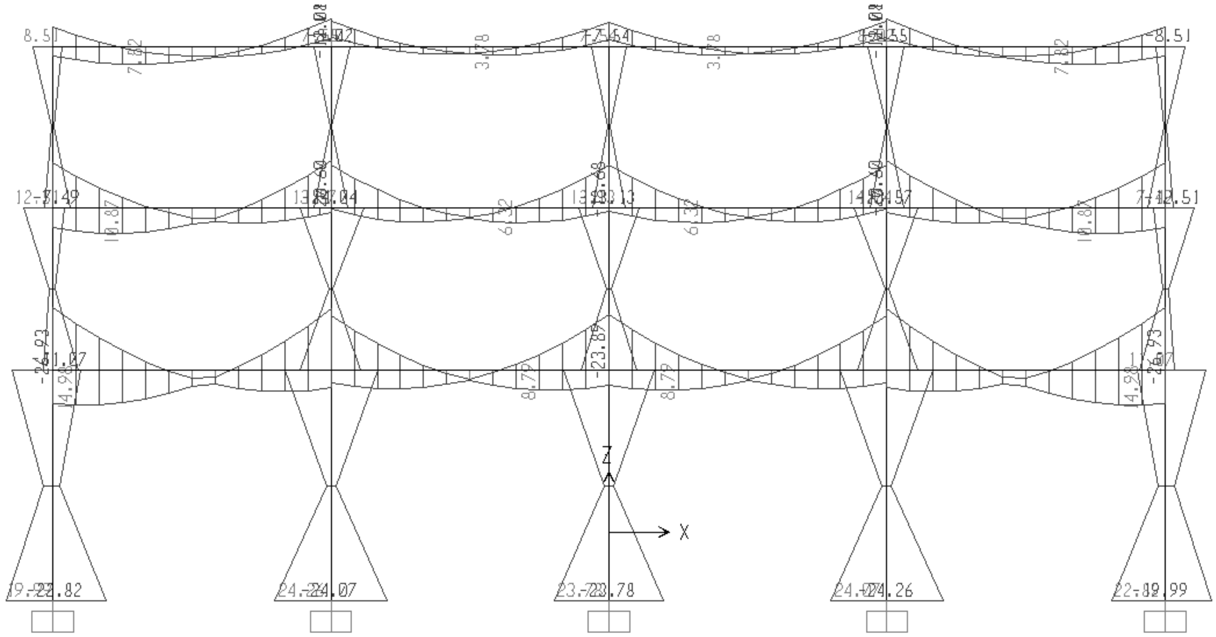


Fig. 4.9 Envelope of bending moment diagram for zone 1, no slab

4.6.4 Zone 3, Slab Thickness of 10 mm

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in Appendix B, Table B.10

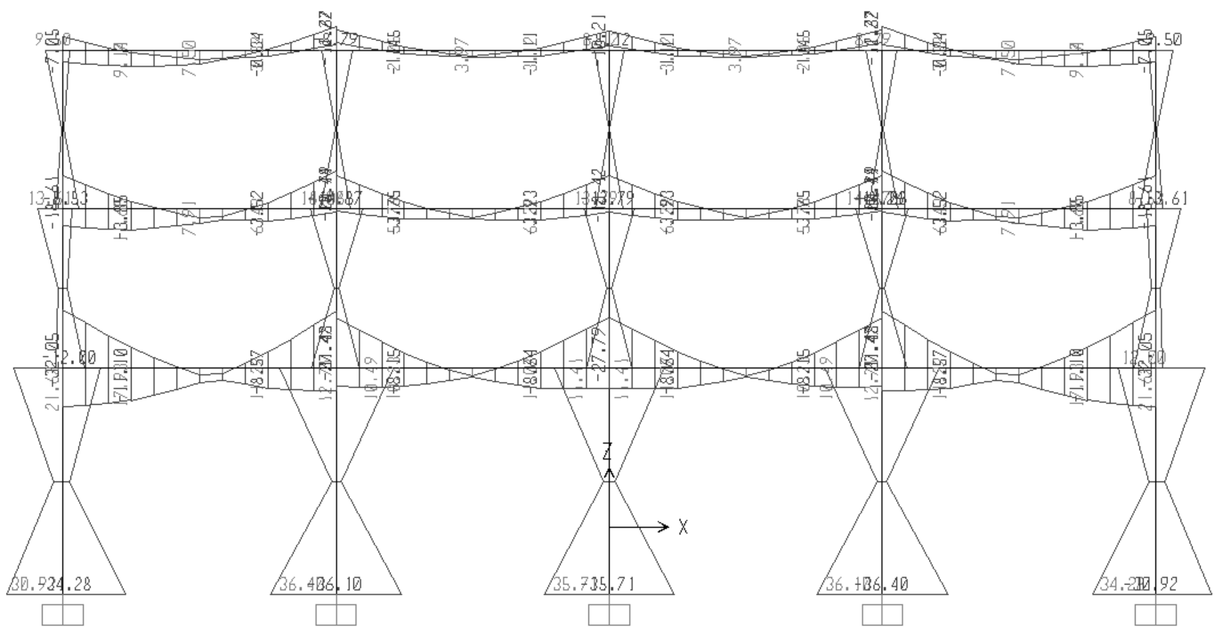


Fig. 4.10 Envelope of bending moment diagram for zone 3, slab thick. = 10 mm

4.6.5 Zone 3, Slab Thickness of 160 mm

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in Appendix B, Table B.11

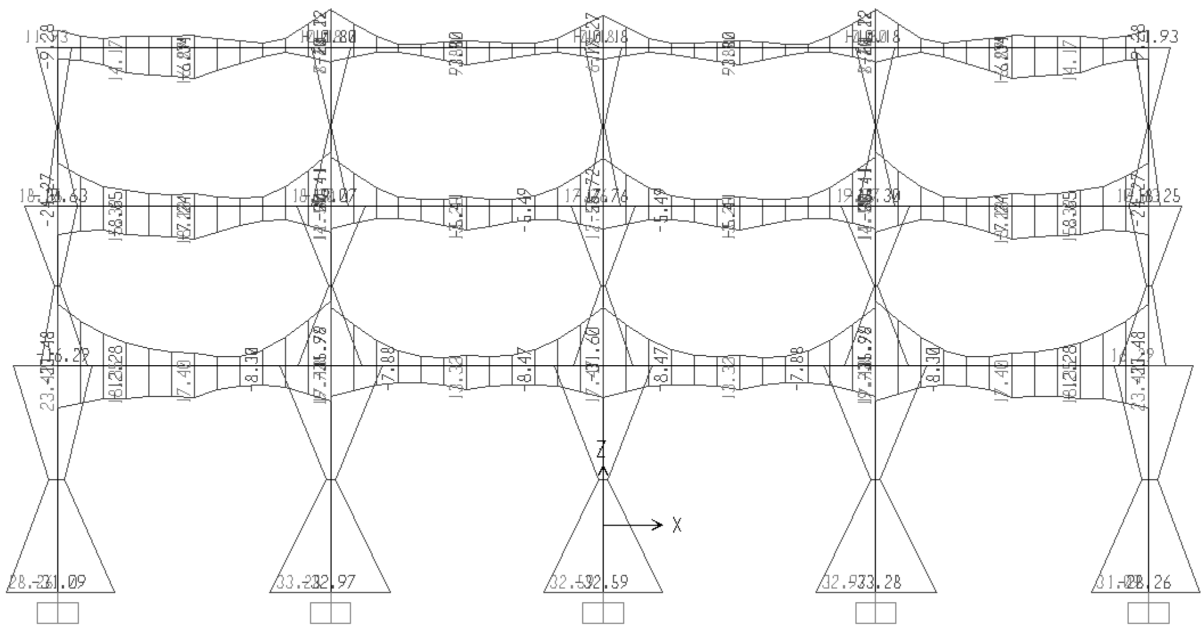


Fig. 4.11 Envelope of bending moment diagram for zone 3, slab thick. = 160 mm

4.6.6 Zone 3, No Slab

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in appendix B, Table B.12

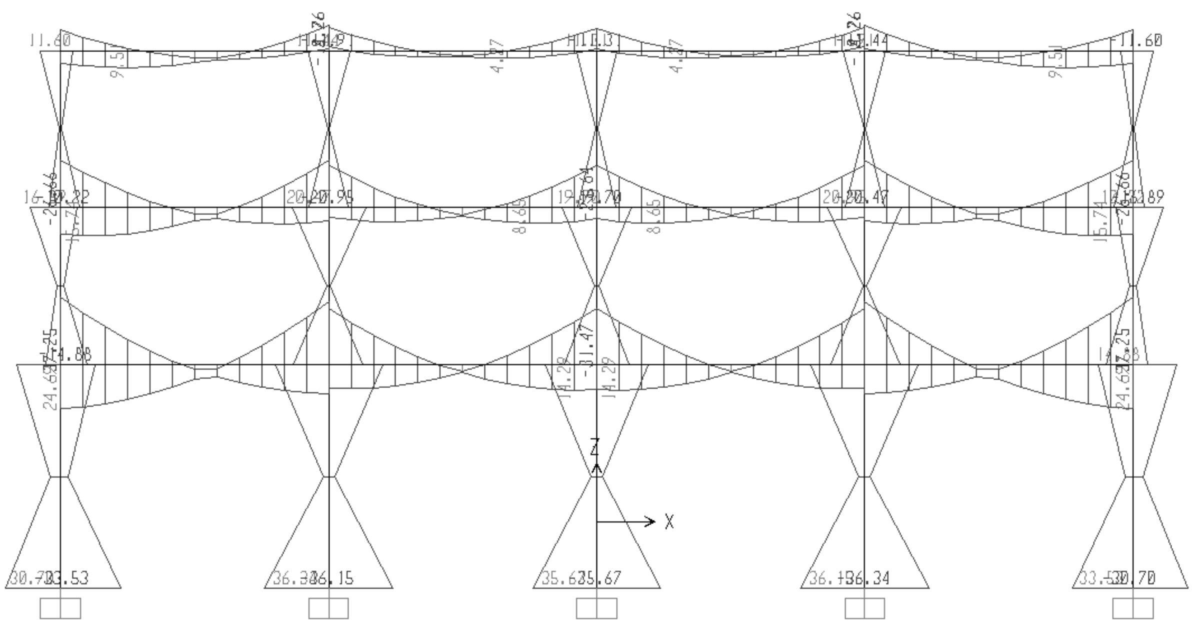


Fig. 4.12 Envelope of bending moment diagram for zone 3, no Slab

4.6.7 Zone 5, Slab Thickness of 10 mm

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in Appendix B, Table B.13

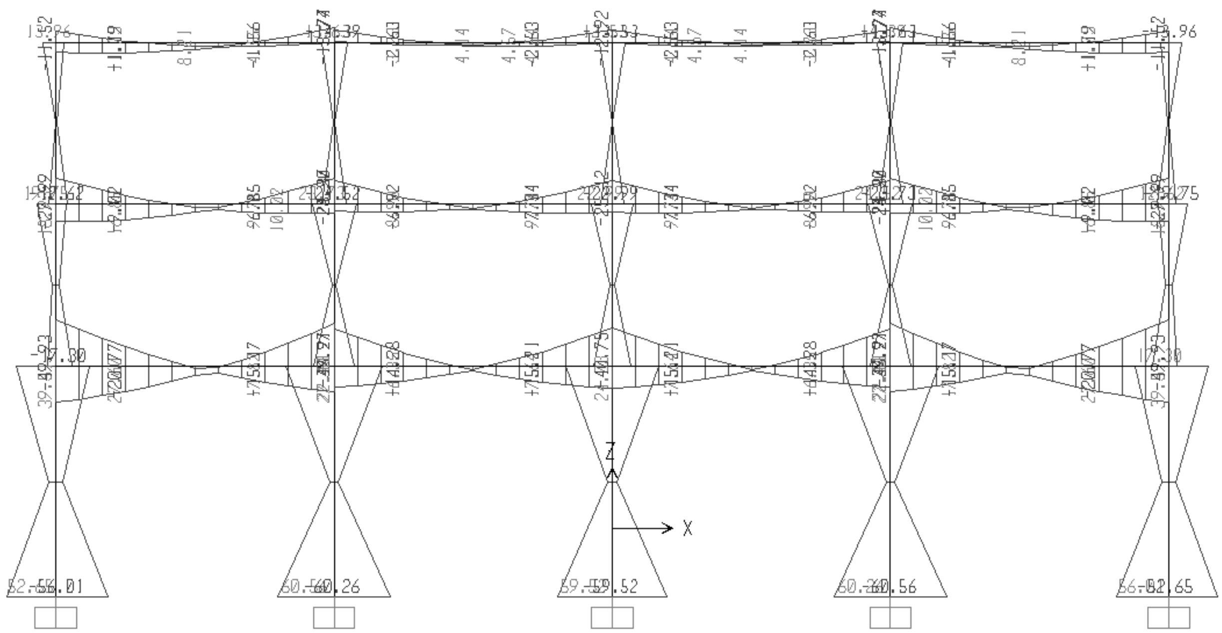


Fig. 4.13 Envelope of bending moment diagram for zone 5, slab thick. = 10 mm

4.6.8 Zone 5, Slab Thickness of 160 mm

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in Appendix B, Table B.14

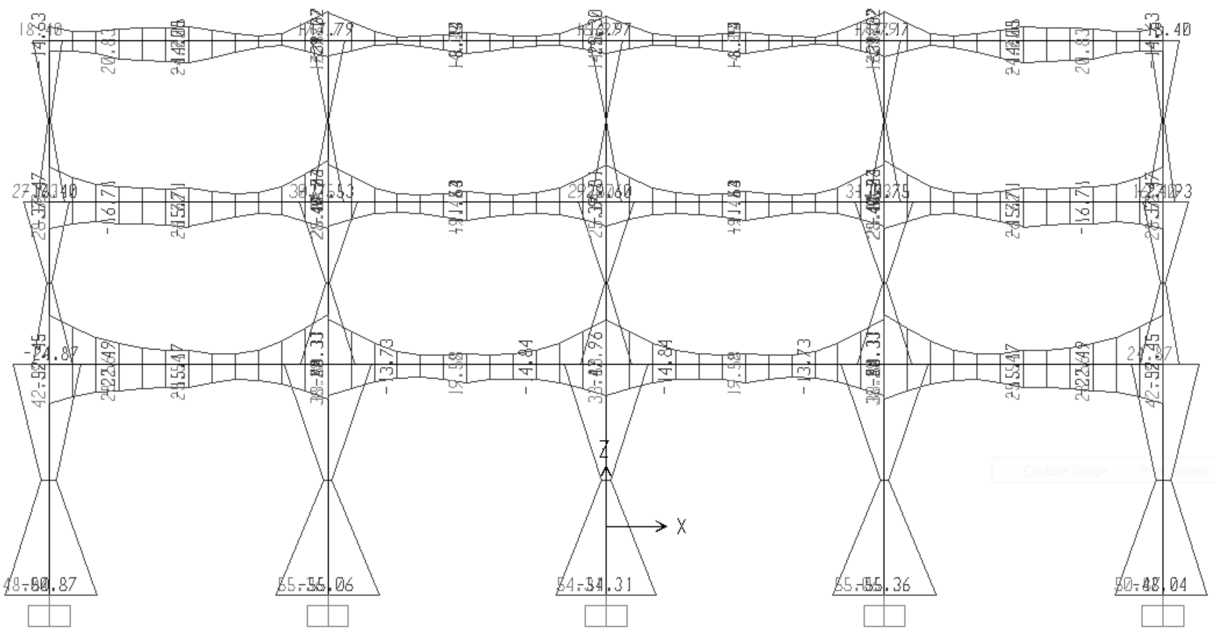


Fig. 4.14 Envelope of bending moment diagram for zone 5, slab thick. = 160 mm

4.6.9 Zone 5, No Slab

The following figure depicts the bending moment values for the frame elements on axis A, as an example. The rest of the analysis results were presented in appendix B, table B.15

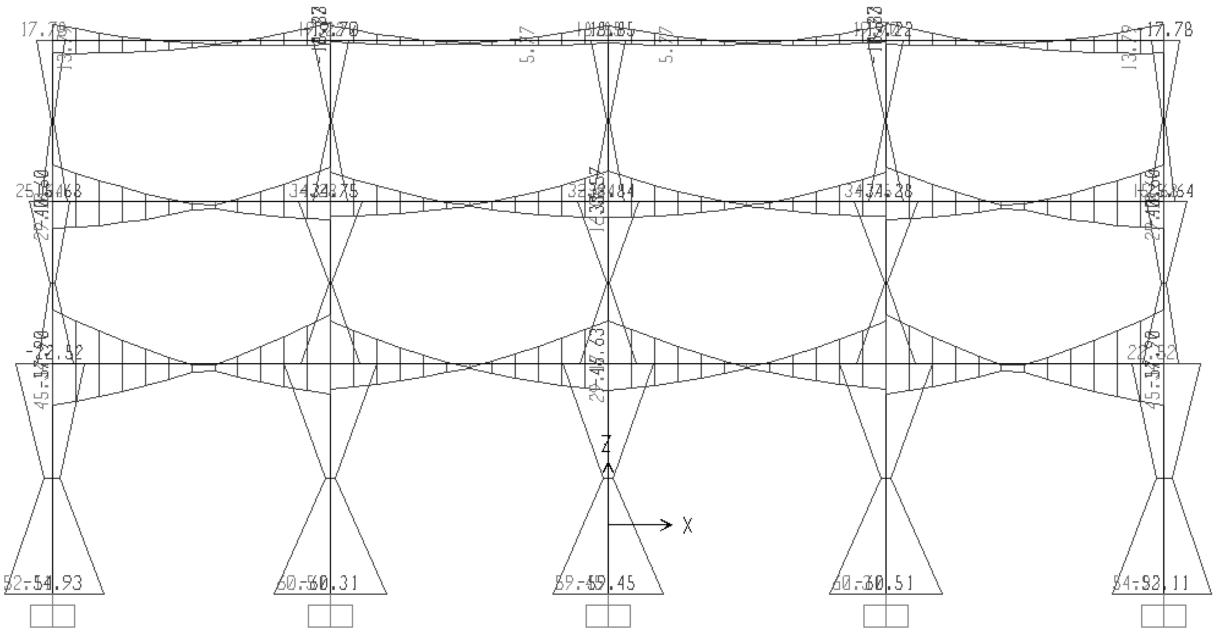


Fig. 4.15 Envelope of bending moment diagram for zone 5, no slab

4.7 Seismic analyses Design

After analyzing the building for every zone, zone 1, 3, and 5, the beam element sections were designed according to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007). The area of reinforcement steel was calculated for every section. The procedure that was followed was identically to what mentioned in the previous chapter. The results were listed as following.

4.7.1 Zone 1, Slab Thickness of 10 mm

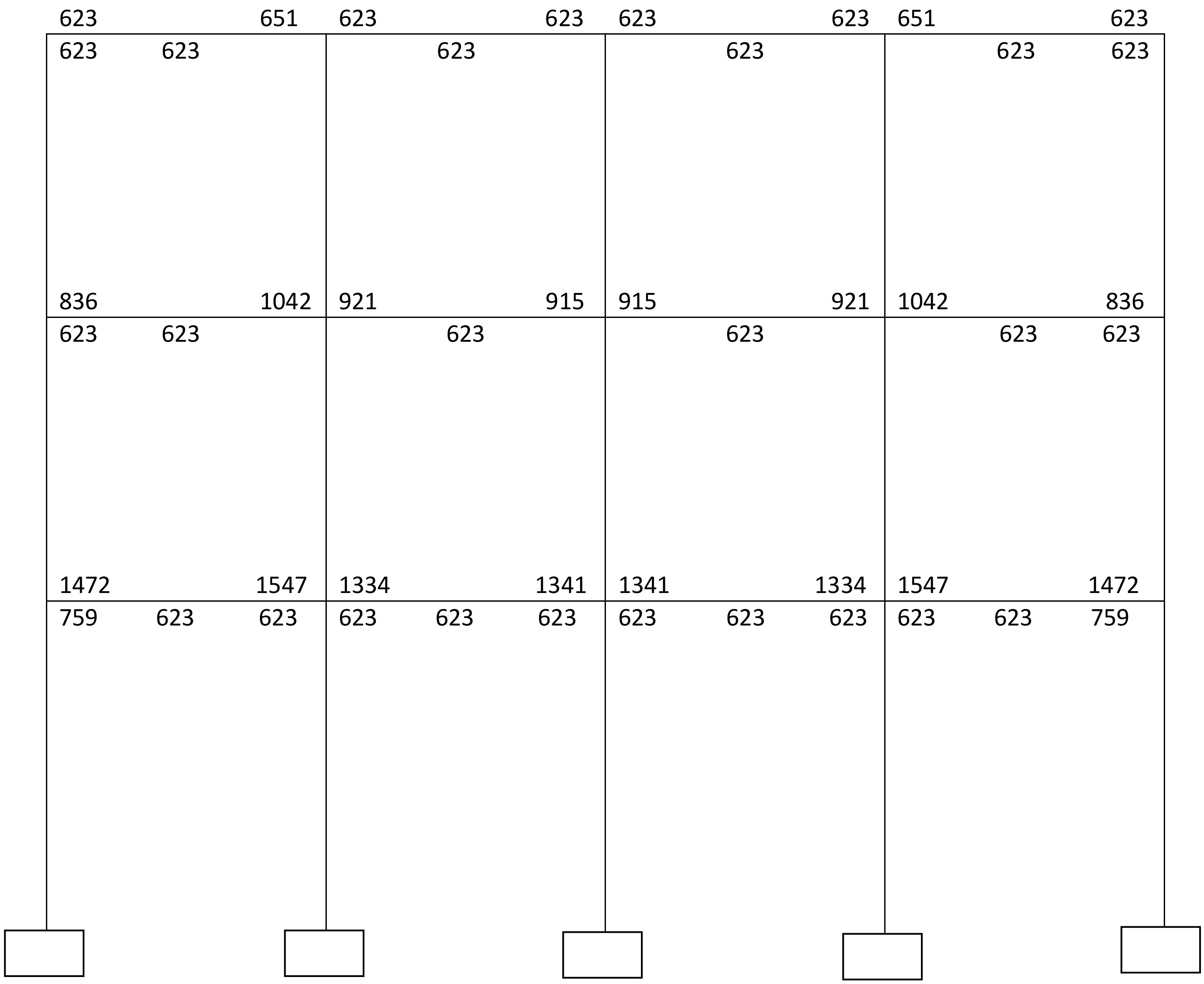


Fig. 4.16 Reinforcement (mm^2) of beams on axes 1 and 4, zone_1 loads, t_s= 10 mm

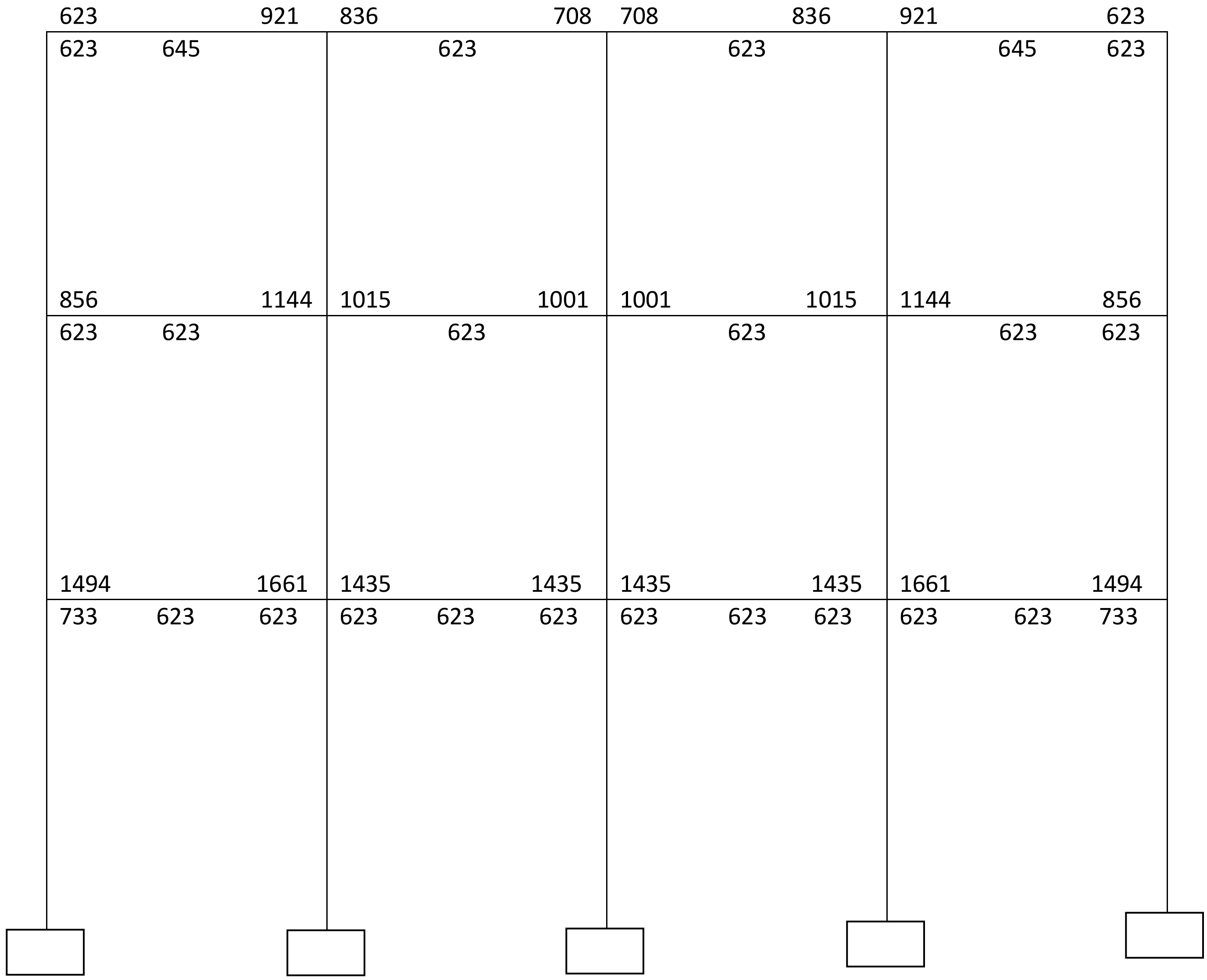


Fig. 4.17 Reinforcement (mm^2) of beams on axes 2and 3, zone_1 loads, t_s= 10 mm

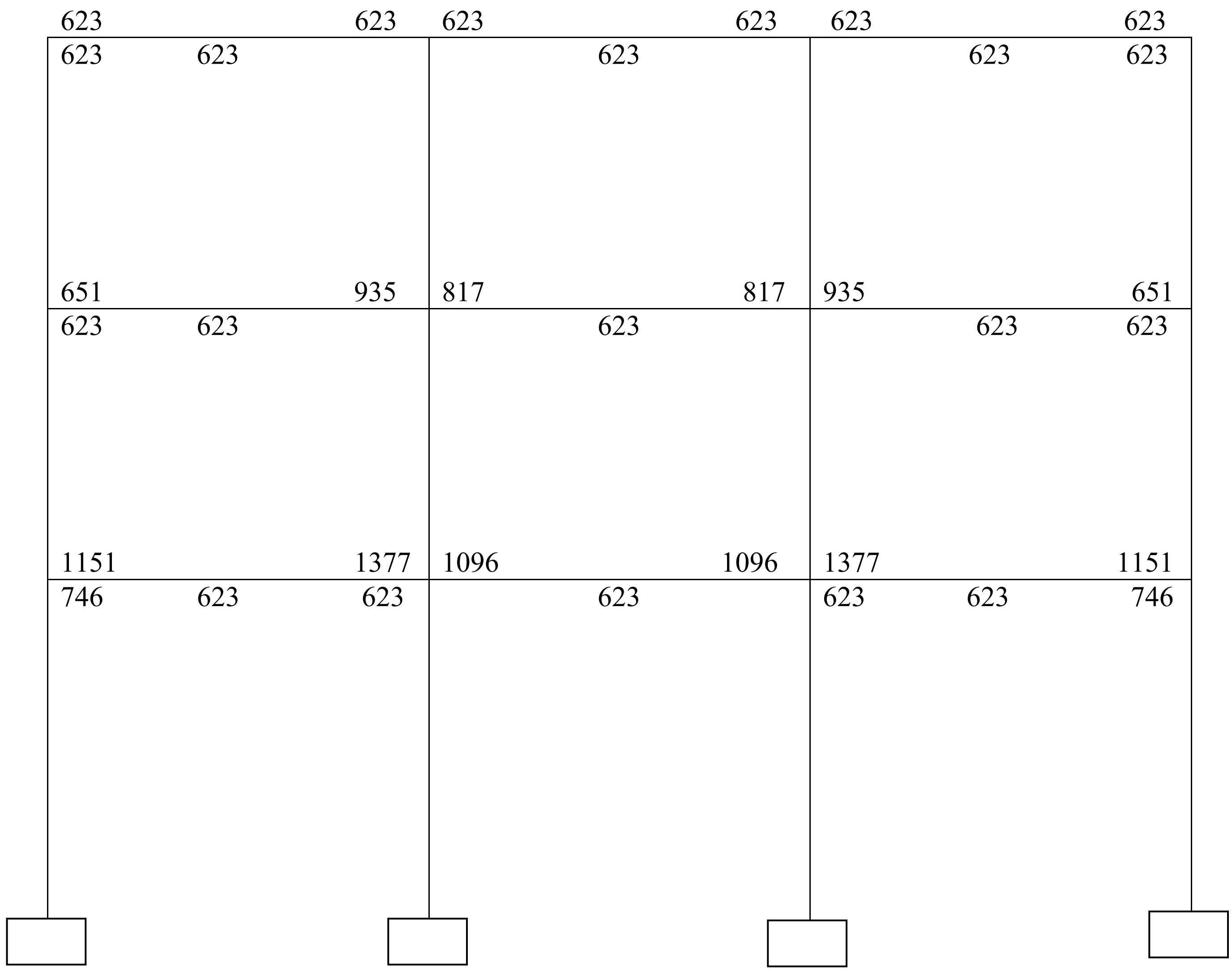


Fig. 4.18 Reinforcement (mm^2) of beams on axes A and E, zone_1 loads, t_s= 10 mm

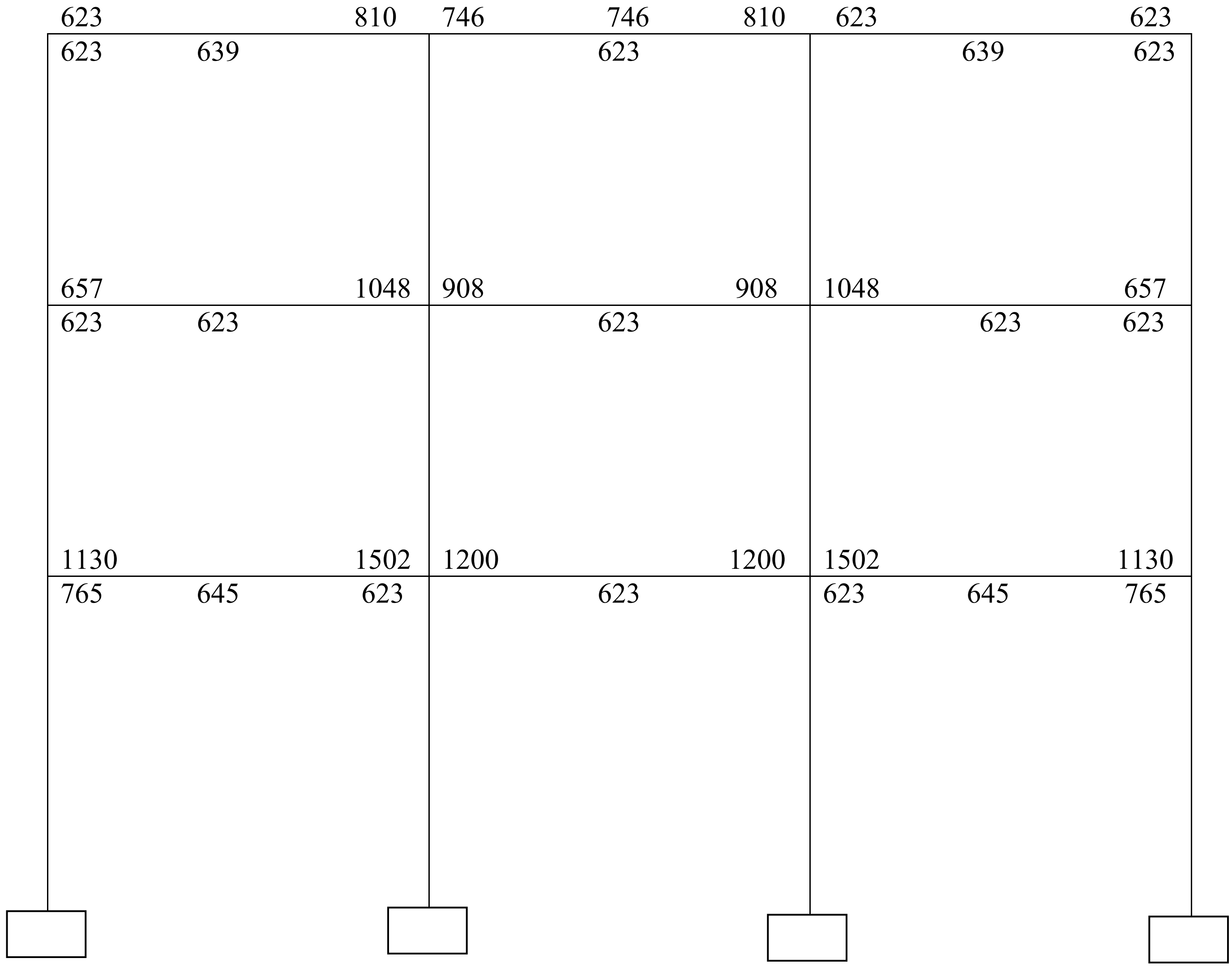


Fig. 4.19 Reinforcement (mm^2) of beams on axes B, C, and D, zone_1 loads, t_s= 10 mm

4.7.2 Zone 1, Slab Thickness of 160 mm

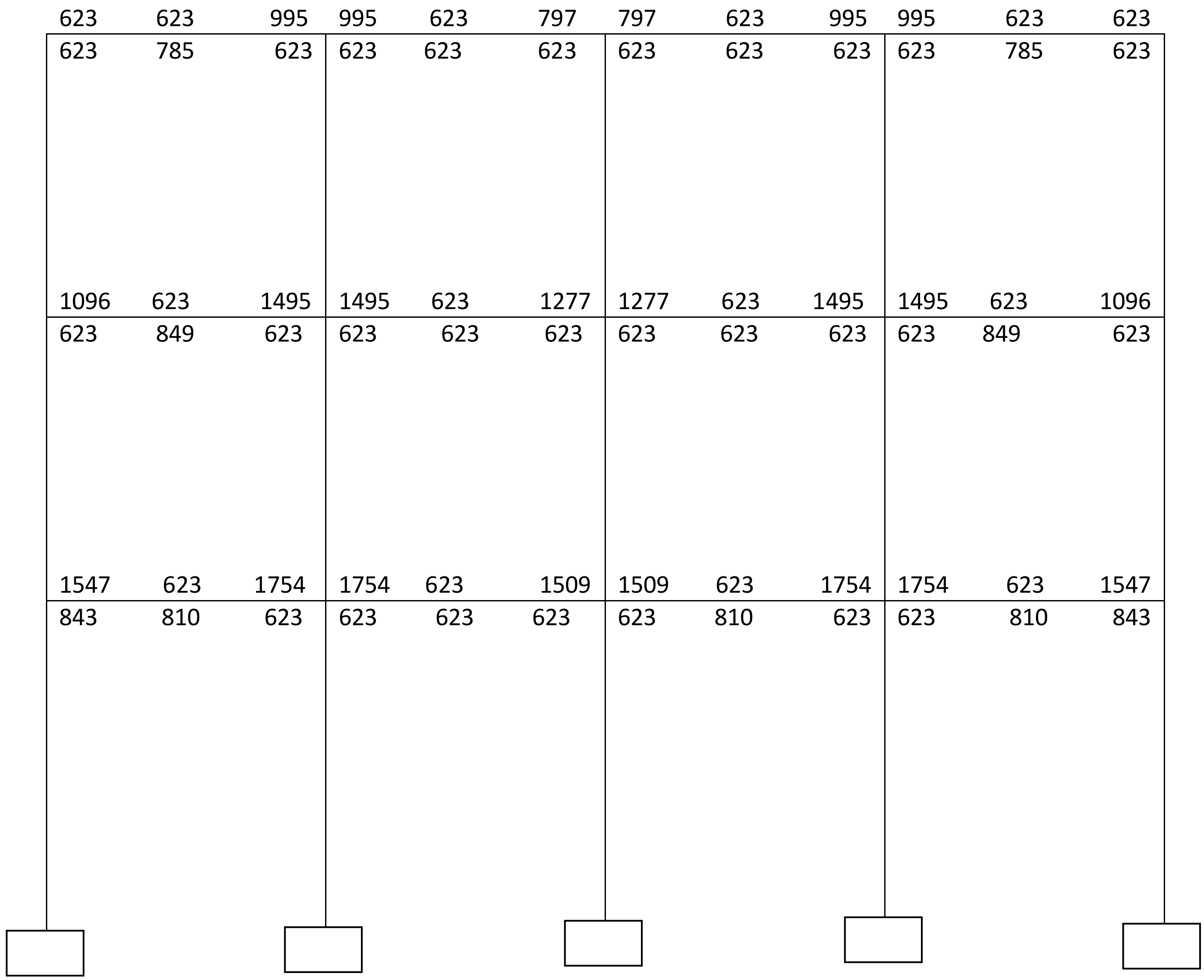


Fig. 4.20 Reinforcement (mm[2]) of beams on axes 1 and 4, zone_1 loads, t_s= 160 mm

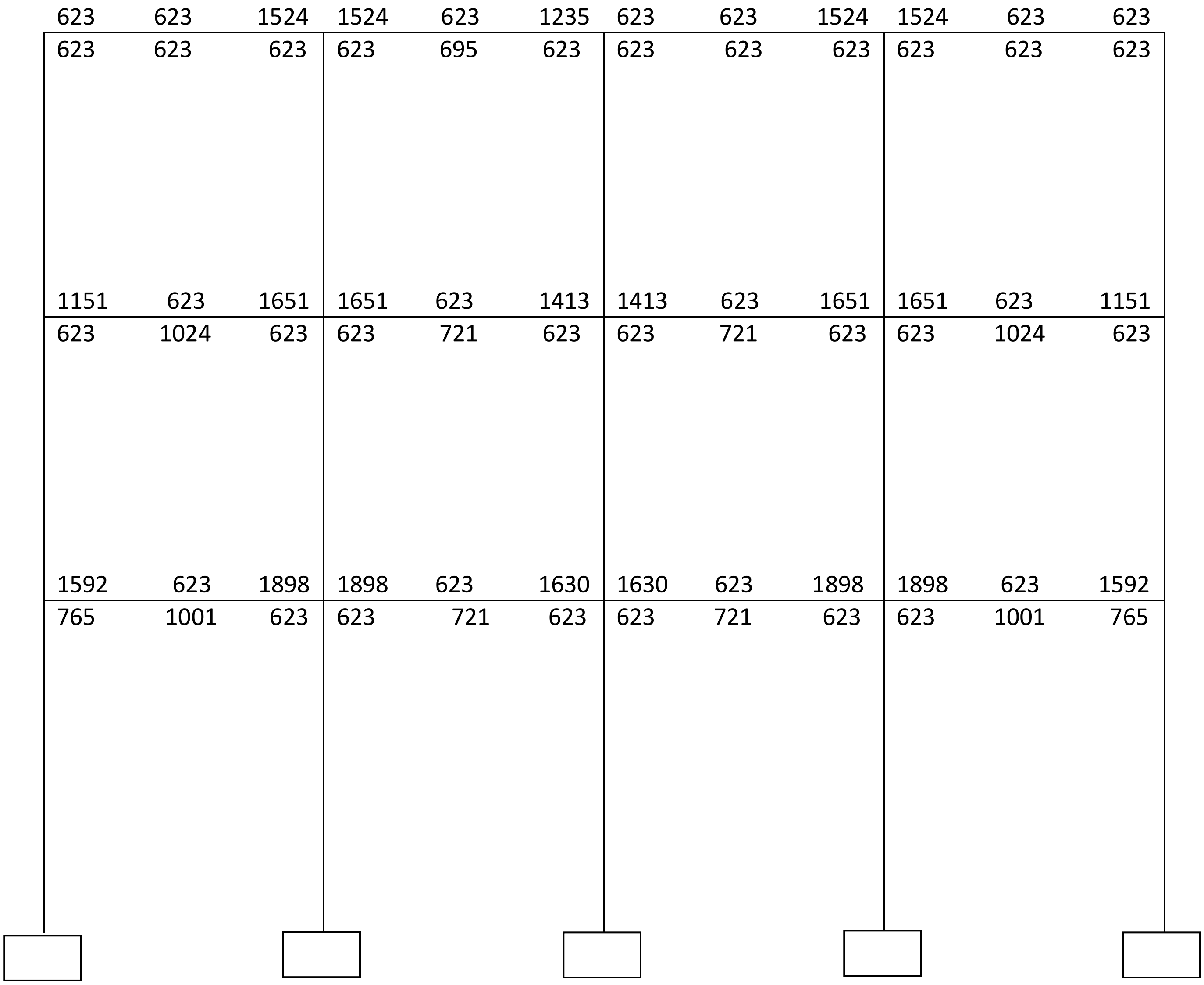


Fig. 4.21 Reinforcement (mm^2) of beams on axes 2and 3, zone_1 loads, t_s= 160 mm

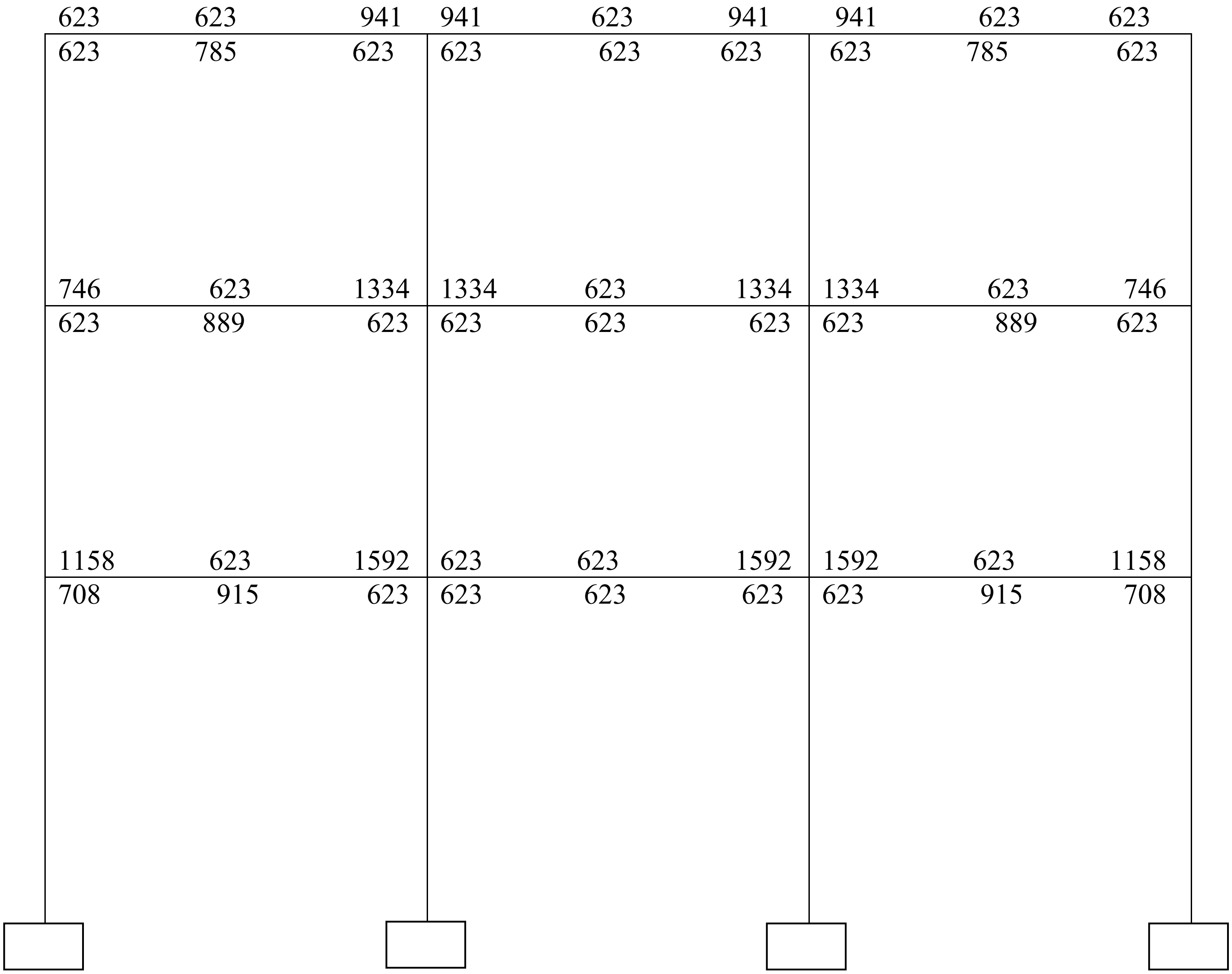


Fig. 4.22 Reinforcement (mm²) of beams on axes A and E, zone_1 loads, t_s= 160 mm

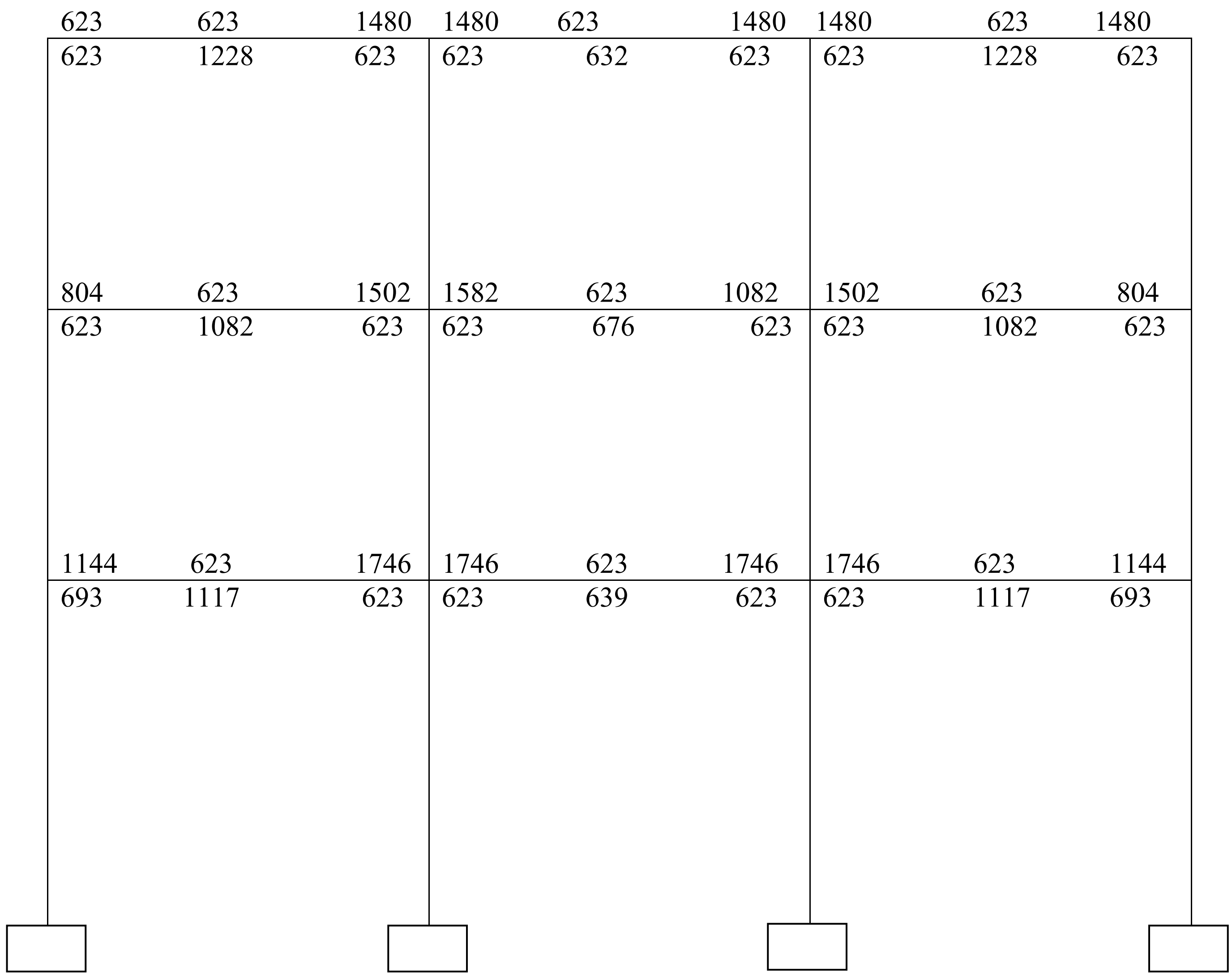


Fig. 4.23 Reinforcement (mm^2) of beams on axes B, C, and D, zone_1 loads, t_s= 160 mm

4.7.3 Zone 1, No Slab

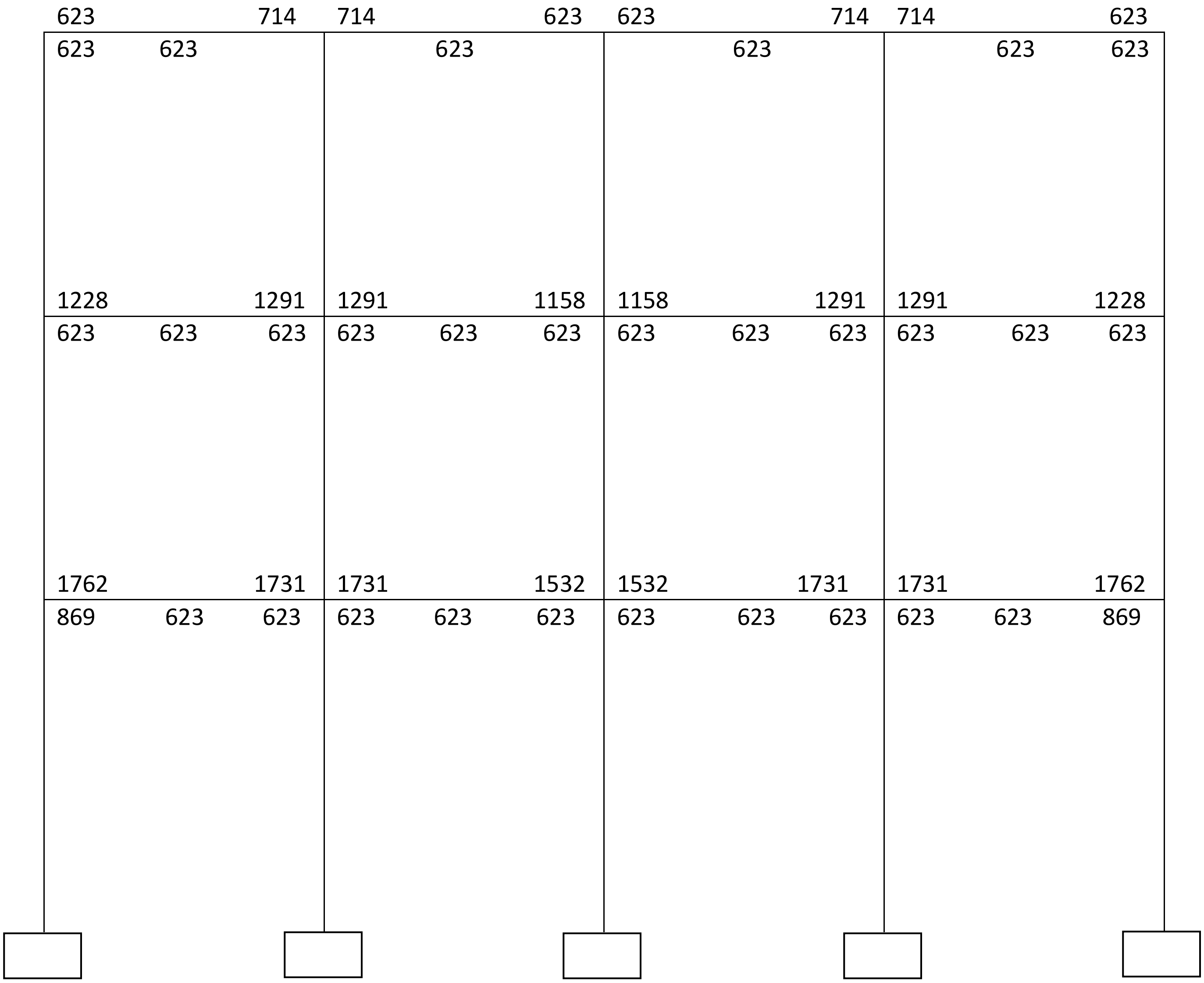


Fig. 4.24 Reinforcement (mm²) of beams on axes 1 and 4, zone_1 loads, no slab

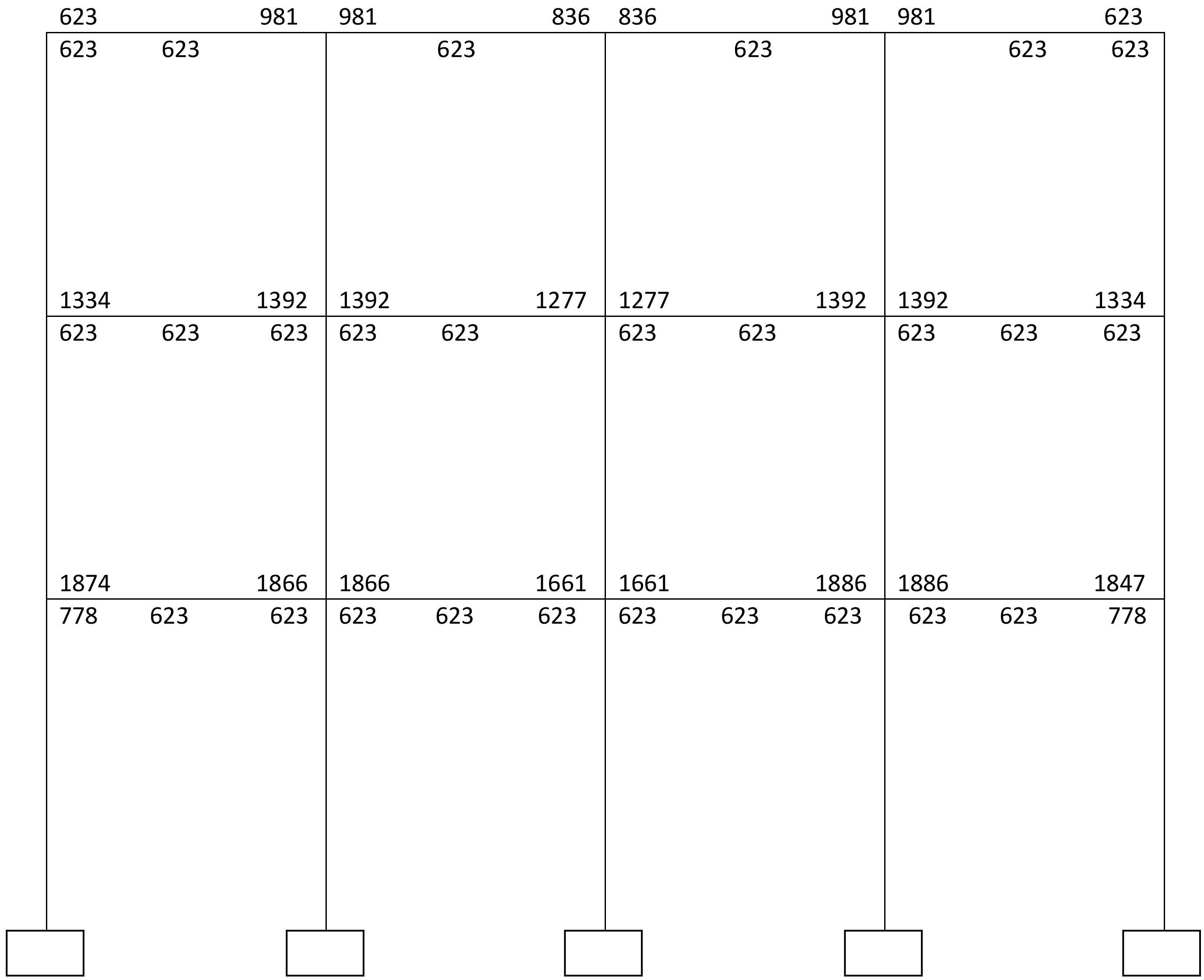


Fig. 4.25 Reinforcement (mm²) of beams on axes 2and 3, zone_1 loads, no slab

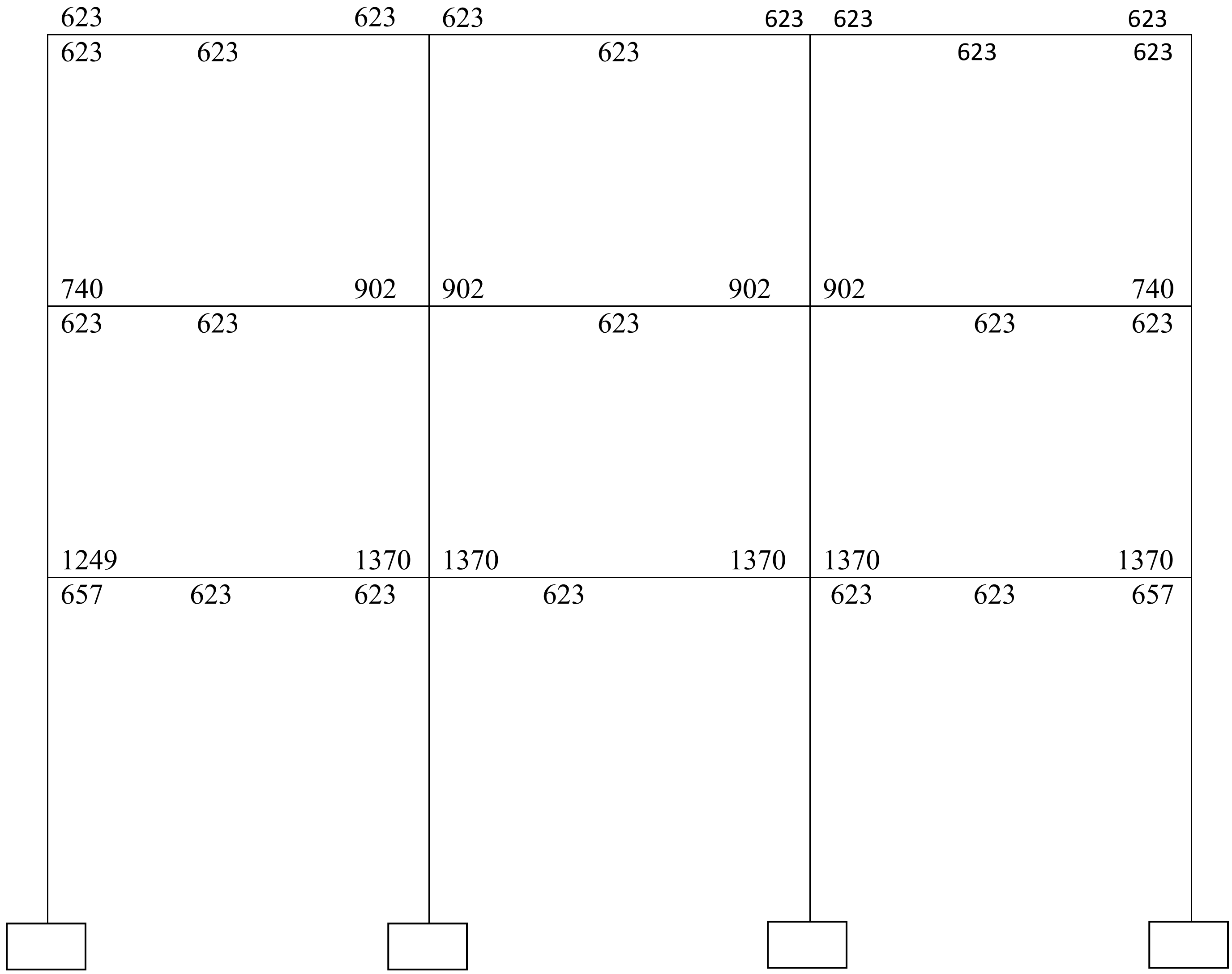


Fig. 4.26 Reinforcement (mm²) of beams on axes A and E, zone_1 loads, no slab

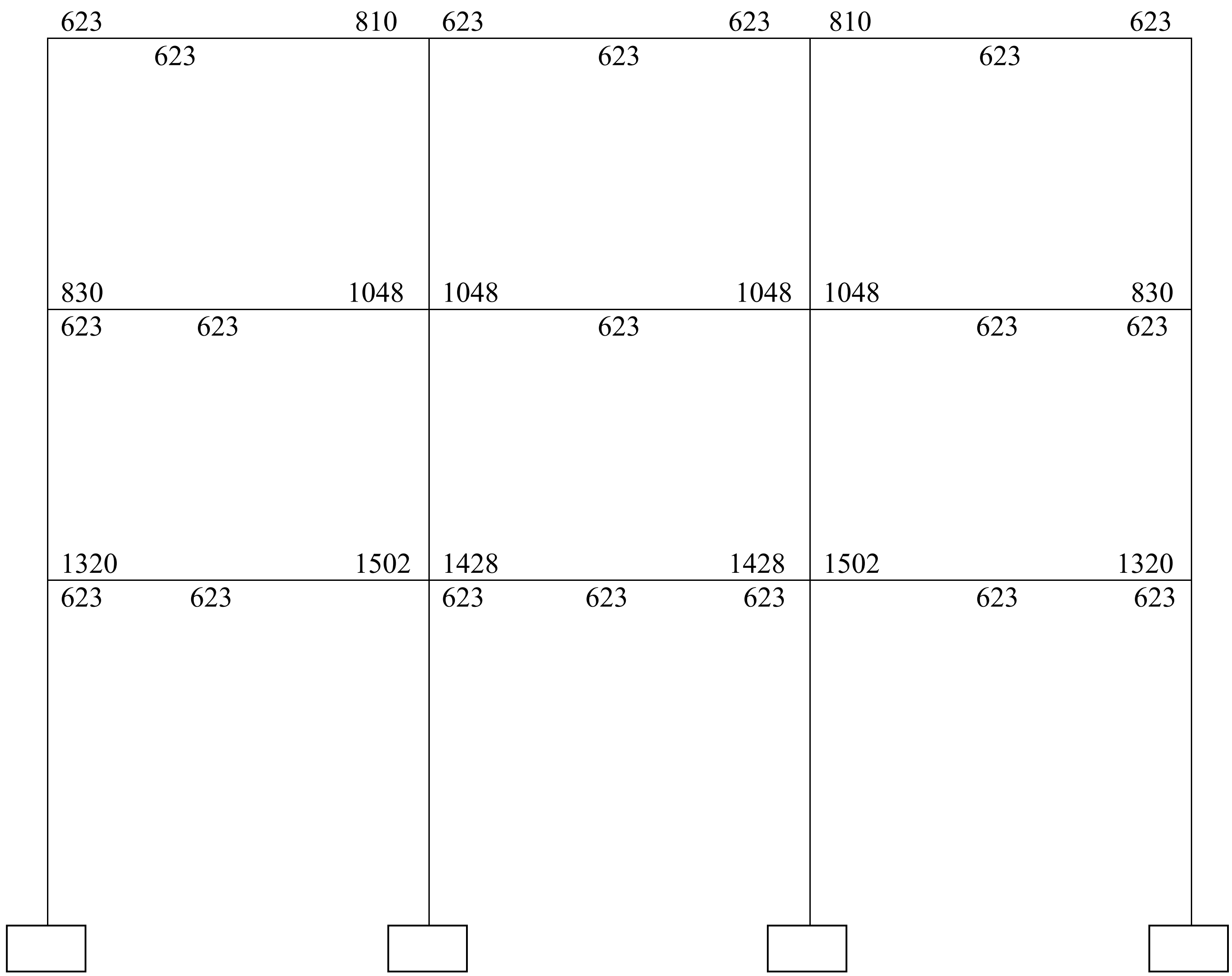


Fig. 4.27 Reinforcement (mm^2) of beams on axes B, C, and D, zone_1 loads, no slab

4.7.4 Zone 3, Slab Thickness of 10 mm

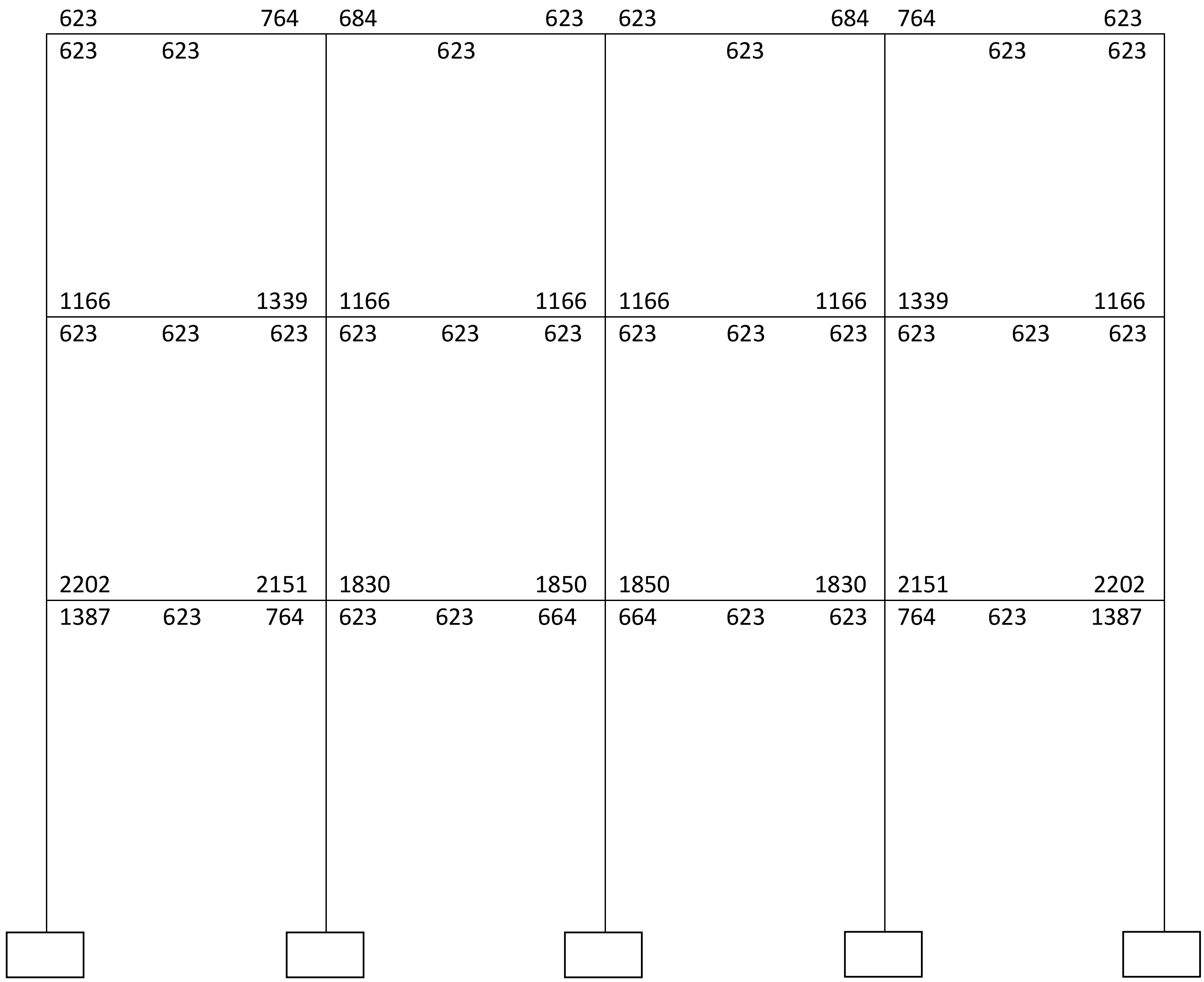


Fig. 4.28 Reinforcement (mm^2) of beams on axes 1 and 4, zone_3 loads, t_s= 10 mm

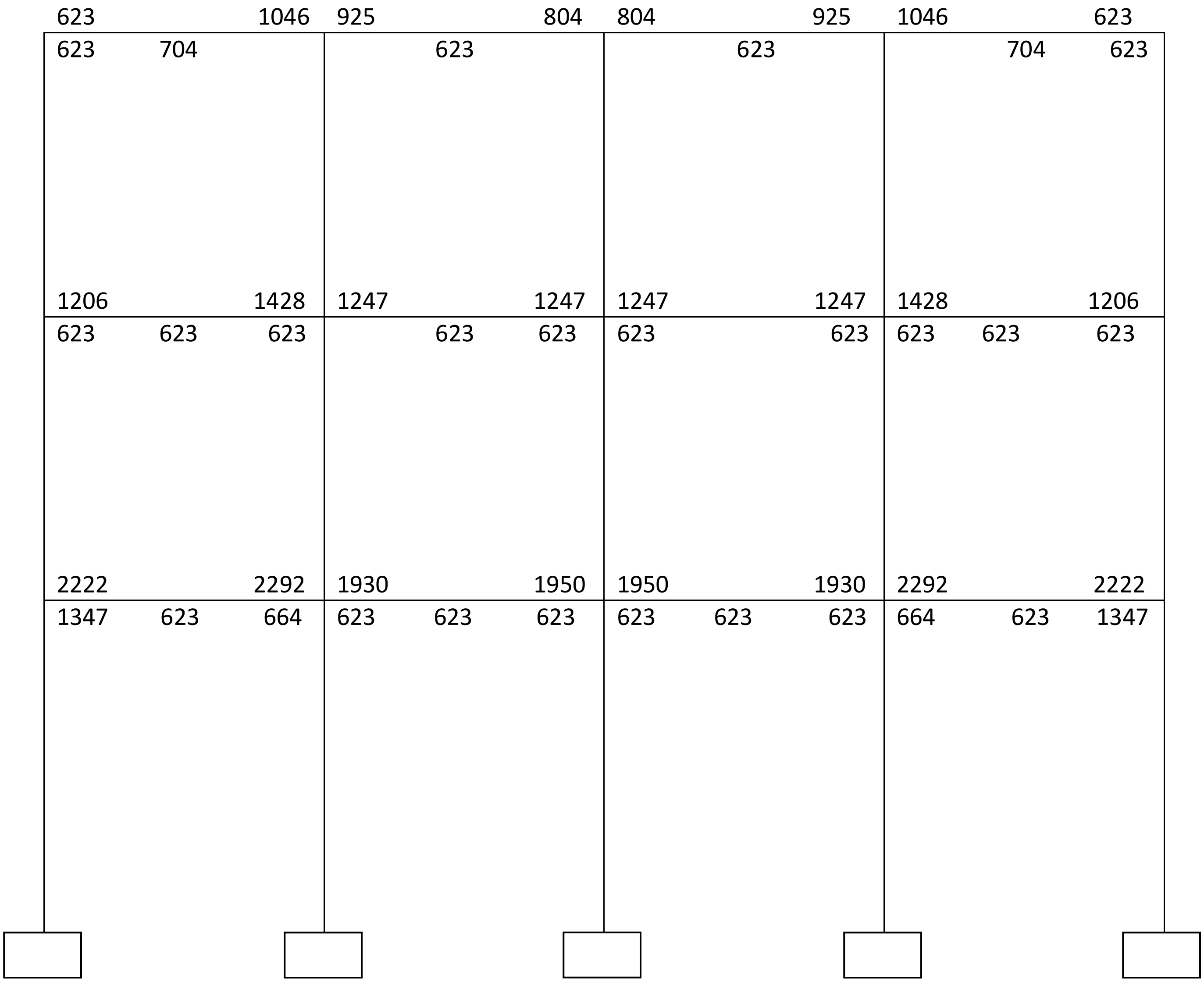


Fig. 4.29 Reinforcement (mm^2) of beams on axes 2and 3, zone_3 loads, t_s= 10 mm

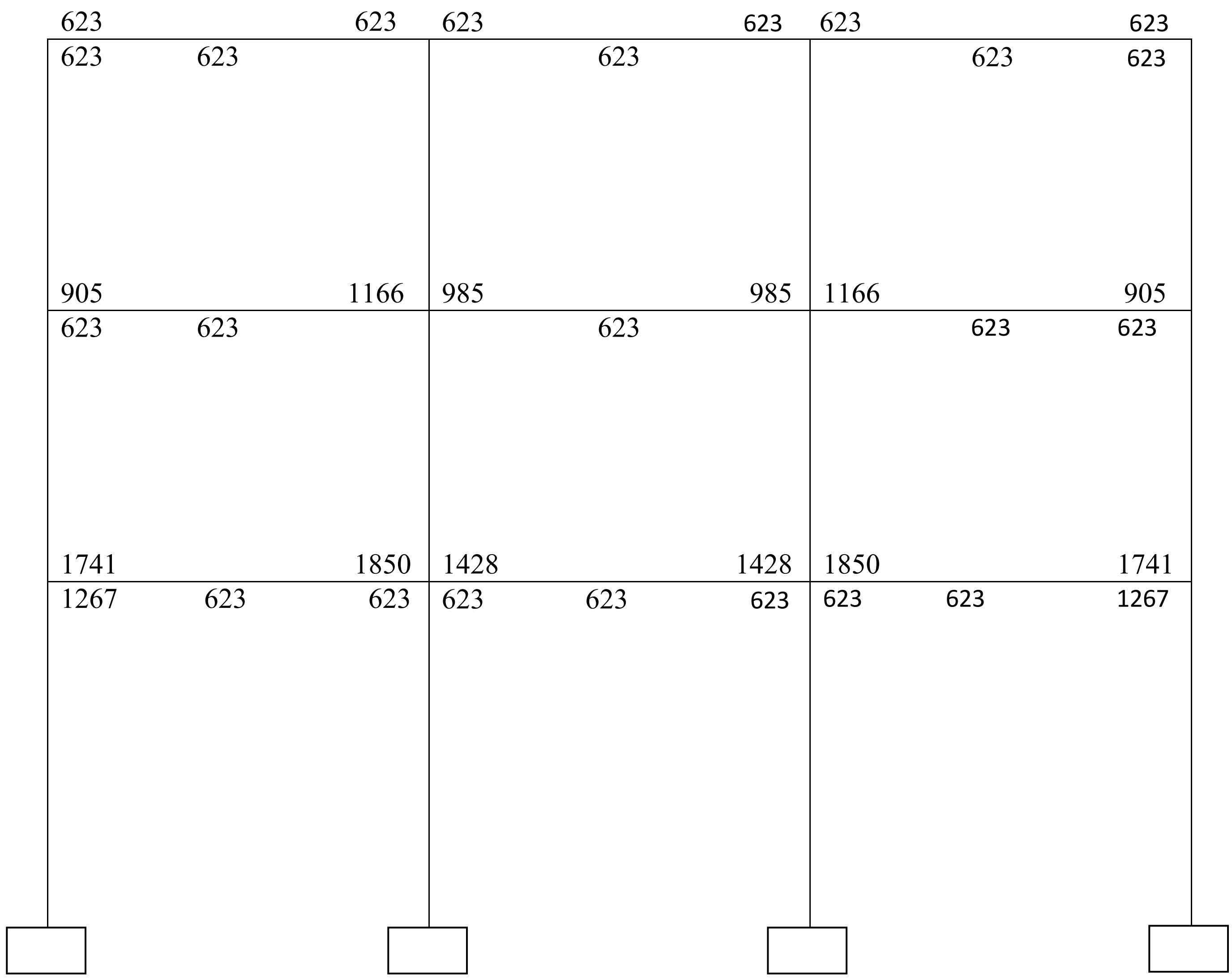


Fig. 4.30 Reinforcement (mm^2) of beams on axes A and E, zone_3 loads, t_s= 10 mm

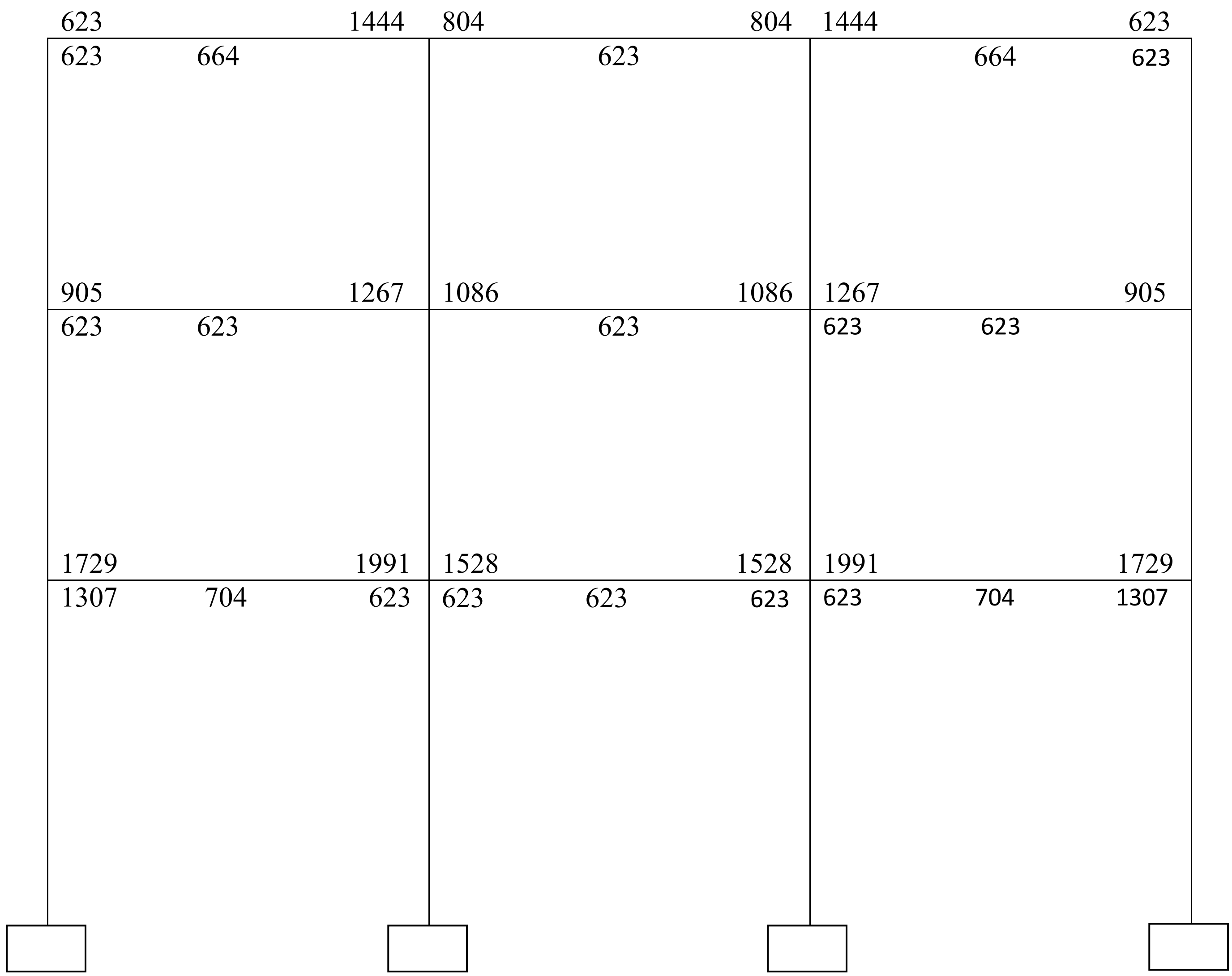


Fig. 4.31 Reinforcement (mm^2) of beams on axes B, C, and D, zone_3 loads, t_s= 10 mm

4.7.5 Zone 3, Slab Thickness of 160 mm

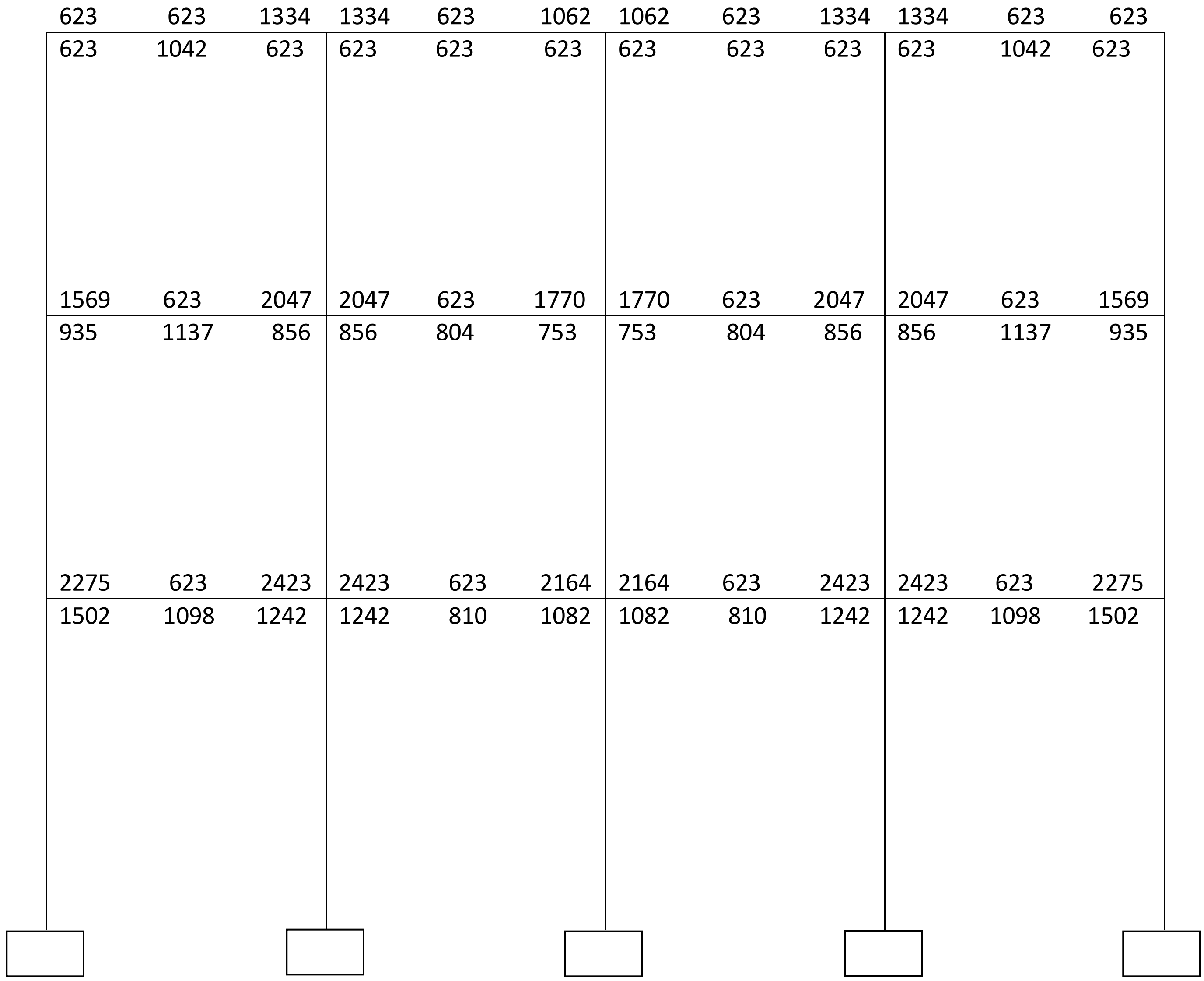


Fig. 4.32 Reinforcement (mm^2) of beams on axes 1 and 4, zone_3 loads, t_s= 160 mm

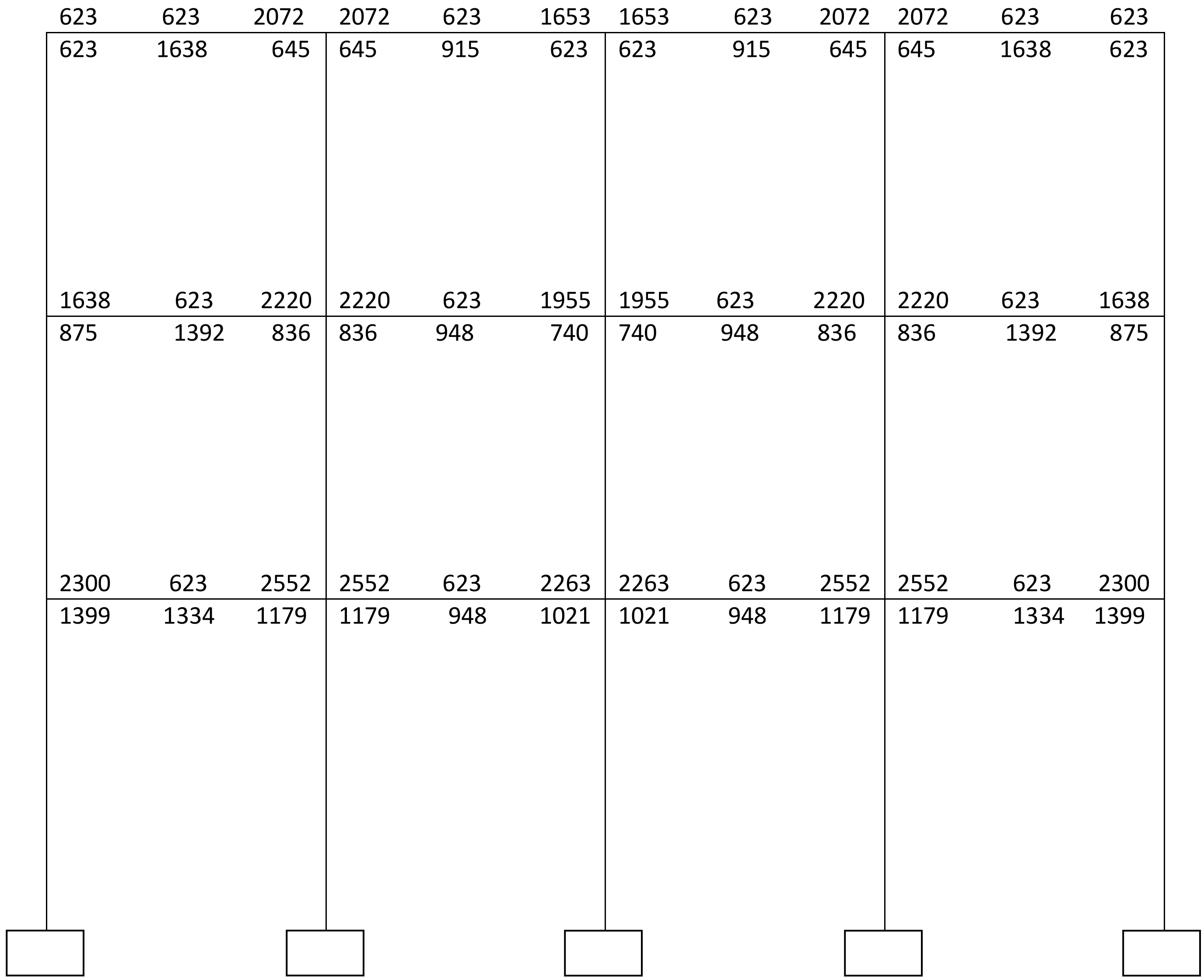


Fig. 4.33 Reinforcement (mm²) of beams on axes 2and 3, zone_3 loads, t_s= 160 mm

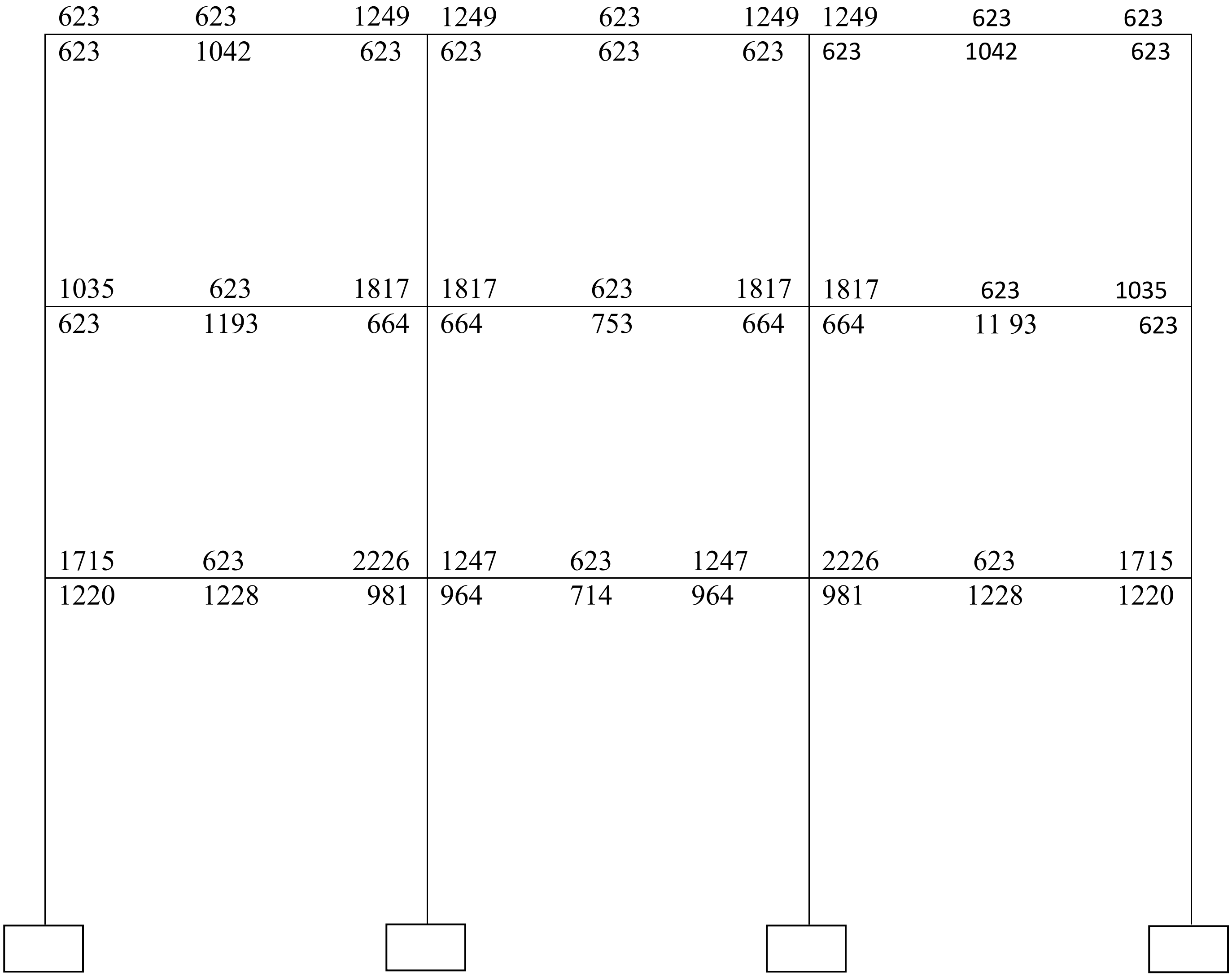


Fig. 4.34 Reinforcement (mm²) of beams on axes A and E, zone_3 loads, t_s= 160 mm

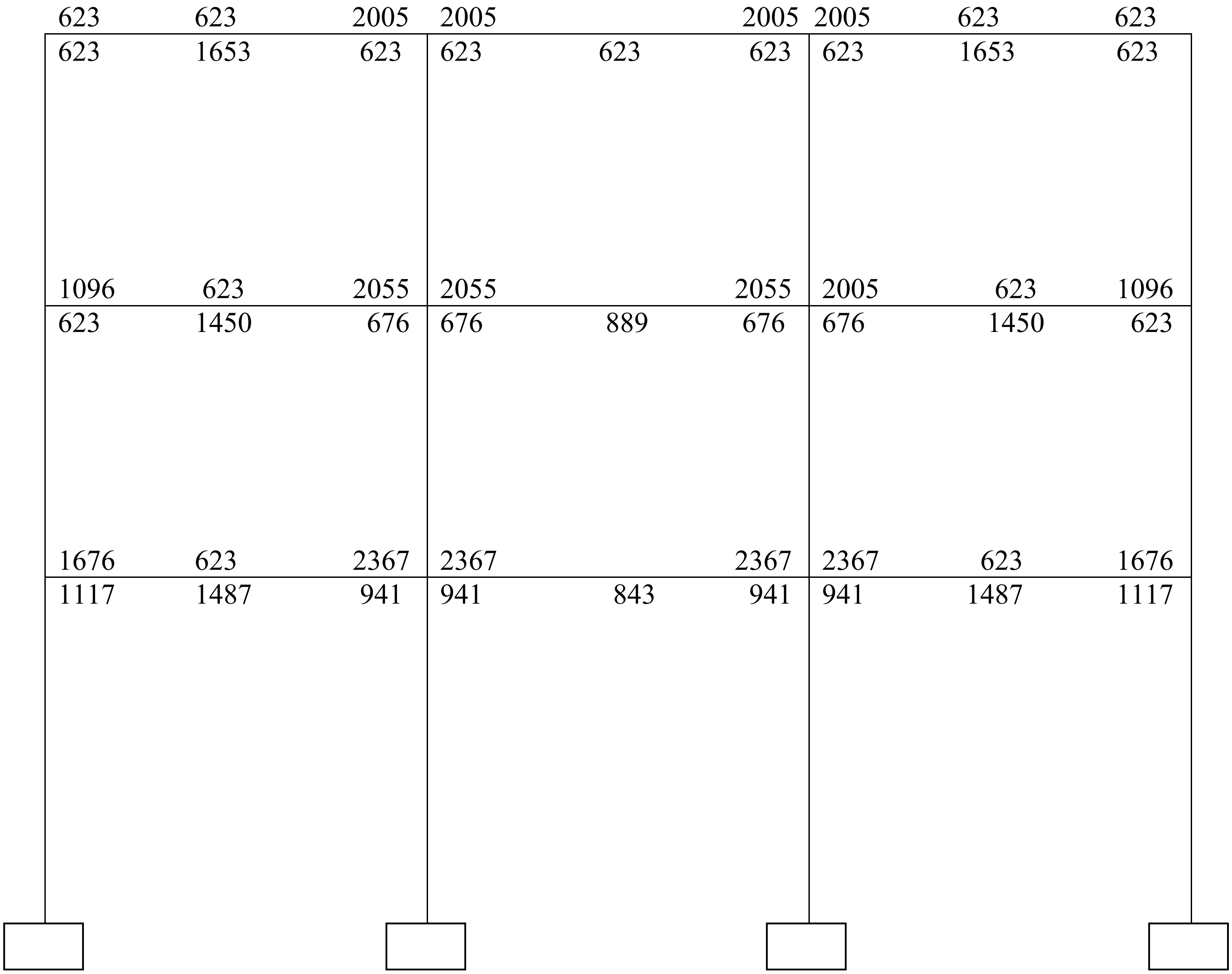


Fig. 4.35 Reinforcement (mm²) of beams on axes B, C, and D, zone_3 loads, t_s= 160 mm

4.7.6 Zone 3, No Slab

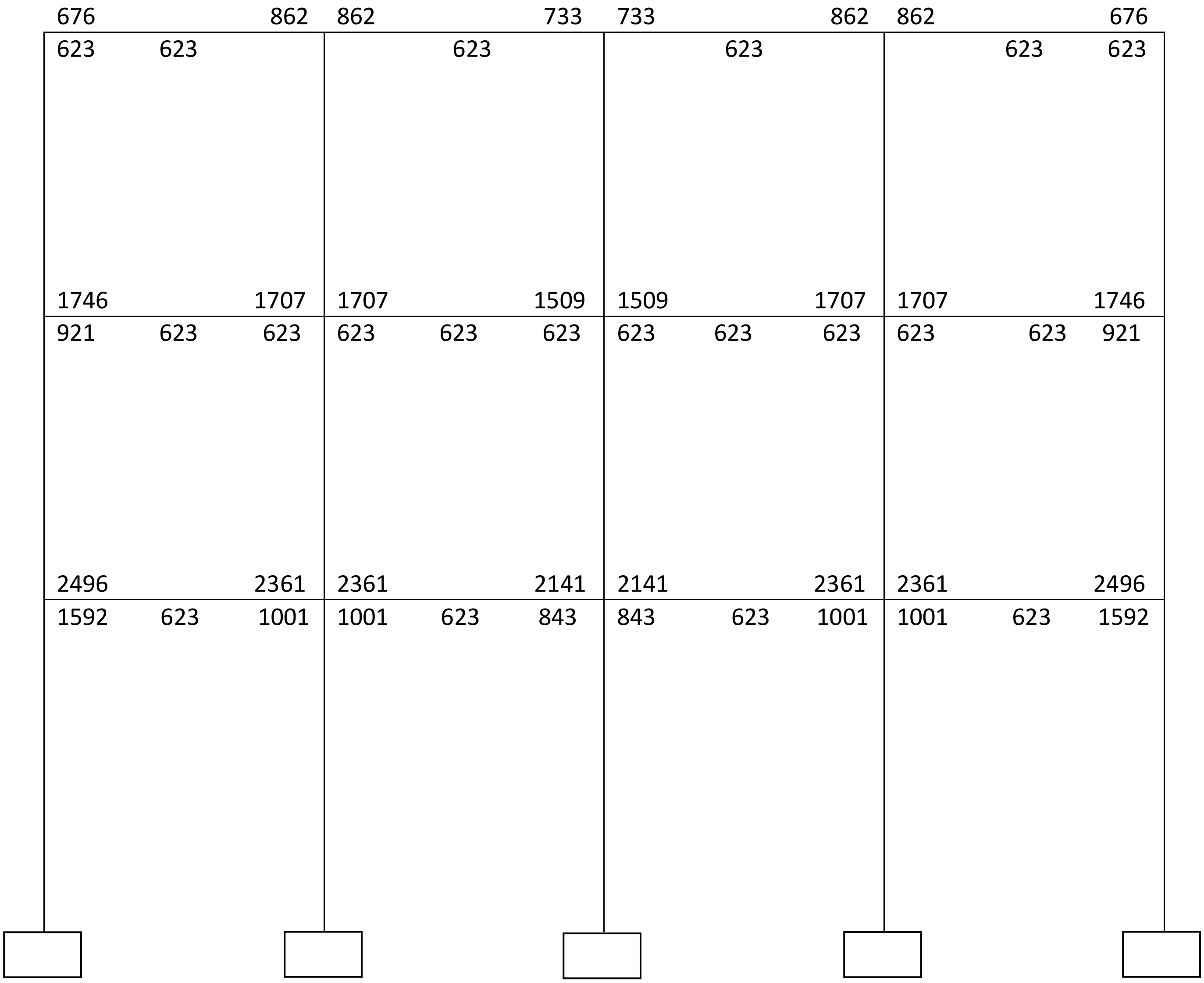


Fig. 4.36 Reinforcement (mm²) of beams on axes 1 and 4, zone_3 loads, no slab

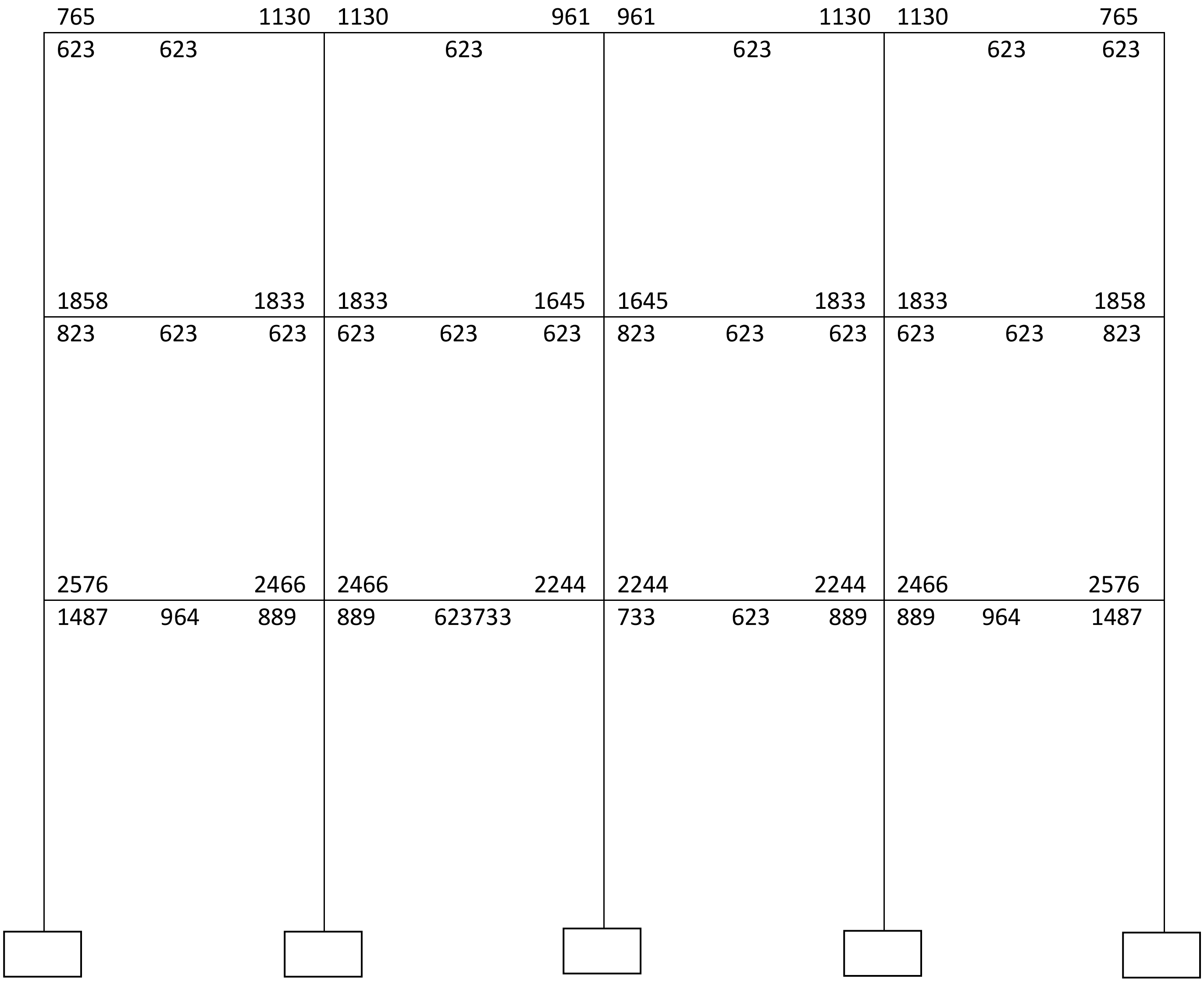


Fig. 4.37 Reinforcement (mm²) of beams on axes 2and 3, zone_3 loads, no slab

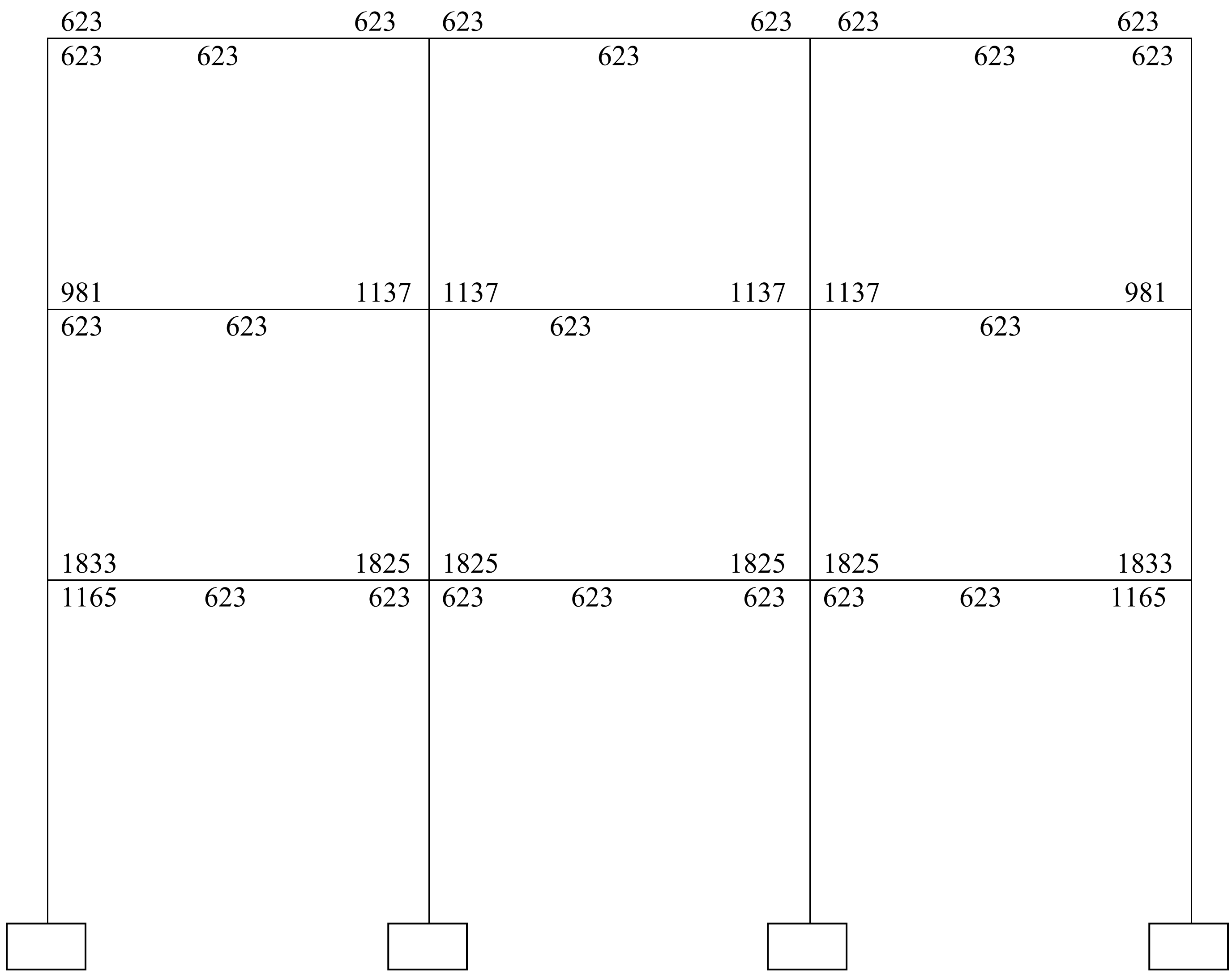


Fig. 4.38 Reinforcement (mm^2) of beams on axes A and E, zone_3 loads, no slab

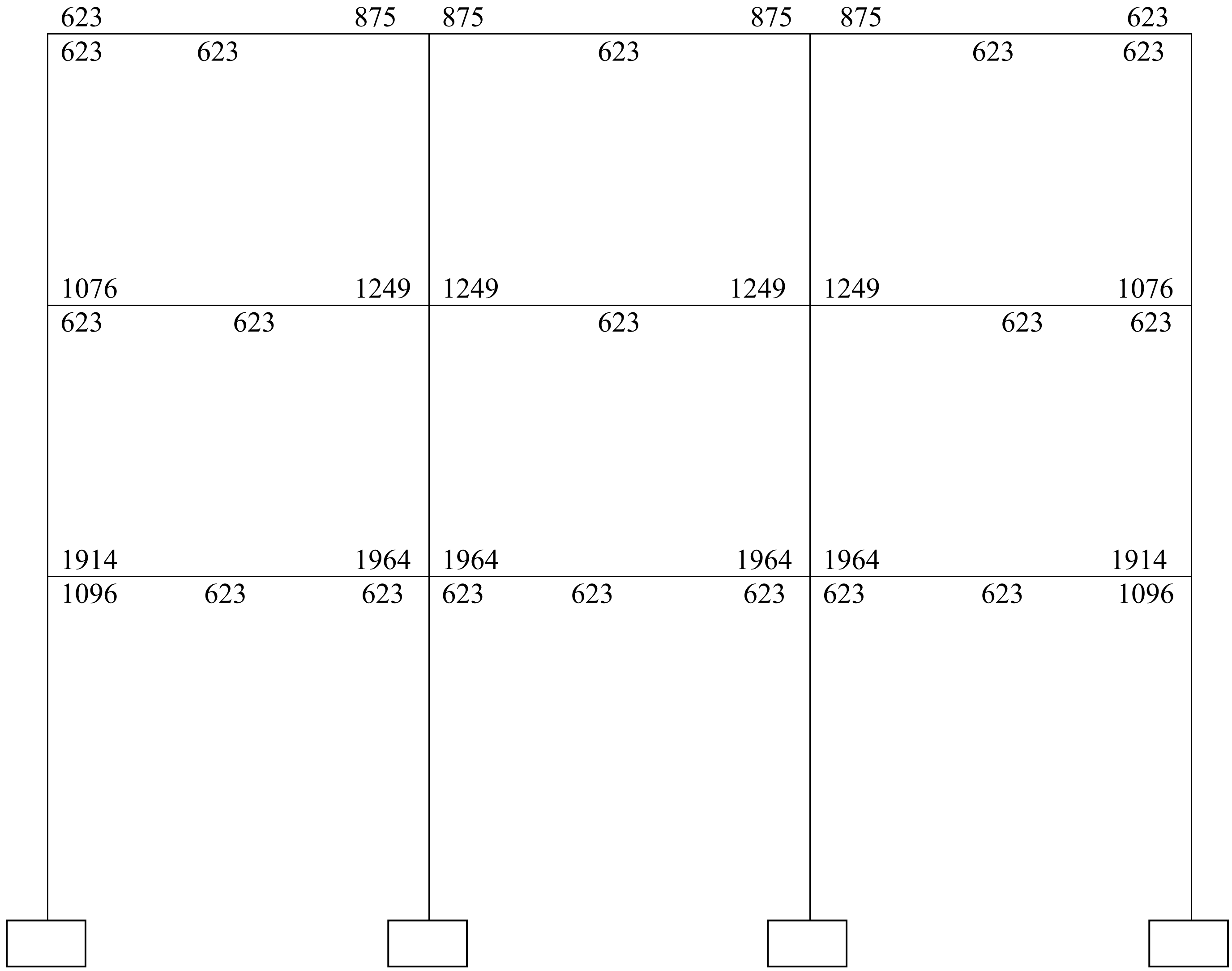


Fig. 4.39 Reinforcement (mm²) of beams on axes B, C, and D, zone_3 loads, no slab

4.7.7 Zone 5, Slab Thickness of 10 mm

As the seismic load can be reversible, so the reinforcement will be symmetric.

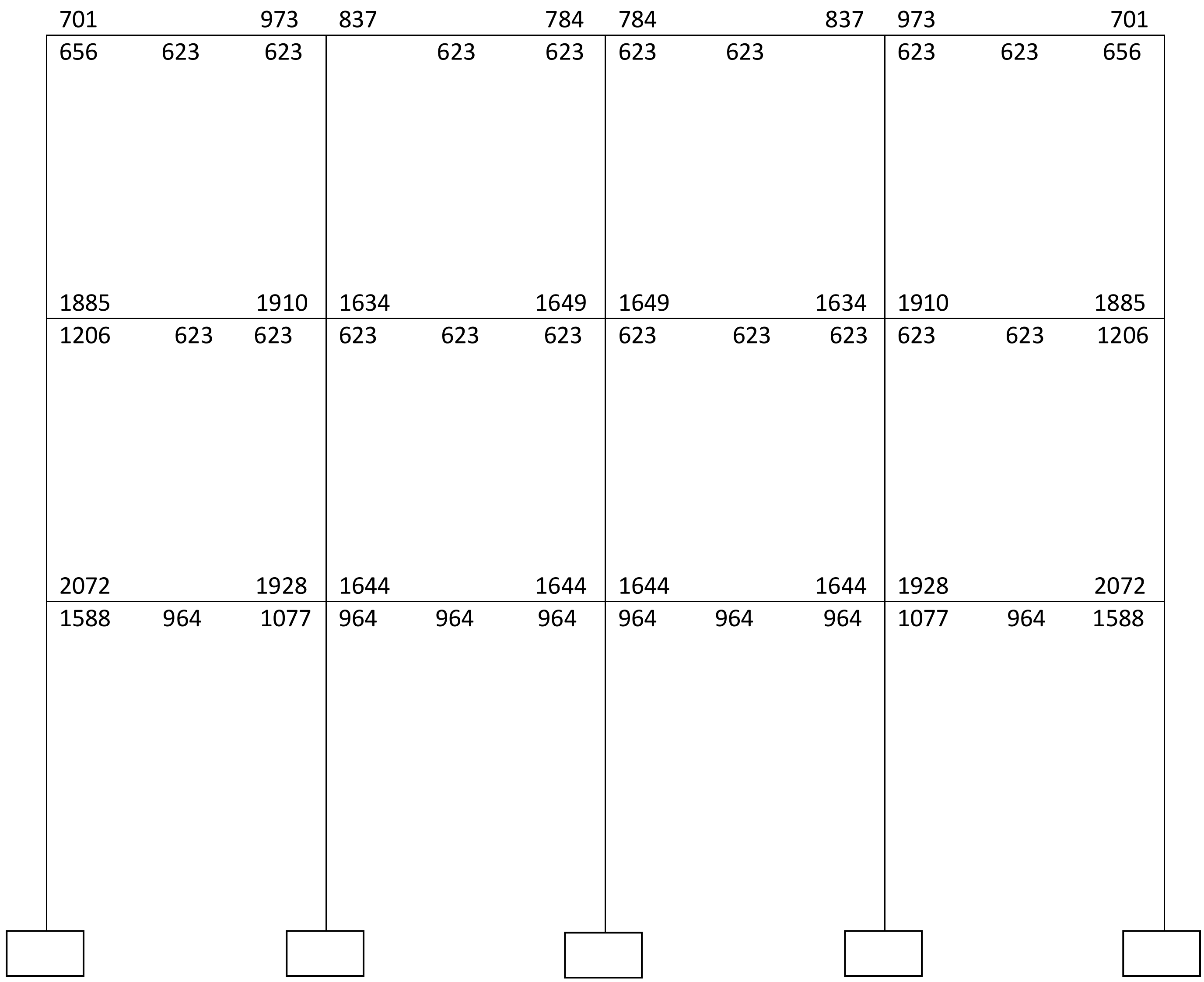


Fig. 4.40 Reinforcement (mm^2) of beams on axes 1 and 4, zone_5 loads, t_s= 10 mm

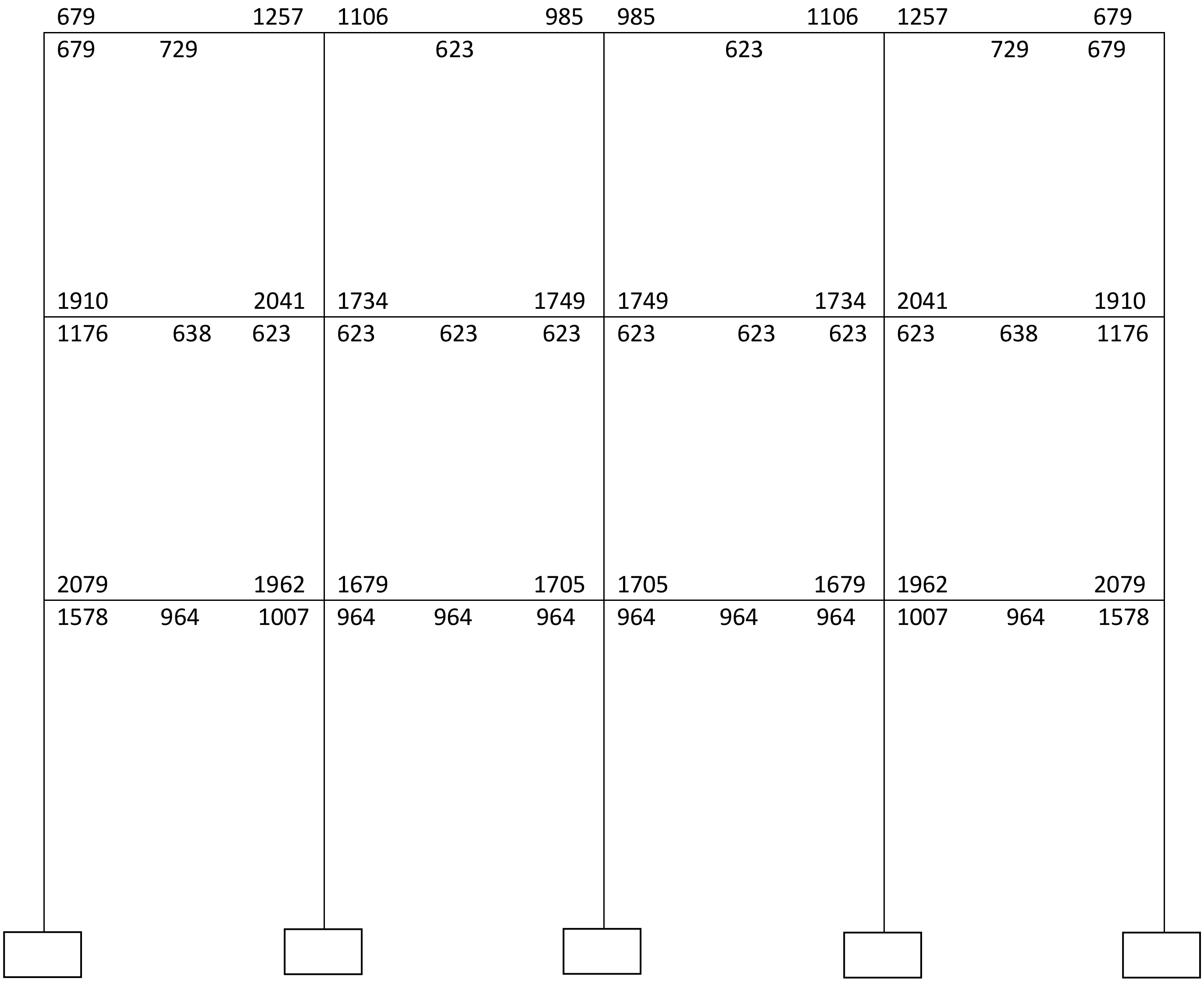


Fig. 4.41 Reinforcement (mm^2) of beams on axes 2and 3, zone_5 loads, t_s= 10 mm

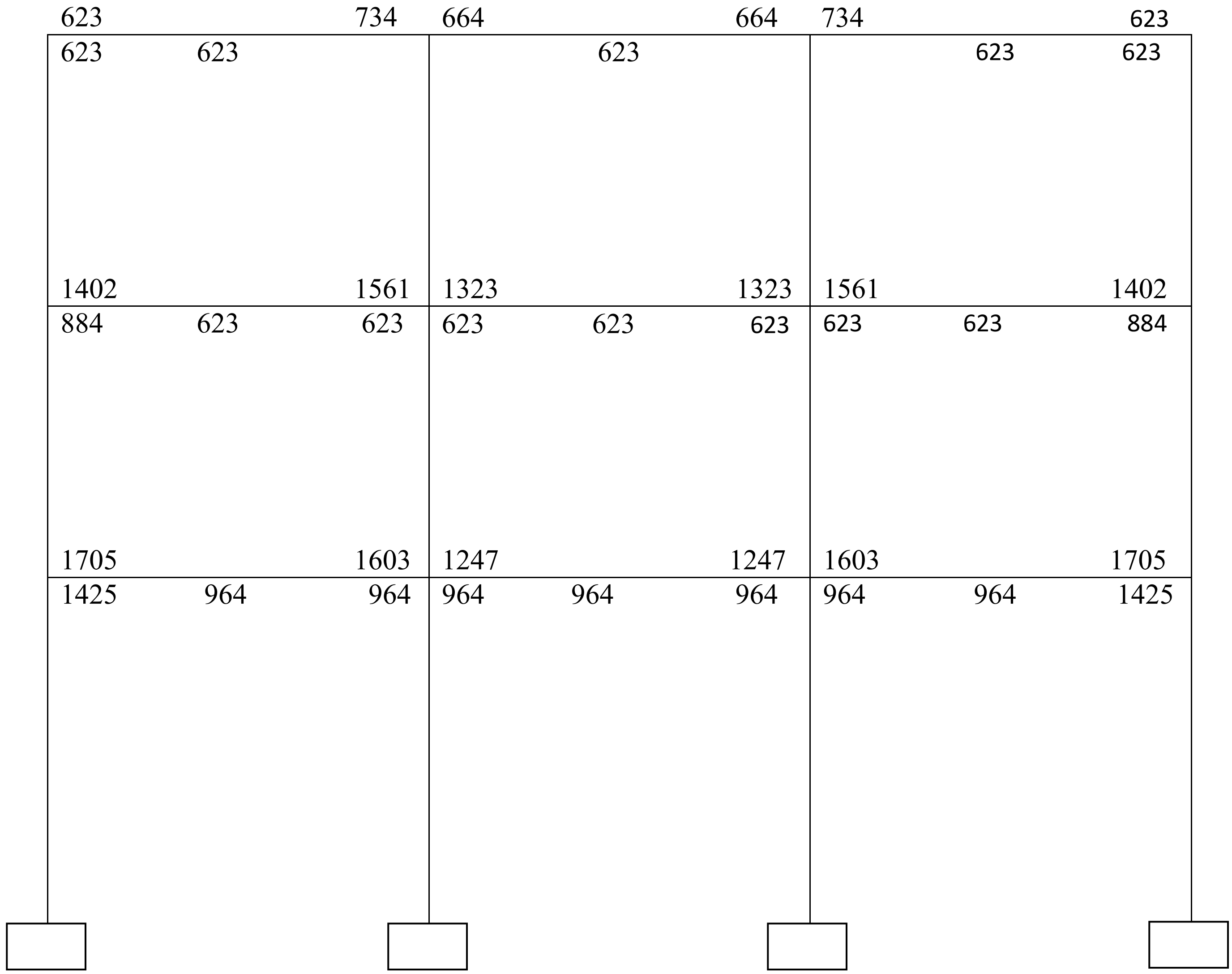


Fig. 4.42 Reinforcement (mm^2) of beams on axes A and E, zone_5 loads, t_s= 10 mm

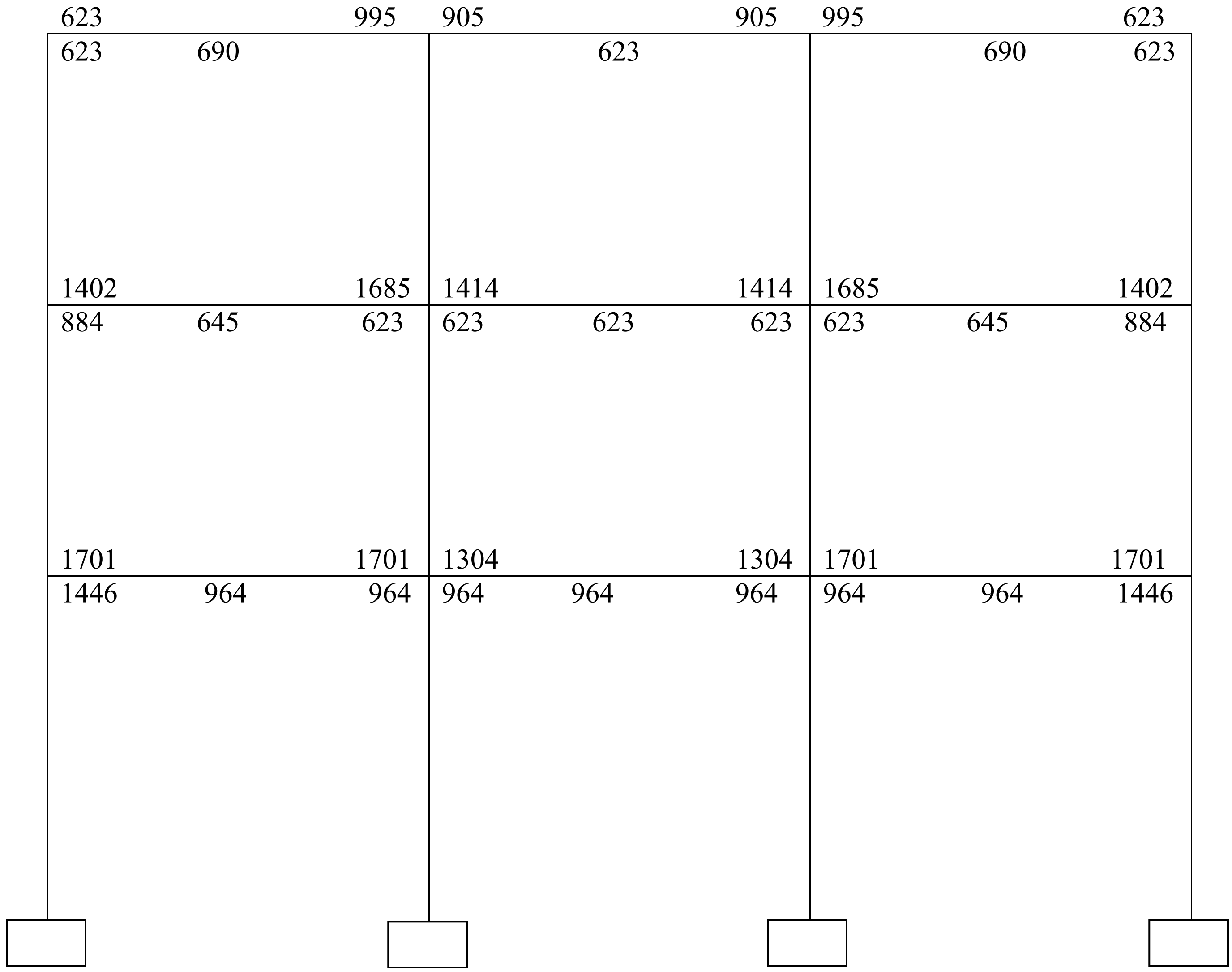


Fig. 4.43 Reinforcement (mm^2) of beams on axes B, C, and D, zone_5 loads, t_s= 10 mm

4.7.8 Zone 5, Slab Thickness of 160 mm

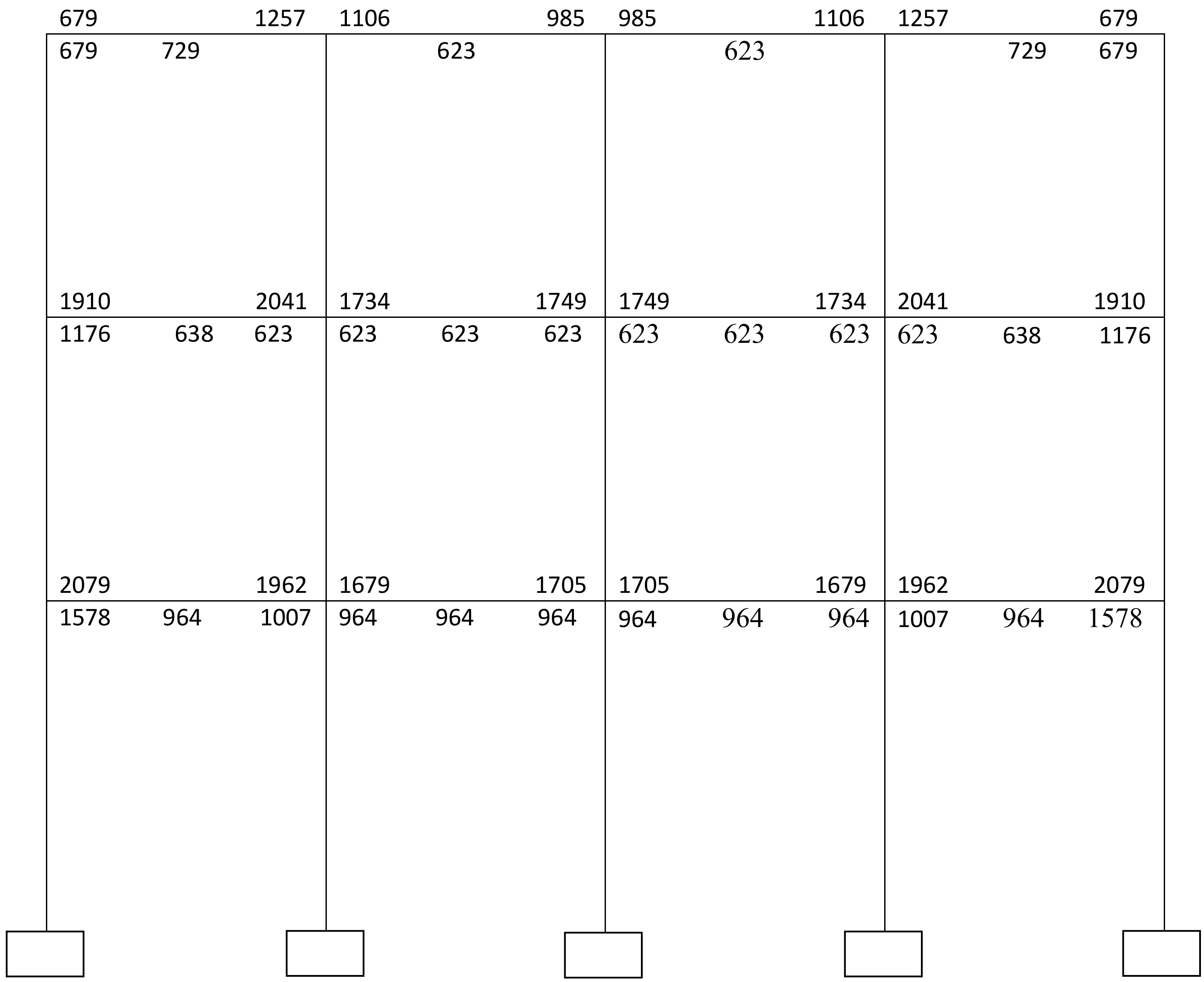


Fig. 4.44 Reinforcement (mm²) of beams on axes 1 and 4, zone_5 loads, t_s= 160 mm

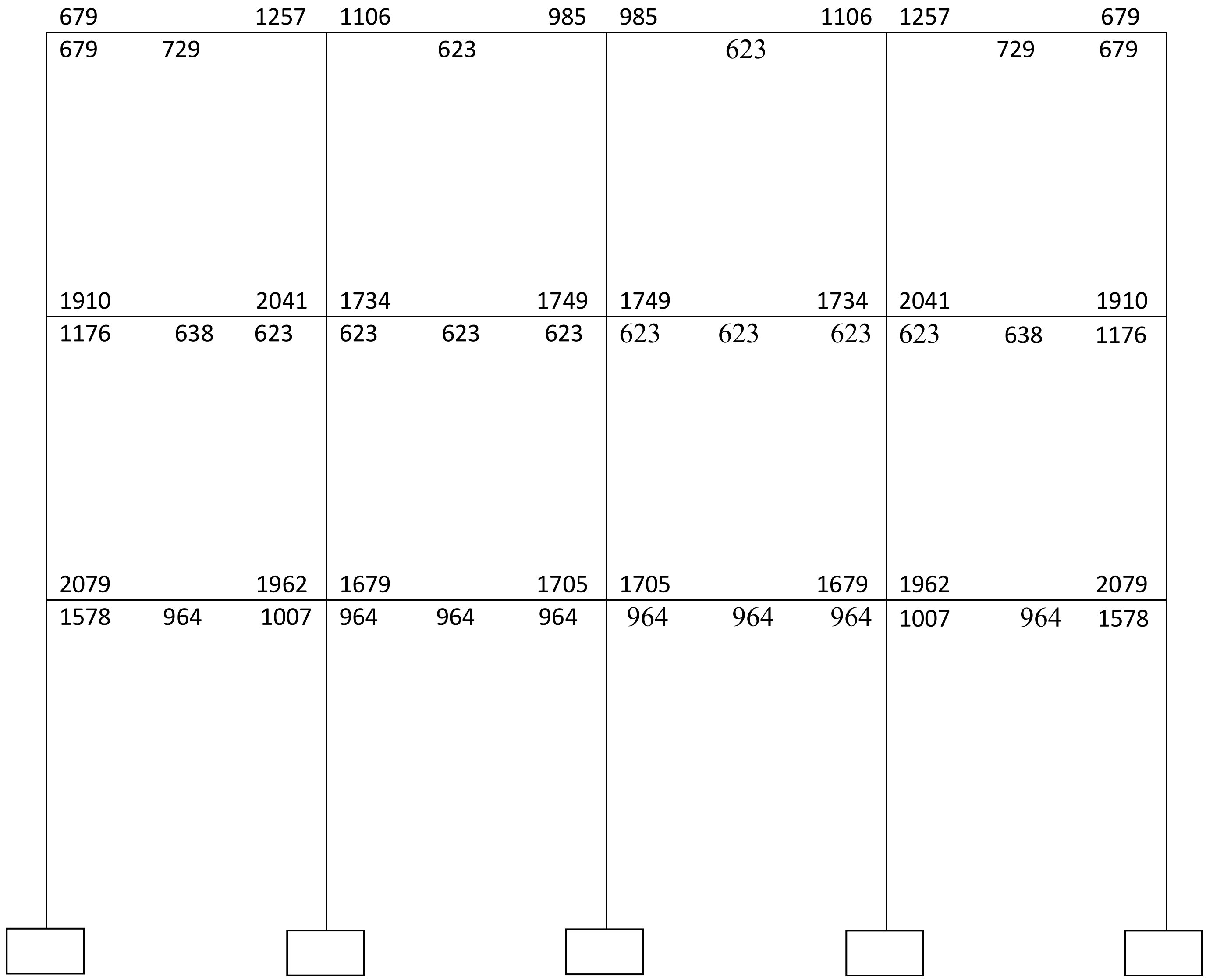


Fig. 4.45 Reinforcement (mm^2) of beams on axes 2and 3, zone_5 loads, t_s= 160 mm

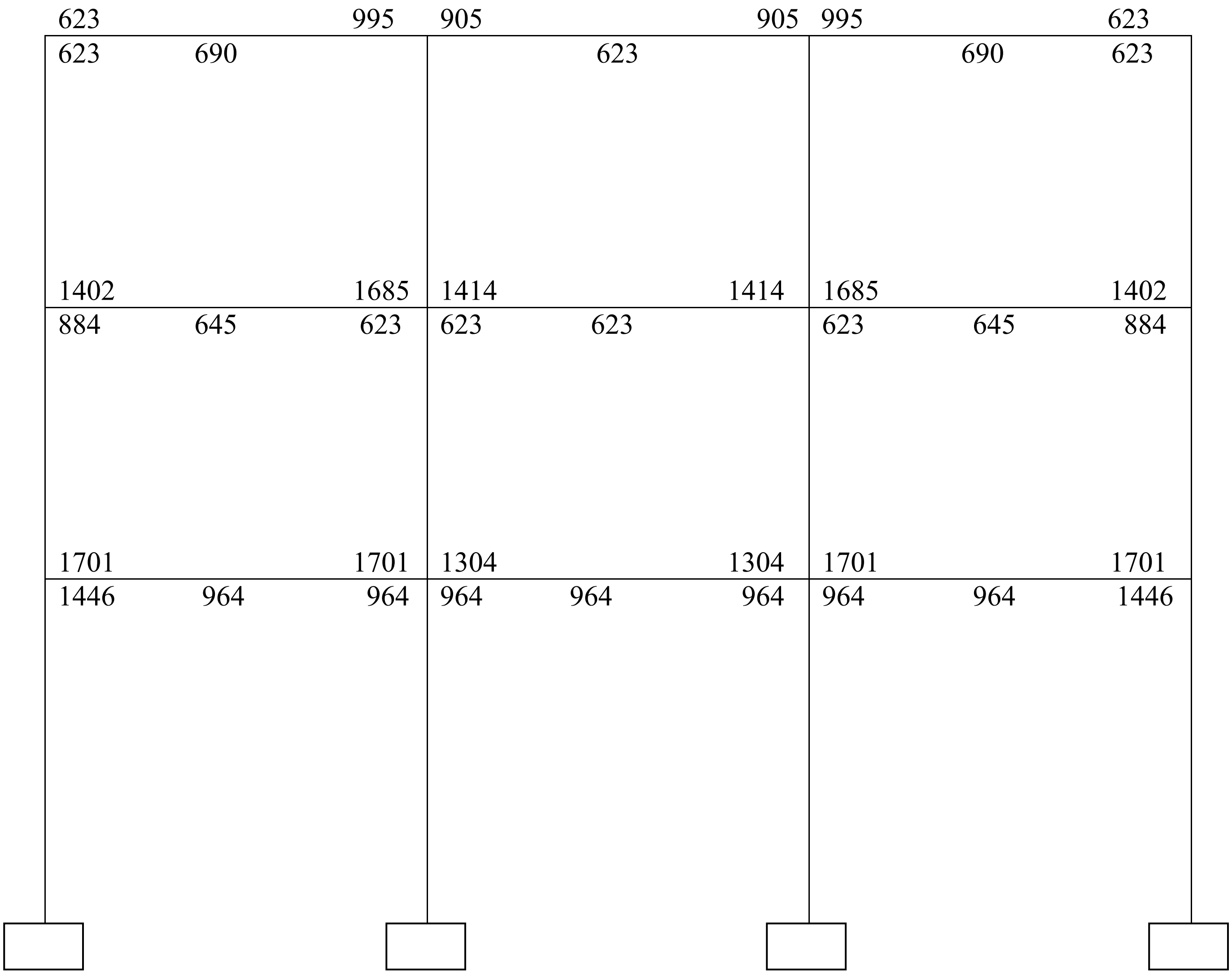


Fig. 4.46 Reinforcement (mm^2) of beams on axes A and E, zone_5 loads, t_s= 160 mm

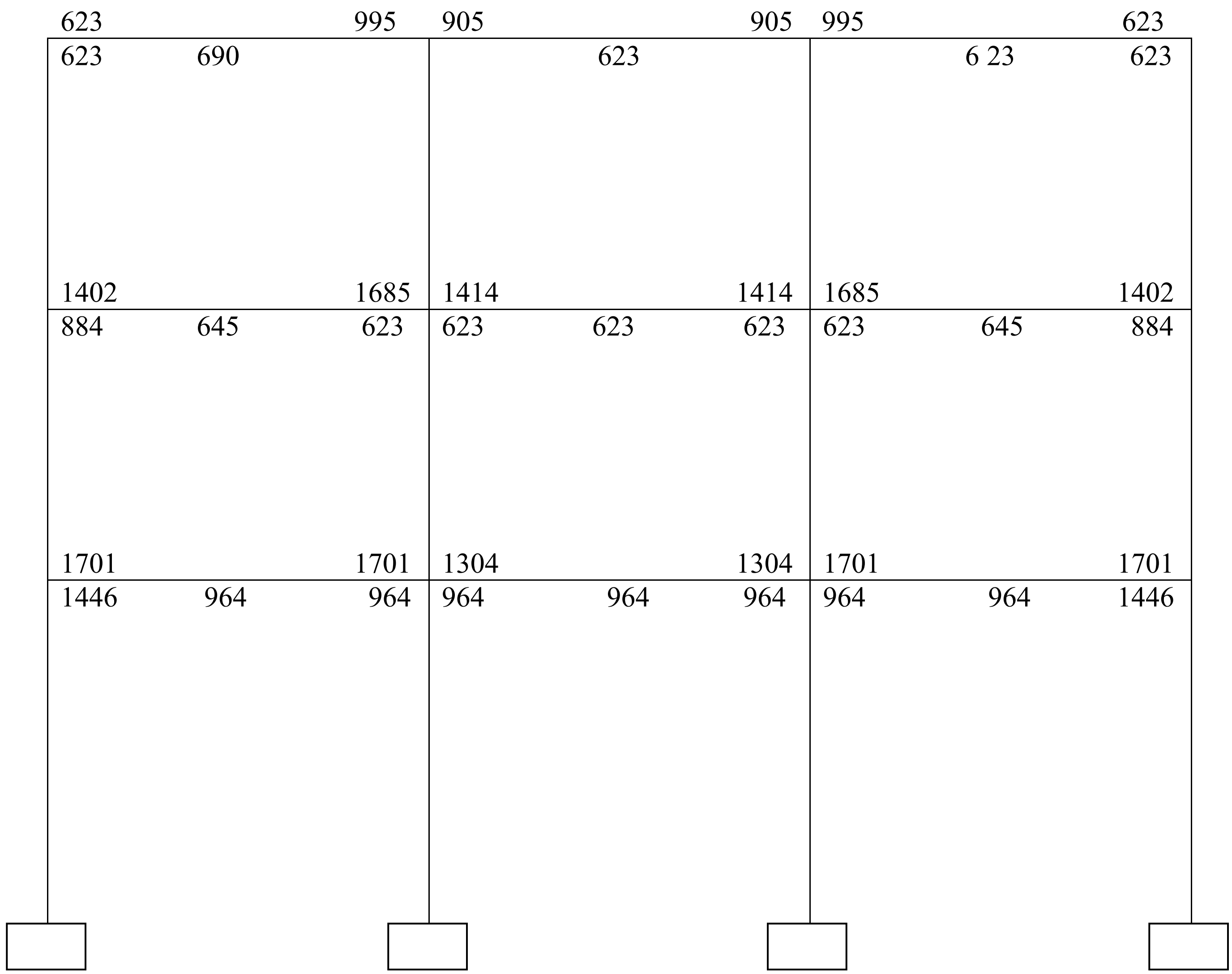


Fig. 4.47 Reinforcement (mm^2) of beams on axes B, C, and D, zone_5 loads, t_s= 160 mm

4.7.9 Zone 5, No Slab

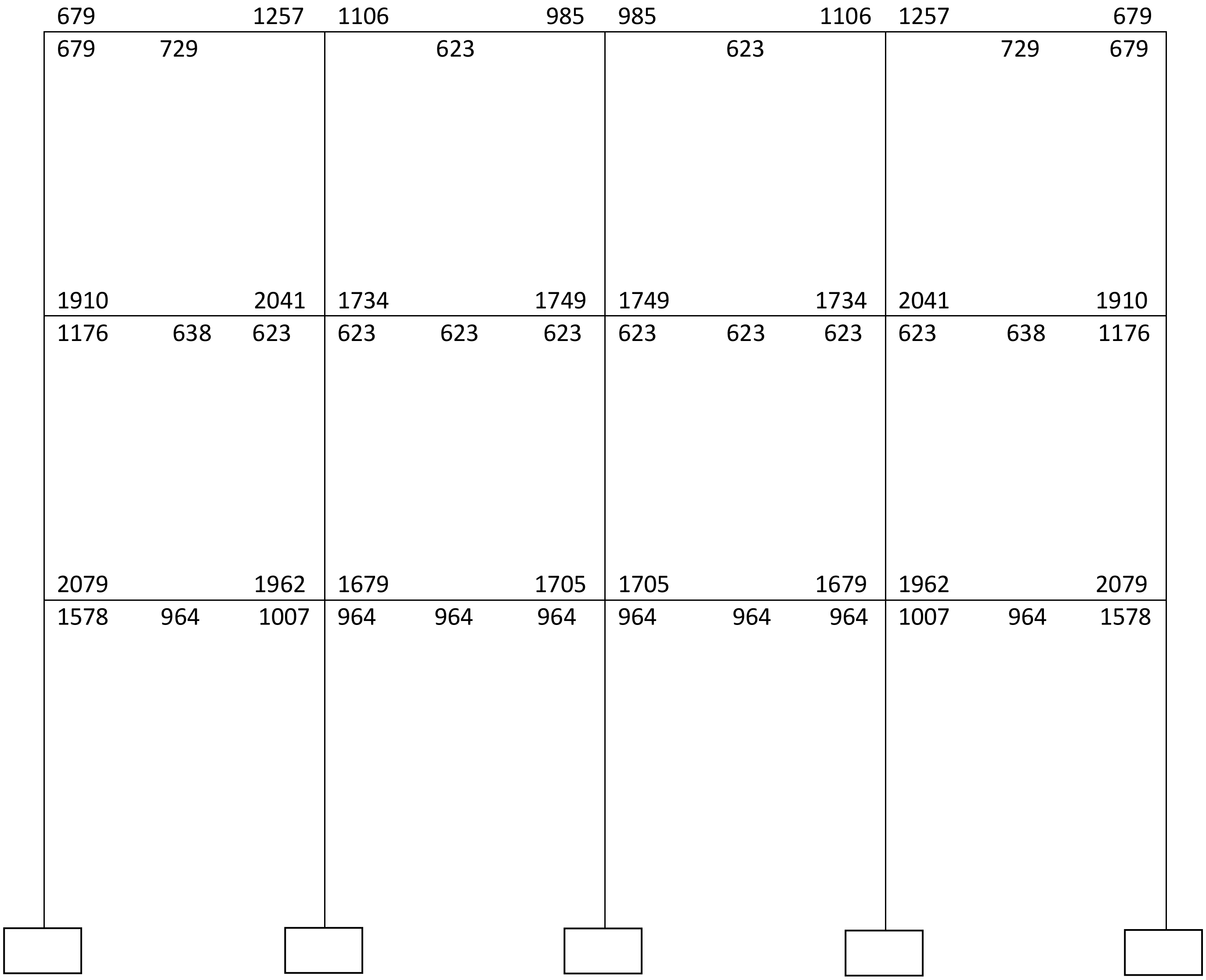


Fig. 4.48 Reinforcement (mm²) of beams on axes 1 and 4, zone_5 loads, no slab

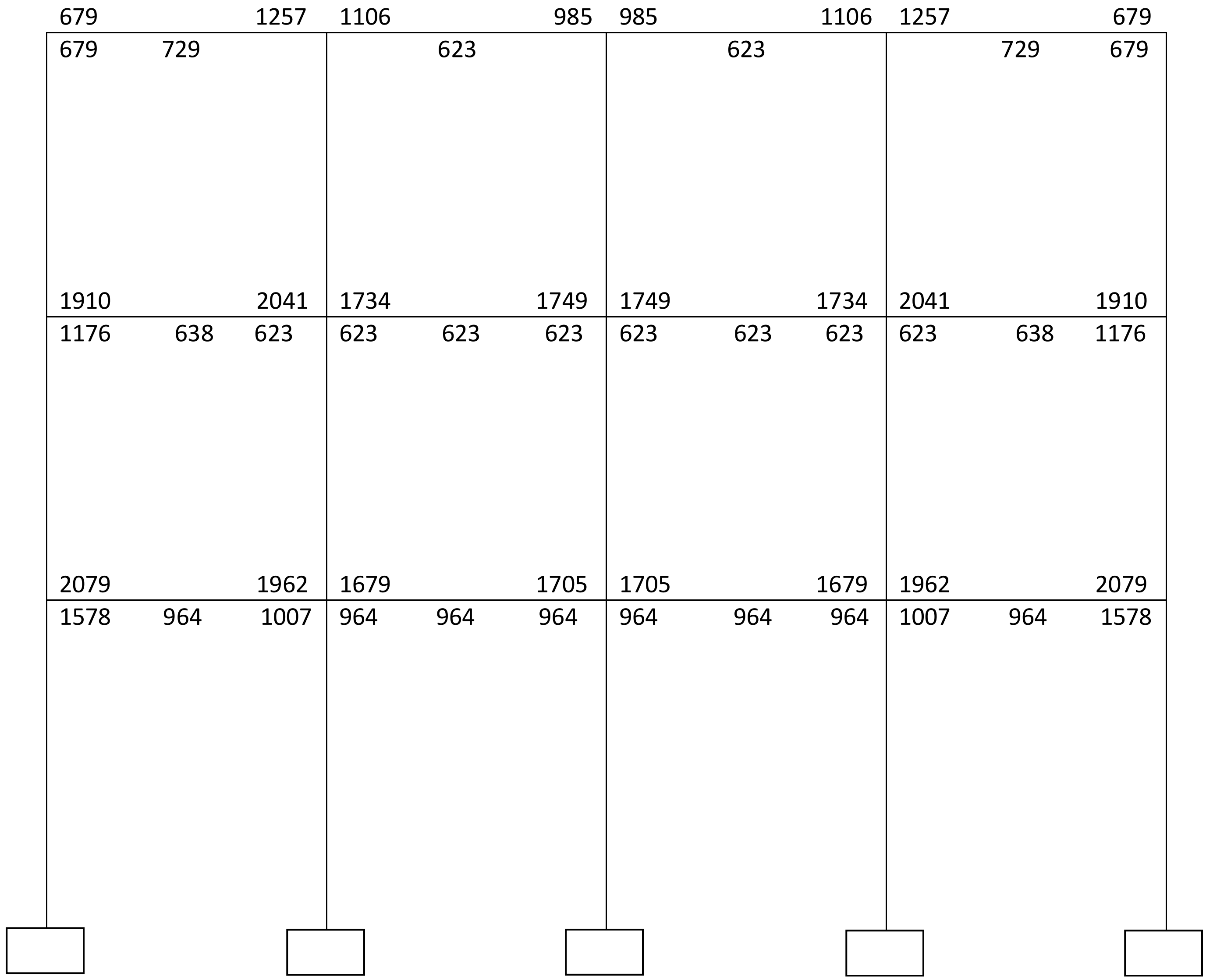


Fig. 4.49 Reinforcement (mm^2) of beams on axes 2and 3, zone_5 loads, no slab

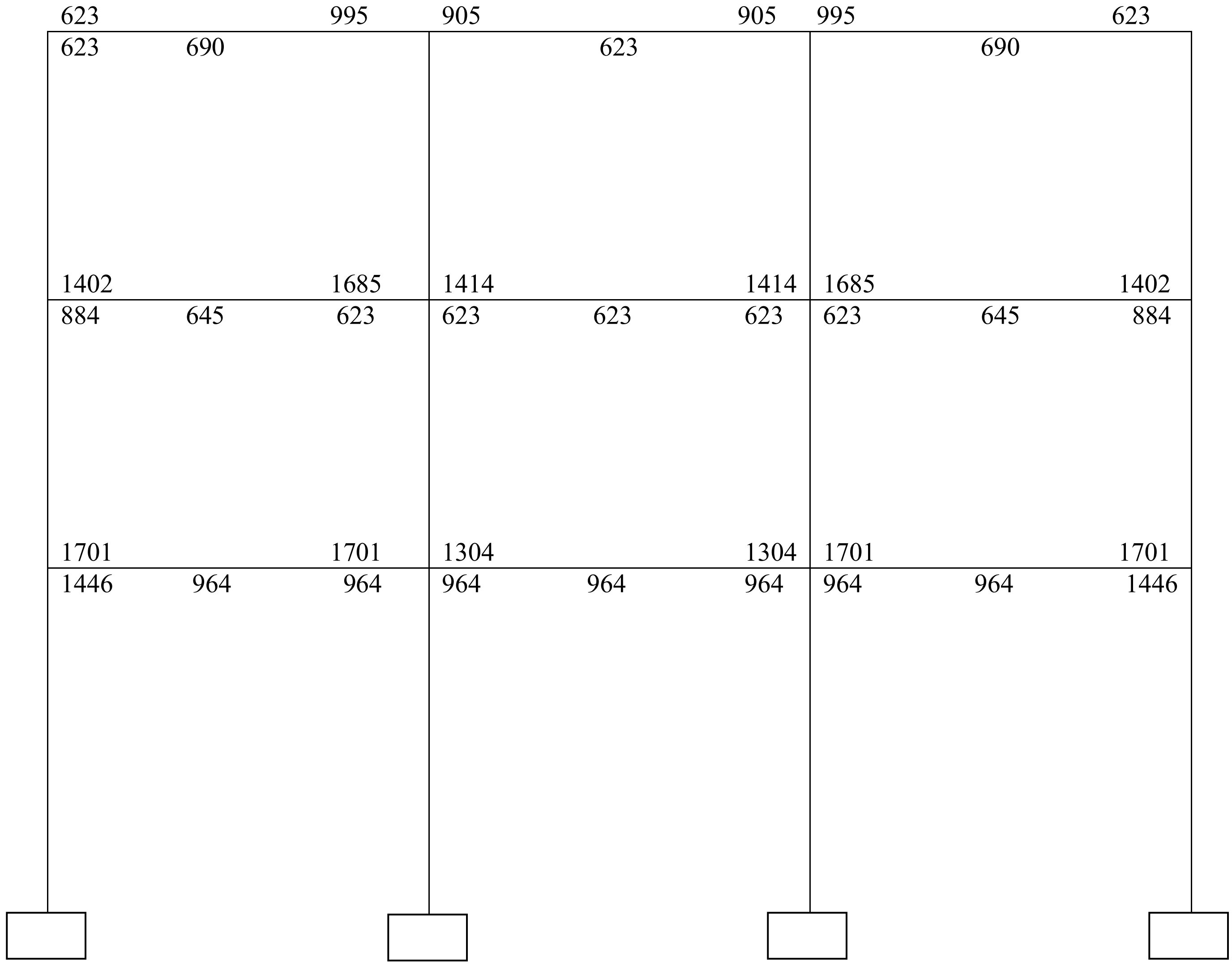


Fig. 4.50 Reinforcement (mm^2) of beams on axes A and E, zone_5 loads, no slab

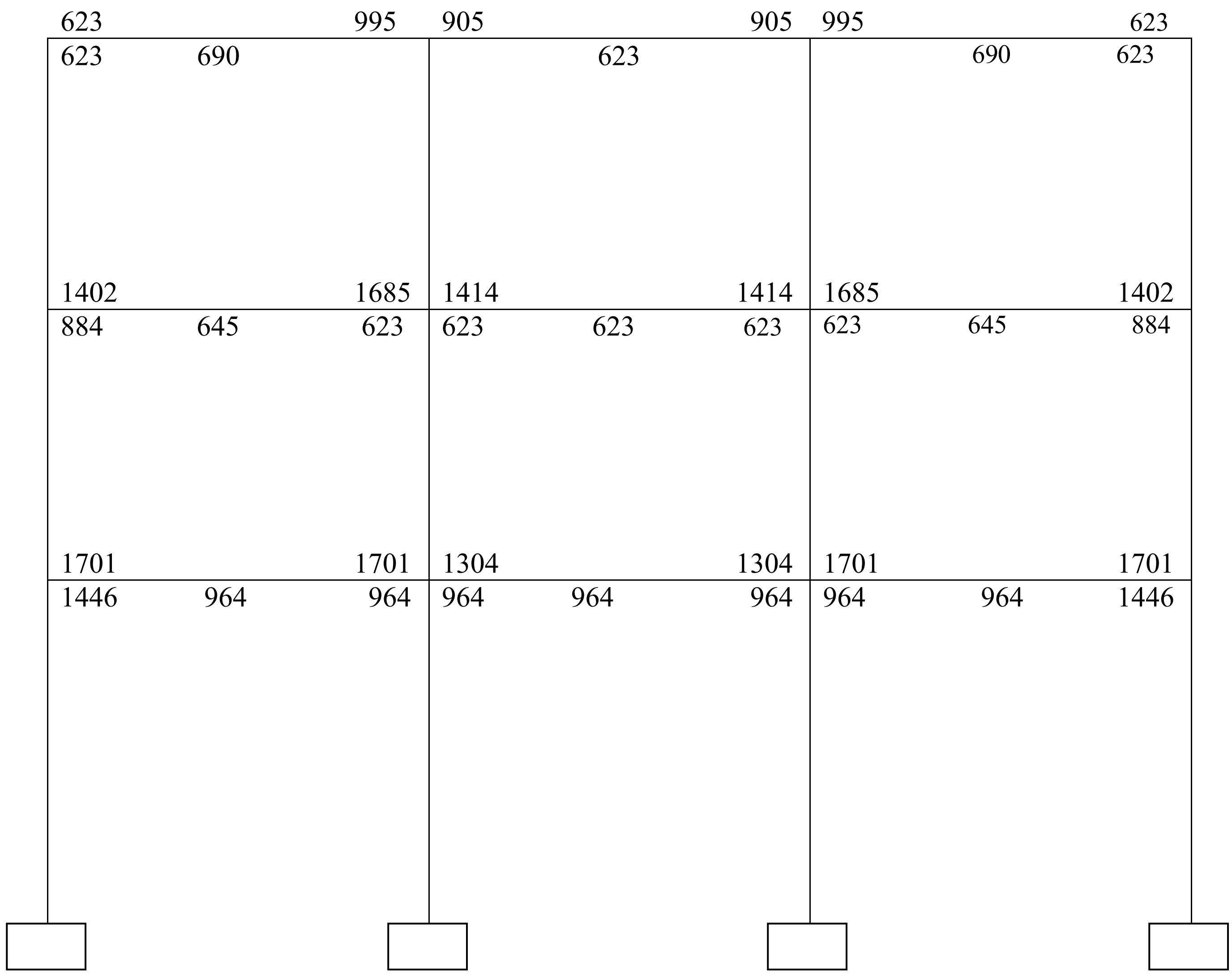


Fig. 4.51 Reinforcement (mm^2) of beams on axes B, C, and D, zone_5 loads, no slab

CHAPTER 5

PROGRESSIVE COLLAPSE ANALYSES AND DESIGN

5.1 Introduction

There are many definitions for progressive collapse phenomenon; here are some of these definitions. The progressive collapse is defined in the commentary of the American Society of Civil Engineers Standard 7-02 *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-02) as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". Another definition for progressive collapse is "a situation where local failure of a primary structural component leads to the collapse of adjoining members, which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause" (FEMA, Building Design Guidance 3.2).

After the seismic analyses and design were done, the progressive collapse analyses were carried out. Because there is no code in Egypt dealing with this subject, the researcher followed the General Service Administration, GSA, recommendations. The GSA recommendations state the following scenarios:

1. Remove a load bearing element near or at the middle of a longer side.

2. Remove a load bearing element near or at the middle of a shorter side.

3. Remove a load bearing element at the corner.

As the building consisting of four and three bays in the X and Y directions, so there is a column at the middle of the longer side, but there is a column near the middle of the shorter side.

Also the GSA states the loads that are applied in either linear static analysis or nonlinear static analysis to be equal to 2(DL + 0.25 LL), where DL is the dead loads and LL is the imposed live loads. In this research, the researcher used the nonlinear static analysis to observe the formulation of plastic hinges through the structure, and the failed beam elements.

5.2Applied Loads

According to General Service Administration, GSA, guidelines, the applied loads that were applied for using nonlinear static analysis method equal to be 2(dead loads + 0.25 live loads).

By applying these loads, the SAP2000 has the capability of doing such load combination, and removing the required column for each case of slab thickness/ existing and seismic zones, according to GSA guidelines the progressive collapse analyses were run. The detailed steps were represented in the following procedure.

5.3 Procedures, Analyses and Results

The following procedures were conducted for every scenario that is shown in the following table:

Table 5.1 Progressive Collapse Scenarios

<table>
<tr><td colspan="9">Zone one</td><td colspan="9">Zone three</td><td colspan="9">Zone five</td></tr>
<tr><td colspan="3">ts = 10 mm</td><td colspan="3">ts = 160 mm</td><td colspan="3">No slab</td><td colspan="3">ts = 10 mm</td><td colspan="3">ts = 160 mm</td><td colspan="3">No slab</td><td colspan="3">ts = 10 mm</td><td colspan="3">ts = 160 mm</td><td colspan="3">No slab</td></tr>
<tr><td>C</td><td>L</td><td>S</td><td>C</td><td>L</td><td>S</td><td>C</td><td>L</td><td>S</td><td>C</td><td>L</td><td>S</td><td>C</td><td>L</td><td>S</td><td>C</td><td>L</td><td>S</td><td>C</td><td>L</td><td>S</td><td>C</td><td>L</td><td>S</td><td>C</td><td>L</td><td>S</td></tr>
</table>

C: removing a column at the corner

L: removing a column at the middle in the longer side

S: removing a column near the middle in the shorter side

Thus there were 27 scenarios, and the procedure was as following:

1. The reinforcement steel area, A_s, for every cross section for every beam element was known.
2. According to A_s , the ultimate moment capacity was calculated.
3. Three plastic hinges were inserted for every beam elements, one at both ends of the beam element and the third at the middle of it.
4. The SAP2000was fed by the loads according to GSA requirements.

5. The concerned load bearing element, column, was removed
6. Nonlinear static analysis was performed via SAP2000.
7. Observing the formulation of plastic hinges through the entire building, i.e. through the 93 beam elements.
8. Observing the formulation of failed beam elements through the entire building, i.e. through the 93 beam elements.

The above procedure was conducted for every scenario listed in the above table, and the following are the details for the procedure.

5.3.1Ultimate Moment Capacity

According to the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007), the ultimate moment capacity, Mu, for any section can be calculated from either of the two ultimate moment capacities for the section

$Mu = A_s * f_y *(d- a/2)$ or

$Mu = 0.67 * f_{cu} / \gamma_c * a* b * (d- a/2)$, where

a is the actual length of the equivalent stress block,

b is the cross section breadth,

d is the effective cross section depth,

$A_s$the reinforcement area that was calculated for a specific seismic zone,

$f_y$is the yield strength or proof strength for reinforcement,

$f_{cu}$is the characteristic strength of concrete, and

γ_c is the strength reduction factor.

5.3.2 Plastic Hinge

After calculating the ultimate moment capacity for every cross section, a corresponding plastic hinge was fed to (SAP2000) program. For ultimate moment capacity there was one of the following cases:

- The section had only positive bending moment; in that case the section was provided by compression reinforcement with ratio of ten percent.
- The section had both positive and negative bending moment; in that case the section was designed for both bending moments.
- The section had only negative bending moment; in that case the section was provided by compression reinforcement with ratio of ten percent or according to the Egyptian code of practice the reinforcement details.

5.3.3 Assigning Plastic Hinges

Because this step is prerequisite for the analysis of progressive collapse via SAP2000 using nonlinear static analysis, and the expected of the failure of beam elements is not predefined, so the plastic hinges are assigned for the whole cross beam sections, i.e. for 279 sections a plastic hinge is assigned for every seismic zone for the three cases; slab thickness 10 mm, 160 mm, and no slab, with a total of 2511assigned plastic hinges.

The following figures depict the interface of the SAP2000 when the author defining a plastic hinge type and assigning the plastic hinge values.

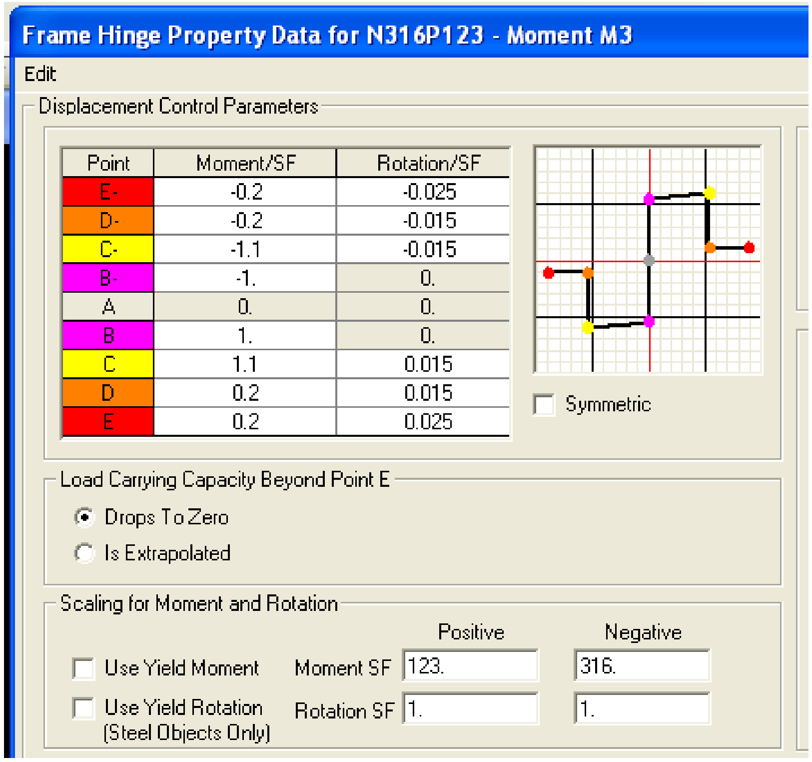


Fig.5.1 defining a plastic hinge type

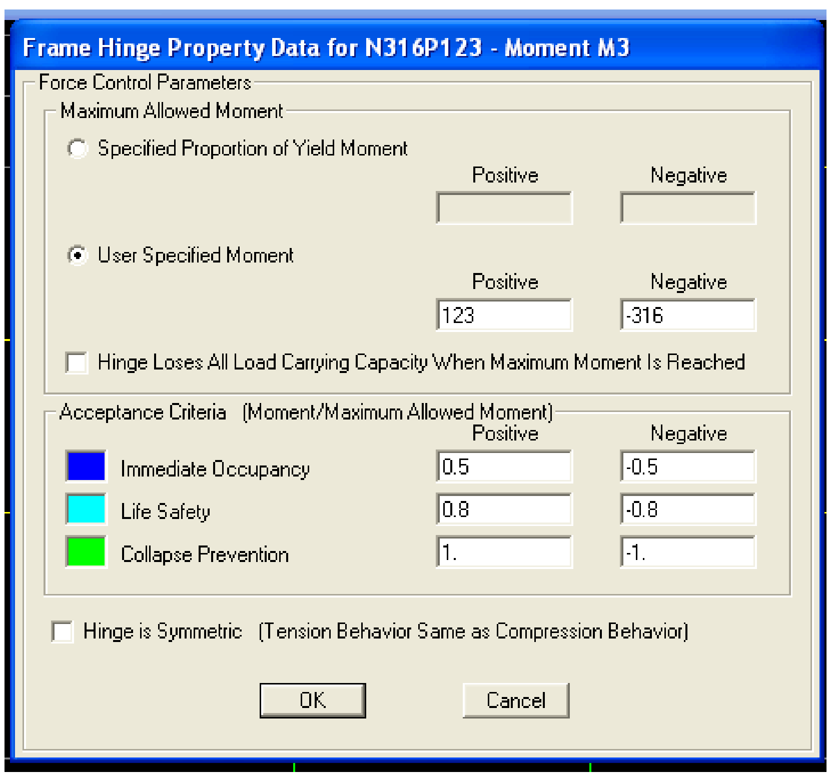


Fig.5.2 assigning ultimate bending moment values for a hinge

5.3.4 Removing a Column at the Middle of the Longer Side

As the longer side of the RC building consists of 4 bays with span of 6.00 m for each, therefore the longer side has a column in the middle. The analysis done by following the next steps:

1. The column; that is on axes one and C and labeled 25, of the First floor in the middle of the frame was removed.
2. For the basic model a modification for load values were done to comply the GSA guidelines, i.e. the applied loads = 2(dead loads + 0.25 live loads).
3. Fed SAP2000 to run in the nonlinear static analysis mode.
4. After analysis was done, observations for formulation of plastic hinges through all the beam elements were examined and recorded. Also the failed beam elements were recorded.

5.3.5 Removing a Column near the Middle of the Shorter Side

As the shorter side of the RC building consists of 3 bays with span of 6.00 m for each, therefore the shorter side does not have a column in the middle, instead it has a column near the middle of the shorter side. The analysis done by following the next steps:

1. The column; that is on axes two and A and labeled 4, of the First floor near the middle of the frame was removed.
2. For the basic model a modification for load values were done to comply the GSA guidelines, i.e. the applied loads = 2(dead loads + 0.25 live loads).
3. Fed SAP2000 to run in the nonlinear static analysis mode.
4. After analysis was done, observations for formulation of plastic hinges through all the beam elements were examined and recorded. Also the failed beam elements were recorded.

5.3.6 Removing a Column at the Corner

As the RC building has four corner columns, one of them was chosen and the analysis done by following the next steps:

1. The column; that is on axes one and A and labeled 1, of the First floor at the corner of the frame was removed.
2. For the basic model a modification for load values were done to comply the GSA guidelines, i.e. the applied loads = 2(dead loads + 0.25 live loads).
3. Fed SAP2000 to run in the nonlinear static analysis mode.
4. After analysis was done, observations for formulation of plastic hinges through all the beam elements were examined and recorded. Also the failed beam elements were recorded.

5.4 Analyses Results

As the researcher has twenty seven scenarios for progressive collapse analyses, hence the results yield twenty seven cases for formulation of plastic hinges and failed beam elements. The following figures depict the results for every scenario. The researcher preferred to illustrate the formulation of plastic hinges and failed beam elements through every frame on every axis, so every figure has nine frames that shows the formulation of plastic hinges. After that there is a summary for the number of formulated plastic hinges and the number of failed beam elements.

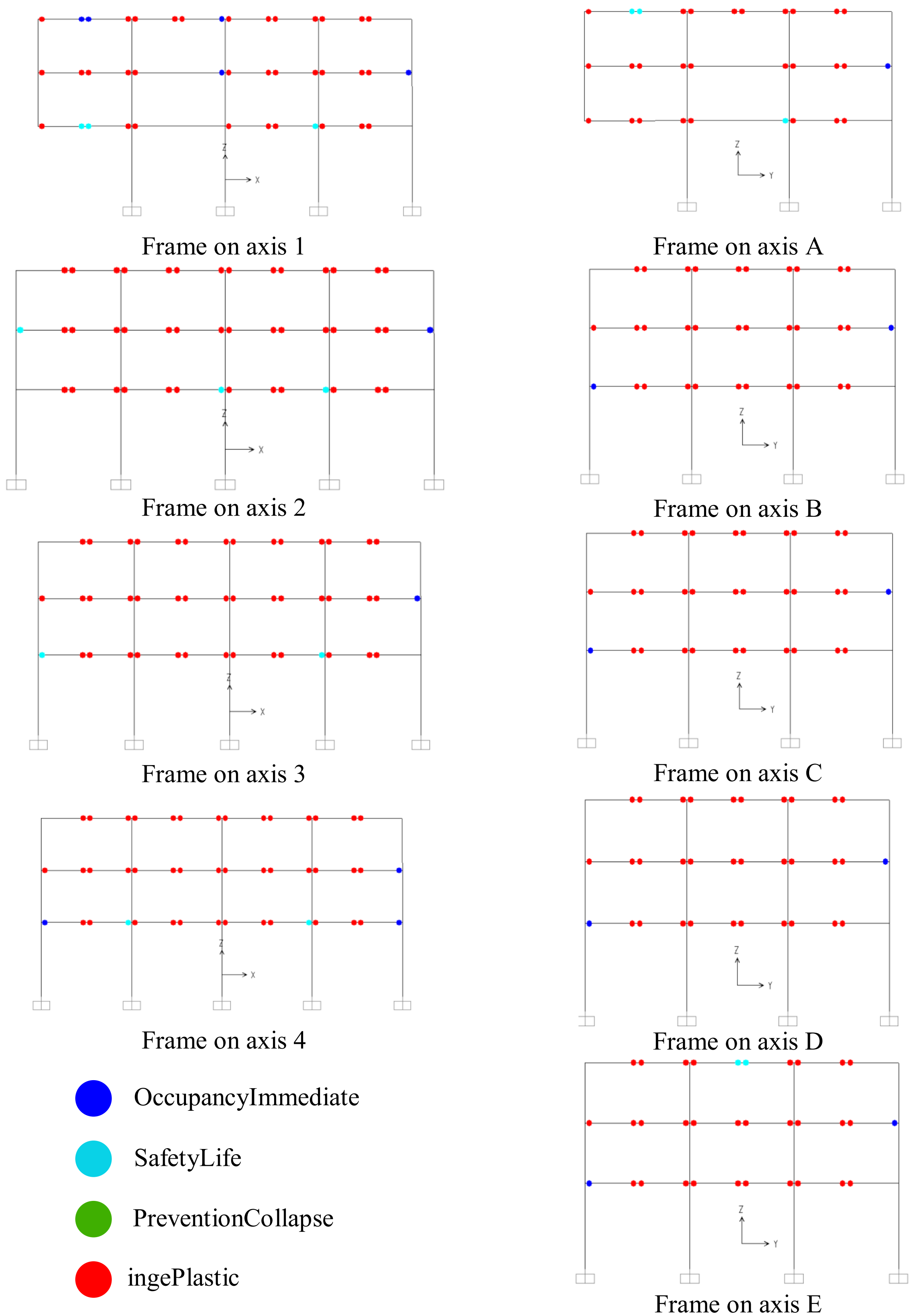


Fig. 5.3 Formulation of Plastic Hinges, Zone 1, t_s= 10 mm, Col. Removed at Corner

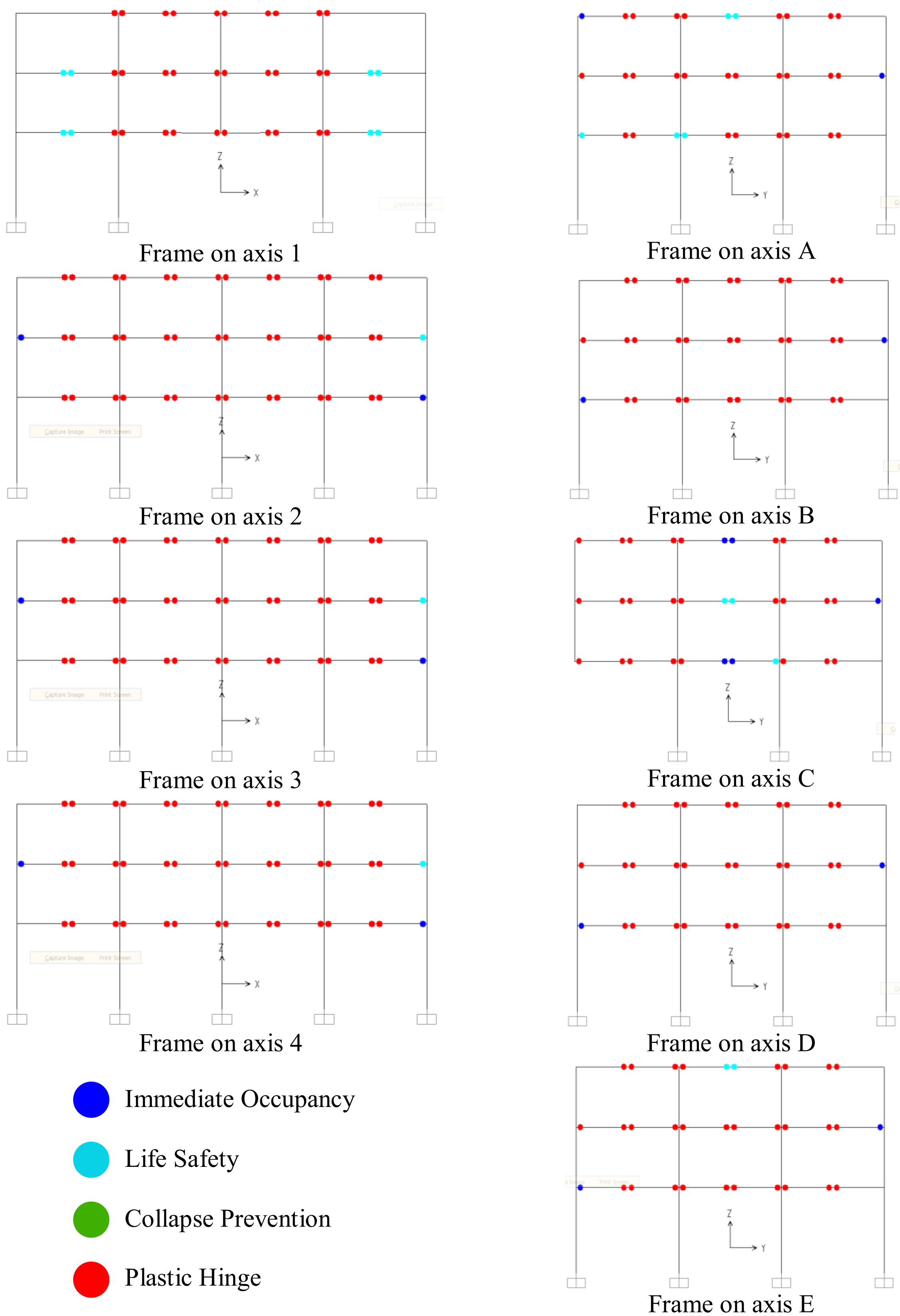


Fig. 5.4 Formulation of Plastic Hinges, Zone 1, ts = 10 mm, Col. Removed in the Longer Side

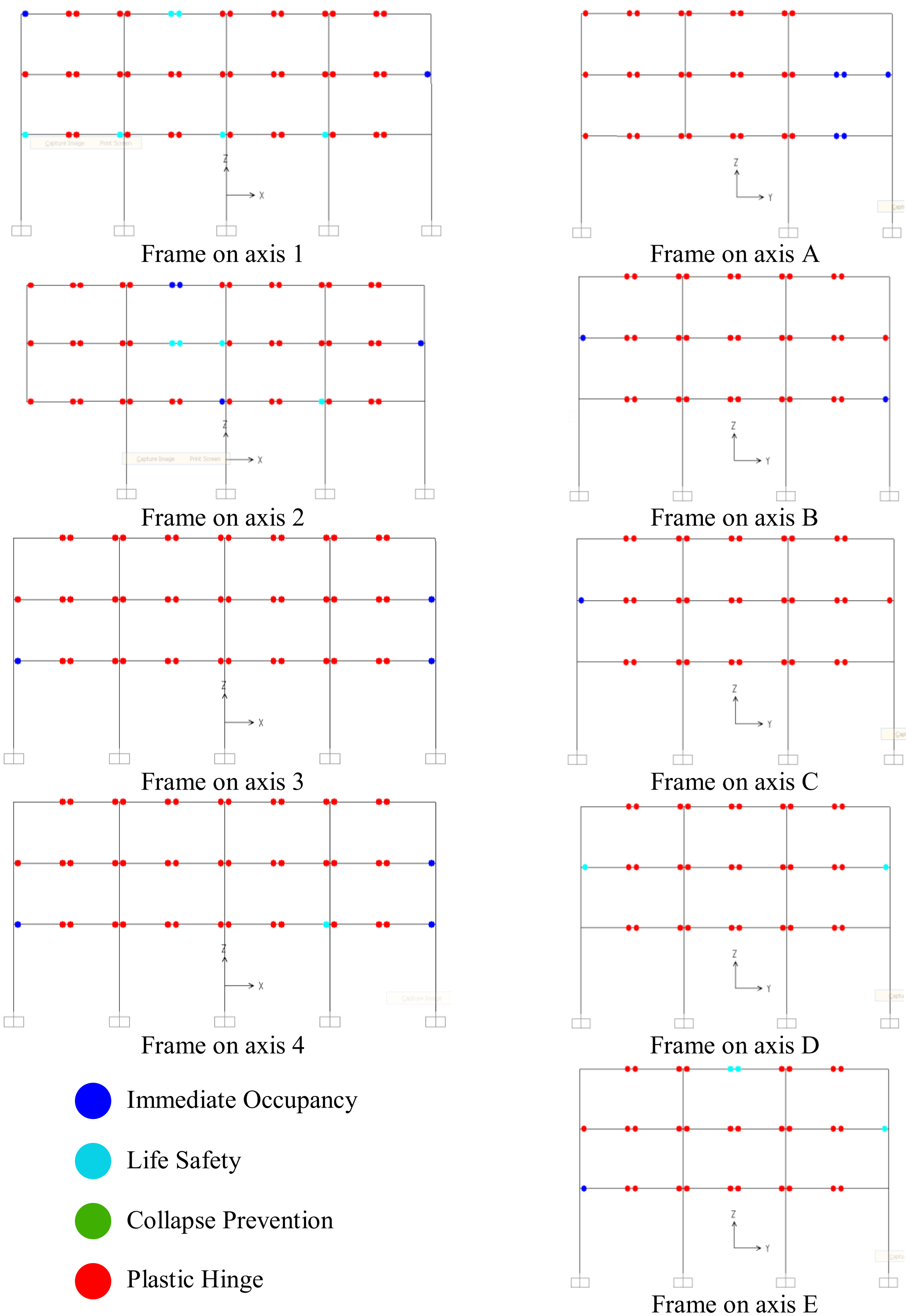


Fig. 5.5 Formulation of Plastic Hinges, Zone 1, ts = 10 mm, Col. Removed in the Short Side

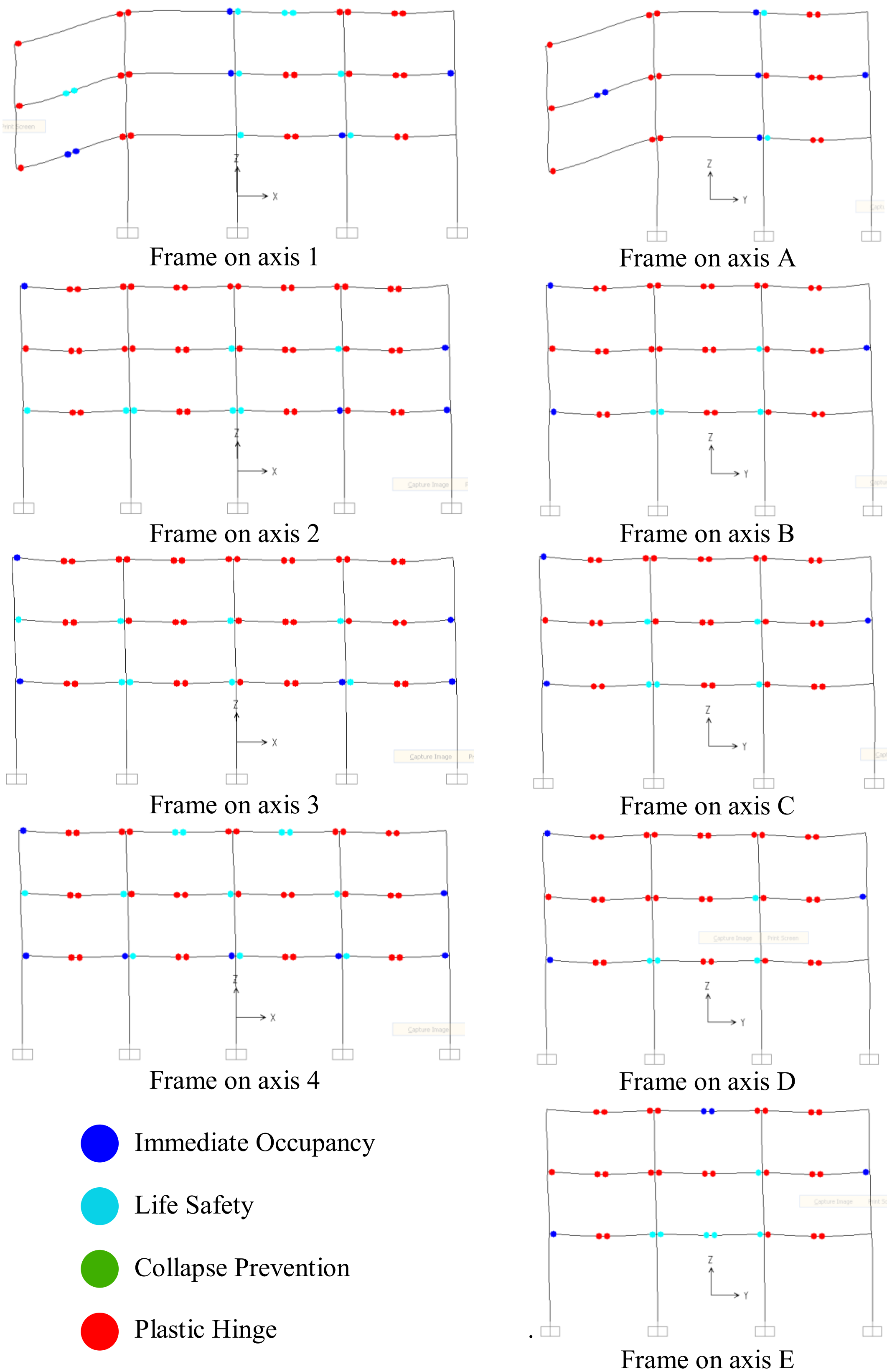


Fig. 5.6 Formulation of Plastic Hinges, Zone 1, ts = 160 mm, Col. Removed at Corner

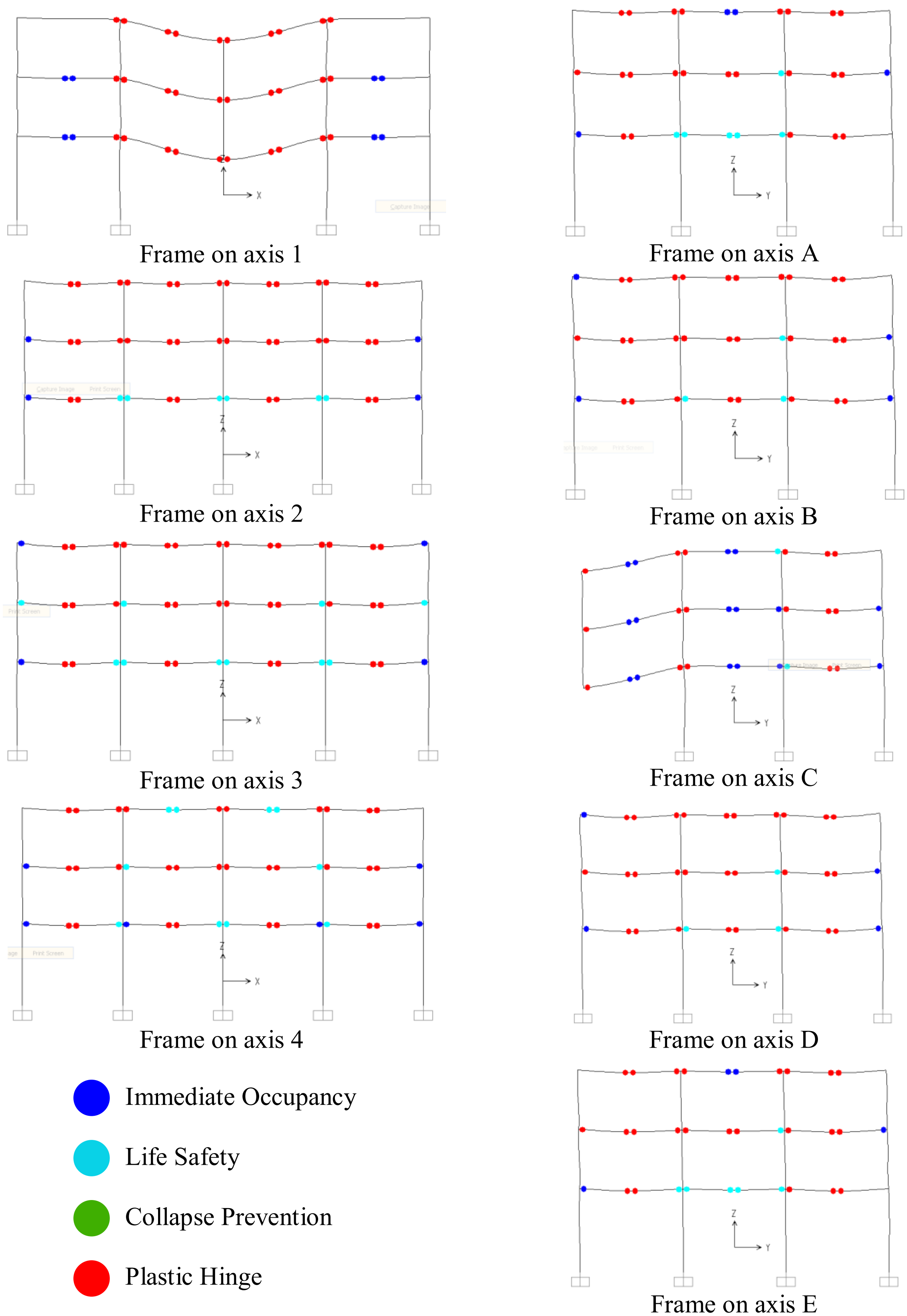


Fig. 5.7 Formulation of Plastic Hinges, Zone 1, ts = 160 mm, Col. Removed in the Longer Side

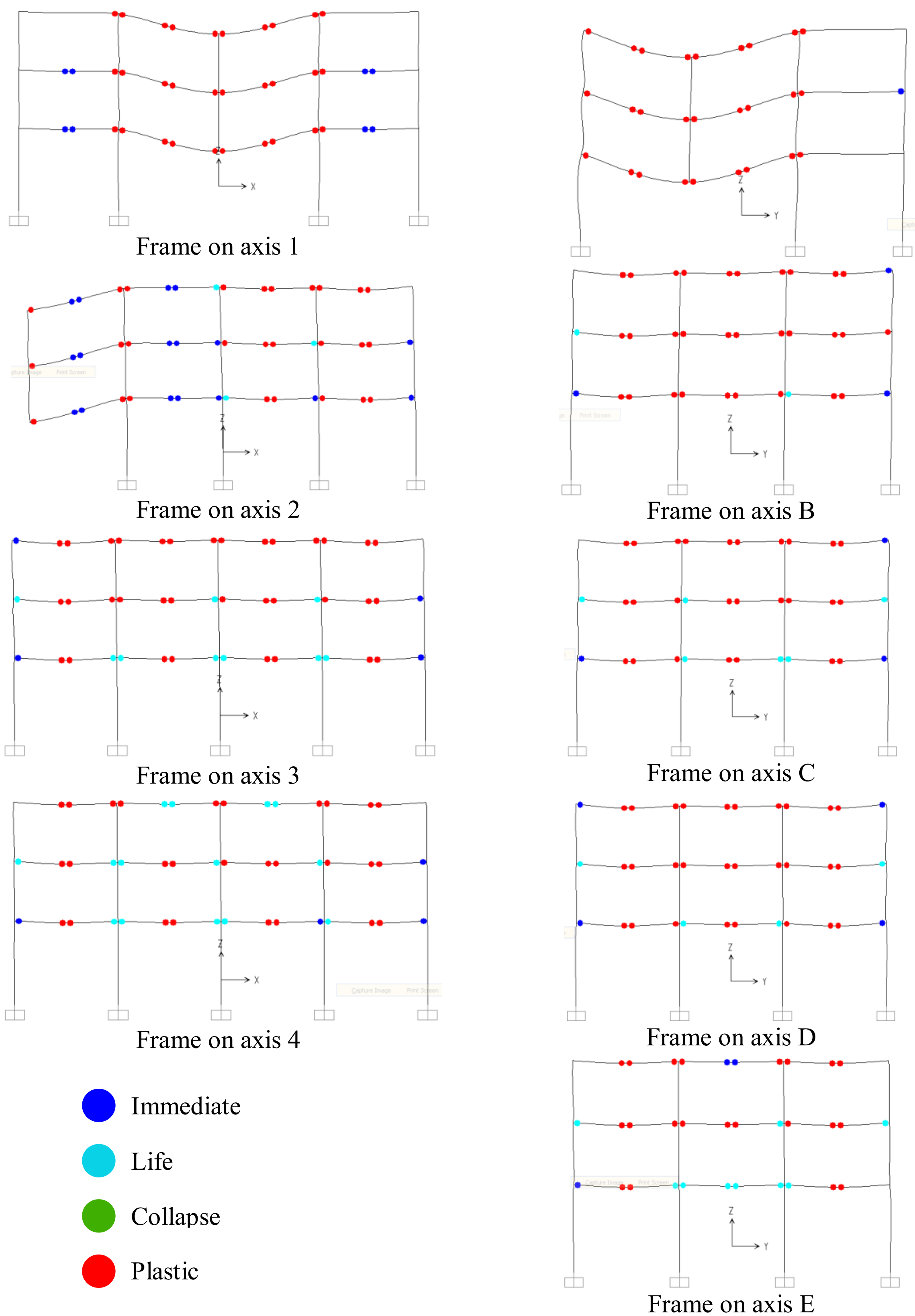


Fig. 5.8 Formulation of Plastic Hinges, Zone 1, ts = 160 mm, Col. Removed in the Short Side

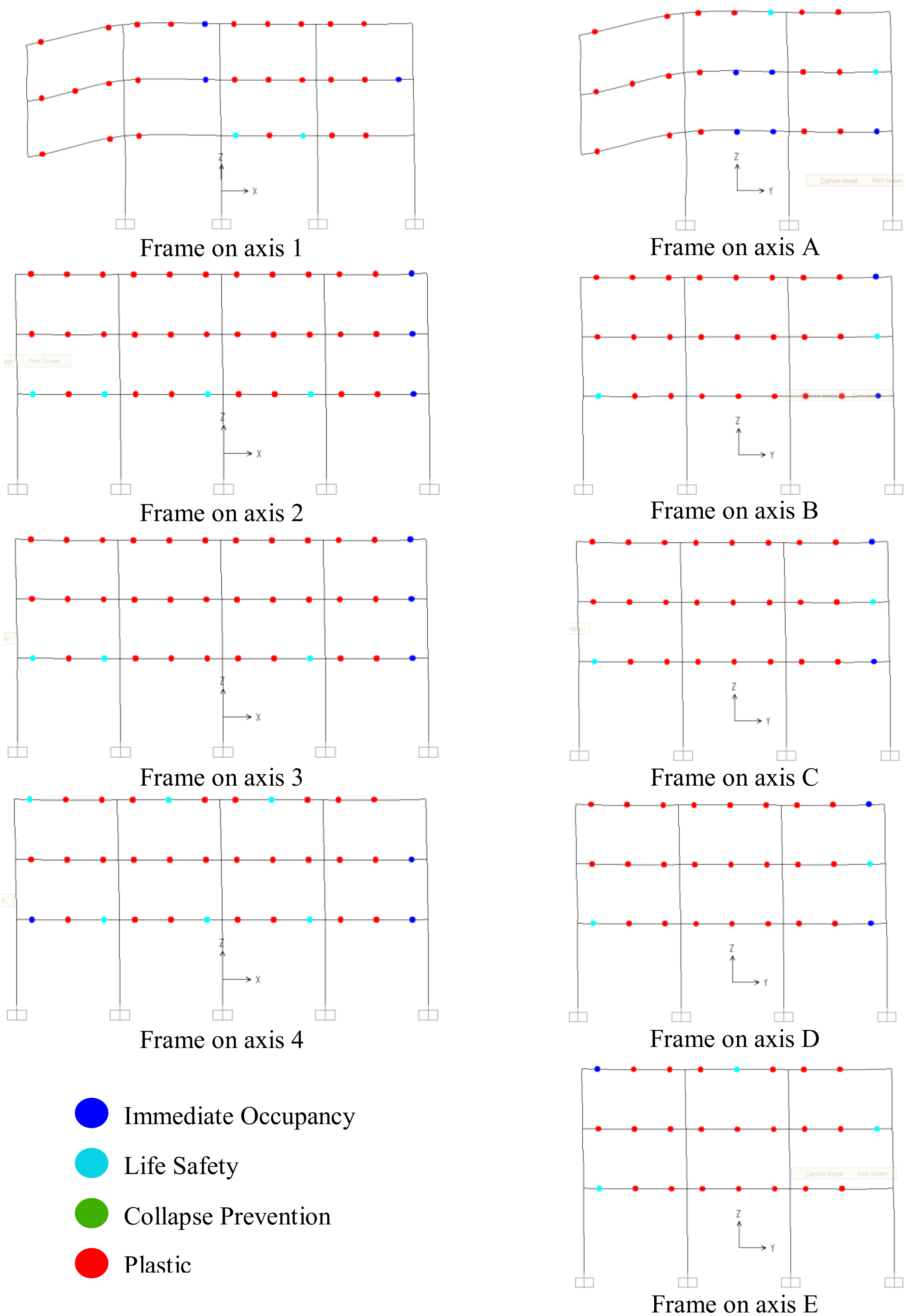


Fig. 5.9 Formulation of Plastic Hinges, Zone 1, No Slab, Col. Removed at the Corner

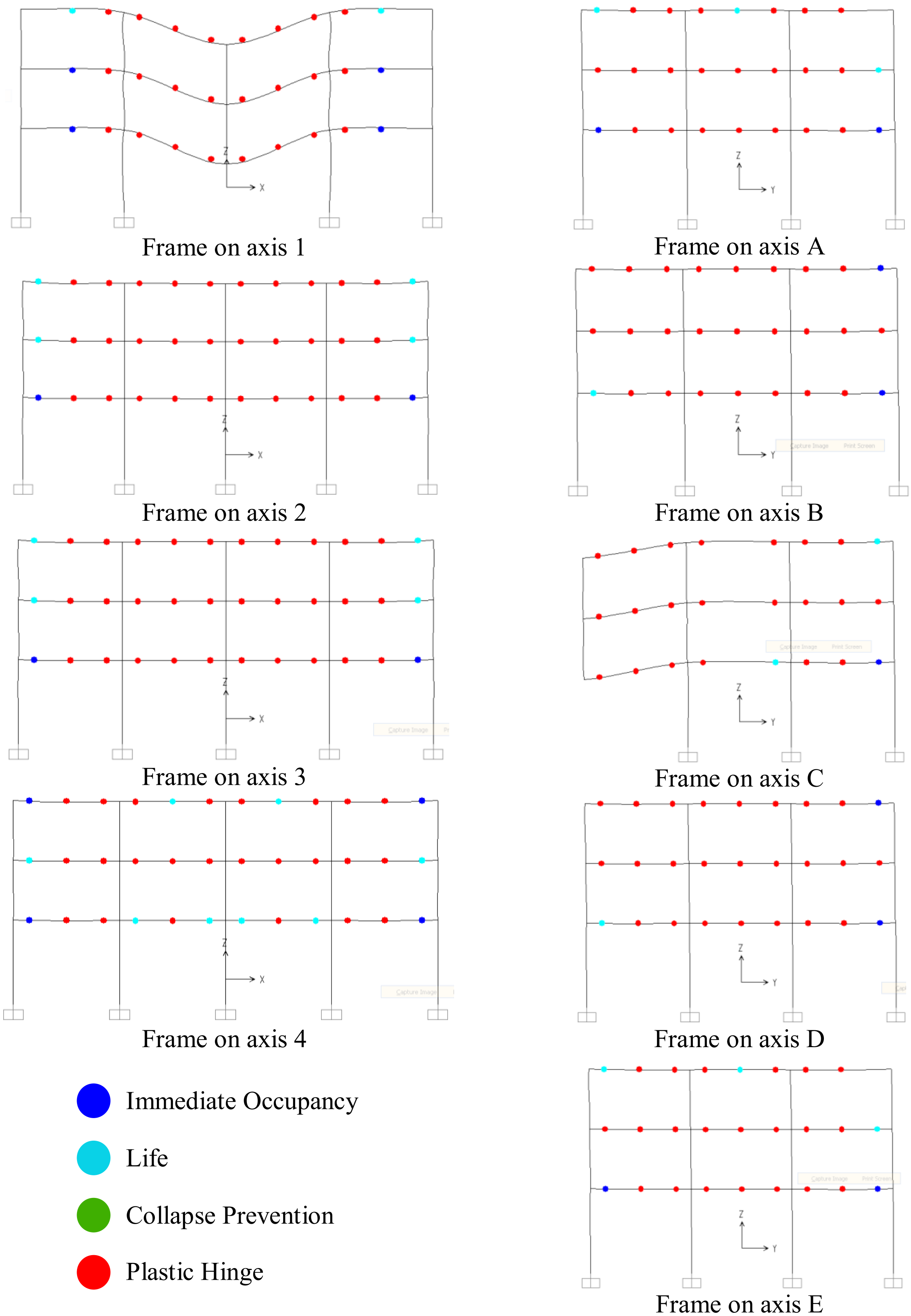


Fig. 5.10 Formulation of Plastic Hinges, Zone 1, No Slab, Col. Removed in the Longer Side

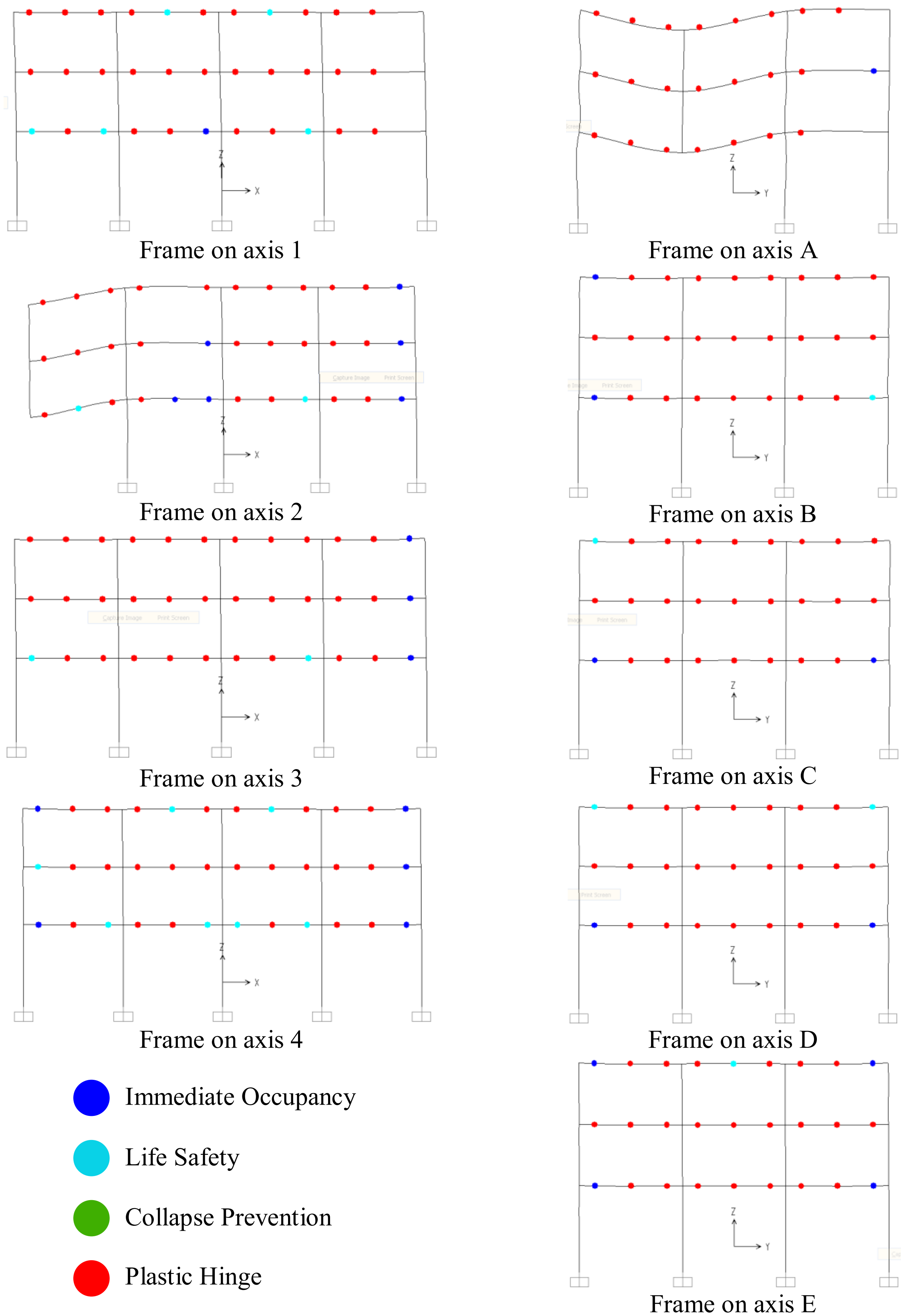


Fig. 5.11 Formulation of Plastic Hinges, Zone 1, No Slab, Col. Removed in the Shorter

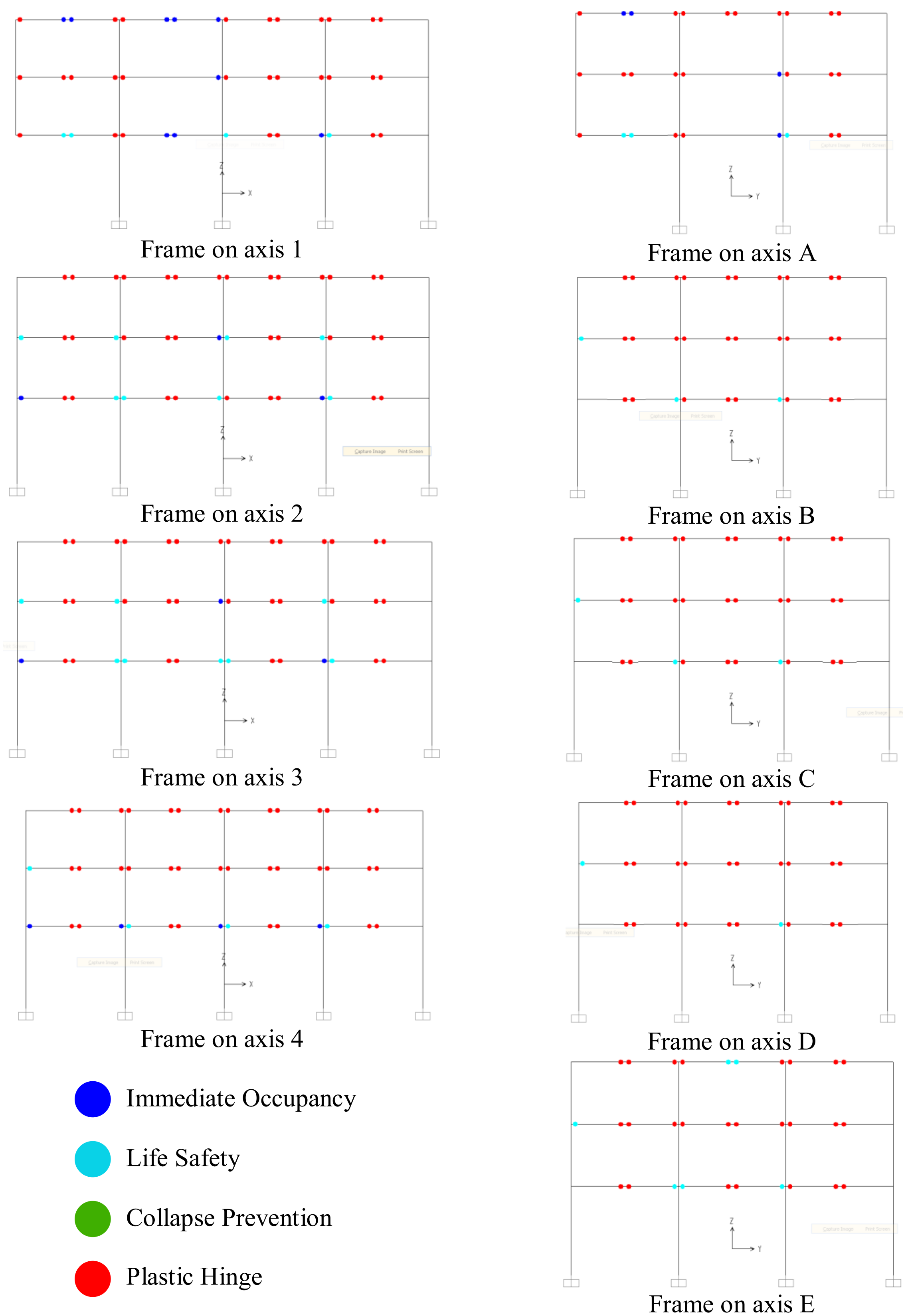


Fig. 5.12 Formulation of Plastic Hinges, Zone 3, ts = 10 mm, Col. Removed at Corner

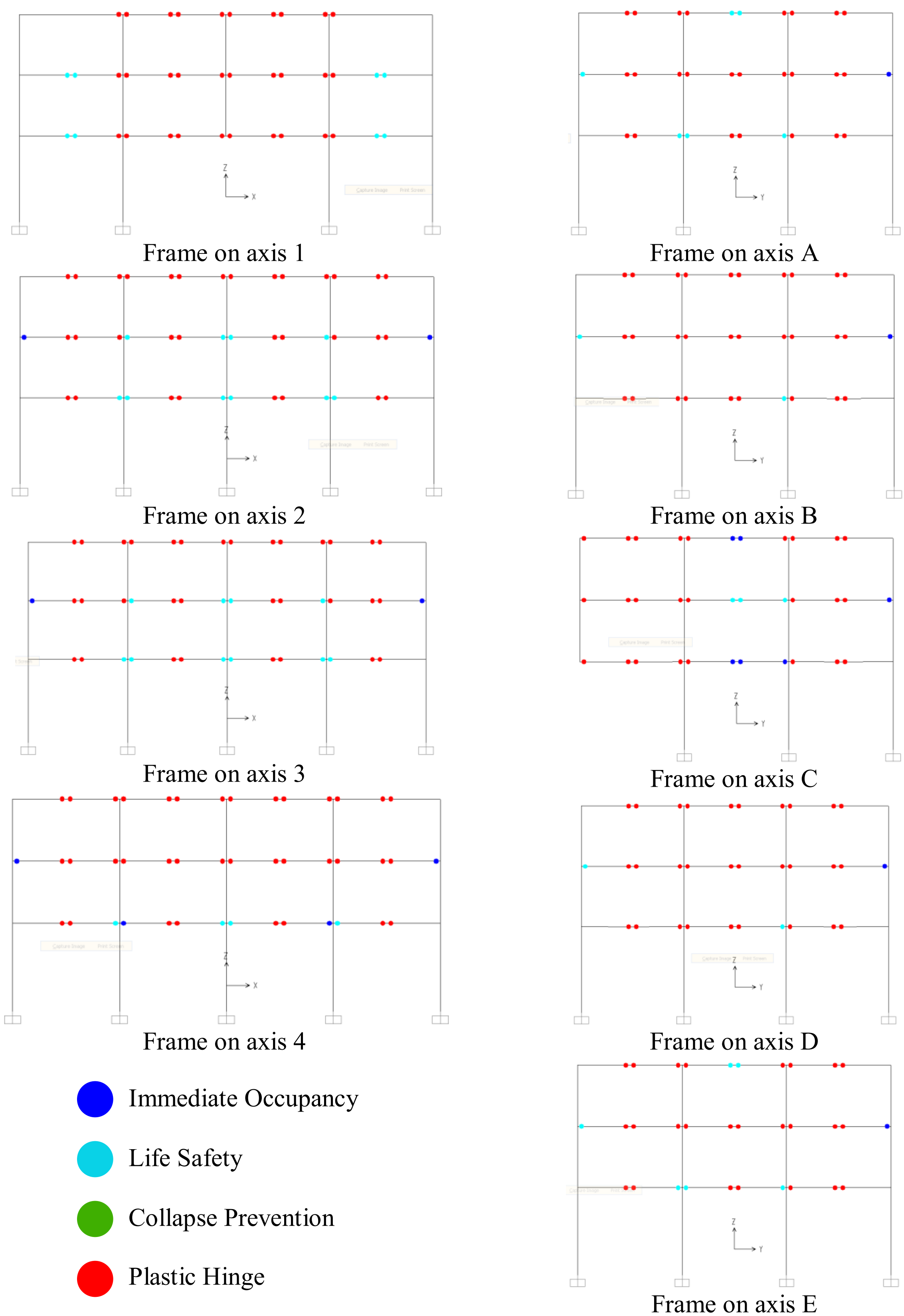


Fig. 5.13 Formulation of Plastic Hinges, Zone 3, ts = 10 mm, Col. Removed in the Longer Side

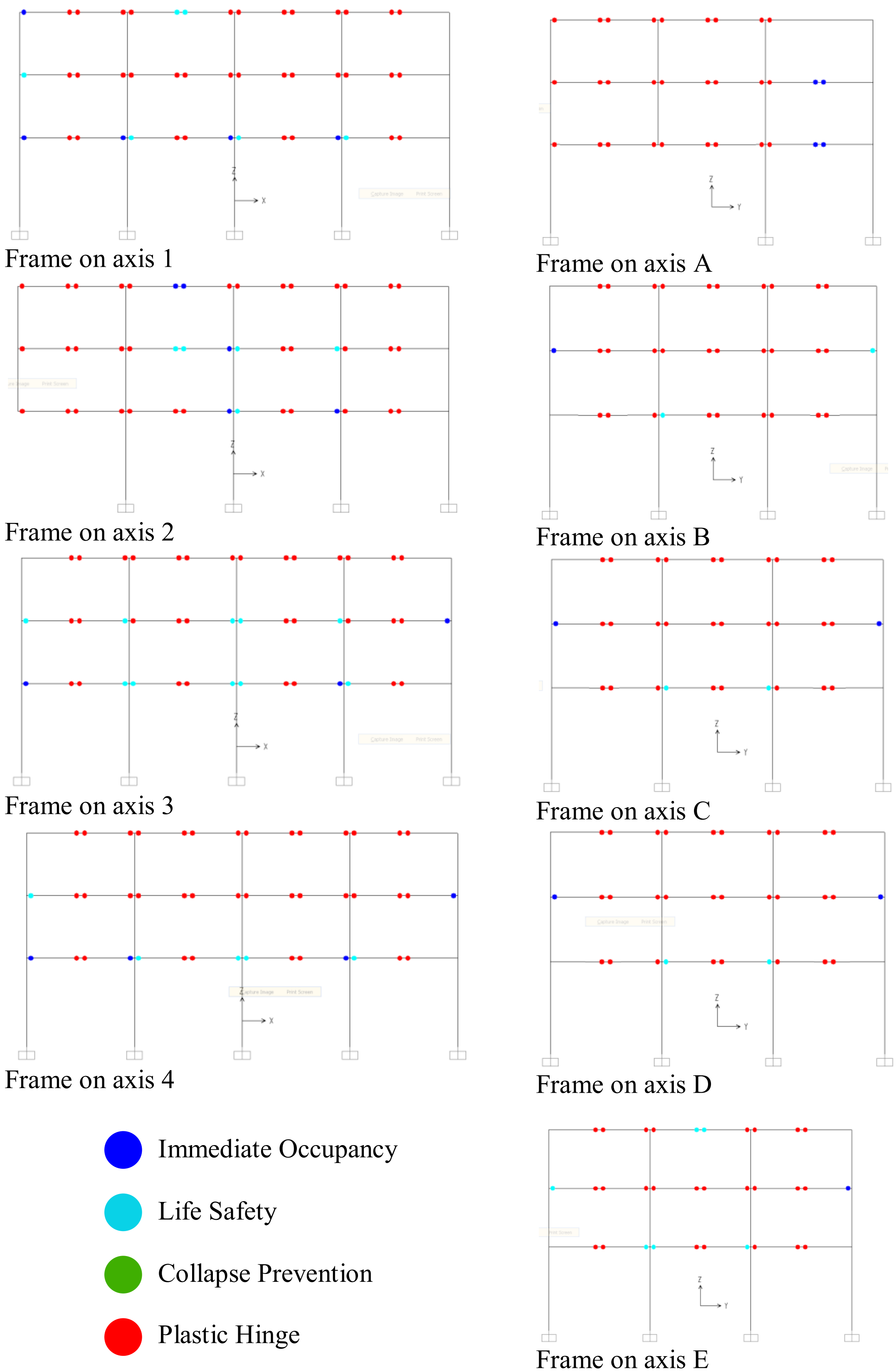


Fig. 5.14 Formulation of Plastic Hinges, Zone 3, ts = 10 mm, Col. Removed in the Shorter Side

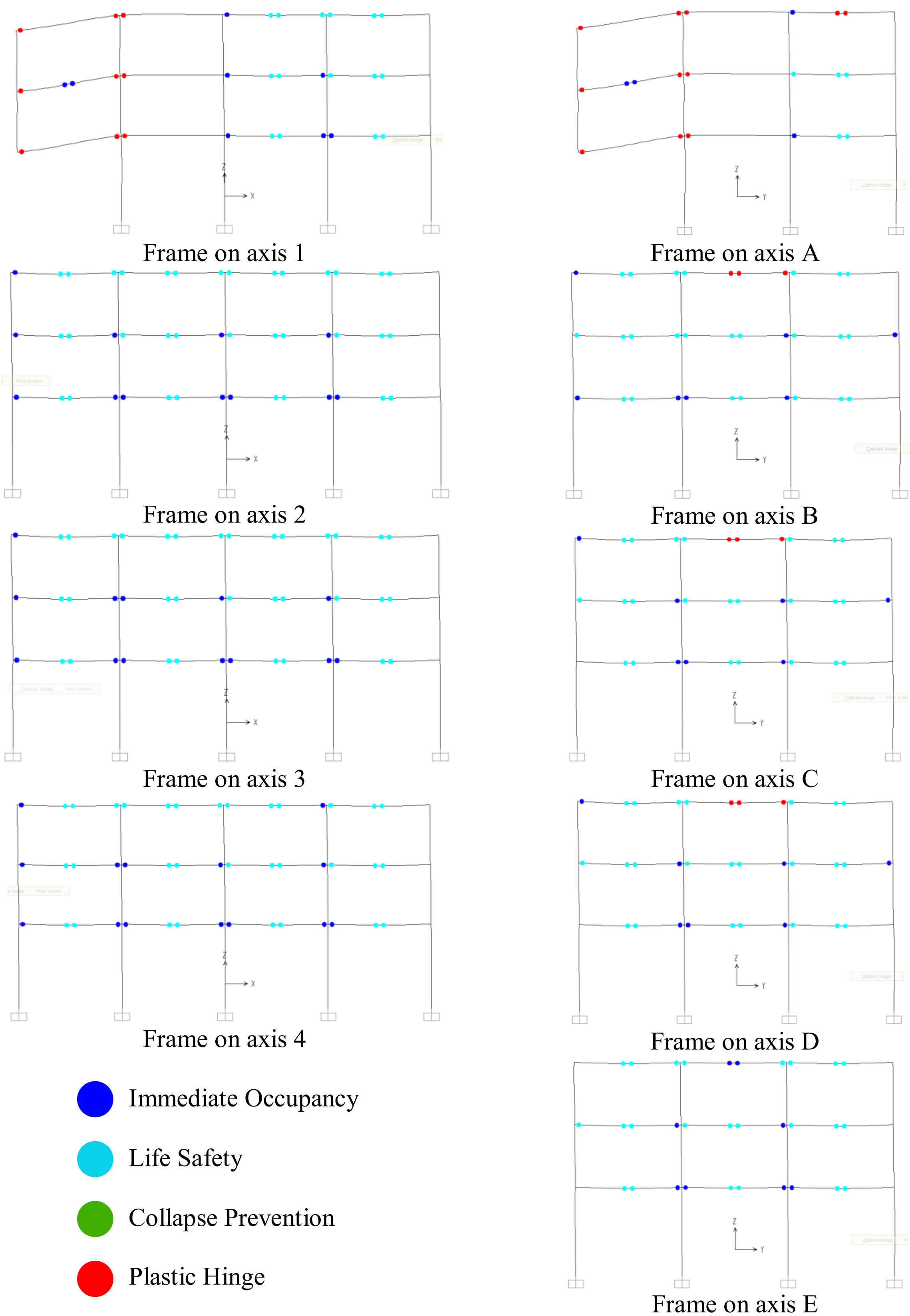
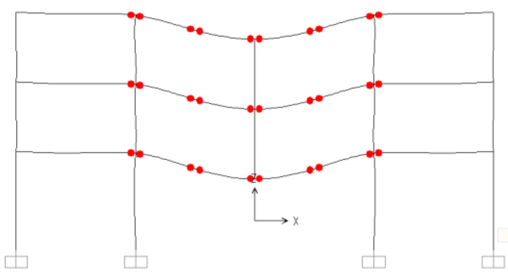
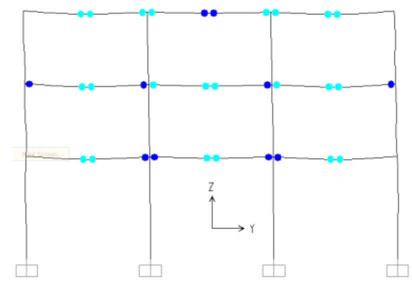


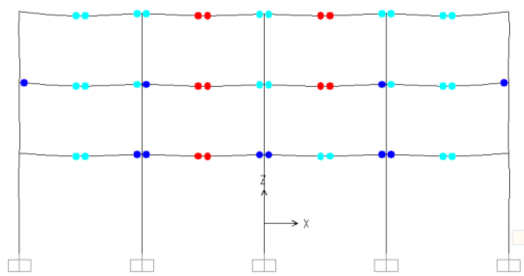
Fig. 5.15 Formulation of Plastic Hinges, Zone 3, ts = 160 mm, Col. Removed at Corner



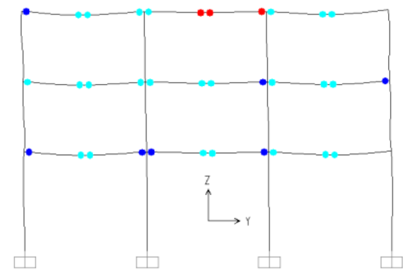
Frame on axis 1



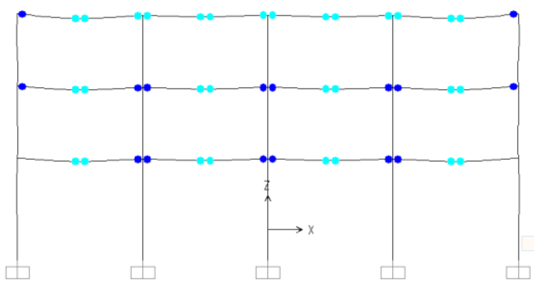
Frame on axis A



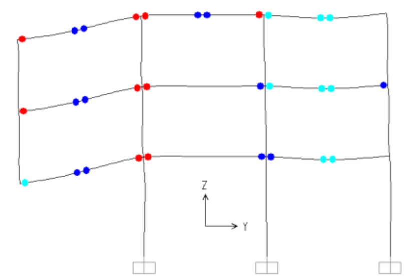
Frame on axis 2



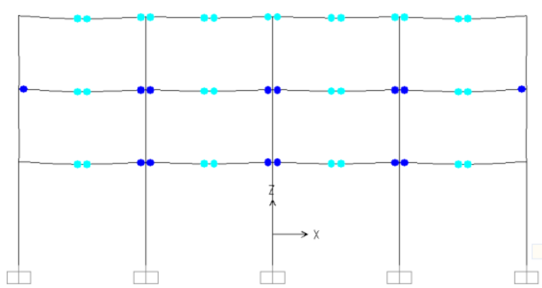
Frame on axis B



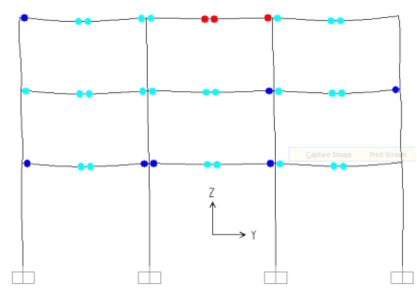
Frame on axis 3



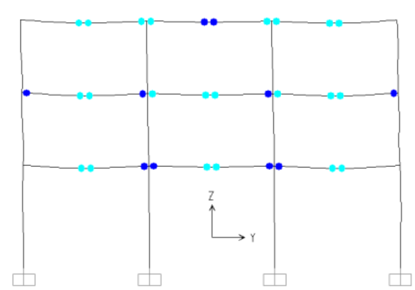
Frame on axis C



Frame on axis 4

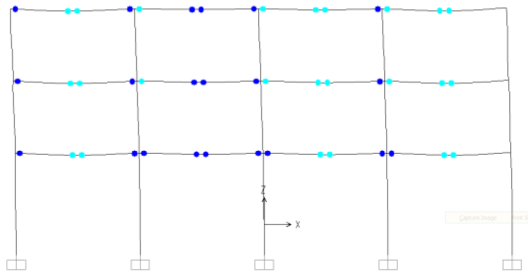


Frame on axis D

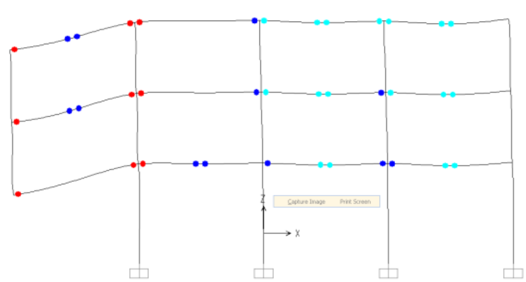


Frame on axis E

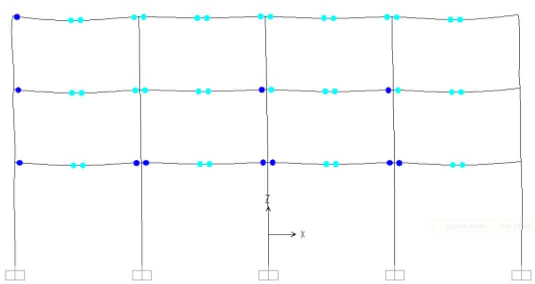
Fig. 5.16 Formulation of Plastic Hinges, zone 3, ts = 160 mm, Col. Removed in the Longer Side



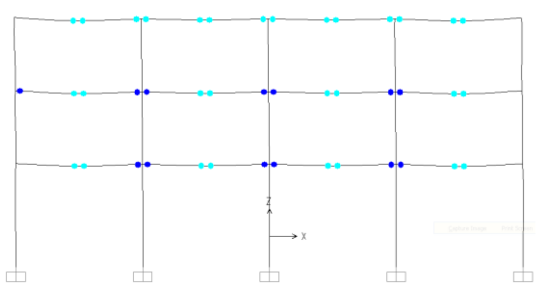
Frame on axis 1



Frame on axis 2



Frame on axis 3



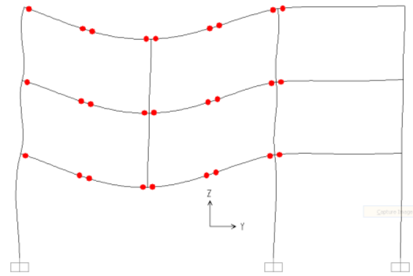
Frame on axis 4

Immediate Occupancy

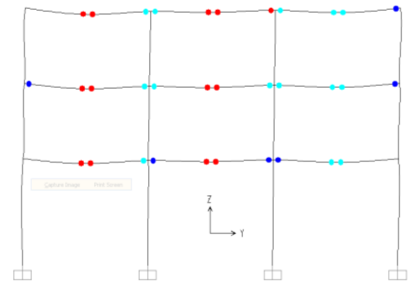
Life Safety

Collapse Prevention

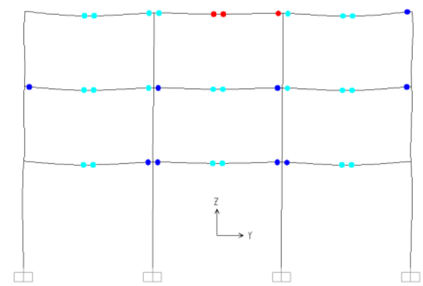
Plastic Hinge



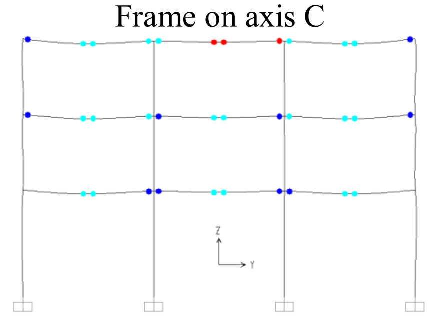
Frame on axis A



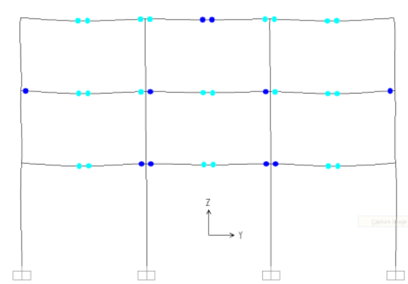
Frame on axis B



Frame on axis C

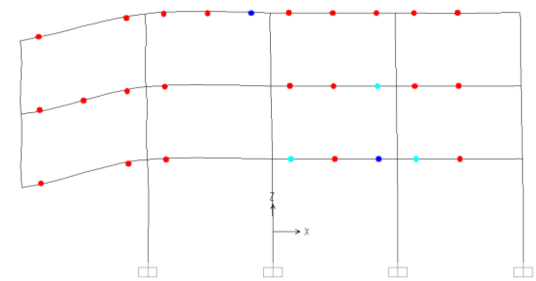


Frame on axis D

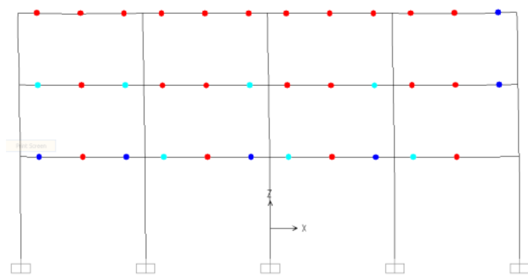


Frame on axis E

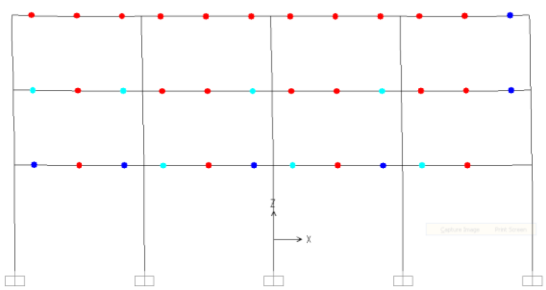
Fig. 5.17 Formulation of Plastic Hinges, , Zone 3, No Slab, Col. Removed in the longer side



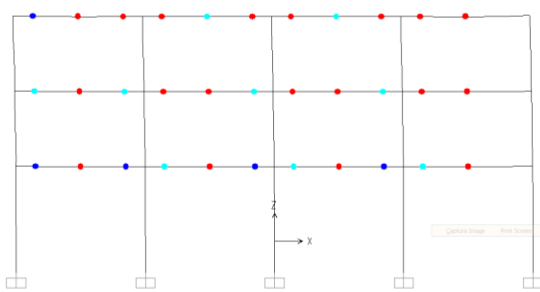
Frame on axis 1



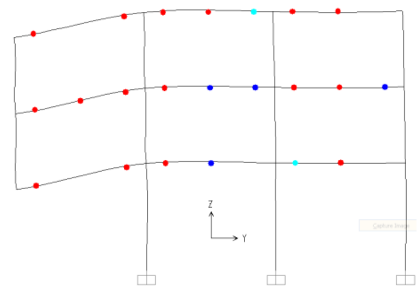
Frame on axis 2



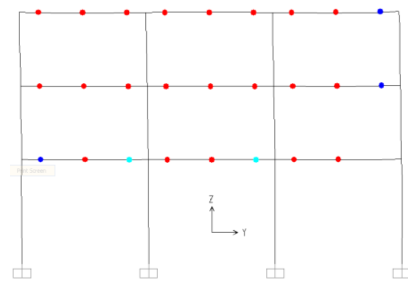
Frame on axis 3



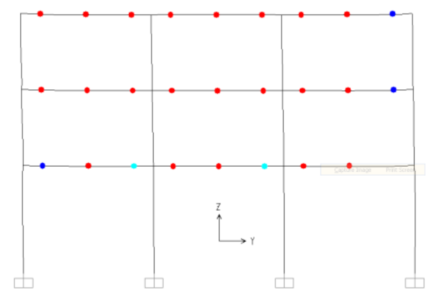
Frame on axis 4



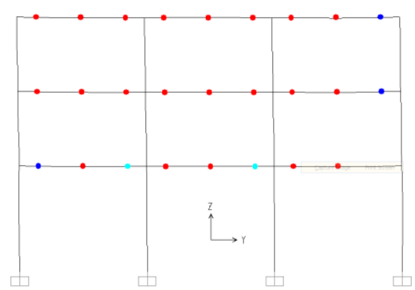
Frame on axis A



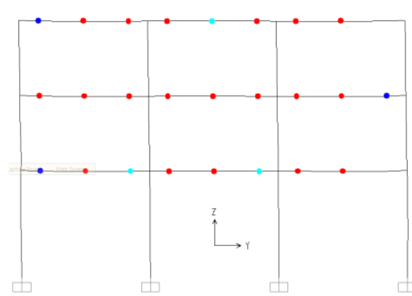
Frame on axis B



Frame on axis C



Frame on axis D



Frame on axis E

- Immediate Occupancy
- Life Safety
- Collapse Prevention
- Plastic Hinge

Fig. 5.18 Formulation of Plastic Hinges, Zone 3, No Slab, Col. Removed at the Corner

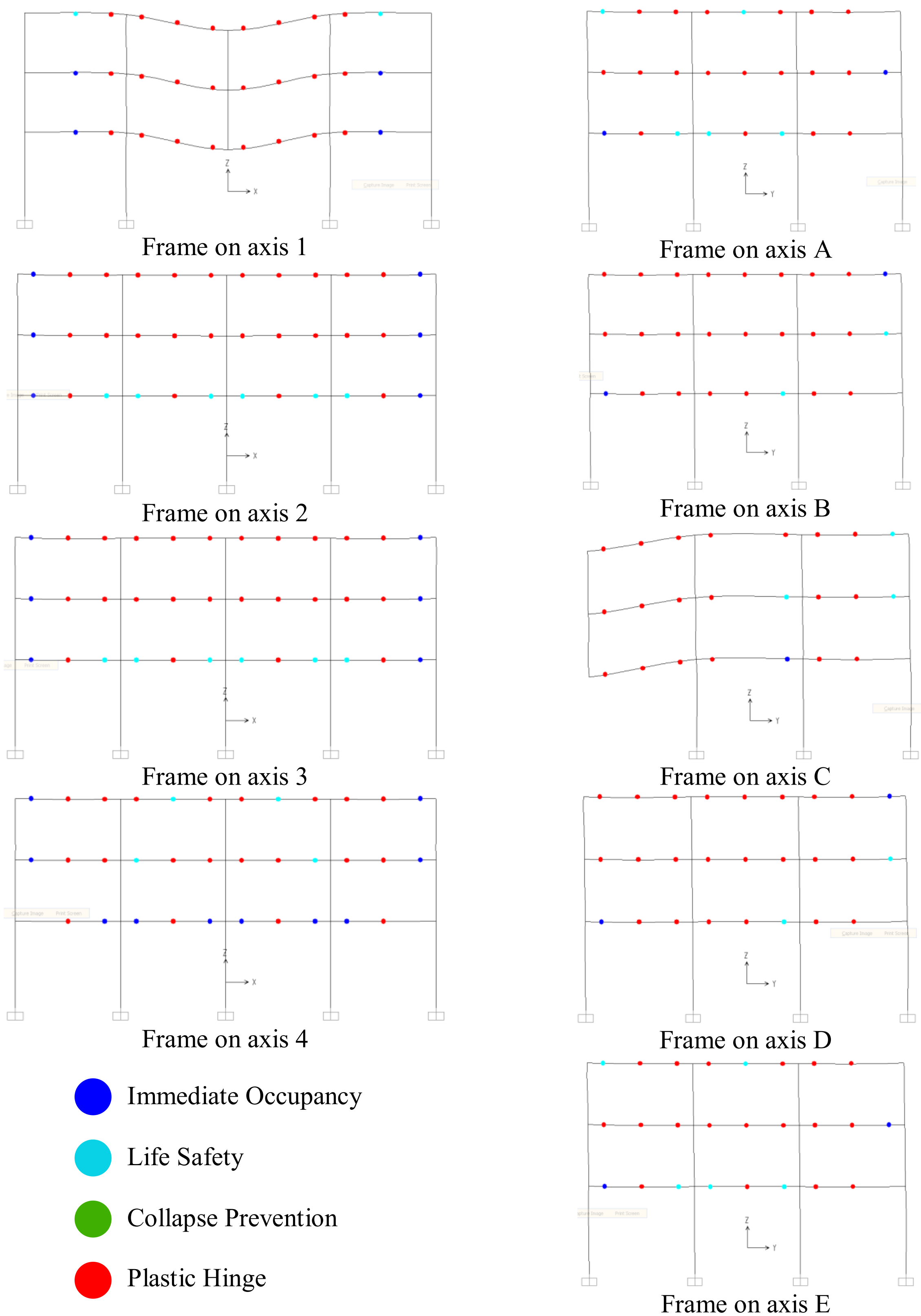


Fig. 5.19 Formulation of Plastic Hinges, Zone 3, No Slab, Col. Removed in the Longer Side

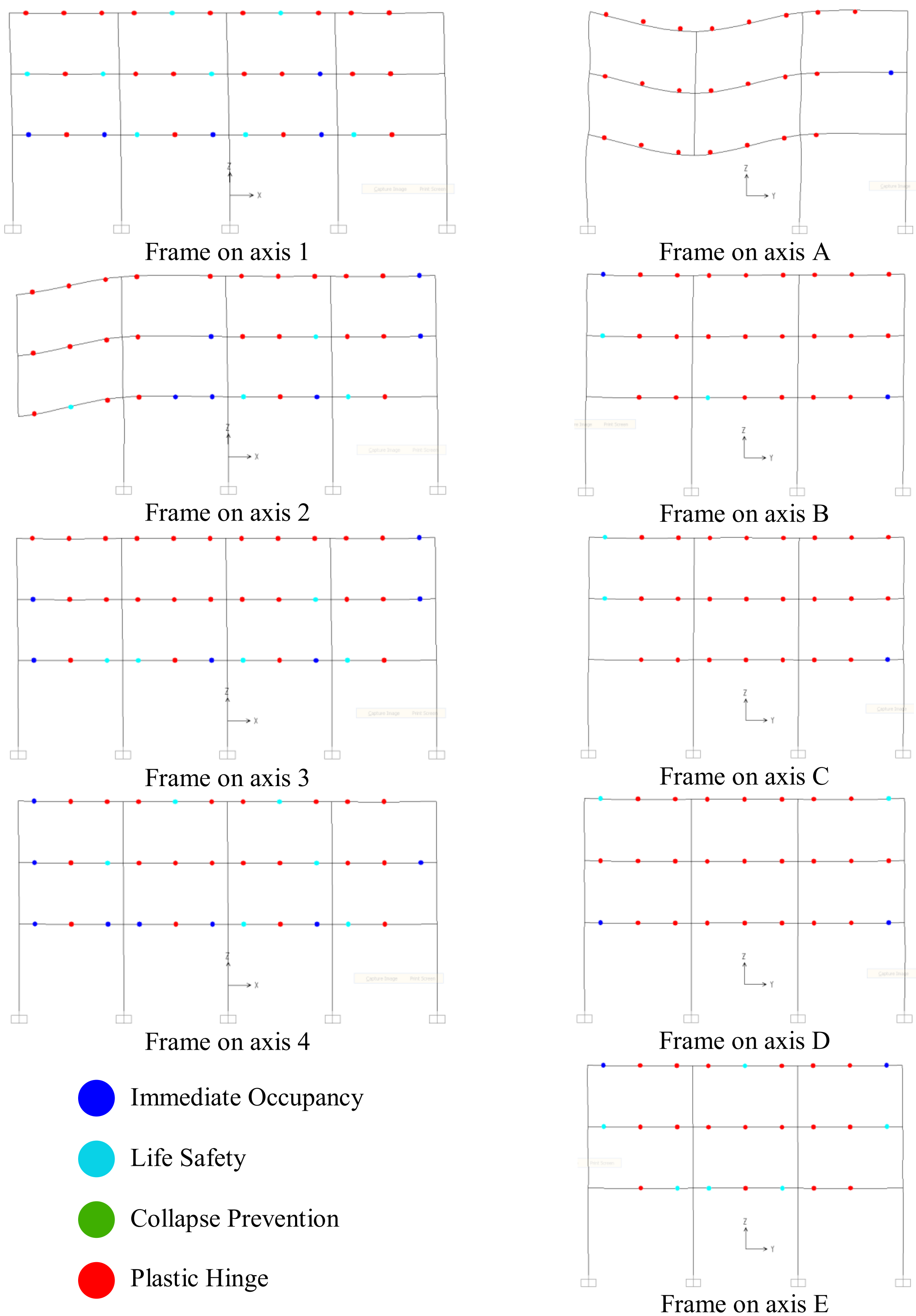


Fig. 5.20 Formulation of Plastic Hinges, Zone 3, No Slab, Col. Removed in the Shorter

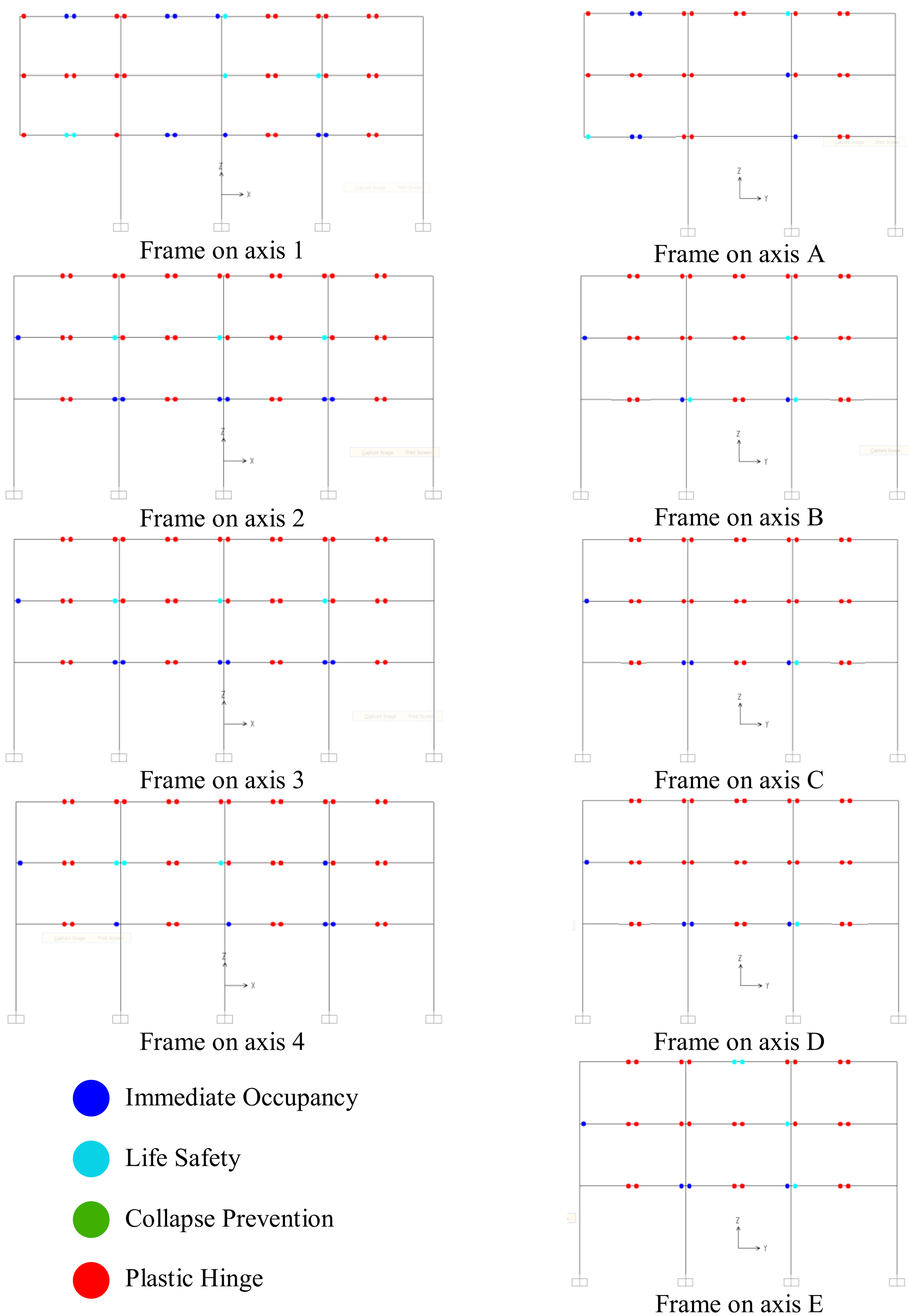
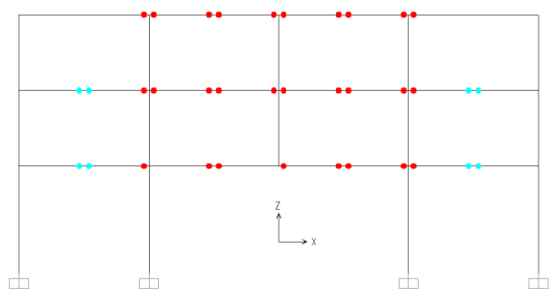
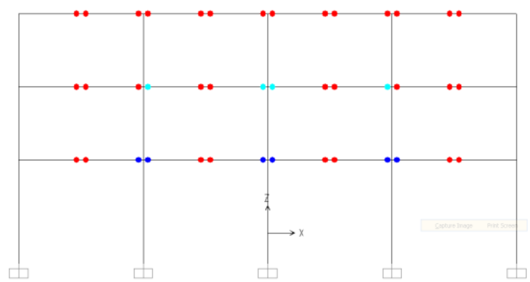


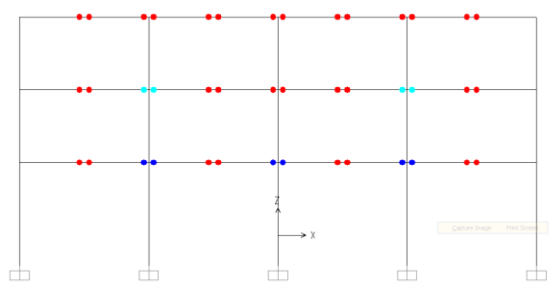
Fig. 5.21 Formulation of Plastic Hinges, Zone 5, ts = 10 mm, Col. Removed at Corner



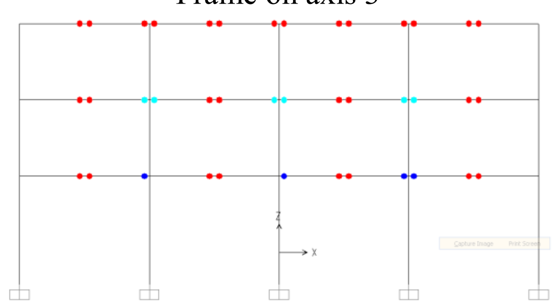
Frame on axis 1



Frame on axis 2



Frame on axis 3



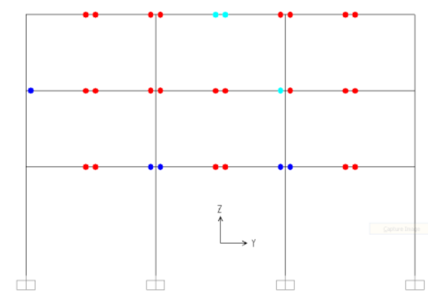
Frame on axis 4

Immediate Occupancy

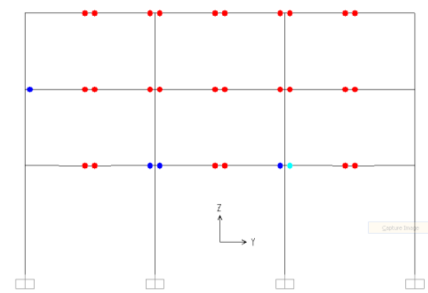
Life Safety

Collapse Prevention

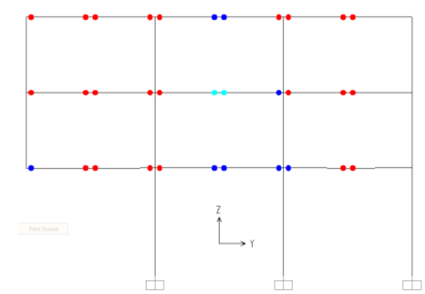
Plastic Hinge



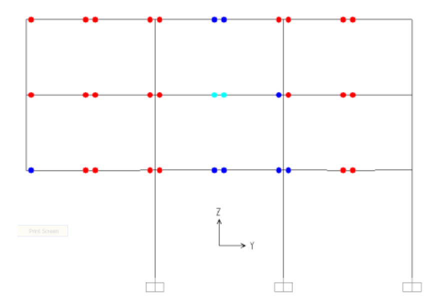
Frame on axis A



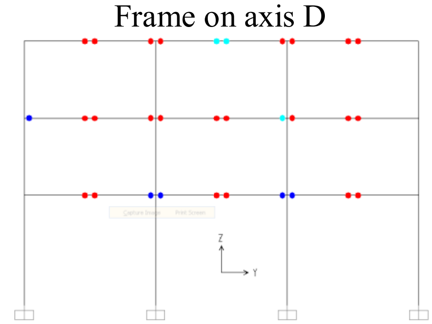
Frame on axis B



Frame on axis C



Frame on axis D



Frame on axis E

Fig. 5.22 Formulation of Plastic Hinges, Zone 5, ts = 10 mm, Col. Removed in the Longer Side

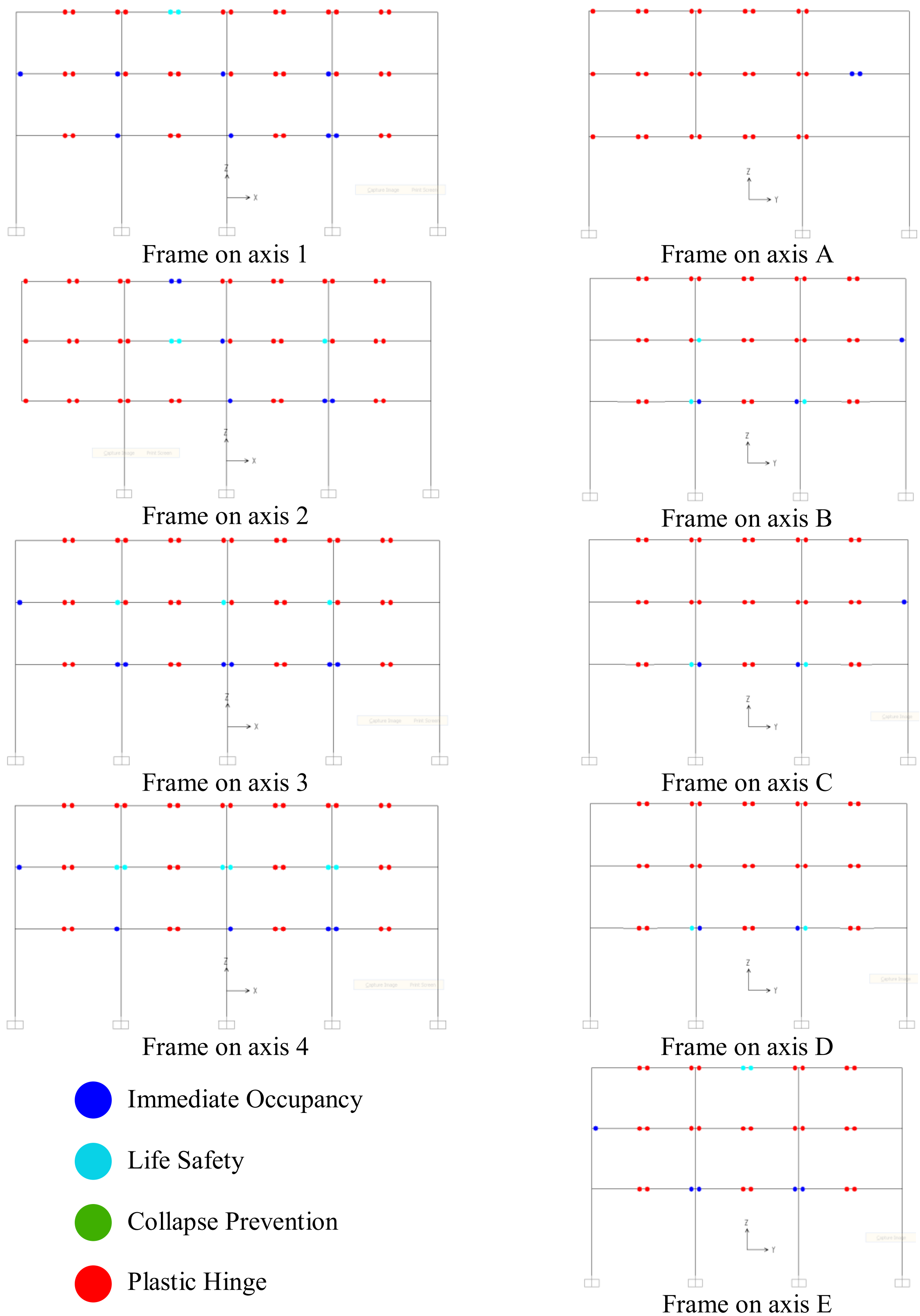


Fig. 5.23 Formulation of Plastic Hinges, Zone 5, ts = 10 mm, Col. Removed in the Short Side

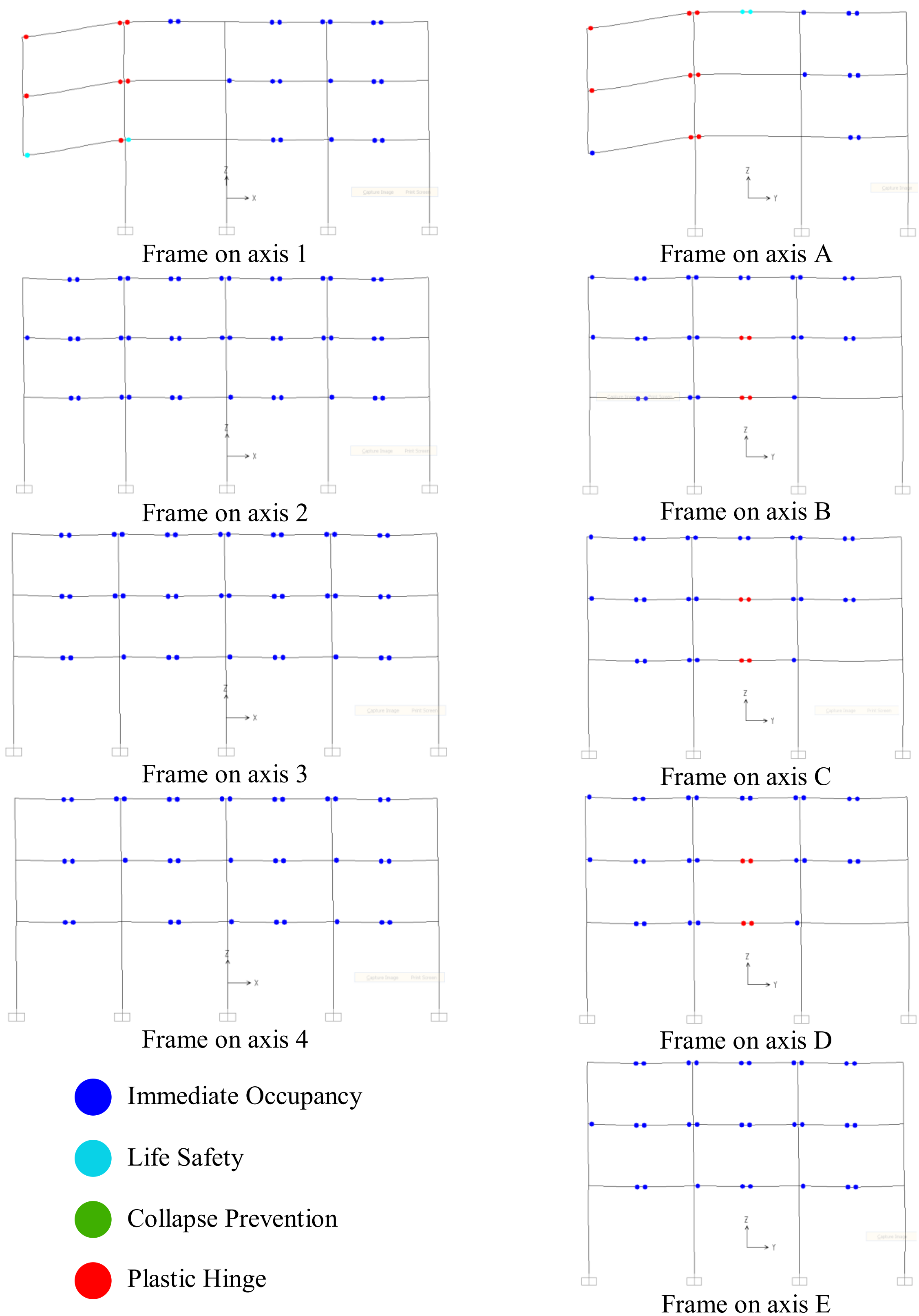


Fig. 5.24 Formulation of Plastic Hinges, Zone 5, ts = 160 mm, Col. Removed at Corner

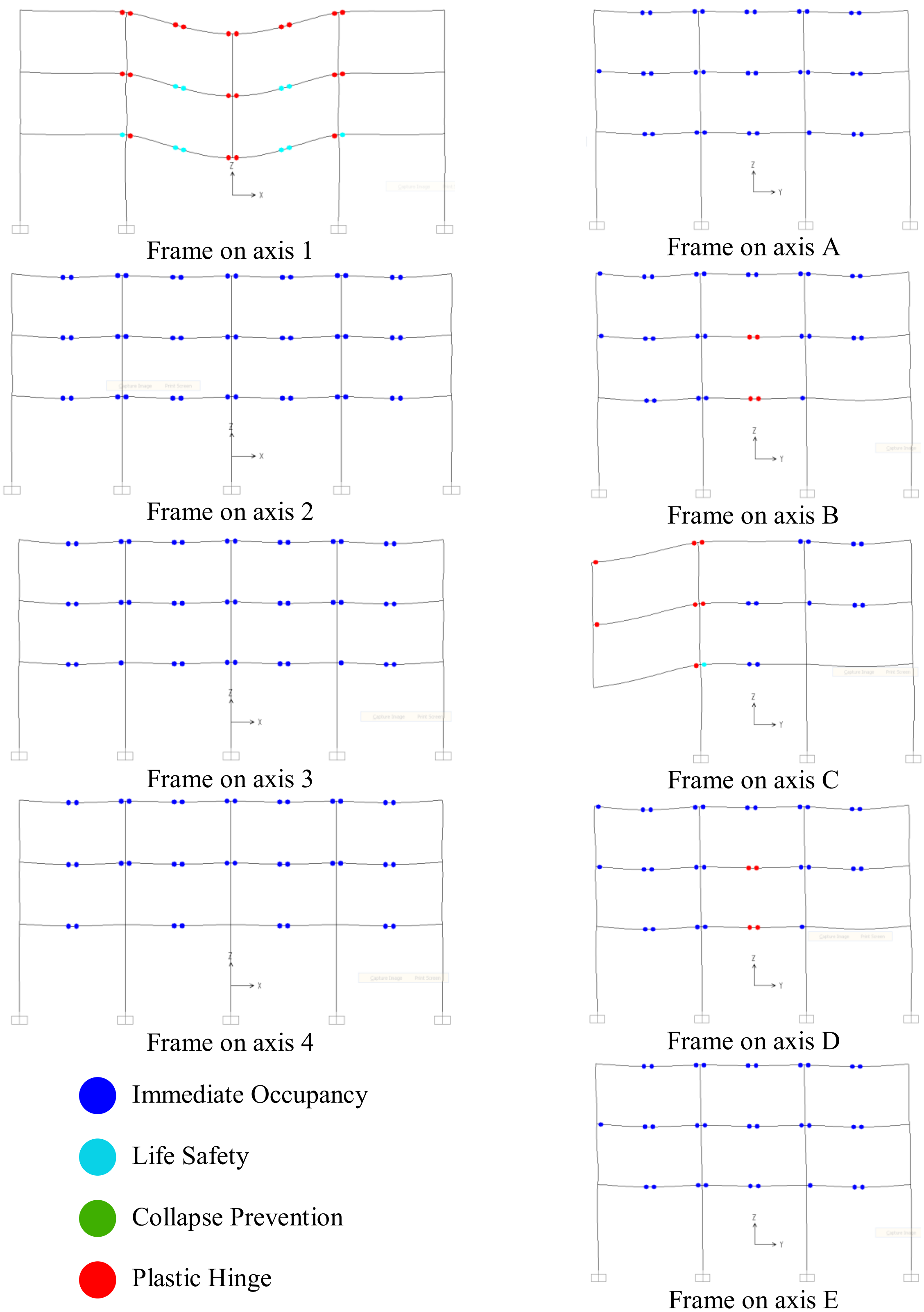


Fig. 5.25 Formulation of Plastic Hinges, Zone 5, ts = 160 mm, Col. Removed in the Longer Side

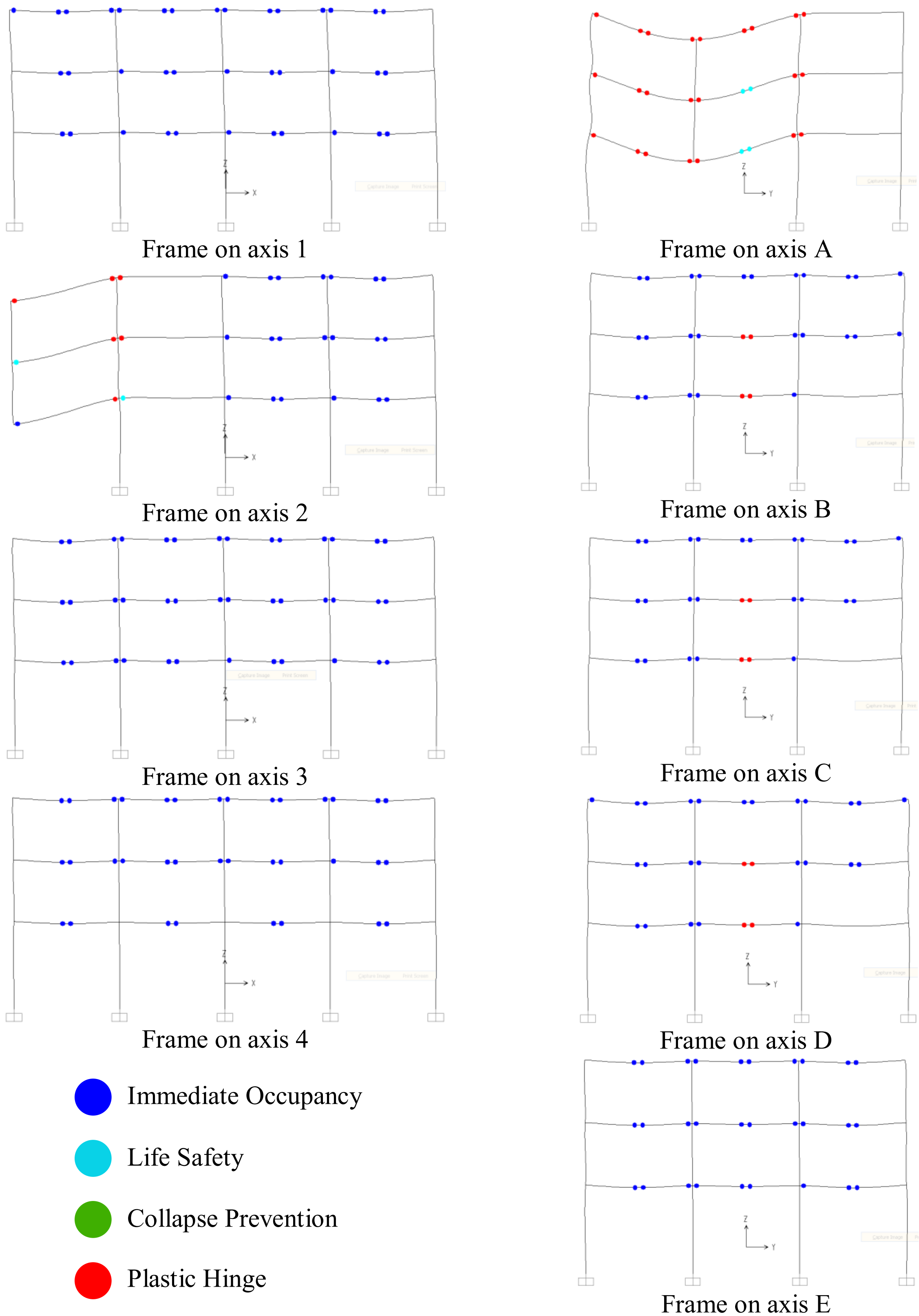


Fig. 5.26 Formulation of Plastic Hinges, Zone 5, ts = 160 mm, Col. Removed in the Short Side

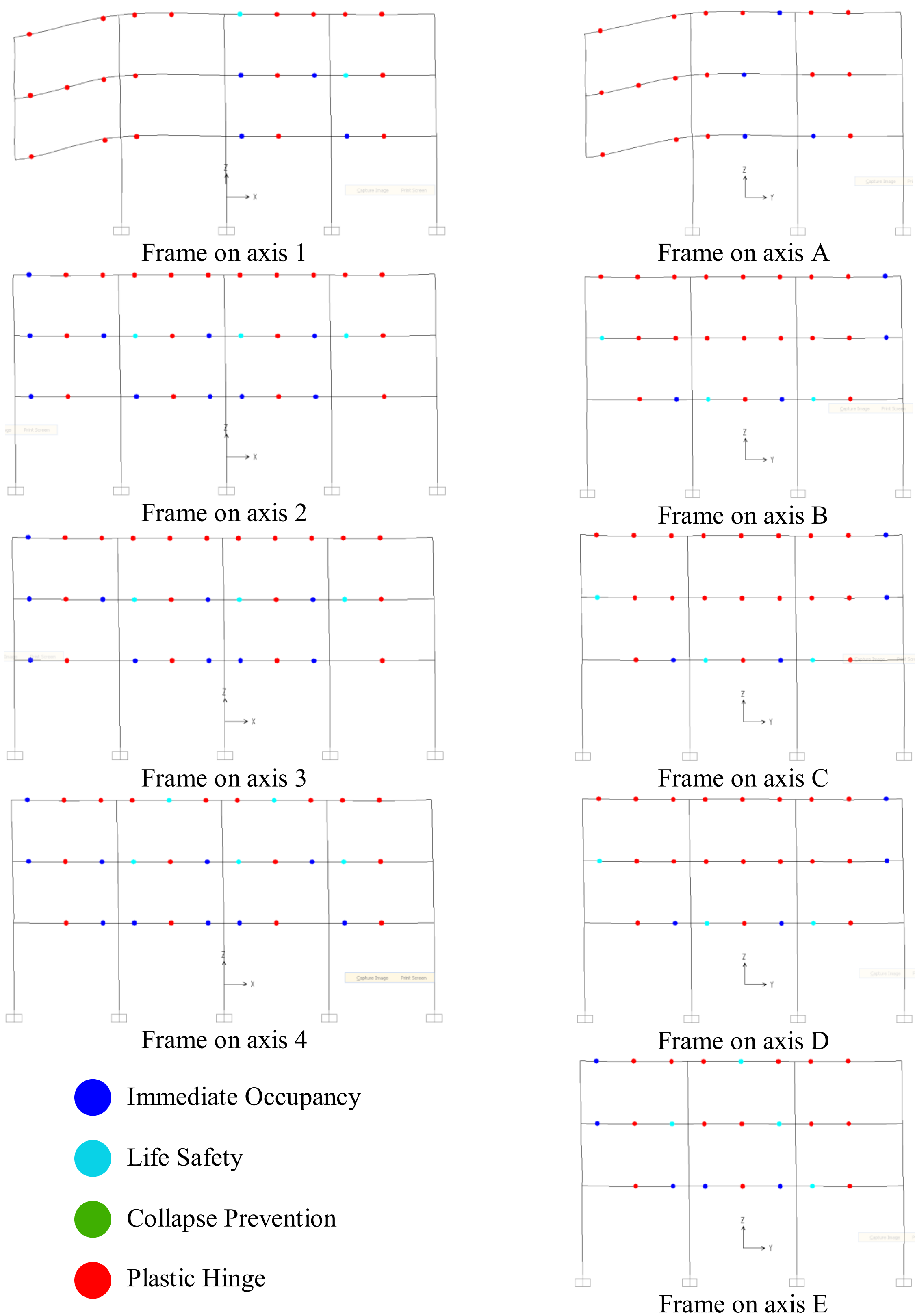


Fig. 5.27 Formulation of Plastic Hinges, Zone 5, No Slab, Col. Removed at the Corner

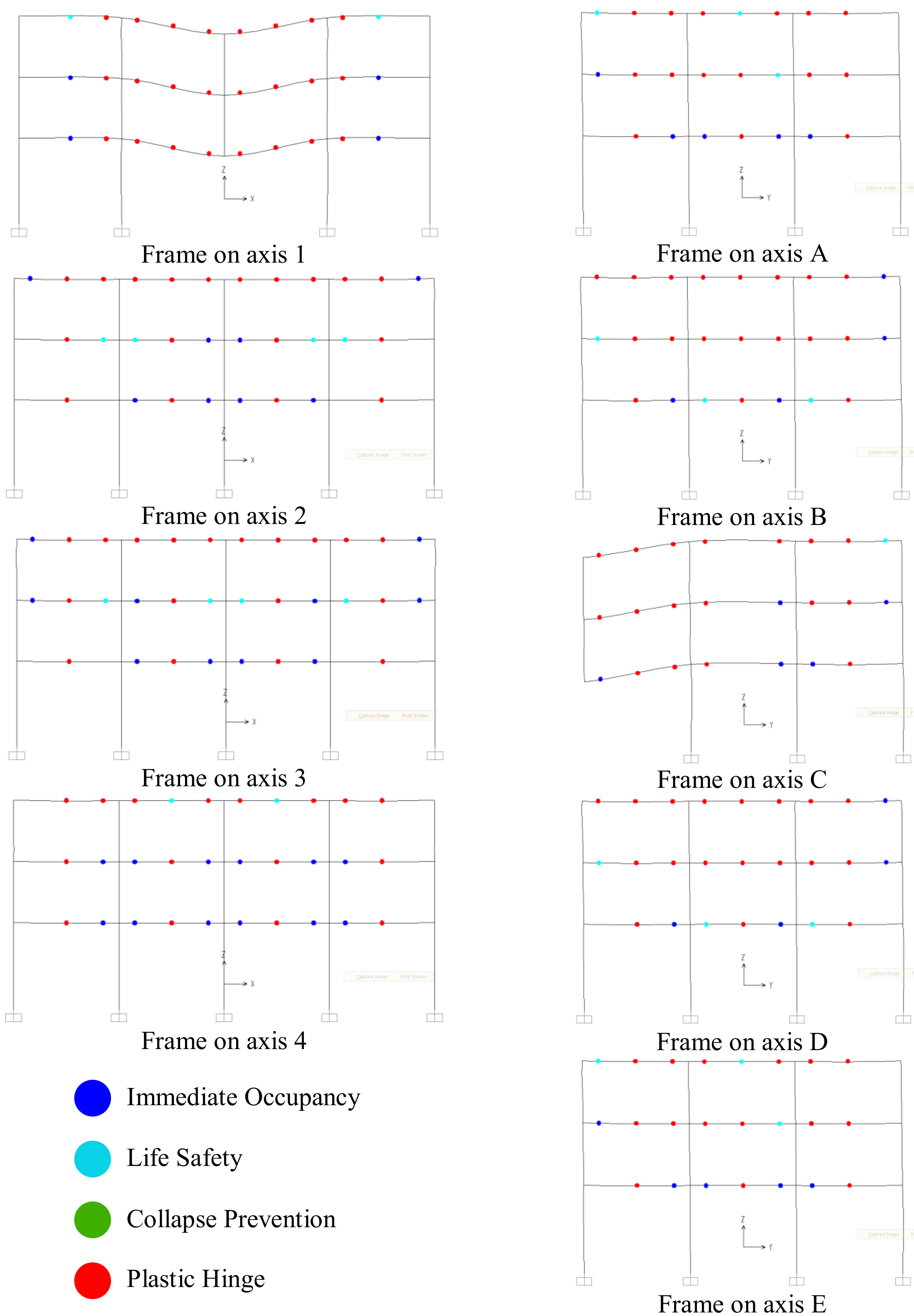
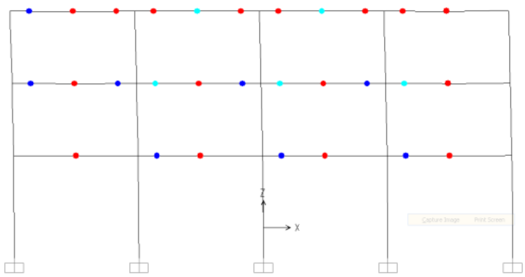
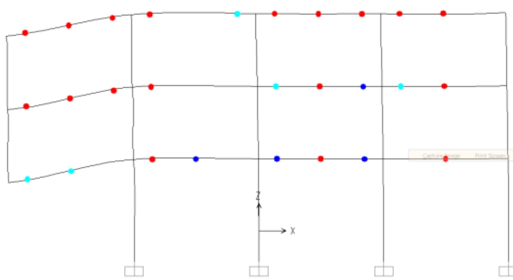


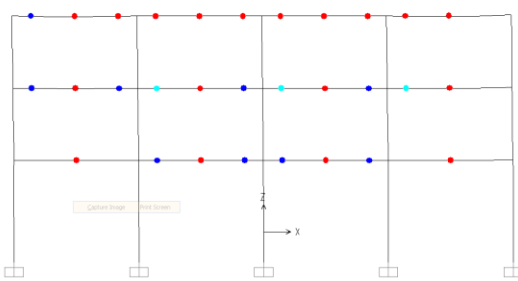
Fig. 5.28 Formulation of Plastic Hinges, Zone 5, No Slab, Col. Removed in the Longer Side



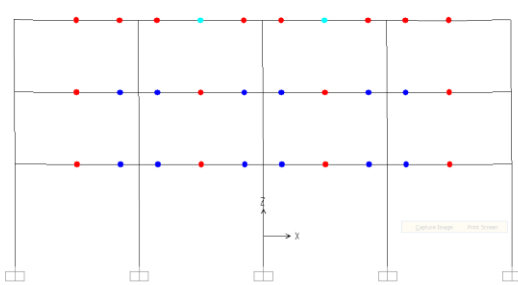
Frame on axis 1



Frame on axis 2



Frame on axis 3



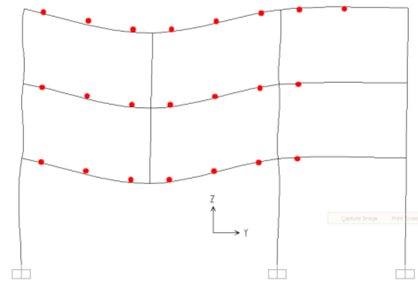
Frame on axis 4

Immediate Occupancy

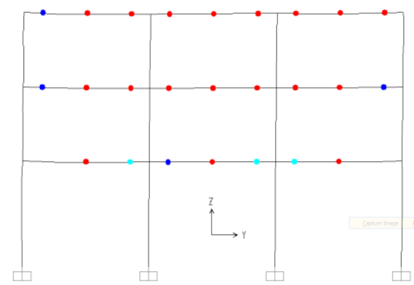
Life Safety

Collapse Prevention

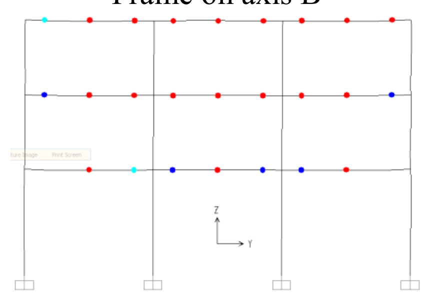
Plastic Hinge



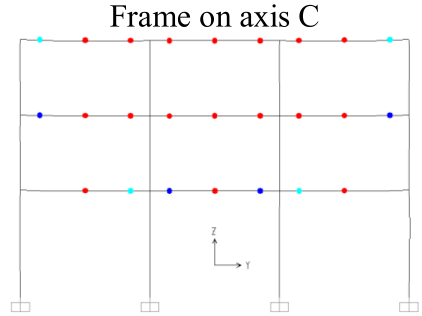
Frame on axis A



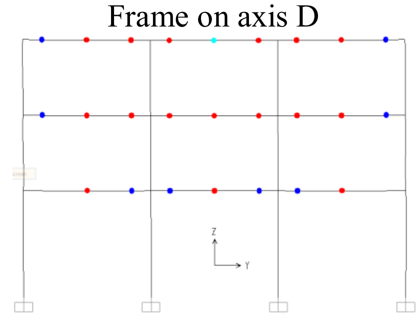
Frame on axis B



Frame on axis C



Frame on axis D



Frame on axis E

Fig. 5.29 Formulation of Plastic Hinges, Zone 5, No Slab, Col. Removed in the Shorter

5.4.1 Summary of Analyses Results

The following table depicts the number of plastic hinges that were formulated and the number of failed beam elements in every scenario.

Table 5.2 No. of Formulated Plastic Hinge and No. of Failed Beam Elements

Scenario No.	Scenario Name	No. of Formulated Plastic Hinge	No. of Failed Beam Elements
1	Zone 1, ts = 10 mm, corner	219	40
2	Zone 1, ts = 10 mm, long	218	40
3	Zone 1, ts = 10 mm, short	223	42
4	Zone 1, ts = 160 mm, corner	165	17
5	Zone 1, ts = 160 mm, long	174	21
6	Zone 1, ts = 160 mm, short	172	19
7	Zone 1, beam, corner	216	42
8	Zone 1, beam, long	219	42
9	Zone 1, beam, short	223	45
10	Zone 3, ts = 10 mm, corner	180	24
11	Zone 3, ts = 10 mm, long	181	23
12	Zone 3, ts = 10 mm, short	188	26
13	Zone 3, ts = 160 mm, corner	25	6
14	Zone 3, ts = 160 mm, long	42	9
15	Zone 3, ts = 16 cm, short	41	9
16	Zone 3, beam, corner	183	27
17	Zone 3, beam, long	195	29
18	Zone 3, beam, short	194	31
19	Zone 5, ts = 10 mm, corner	160	17
20	Zone 5, ts = 10 mm, long	161	19
21	Zone 5, ts = 10 mm, short	169	21
22	Zone 5, ts = 160 mm, corner	21	6
23	Zone 5, ts = 160 mm, long	29	9
24	Zone 5, ts = 160 mm, short	31	9
25	Zone 5, beam, corner	155	19
26	Zone 5, beam, long	159	19
27	Zone 5, beam, short	159	20

The number of failed beam elements due to instability when the column at the corner was removed was 6, when the column at the middle in the longer side was removed was 9, and when the column near the middle in the shorter side was removed was 9.

5.4.2 Relationship between the Seismic zones, Slab thickness, and Removed Column

In summary, the following tables summarize the relationships between the seismic zones, slab thickness/ existing, removed column locations, and the number of formulated plastic hinges for all the 27 scenarios that were conducted.

Table 5.3 No. of plastic hinges versus removed column locations and seismic zones

Seismic zone	Slab case	Corner	Longer side	Shorter side
Zone 1	ts = 10 mm	219	218	223
	ts = 160 mm	165	174	172
	No slab (beam)	216	219	223
Zone 3	ts = 10 mm	180	181	188
	ts = 160 mm	29	42	41
	No slab (beam)	183	195	194
Zone 5	ts = 10 mm	160	161	169
	ts = 160 mm	21	29	31
	No slab (beam)	155	159	159

Table 5.4 No. of plastic hinges versus seismic zones and slab thickness/ existing

Slab case	Removed column	Zone 1	Zone 3	Zone 5
ts = 10 mm	Corner	219	180	160
	Longer side	218	181	161
	Shorter side	223	188	169
ts = 160 mm	Corner	165	25	21
	Longer side	174	42	29
	Shorter side	172	41	31
No slab (beam)	Corner	216	183	155
	Longer side	219	195	159
	Shorter side	223	194	159

Table 5.5 No. of plastic hinges versus slab thickness/ existing and removed column locations

Slab case	Removed column	ts = 10 mm	ts = 160 mm	No slab (beam)
Corner	Zone 1	219	165	216
	Zone 3	180	29	183
	Zone 5	160	25	155
Longer side	Zone 1	218	174	219
	Zone 3	181	42	195
	Zone 5	161	29	159
Shorter side	Zone 1	223	172	223
	Zone 3	188	41	194
	Zone 5	169	31	159

When three plastic hinges are formulated in a beam element, that beam element is accounted asa failed beam. Hence, the researcher summarized the relationships between the seismic zones, slab thickness/ existing, removed column locations, and the number of failed beam elements for all the 27 scenarios that were conducted in the following tables.

Table 5.6 No. of failed beam elements versus removed column locations and seismic zones

Seismic zone	Removed column	Corner	Longer side	Shorter side
Zone 1	ts = 10 mm	40	40	42
	ts = 160 mm	17	21	19
	No slab (beam)	42	42	45
Zone 3	ts = 10 mm	24	23	26
	ts = 160 mm	6	9	9
	No slab (beam)	27	29	31
Zone 5	ts = 10 mm	17	19	21
	ts = 160 mm	6	9	9
	No slab (beam)	19	19	20

Table 5.7 No. of failed beam elements versus seismic zones and slab thickness/ existing

Slab case	Seismic Zone	Zone 1	Zone 3	Zone 5
ts = 10 mm	Corner	40	24	17
	Longer side	40	23	19
	Shorter side	42	26	21
ts = 160 mm	Corner	17	6	6
	Longer side	21	9	9
	Shorter side	19	9	9
No slab (beam)	Corner	42	27	19
	Longer side	42	29	19
	Shorter side	45	31	20

Table 5.8 No. of failed beam elements versus slab thickness/ existing and removed column locations

Removed column	Slab case	ts = 10 mm	ts = 160 mm	No slab (beam)
Corner	Zone 1	40	17	42
	Zone 3	24	6	27
	Zone 5	17	6	19
Longer side	Zone 1	40	21	42
	Zone 3	23	9	29
	Zone 5	19	9	19
Shorter side	Zone 1	42	19	45
	Zone 3	26	9	31
	Zone 5	21	9	20

And in the form of figures, the following figures summarize the relationships between the seismic zones, slab thickness/ existing, removed column locations, and the number of formulated plastic hinges for all the 27 scenarios that were conducted.

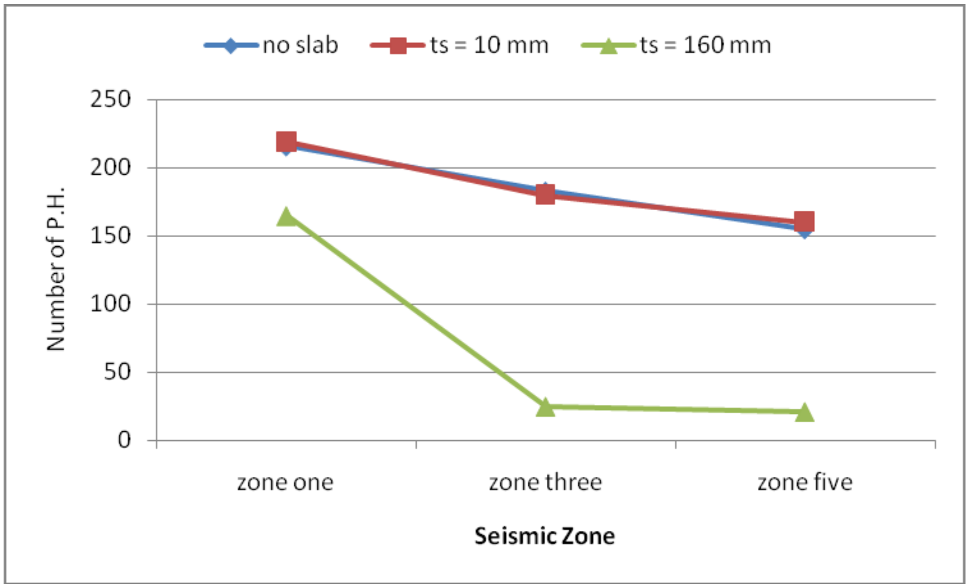


Fig 5.30 Relationship between number of plastic hinge and seismic zone when a column removed at the corner

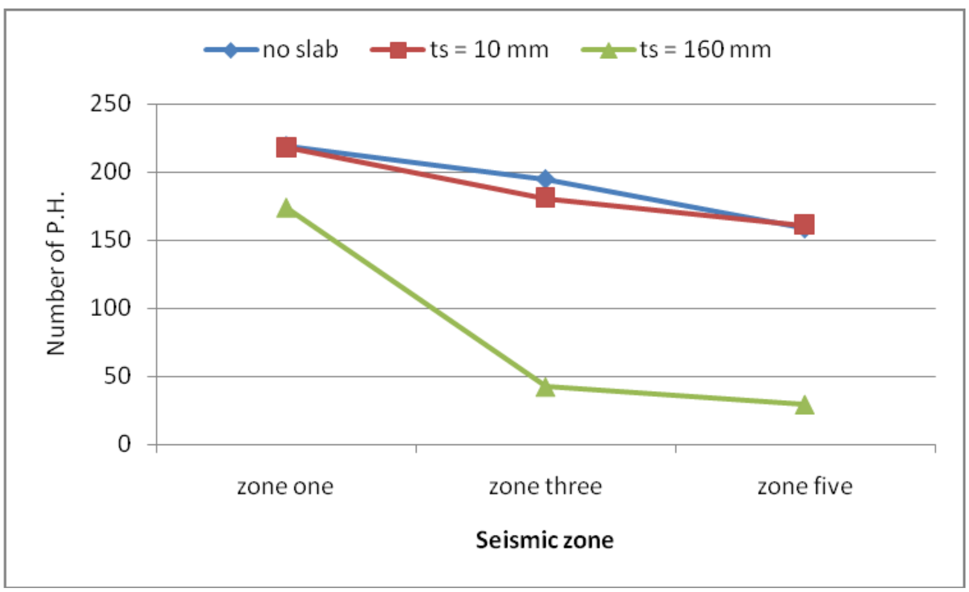


Fig 5.31 Relationship between number of plastic hinge and seismic zone when a column removed in the middle of the longer side.

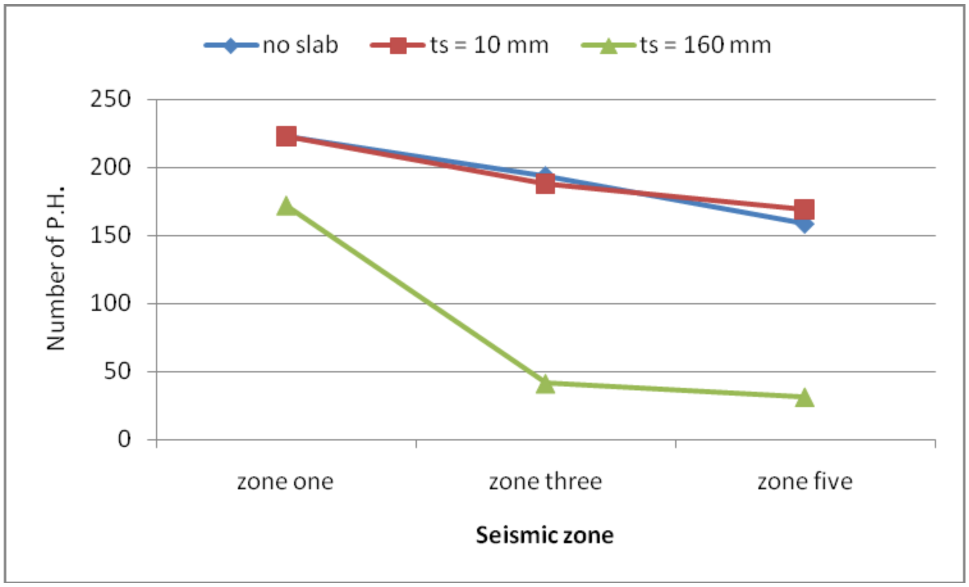


Fig. 5.32 Relationship between number of plastic hinge and seismic zone when a column removed near the middle of the shorter side.

5.4.3 Relationship between the Failed Area to the Total Area

As the building has a dimension of 18.0 m by 24.0 m in plan, hence the area in each floor is 432 square meters, and the total area for the three floors is 1296 square meters. The following figures depict the relationship between the failed areas to the total area for all the 27 scenarios.

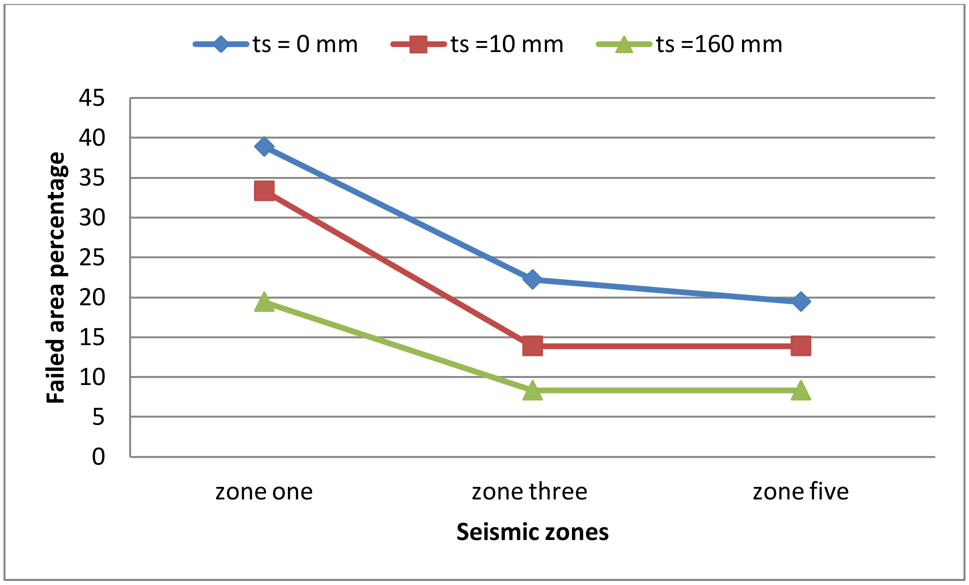


Fig. 5.32 percentage of failed to total area, column removed at the corner.

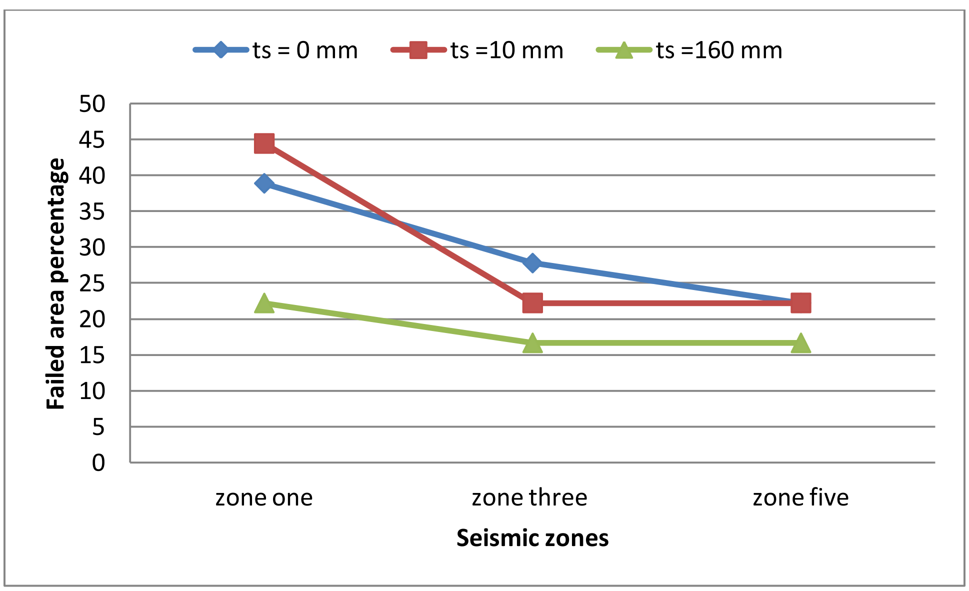


Fig. 5.33 percentage of failed to total area, column removed in the middle of the longer side.

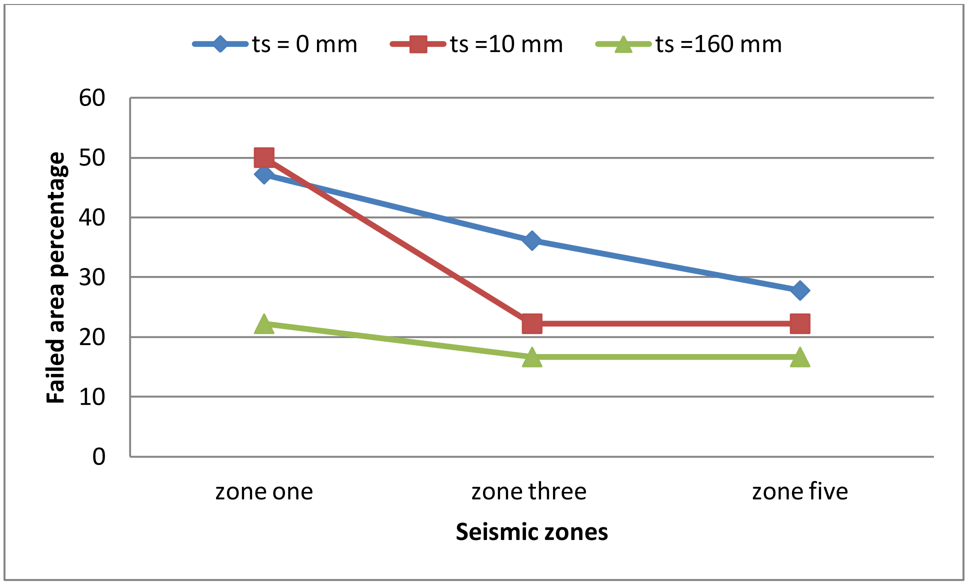


Fig. 5.34 percentage of failed to total area, column removed near the middle of the shorter side.

CHAPTER 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

6.1 Summary

The analysis of multistory multi-bay RC building frame system against progressive collapse is presented. Since design against progressive collapse is not mandated by existing codes and due to the fact that some already existing buildings may become subjected to high risk from nearby accidental explosion, it becomes important to evaluate the performance of such buildings against progressive collapse. Most of existing buildings are designed for seismic resistance according to existing codes of practice. The effect of seismic design level (seismic zone) and reinforcement details on the performance of existing RC frames during progressive collapse analysis is considered in this research. The model building was designed for different seismic zones according to the Egyptian Code for Calculating Loads and Forces on Structures and Buildings (201_2003), and the Egyptian Code of Practice for Design and Construction of Concrete Structures (ECP203-2007).

The factors considered in this research include: the location of failed column, the seismic zone, seismic design level, the reinforcement details, and the contribution of RC slab in resisting progressive collapse. A total of 27 design cases were considered. Three dimensional static nonlinear analyses of all frames were conducted where plastic hinges were allowed to form in all beams at selected locations where the maximum bending moment values were expected. The building performance during progressive collapse was evaluated in terms of the number of formed plastic hinges, the number of failed structural members, and the percentage collapsed of the total built up area of the building.

6.2 Conclusions

From the previous results it could be conducted the following remarks:

1. The structures designed for seismic zone one is the most vulnerable to progressive collapse when removing a load bearing element. The vulnerability for progressive collapse becomes less for structures designed for seismic zone three. The best performance against

progressive collapse was achieved in building designed for the requirement of seismic zone five.

2. Vulnerability for progressive collapse for the three seismic zones; one, three, and five, is similar with respect to the location of removed load bearing element, and that was opposite to what the researcher expected when he began this research. The minimum of formulated plastic hinges and the number of failed beam elements were accounted when a load bearing element was removed at the corner of the building.

3. The number of formulated plastic hinges and the number of failed beam elements that were observed in the case of slab thickness 10 mm is almost the same as the case of no slab case.

4. Slab thickness is a governing factor for controlling the formulation of plastic hinges and the number of failed beam elements. The slab thickness of 160 mm gives the minimum number of formulation of plastic hinges and the number of failed beam elements. The membrane action of the slab participates significantly in bridging the removed columns and transfer load to adjacent columns.

5. The ductility required for seismic design also serves in improving the performance of existing building during progressive collapse due to accidental explosion. This was evident when comparing the numbers of failed elements and percentage of failure for different seismic zone design.

6.3 Future Work

Due to complexity of the progressive collapse phenomenon, the researcher thinks that it still an open area for more research especially using nonlinear dynamic analysis.

Although there is a research that made a field measurements for strain when a progressive collapse occurs, but that research did not take into account the whole beam elements, so it is recommended to carry out more field and experimental measurements for the progressive collapse phenomenon.

The researcher suggested a study for determining the minimum slab thickness that affects for formulation of plastic hinges.

Study techniques for strengthening existing building not designed against progressive collapse.

Research is needed on the value engineering for different techniques for upgrading existing building and also for designing new building to improve their performance against progressive collapse. This would consider the incremental increase in cost during initial design against the cost of upgrading. This is to be conducted within the framework of complete life cost cycle analysis considering the initial cost, the maintenance cost, and the repair cost throughout the life span of the building.

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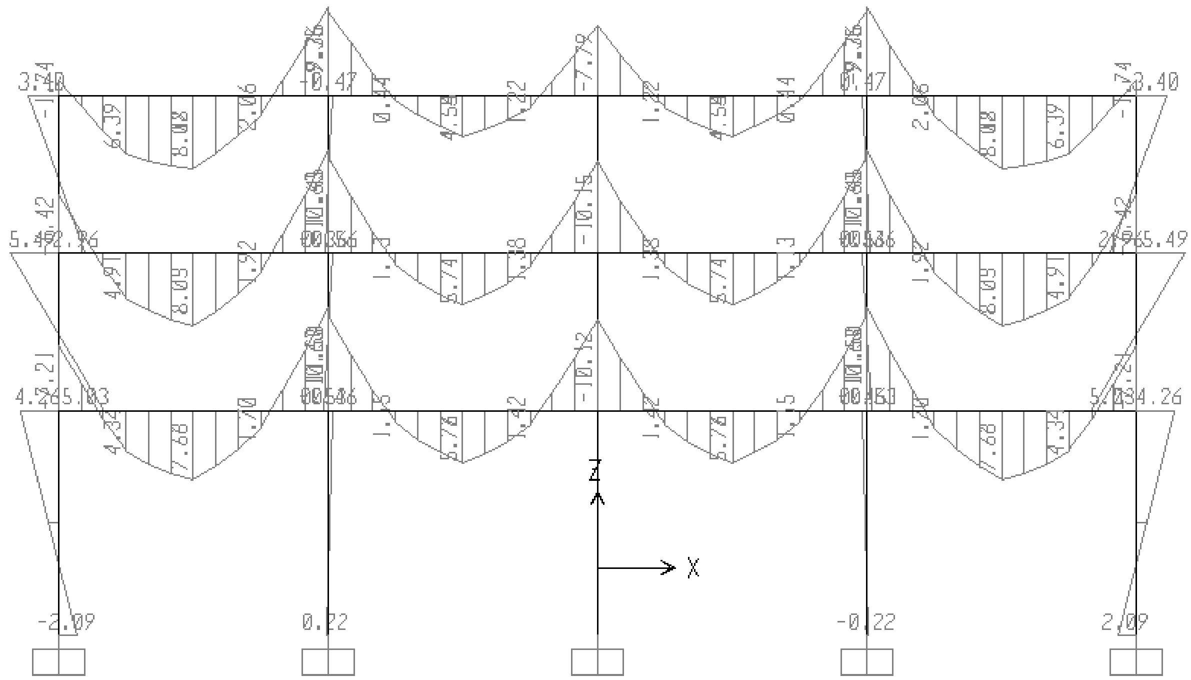
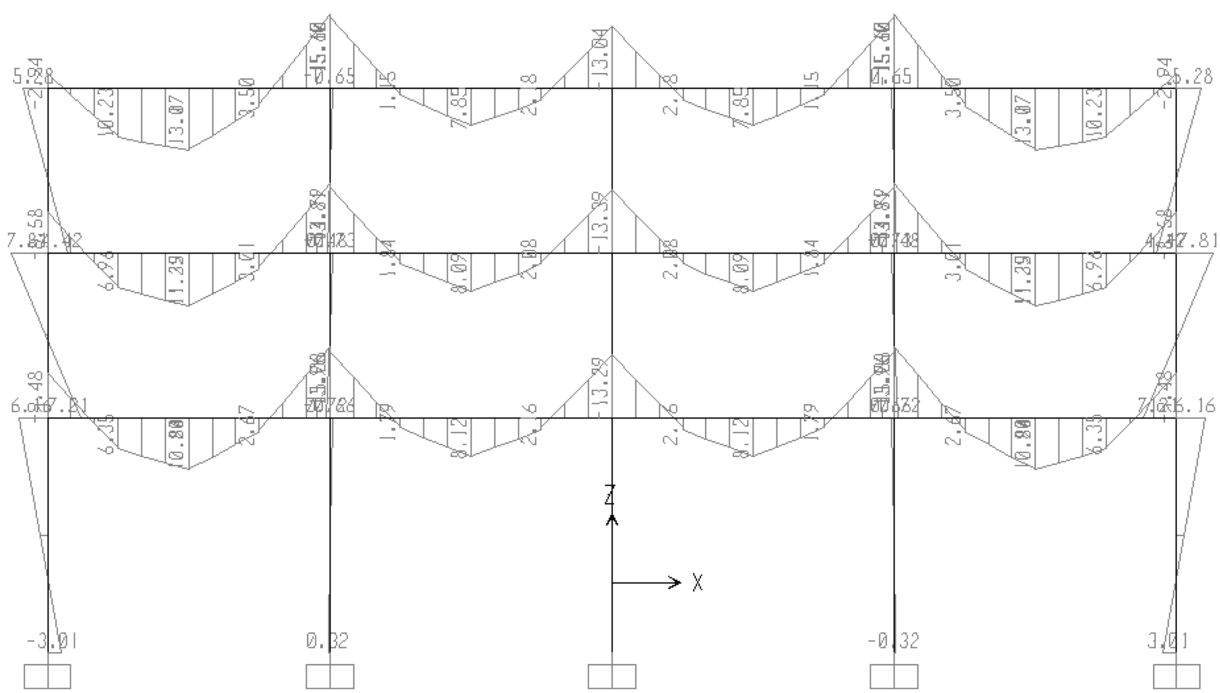
APPENDIX A

BENDING MOMENT DIAGRAMS FOR GRAVITY LOADS,

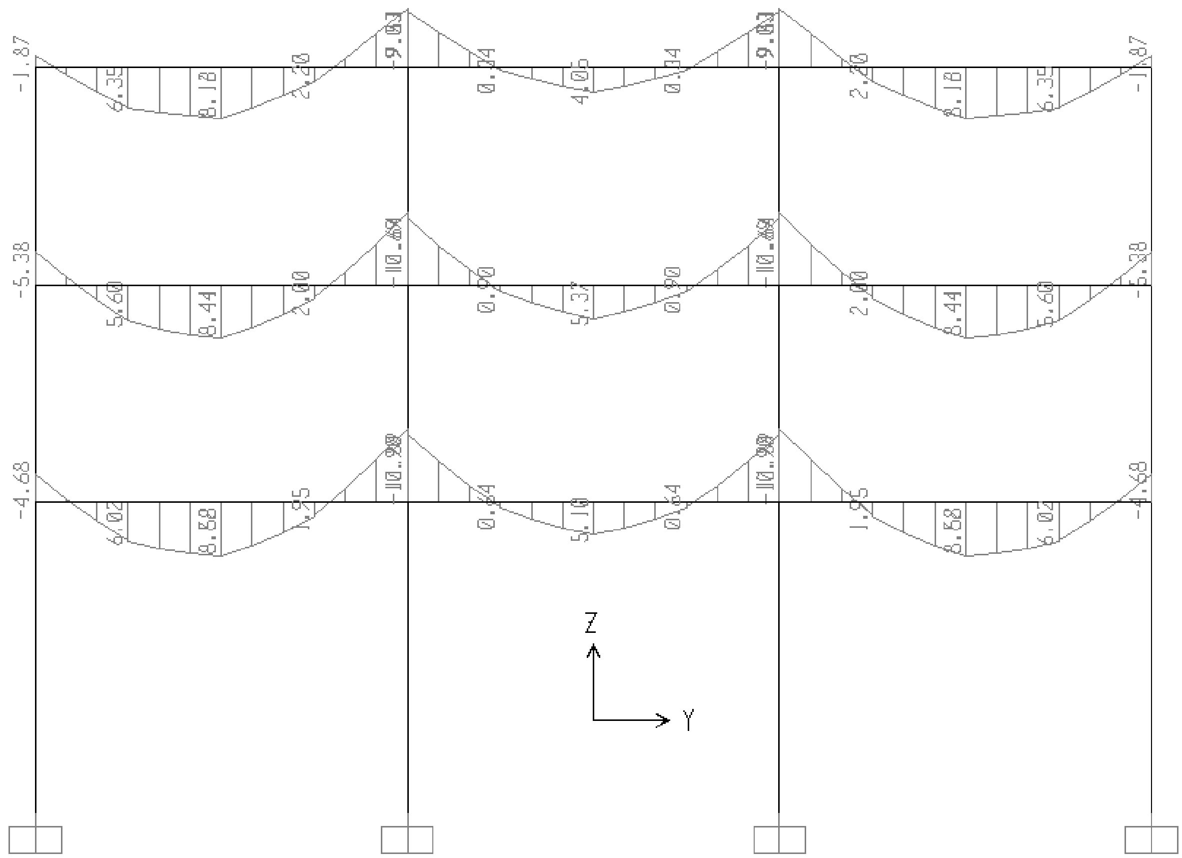
SLAB THICKNESS = 160mm



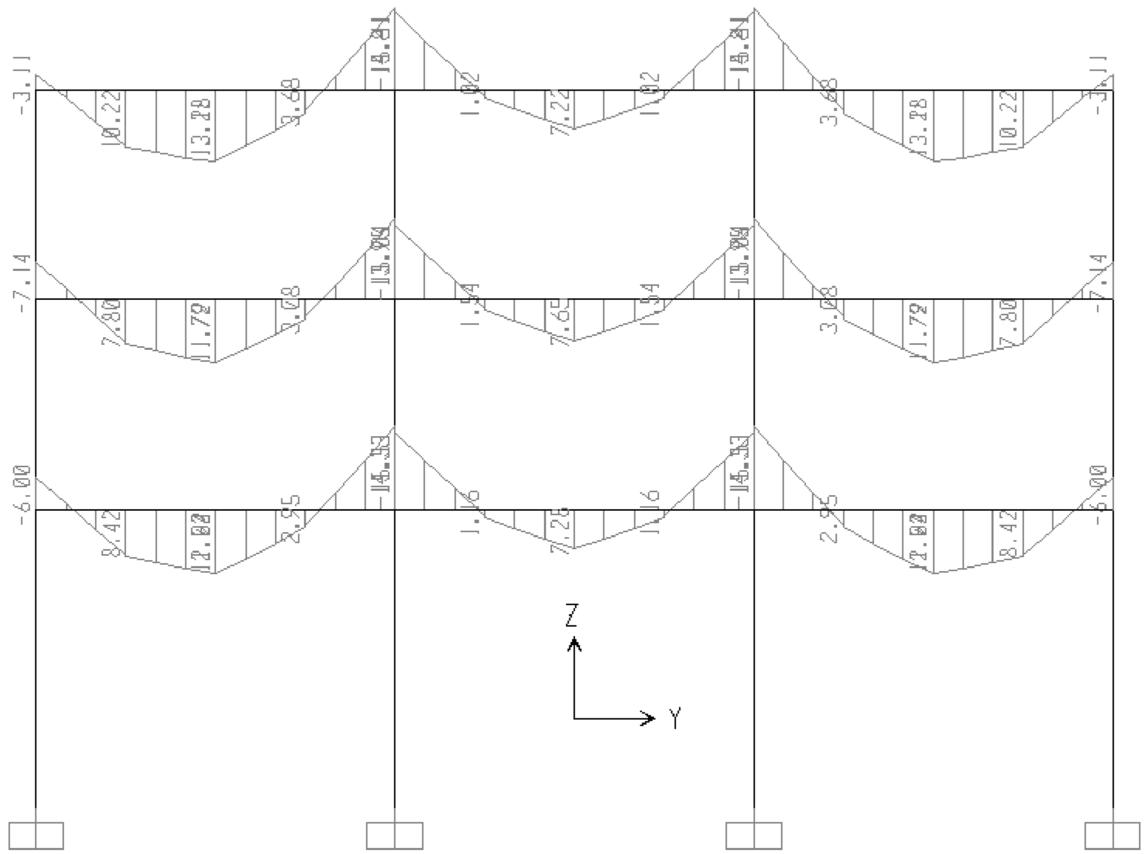
Bending moment diagram, Axes 1and 4



Bending moment diagram, Axes 2and 3



Bending moment diagram, Axes A and E.



Bending moment diagram, Axes B, C, and D.

APPENDIX B

DESIGN AND ELASTIC HORIZONTAL RESPONSE SPECTRUM ANALYSIS

Table B.1 Elastic Horizontal Response Spectrum, zone 1

T	S_e	T	S_e	T	S_e	T	S_e	T	S_e
0	1.472	0.21	3.679	1.1	0.836	2.15	0.428	3.2	0.287
0.01	1.692	0.22	3.679	1.15	0.8	2.2	0.418	3.25	0.283
0.02	1.913	0.23	3.679	1.2	0.766	2.25	0.409	3.3	0.279
0.03	2.134	0.24	3.679	1.25	0.736	2.3	0.4	3.35	0.275
0.04	2.354	0.25	3.679	1.3	0.707	2.35	0.391	3.4	0.27
0.05	2.575	0.3	3.066	1.35	0.681	2.4	0.383	3.45	0.267
0.06	2.796	0.35	2.628	1.4	0.657	2.45	0.375	3.5	0.263
0.07	3.017	0.4	2.299	1.45	0.634	2.5	0.368	3.55	0.259
0.08	3.237	0.45	2.044	1.5	0.613	2.55	0.361	3.6	0.255
0.09	3.458	0.5	1.839	1.55	0.593	2.6	0.354	3.65	0.252
0.1	3.679	0.55	1.672	1.6	0.575	2.65	0.347	3.7	0.249
0.11	3.679	0.6	1.533	1.65	0.557	2.7	0.341	3.75	0.245
0.12	3.679	0.65	1.415	1.7	0.541	2.75	0.334	3.8	0.242
0.13	3.679	0.7	1.314	1.75	0.526	2.8	0.328	3.85	0.239
0.14	3.679	0.75	1.226	1.8	0.511	2.85	0.323	3.9	0.236
0.15	3.679	0.8	1.15	1.85	0.497	2.9	0.317	3.95	0.233
0.16	3.679	0.85	1.082	1.9	0.484	2.95	0.312	4	0.23
0.17	3.679	0.9	1.022	1.95	0.472	3	0.307	4.05	0.227
0.18	3.679	0.95	0.968	2	0.46	3.05	0.302	4.1	0.224
0.19	3.679	1	0.92	2.05	0.449	3.1	0.297	4.15	0.222
0.2	3.679	1.05	0.876	2.1	0.438	3.15	0.292	4.2	0.219

Table B.2Elastic Horizontal Response Spectrum, zone 3

T	S_e	T	S_e	T	S_e	T	S_e	T	S_e
0	2.207	0.21	5.518	1.1	1.254	2.15	0.642	3.2	0.431
0.01	2.538	0.22	5.518	1.15	1.2	2.2	0.627	3.25	0.424
0.02	2.869	0.23	5.518	1.2	1.15	2.25	0.613	3.3	0.418
0.03	3.201	0.24	5.518	1.25	1.104	2.3	0.6	3.35	0.412
0.04	3.532	**0.25**	5.518	1.3	1.061	2.35	0.587	3.4	0.406
0.05	3.863	0.3	4.598	1.35	1.022	2.4	0.575	3.45	0.4
0.06	4.194	0.35	3.942	1.4	0.985	2.45	0.563	3.5	0.394
0.07	4.525	0.4	3.449	1.45	0.951	2.5	0.552	3.55	0.389
0.08	4.856	0.45	3.066	1.5	0.92	2.55	0.541	3.6	0.383
0.09	5.187	0.5	2.759	1.55	0.89	2.6	0.531	3.65	0.378
0.1	5.518	0.55	2.508	1.6	0.862	2.65	0.521	3.7	0.373
0.11	5.518	0.6	2.299	1.65	0.836	2.7	0.511	3.75	0.368
0.12	5.518	0.65	2.122	1.7	0.811	2.75	0.502	3.8	0.363
0.13	5.518	0.7	1.971	1.75	0.788	2.8	0.493	3.85	0.358
0.14	5.518	0.75	1.839	1.8	0.766	2.85	0.484	3.9	0.354
0.15	5.518	0.8	1.724	1.85	0.746	2.9	0.476	3.95	0.349
0.16	5.518	0.85	1.623	1.9	0.726	2.95	0.468	**4**	0.345
0.17	5.518	0.9	1.533	1.95	0.707	3	0.46	4.05	0.341
0.18	5.518	0.95	1.452	2	0.69	3.05	0.452	4.1	0.336
0.19	5.518	1	1.38	2.05	0.673	3.1	0.445	**4.15**	0.332
0.2	5.518	1.05	1.314	2.1	0.657	3.15	0.438	4.2	0.328

Table B.3Elastic Horizontal Response Spectrum, zone 5

T	S_e	T	S_e	T	S_e	T	S_e	T	S_e
0	3.679	0.21	9.197	1.1	2.09	2.15	1.069	3.2	0.719
0.01	4.231	0.22	9.197	1.15	1.999	2.2	1.045	3.25	0.707
0.02	4.782	0.23	9.197	1.2	1.916	2.25	1.022	3.3	0.697
0.03	5.334	0.24	9.197	1.25	1.839	2.3	1	3.35	0.686
0.04	5.886	**0.25**	9.197	1.3	1.769	2.35	0.978	3.4	0.676
0.05	6.438	0.3	7.664	1.35	1.703	2.4	0.958	3.45	0.666
0.06	6.99	0.35	6.569	1.4	1.642	2.45	0.938	3.5	0.657
0.07	7.541	0.4	5.748	1.45	1.586	2.5	0.92	3.55	0.648
0.08	8.093	0.45	5.109	1.5	1.533	2.55	0.902	3.6	0.639
0.09	8.645	0.5	4.598	1.55	1.483	2.6	0.884	3.65	0.63
0.1	9.197	0.55	4.18	1.6	1.437	2.65	0.868	3.7	0.621
0.11	9.197	0.6	3.832	1.65	1.393	2.7	0.852	3.75	0.613
0.12	9.197	0.65	3.537	1.7	1.352	2.75	0.836	3.8	0.605
0.13	9.197	0.7	3.285	1.75	1.314	2.8	0.821	3.85	0.597
0.14	9.197	0.75	3.066	1.8	1.277	2.85	0.807	3.9	0.59
0.15	9.197	0.8	2.874	1.85	1.243	2.9	0.793	3.95	0.582
0.16	9.197	0.85	2.705	1.9	1.21	2.95	0.779	**4**	0.575
0.17	9.197	0.9	2.555	1.95	1.179	3	0.766	4.05	0.568
0.18	9.197	0.95	2.42	2	1.15	3.05	0.754	4.1	0.561
0.19	9.197	1	2.299	2.05	1.122	3.1	0.742	**4.15**	0.554
0.2	9.197	1.05	2.19	2.1	1.095	3.15	0.73	4.2	0.547

Table B.4 Design Horizontal Response Spectrum, zone 1

T	S_d	T	S_d	T	S_d	T	S_d	T	S_d
0	1.472	0.21	0.526	1.1	0.119	2.15	0.061	3.2	0.041
0.01	1.377	0.22	0.526	1.15	0.114	2.2	0.06	3.25	0.04
0.02	1.282	0.23	0.526	1.2	0.109	2.25	0.058	3.3	0.04
0.03	1.188	0.24	0.526	1.25	0.105	2.3	0.057	3.35	0.039
0.04	1.093	**0.25**	0.526	1.3	0.101	2.35	0.056	3.4	0.039
0.05	0.999	0.3	0.438	1.35	0.097	2.4	0.055	3.45	0.038
0.06	0.904	0.35	0.375	1.4	0.094	2.45	0.054	3.5	0.038
0.07	0.809	0.4	0.328	1.45	0.091	2.5	0.053	3.55	0.037
0.08	0.715	0.45	0.292	1.5	0.088	2.55	0.052	3.6	0.036
0.09	0.62	0.5	0.263	1.55	0.085	2.6	0.051	3.65	0.036
0.1	0.526	0.55	0.239	1.6	0.082	2.65	0.05	3.7	0.036
0.11	0.526	0.6	0.219	1.65	0.08	2.7	0.049	3.75	0.035
0.12	0.526	0.65	0.202	1.7	0.077	2.75	0.048	3.8	0.035
0.13	0.526	0.7	0.188	1.75	0.075	2.8	0.047	3.85	0.034
0.14	0.526	0.75	0.175	1.8	0.073	2.85	0.046	3.9	0.034
0.15	0.526	0.8	0.164	1.85	0.071	2.9	0.045	3.95	0.033
0.16	0.526	0.85	0.155	1.9	0.069	2.95	0.045	**4**	0.033
0.17	0.526	0.9	0.146	1.95	0.067	3	0.044	4.05	0.032
0.18	0.526	0.95	0.138	2	0.066	3.05	0.043	4.1	0.032
0.19	0.526	1	0.131	2.05	0.064	3.1	0.042	4.15	0.032
0.2	0.526	1.05	0.125	2.1	0.063	3.15	0.042	4.2	0.031

Table B.5DesignHorizontal Response Spectrum, zone 3

T	S_d	T	S_d	T	S_d	T	S_d	T	S_d
0	2.207	0.21	0.788	1.1	0.179	2.15	0.092	3.2	0.062
0.01	2.065	0.22	0.788	1.15	0.171	2.2	0.09	3.25	0.061
0.02	1.923	0.23	0.788	1.2	0.164	2.25	0.088	3.3	0.06
0.03	1.782	0.24	0.788	1.25	0.158	2.3	0.086	3.35	0.059
0.04	1.64	**0.25**	0.788	1.3	0.152	2.35	0.084	3.4	0.058
0.05	1.498	0.3	0.657	1.35	0.146	2.4	0.082	3.45	0.057
0.06	1.356	0.35	0.563	1.4	0.141	2.45	0.08	3.5	0.056
0.07	1.214	0.4	0.493	1.45	0.136	2.5	0.079	3.55	0.056
0.08	1.072	0.45	0.438	1.5	0.131	2.55	0.077	3.6	0.055
0.09	0.93	0.5	0.394	1.55	0.127	2.6	0.076	3.65	0.054
0.1	0.788	0.55	0.358	1.6	0.123	2.65	0.074	3.7	0.053
0.11	0.788	0.6	0.328	1.65	0.119	2.7	0.073	3.75	0.053
0.12	0.788	0.65	0.303	1.7	0.116	2.75	0.072	3.8	0.052
0.13	0.788	0.7	0.282	1.75	0.113	2.8	0.07	3.85	0.051
0.14	0.788	0.75	0.263	1.8	0.109	2.85	0.069	3.9	0.051
0.15	0.788	0.8	0.246	1.85	0.107	2.9	0.068	3.95	0.05
0.16	0.788	0.85	0.232	1.9	0.104	2.95	0.067	**4**	0.049
0.17	0.788	0.9	0.219	1.95	0.101	3	0.066	4.05	0.049
0.18	0.788	0.95	0.207	2	0.099	3.05	0.065	4.1	0.048
0.19	0.788	1	0.197	2.05	0.096	3.1	0.064	4.15	0.047
0.2	0.788	1.05	0.188	2.1	0.094	3.15	0.063	4.2	0.047

Table B.6Design Horizontal Response Spectrum, zone 5

T	S_d	T	S_d	T	S_d	T	S_d	T	S_d
0	3.679	0.21	1.314	1.1	0.299	2.15	0.153	3.2	0.103
0.01	3.442	0.22	1.314	1.15	0.286	2.2	0.149	3.25	0.101
0.02	3.206	0.23	1.314	1.2	0.274	2.25	0.146	3.3	0.1
0.03	2.969	0.24	1.314	1.25	0.263	2.3	0.143	3.35	0.098
0.04	2.733	**0.25**	1.314	1.3	0.253	2.35	0.14	3.4	0.097
0.05	2.496	0.3	1.095	1.35	0.243	2.4	0.137	3.45	0.095
0.06	2.26	0.35	0.938	1.4	0.235	2.45	0.134	3.5	0.094
0.07	2.023	0.4	0.821	1.45	0.227	2.5	0.131	3.55	0.093
0.08	1.787	0.45	0.73	1.5	0.219	2.55	0.129	3.6	0.091
0.09	1.55	0.5	0.657	1.55	0.212	2.6	0.126	3.65	0.09
0.1	1.314	0.55	0.597	1.6	0.205	2.65	0.124	3.7	0.089
0.11	1.314	0.6	0.547	1.65	0.199	2.7	0.122	3.75	0.088
0.12	1.314	0.65	0.505	1.7	0.193	2.75	0.119	3.8	0.086
0.13	1.314	0.7	0.469	1.75	0.188	2.8	0.117	3.85	0.085
0.14	1.314	0.75	0.438	1.8	0.182	2.85	0.115	3.9	0.084
0.15	1.314	0.8	0.411	1.85	0.178	2.9	0.113	3.95	0.083
0.16	1.314	0.85	0.386	1.9	0.173	2.95	0.111	**4**	0.082
0.17	1.314	0.9	0.365	1.95	0.168	3	0.109	4.05	0.081
0.18	1.314	0.95	0.346	2	0.164	3.05	0.108	4.1	0.08
0.19	1.314	1	0.328	2.05	0.16	3.1	0.106	4.15	0.079
0.2	1.314	1.05	0.313	2.1	0.156	3.15	0.104	4.2	0.078

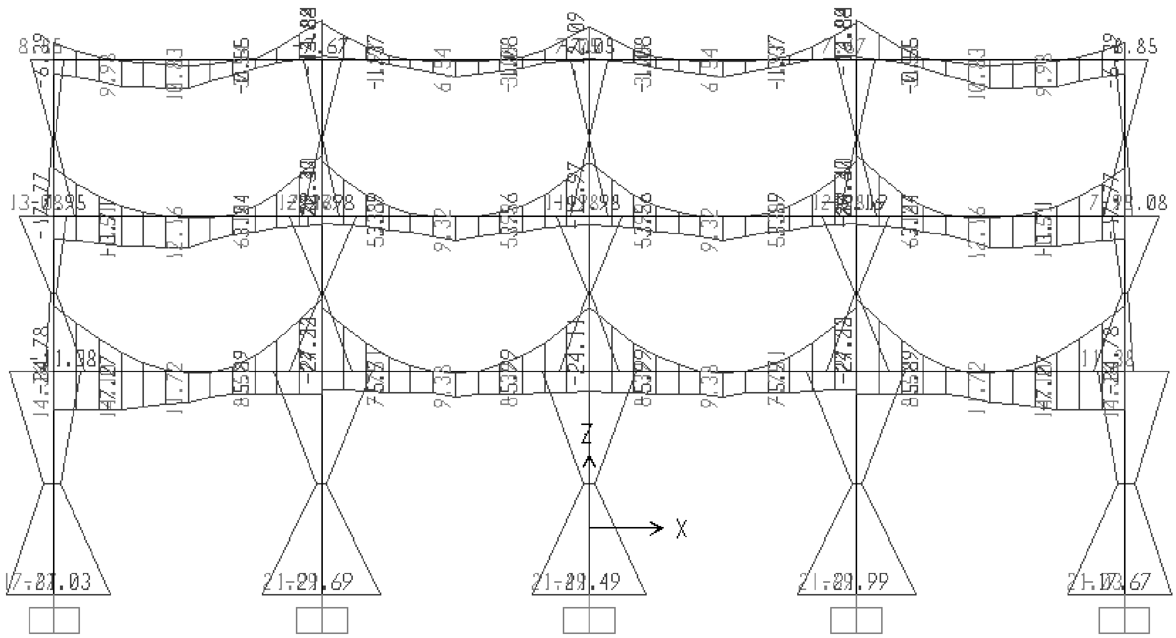


Fig B.1 Envelope of bending moment diagram for zone 1, slab thick. = 10 mm, Axes 1 and 4

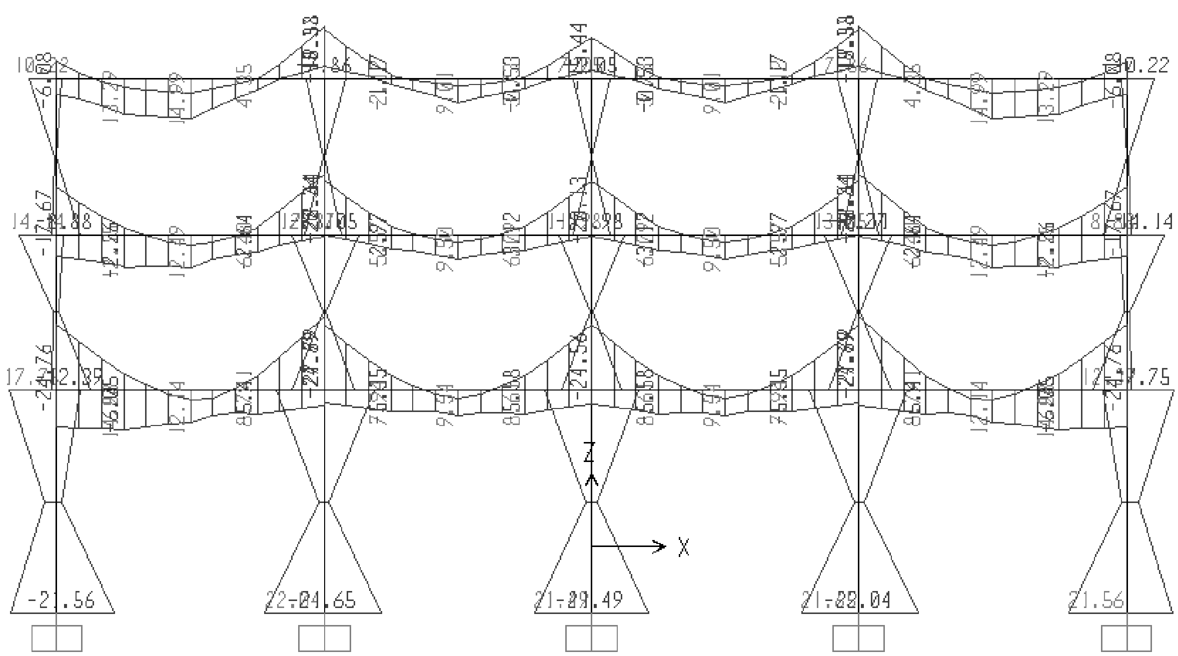


Fig B.2 Envelope of bending moment diagram for zone 1, slab thick. = 10 mm, Axes 2 and 3

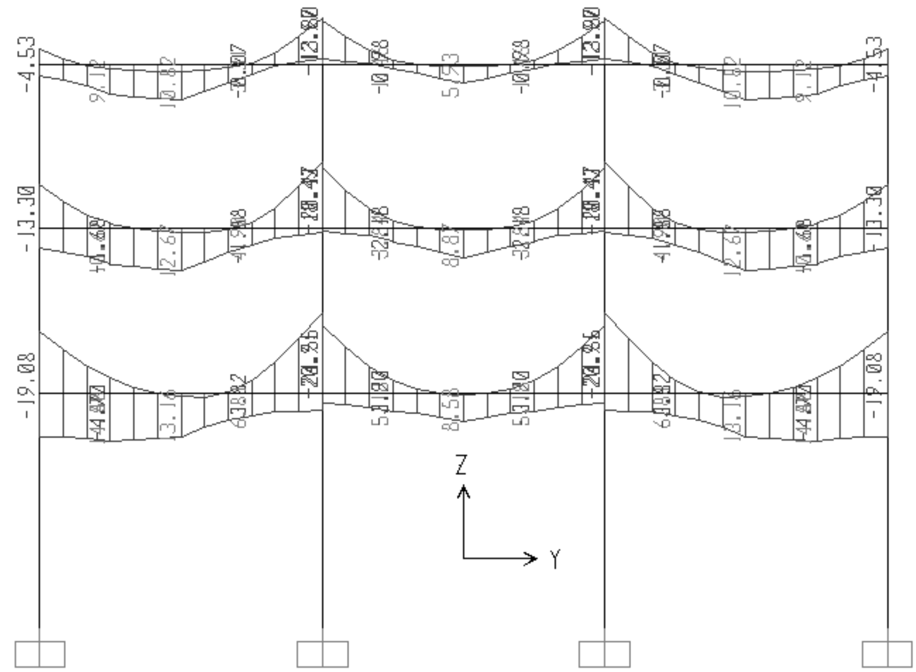


Fig B.3 Envelope of bending moment diagram for zone 1, slab thick. = 10 mm, Axes A and E

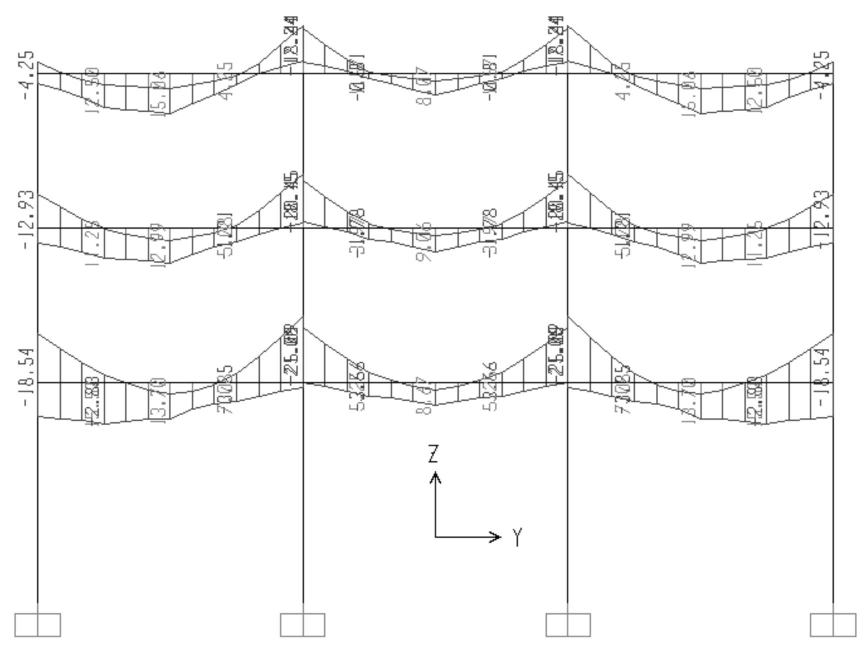


Fig B.4 Envelope of bending moment diagram for zone 1, slab thick. = 10 mm, Axes B, C, and D

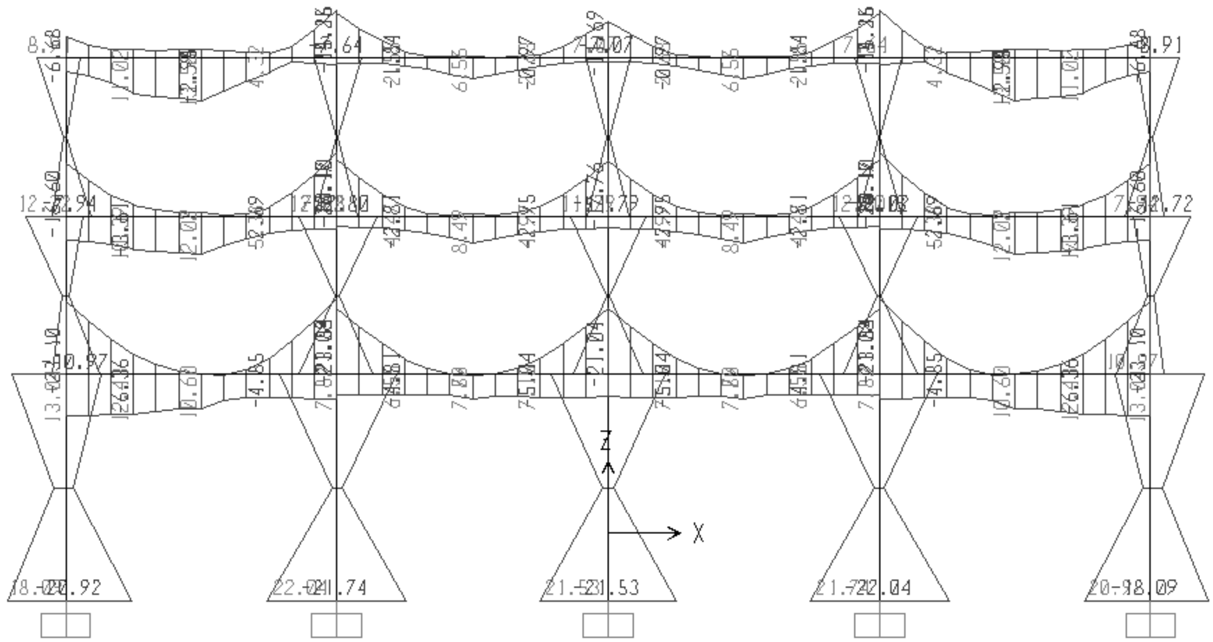


Fig B.5 Envelope of bending moment diagram for zone 1, slab thick. = 160 mm, Axes 1 and 4

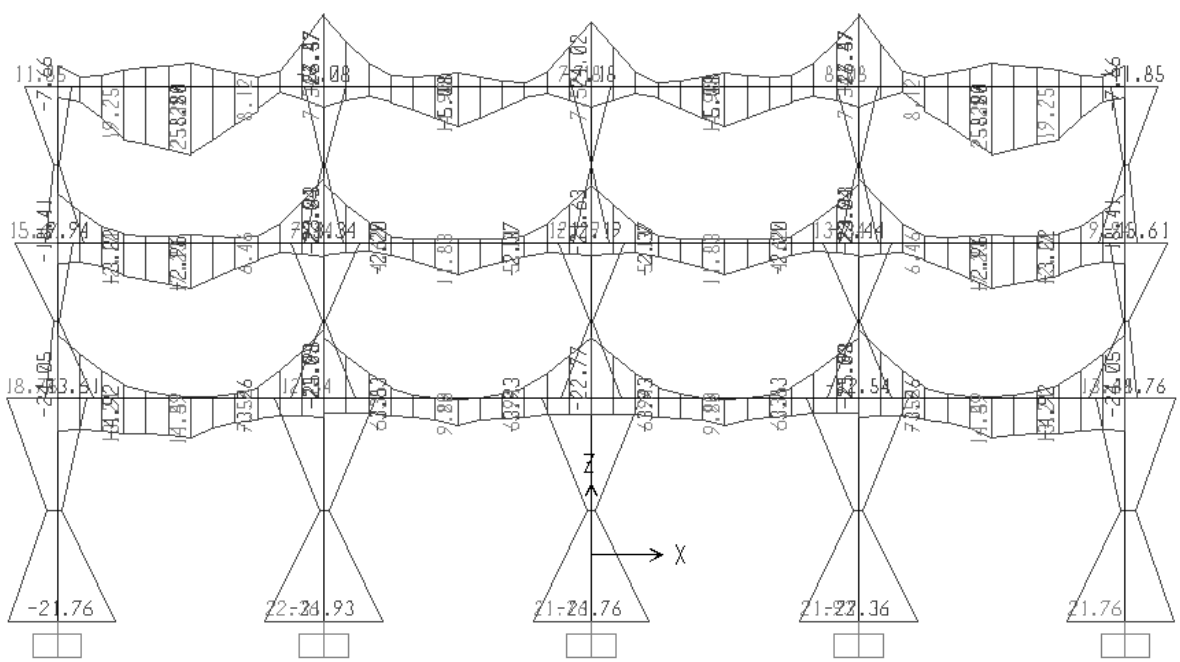


Fig B.6 Envelope of bending moment diagram for zone 1, slab thick. = 160 mm, Axes 2 and 3

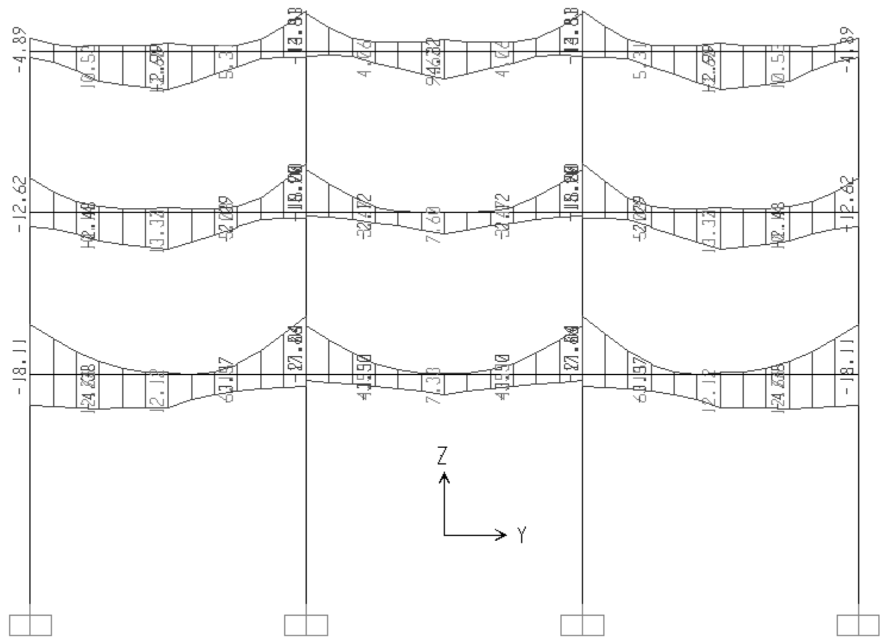


Fig B.7 Envelope of bending moment diagram for zone 1, slab thick. = 160 mm, Axes A and E

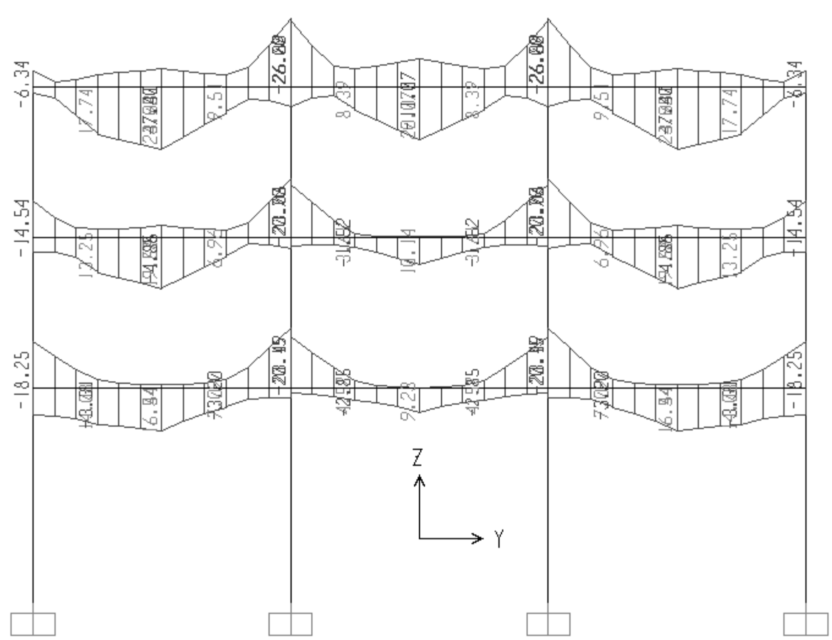


Fig B.8 Envelope of bending moment diagram for zone 1, slab thick. = 160 mm, Axes B, C, and D

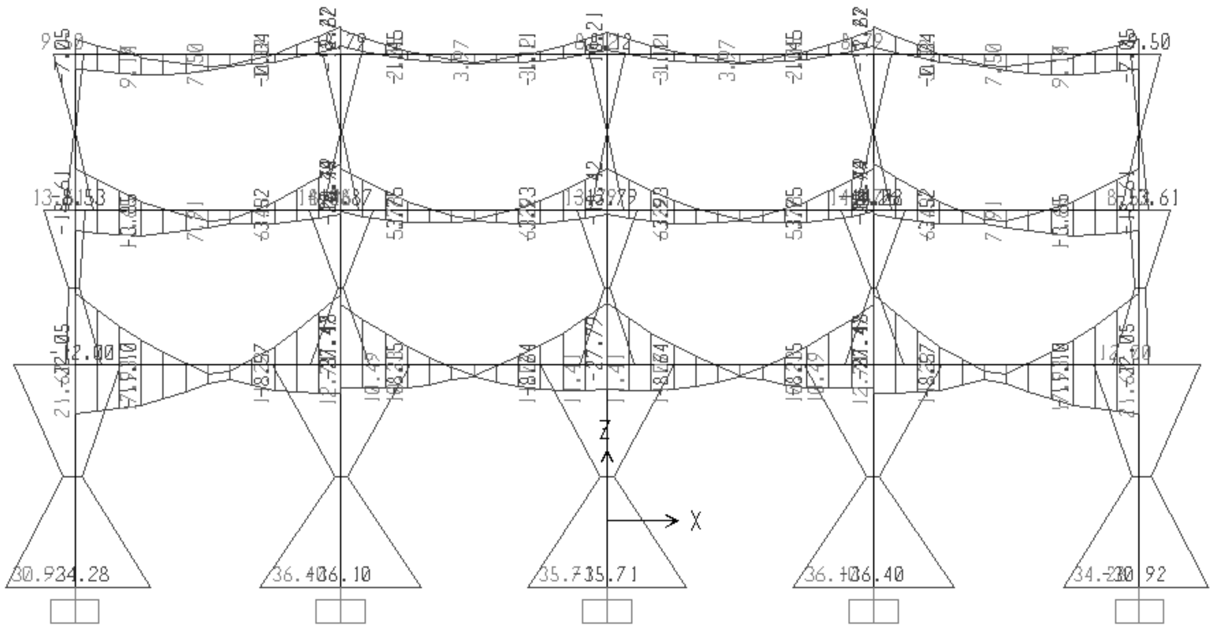


Fig B.9 Envelope of bending moment diagram for zone 3, slab thick. = 10 mm, Axes 1 and 4

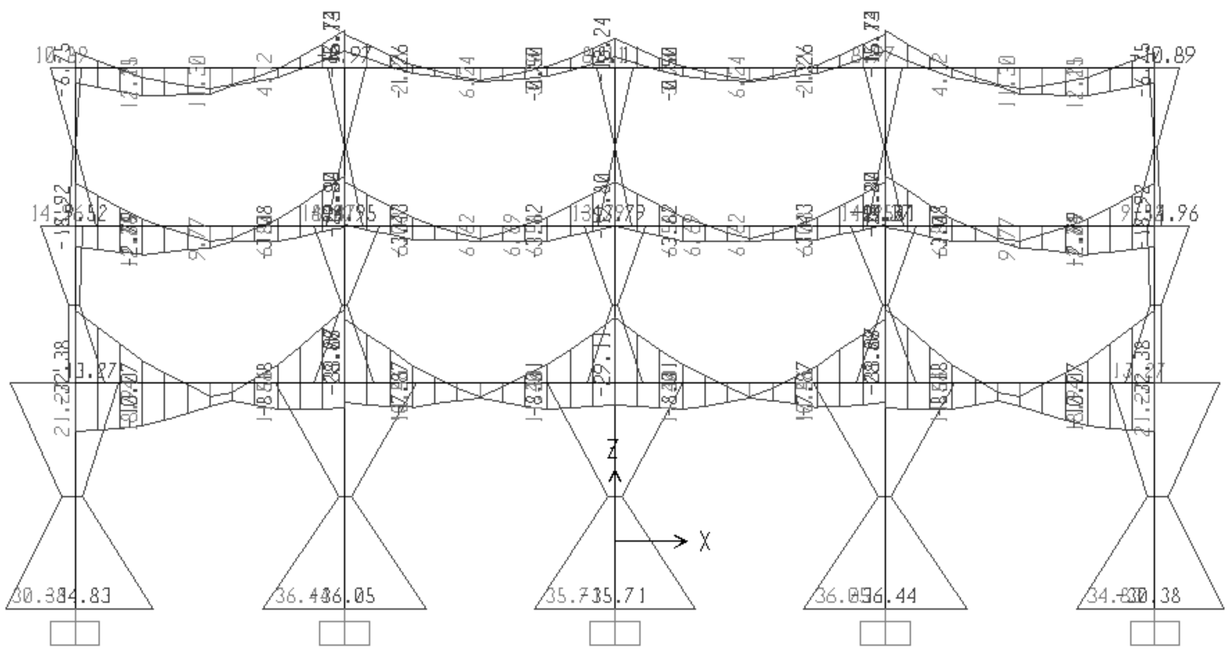


Fig B.10 Envelope of bending moment diagram for zone 3, slab thick. = 10 mm, Axes 2 and 3

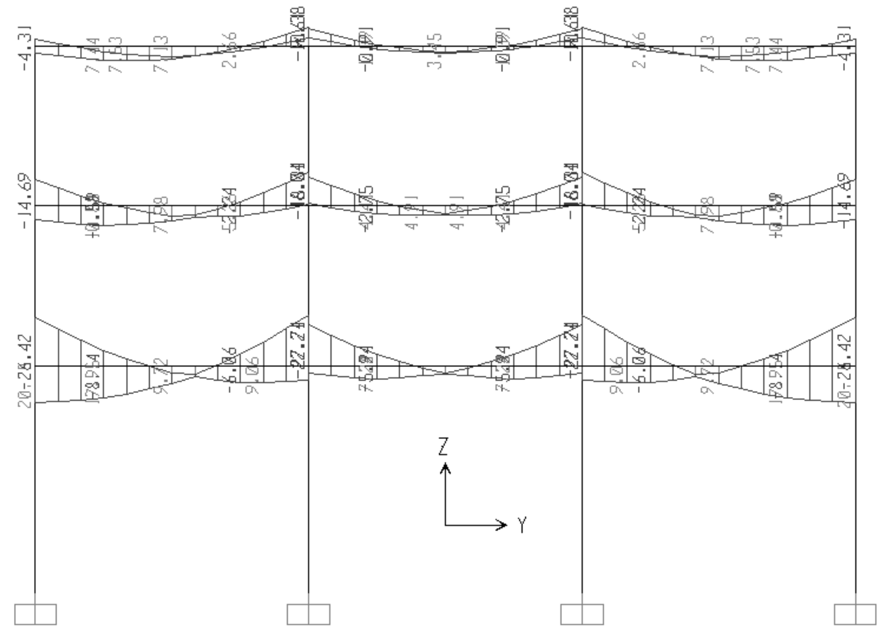


Fig B.11 Envelope of bending moment diagram for zone 3, slab thick. = 10 mm, Axes A and E

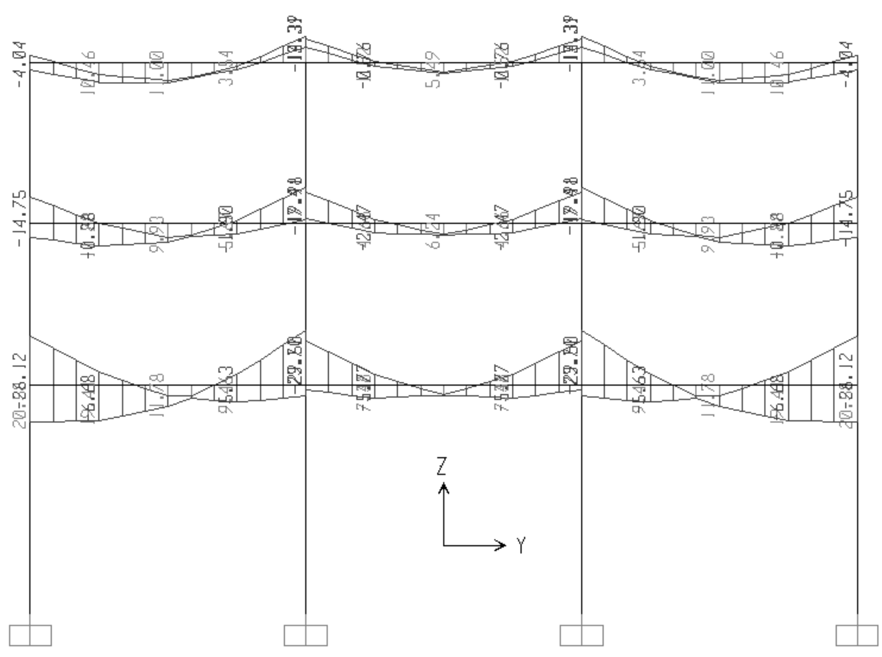


Fig B.12 Envelope of bending moment diagram for zone 3, slab thick. = 10 mm, Axes B, C, and D

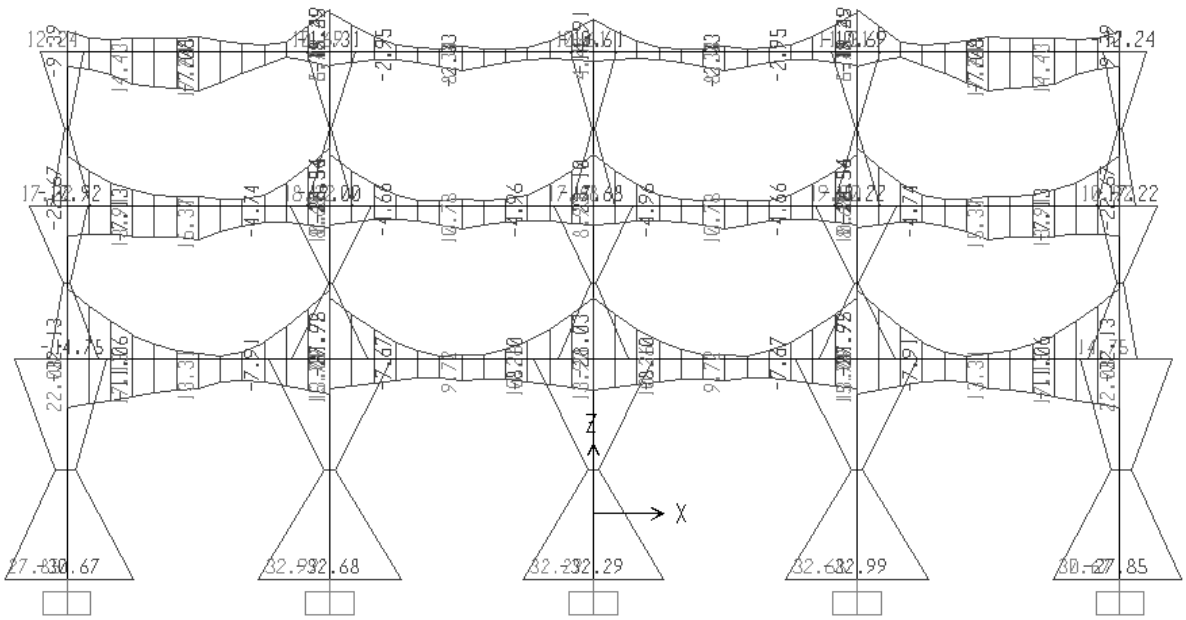


Fig B.13 Envelope of bending moment diagram for zone 3, slab thick. = 160 mm, Axes 1 and 4

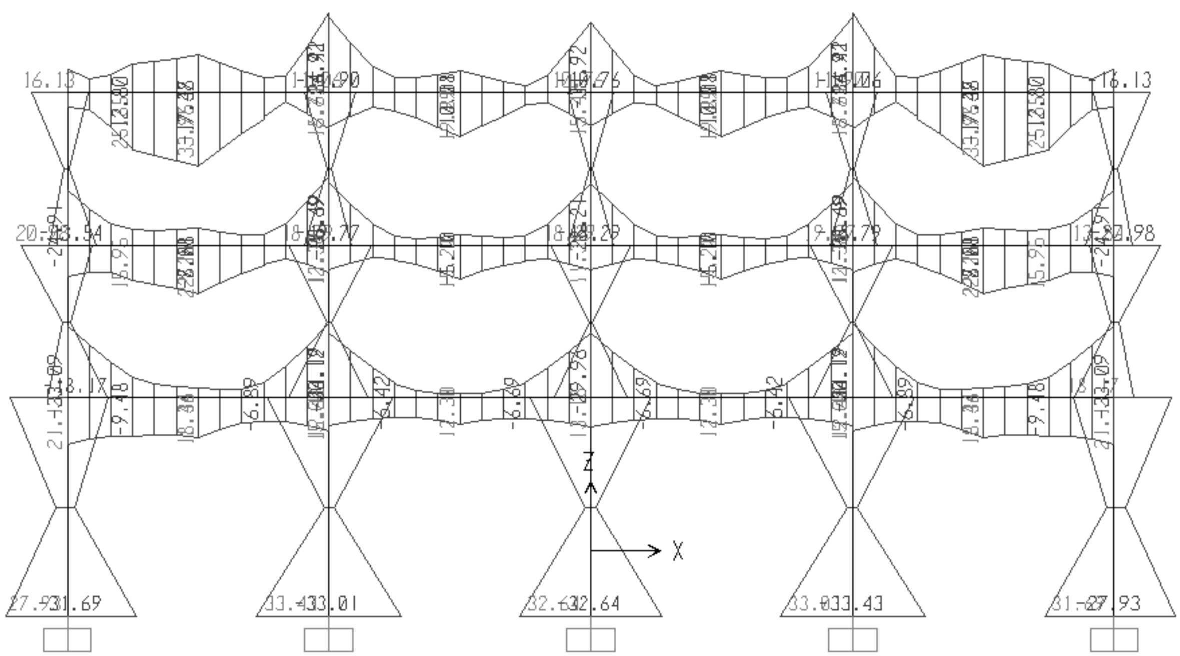


Fig B.14 Envelope of bending moment diagram for zone 3, slab thick. = 160 mm, Axes 2 and 3

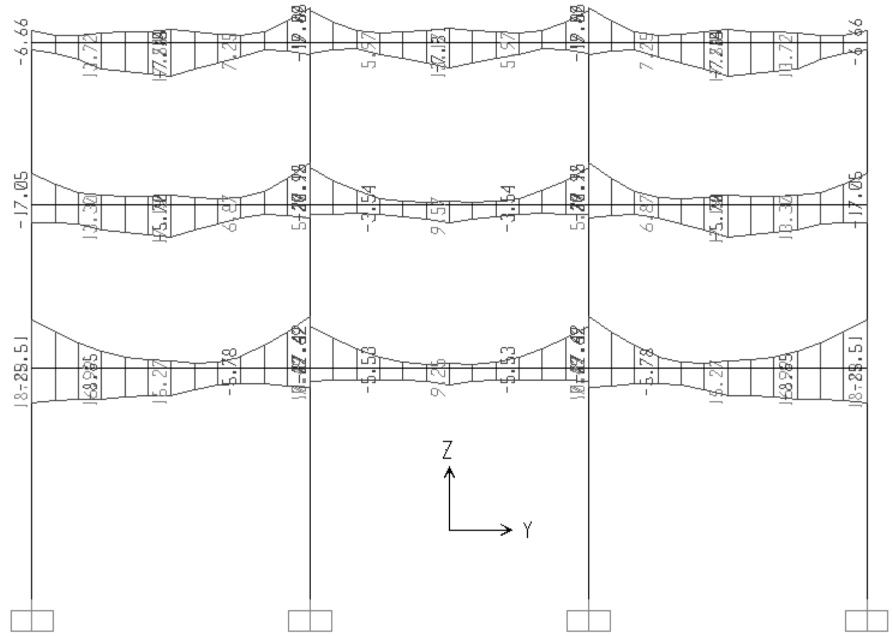


Fig B.15 Envelope of bending moment diagram for zone 3, slab thick. = 160 mm, Axes A and E

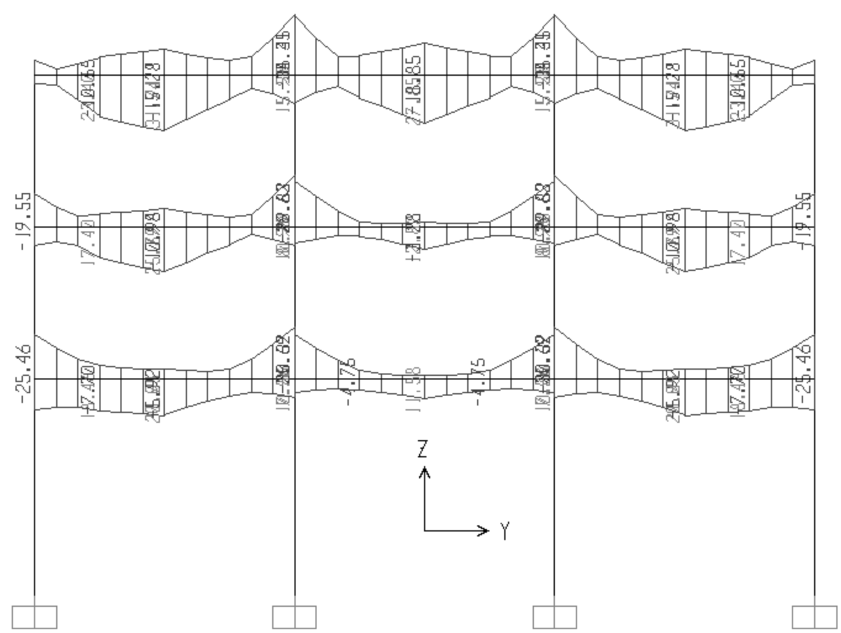


Fig B.16 Envelope of bending moment diagram for zone 3, slab thick. = 160 mm, Axes B, C, and D

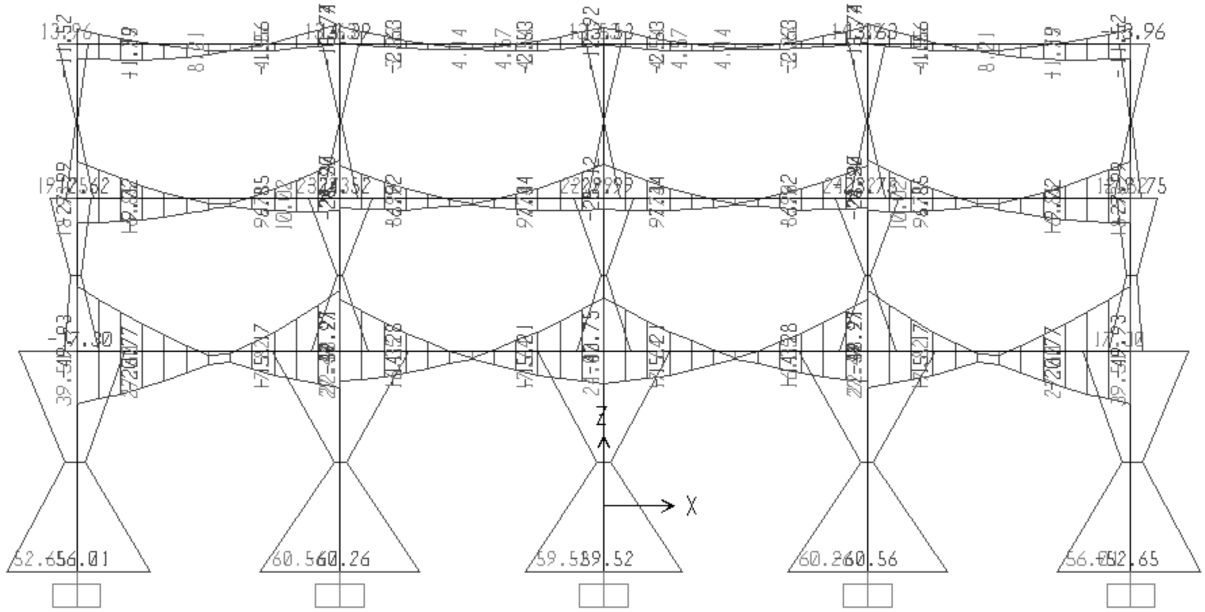


Fig B.17 Envelope of bending moment diagram for zone 5, slab thick. = 10 mm, Axes 1 and 4

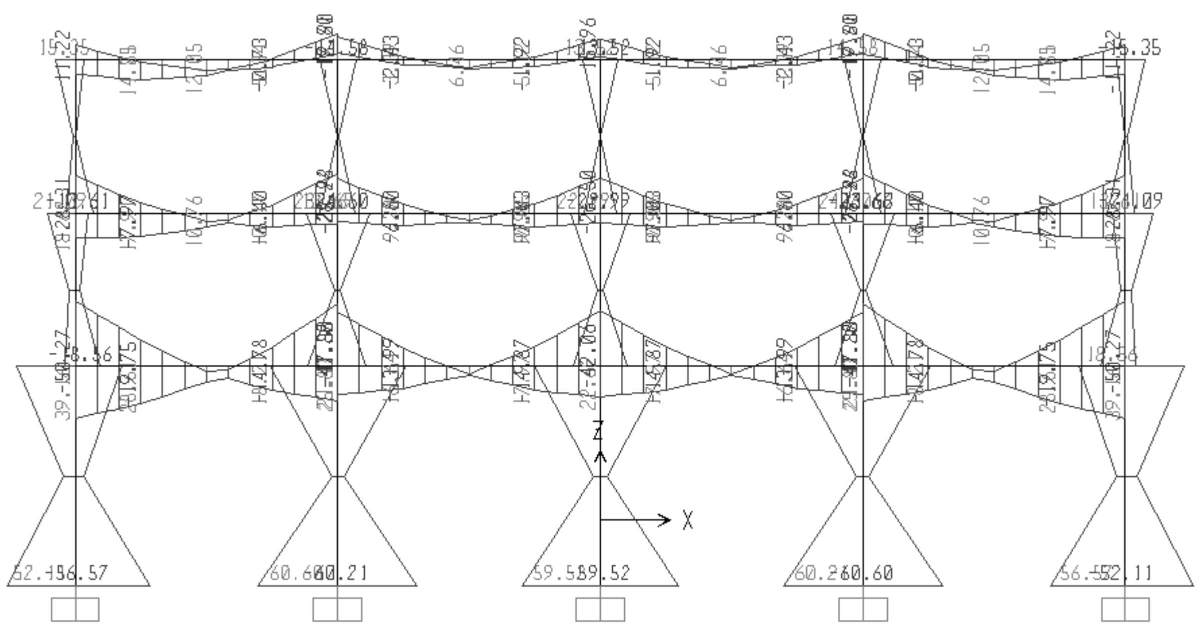


Fig B.18 Envelope of bending moment diagram for zone 5, slab thick. = 10 mm, Axes 2 and 3

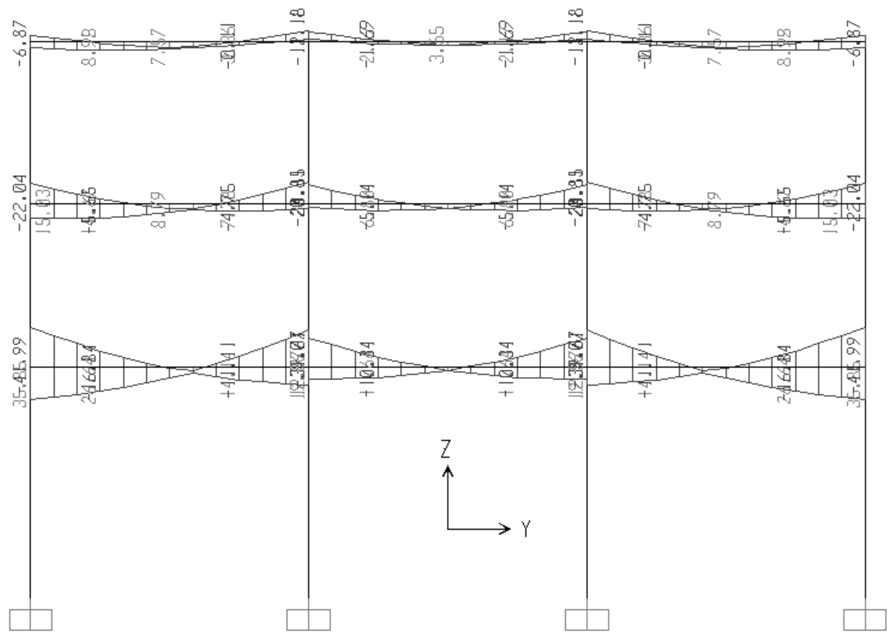


Fig B.19 Envelope of bending moment diagram for zone 5, slab thick. = 10 mm, Axes A and E

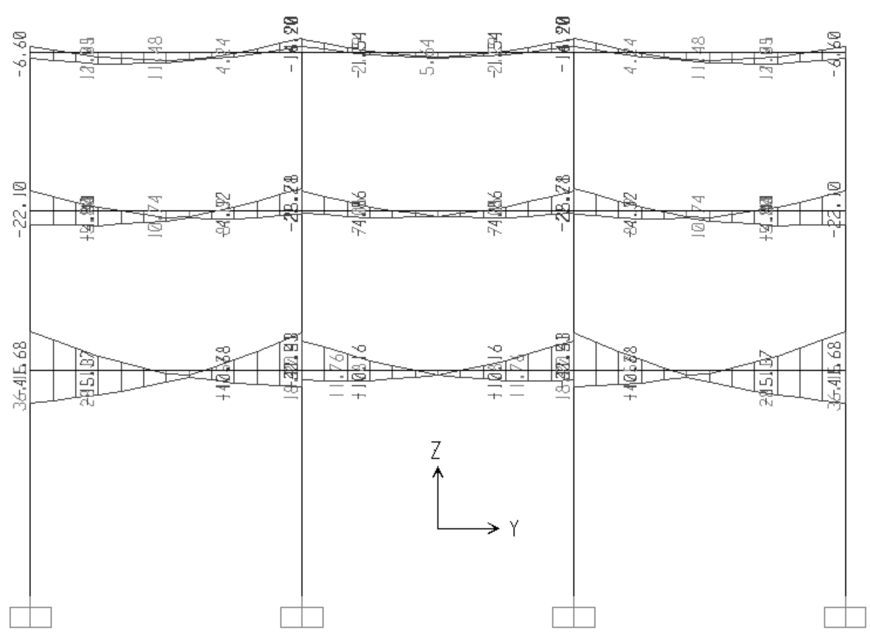


Fig B.20 Envelope of bending moment diagram for zone 5, slab thick. = 10 mm, Axes B, C, and D

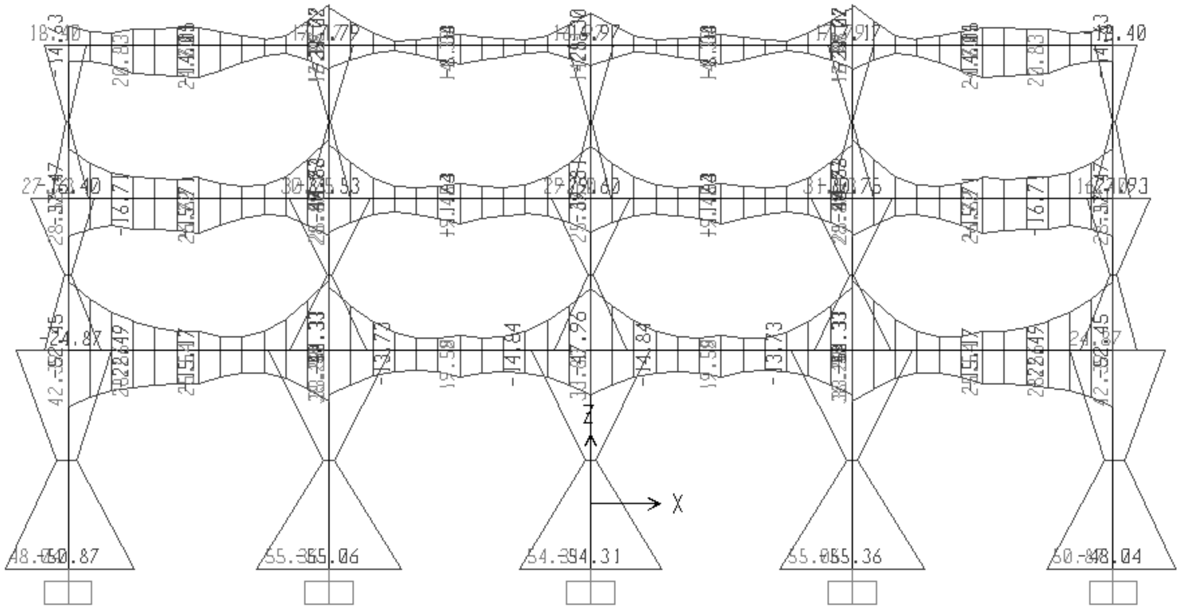


Fig B.21 Envelope of bending moment diagram for zone 5, slab thick. = 160 mm, Axes 1 and 4

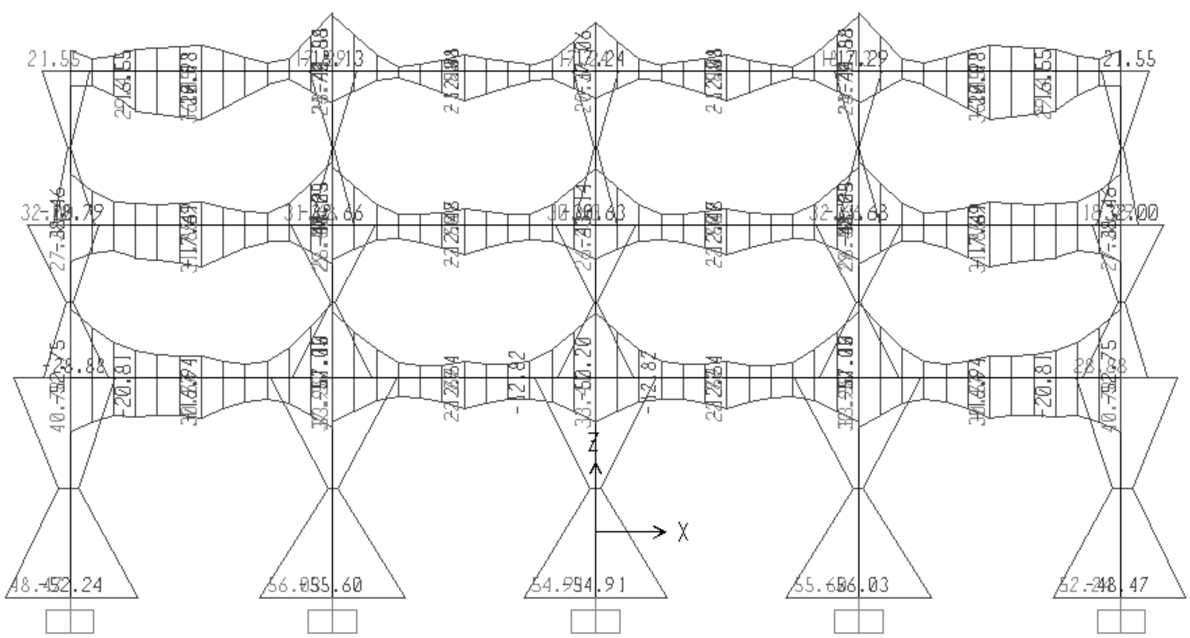


Fig B.22 Envelope of bending moment diagram for zone 5, slab thick. = 160 mm, Axes 2 and 3

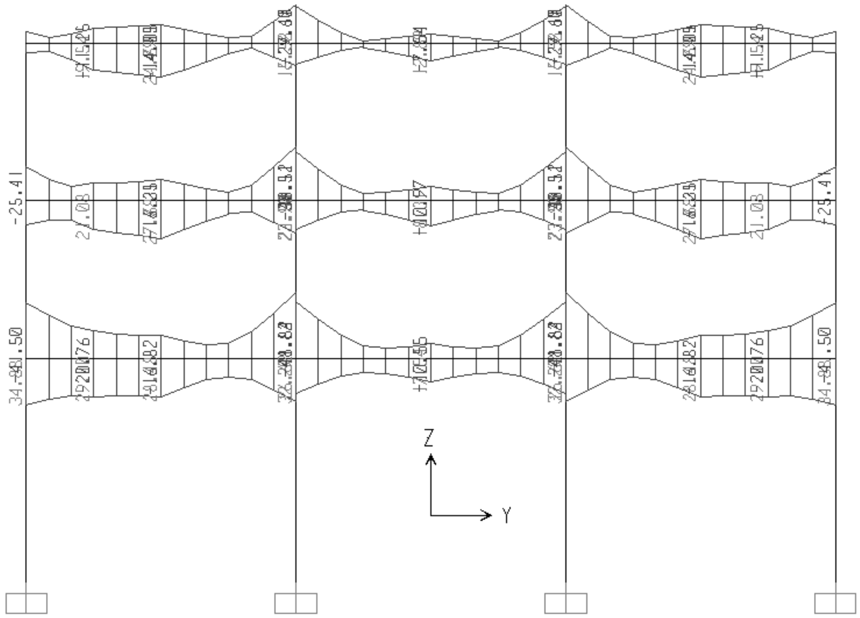


Fig B.23 Envelope of bending moment diagram for zone 5, slab thick. = 160 mm, Axes A and E

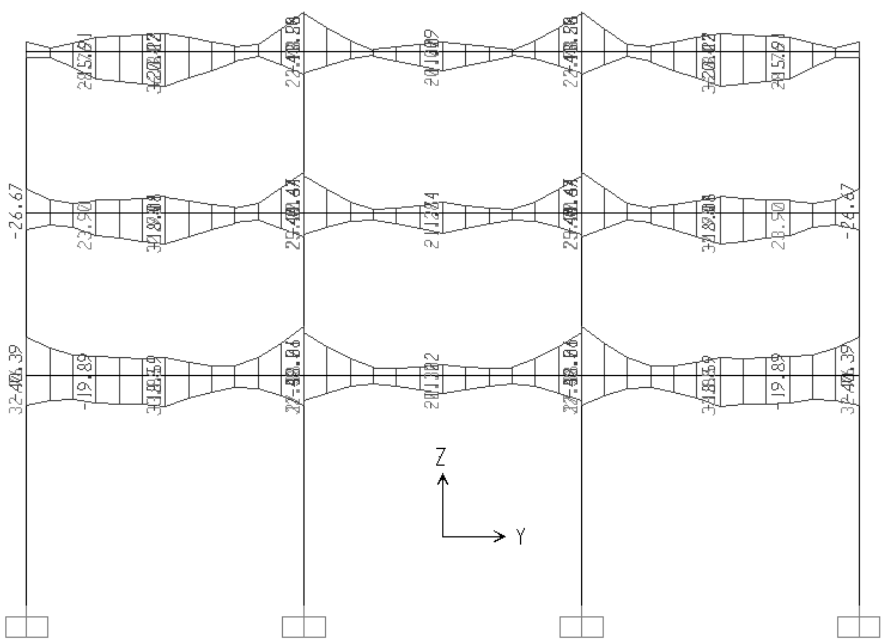


Fig B.24 Envelope of bending moment diagram for zone 5, slab thick. = 160 mm, Axes B, C, and D

APPENDIX C

MODE SHAPES OF VIBRATIONS

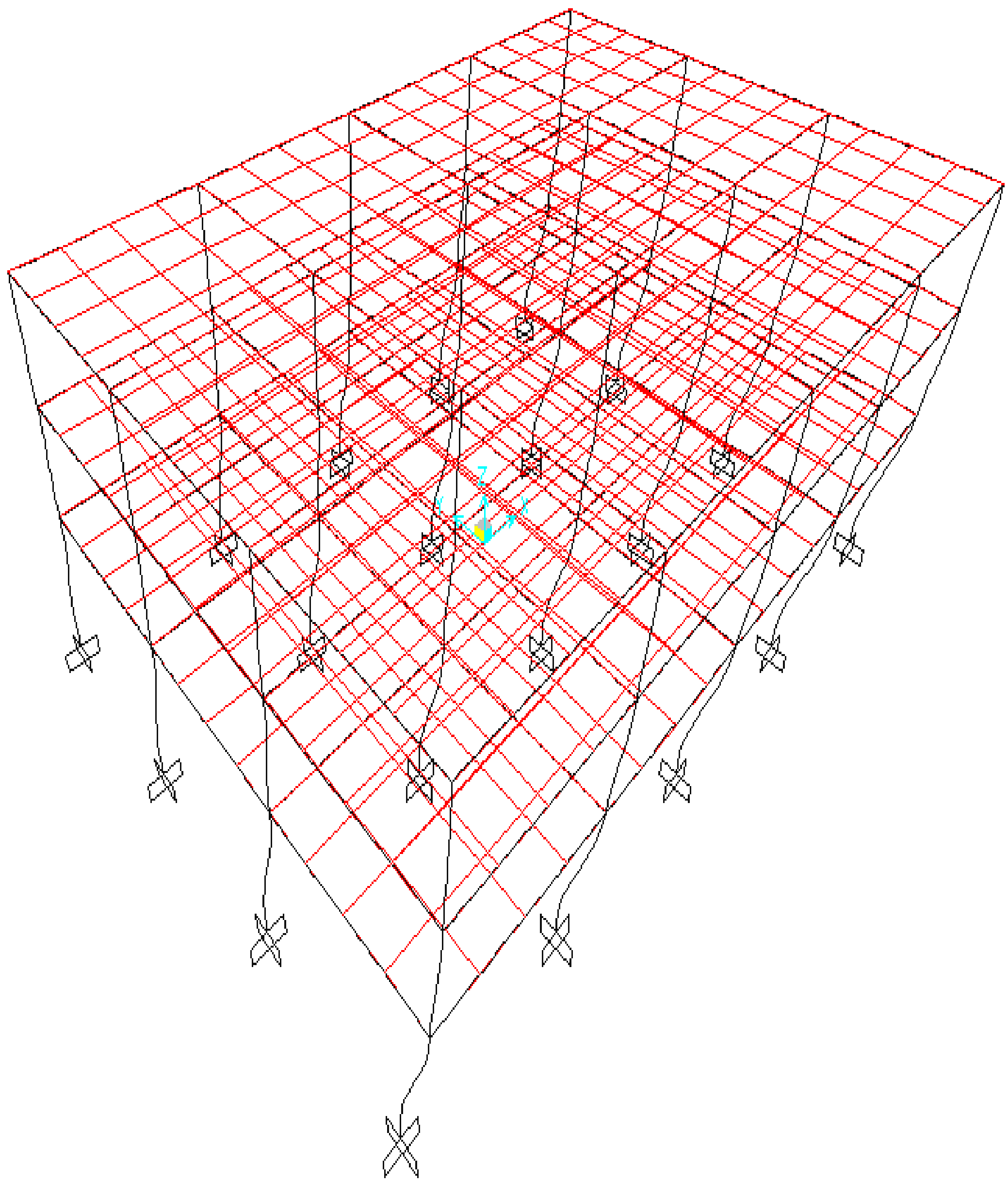


Fig. C.1 Mode 1, period = 1.2861s.

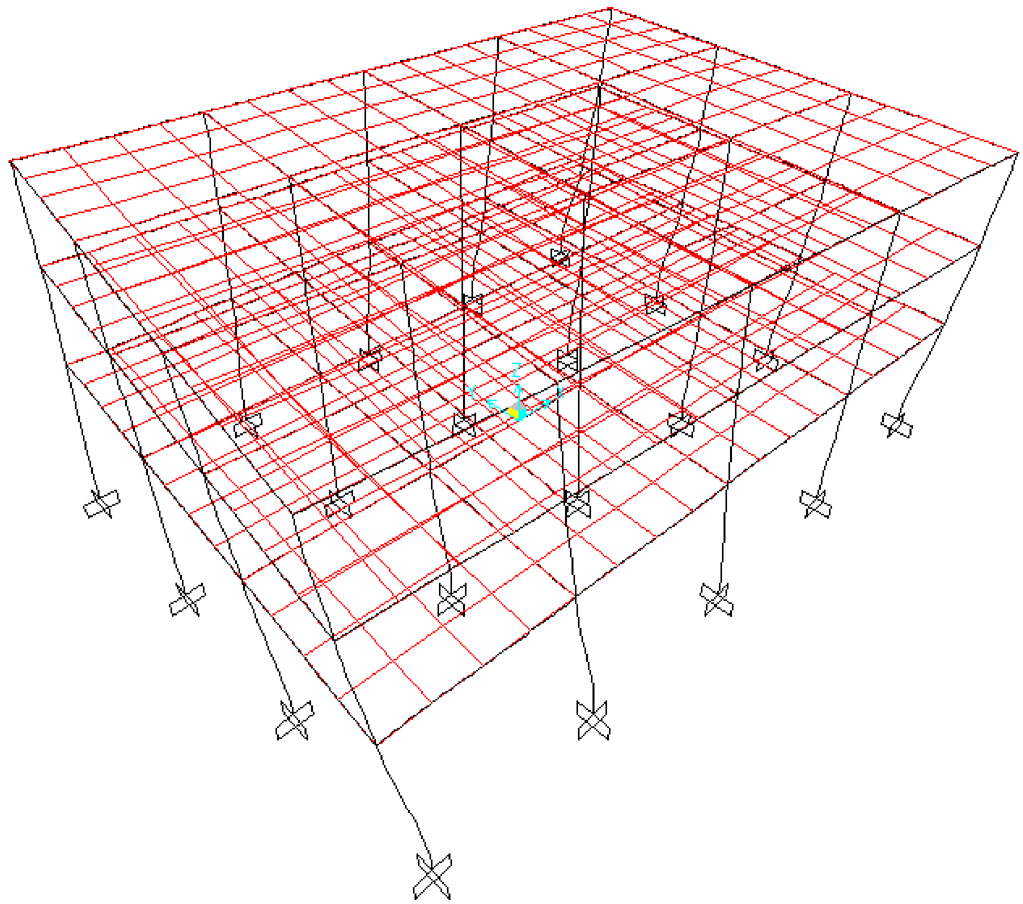


Fig. C.2 Mode 2, period =0.9458s.

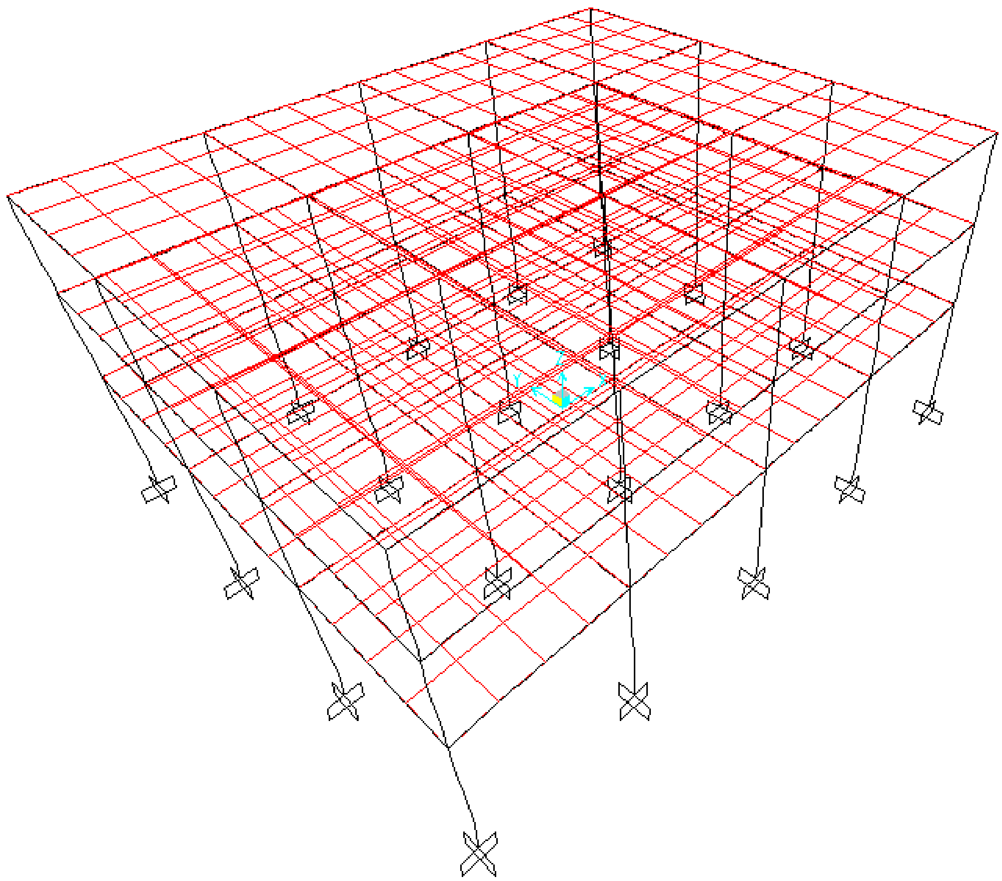


Fig. C.3 Mode 3, period = 0.9307s.

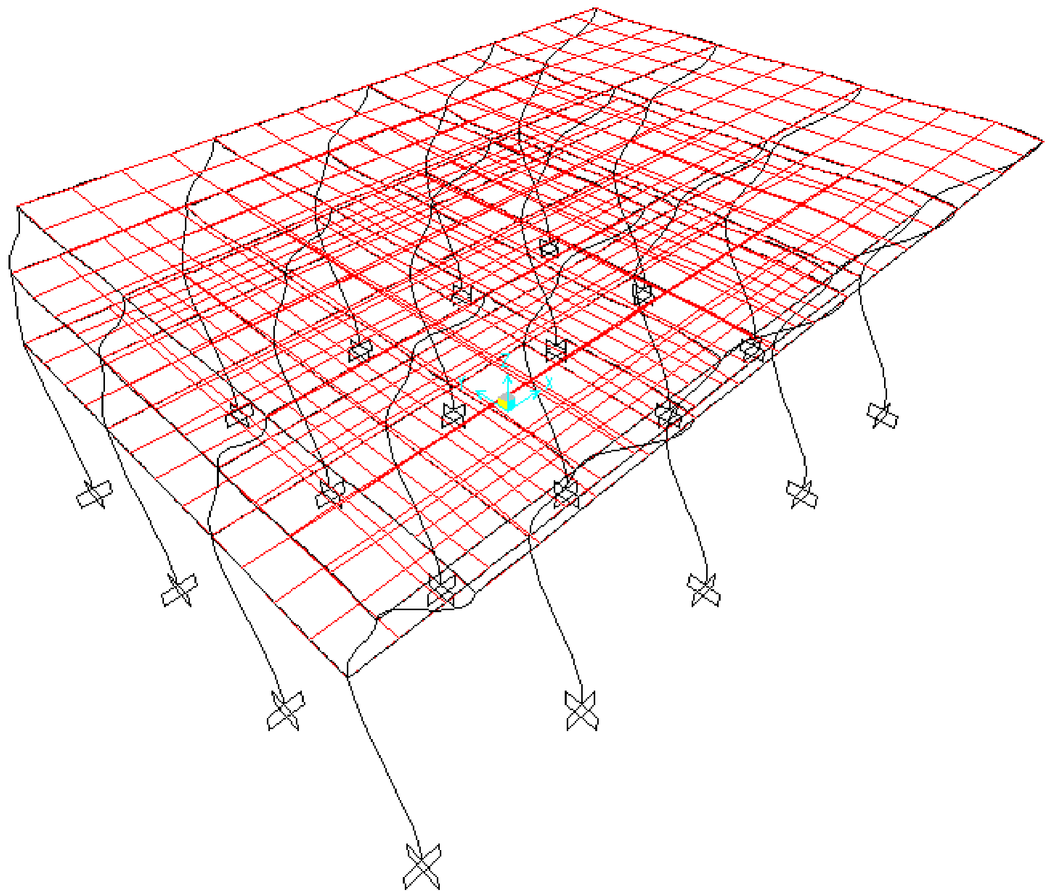


Fig. C.4 Mode 4, period = 0.4160s.

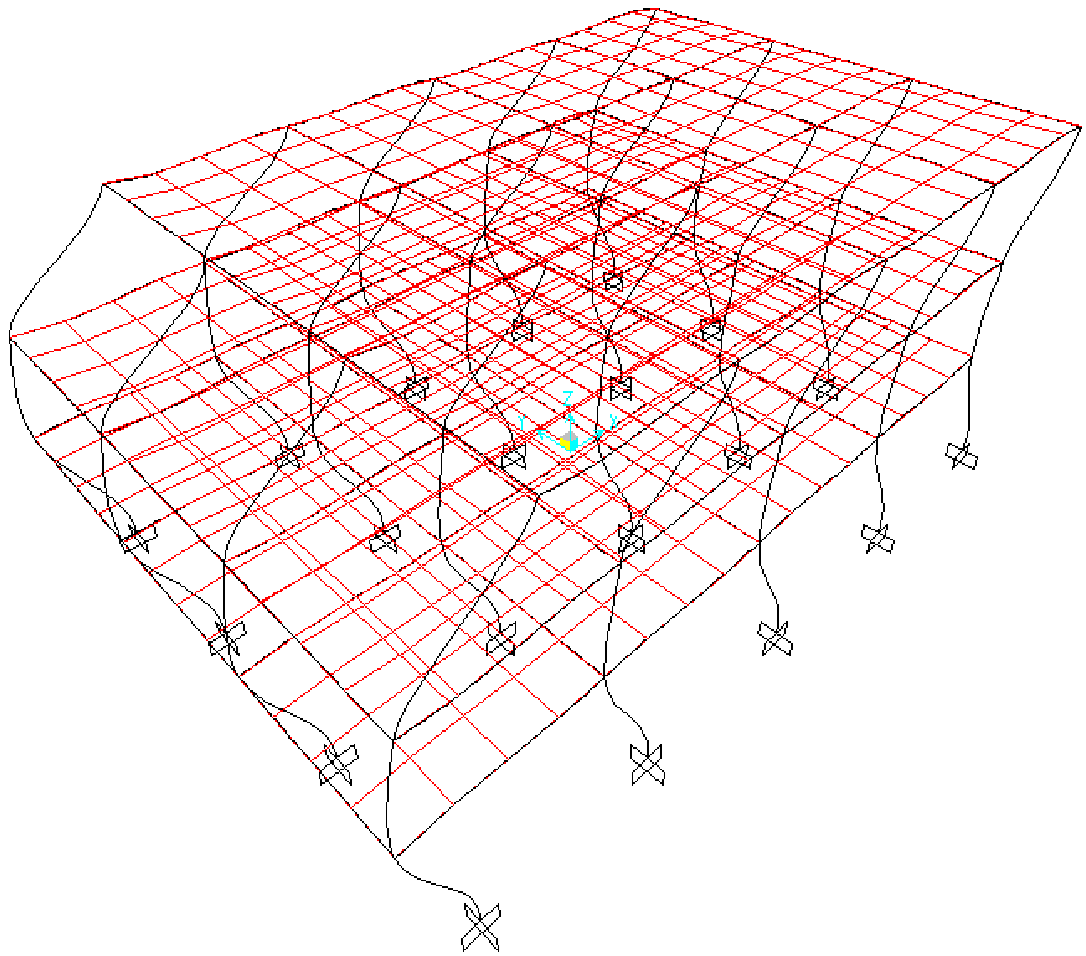


Fig. C.5 Mode 5, period = 0.3599s.

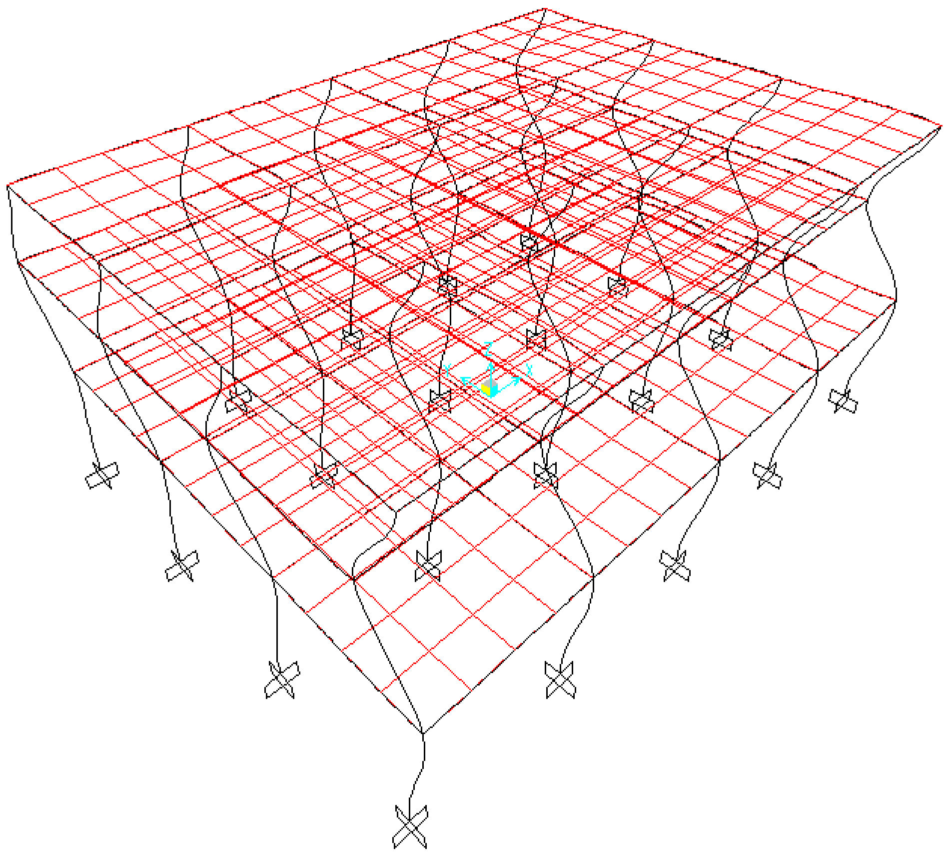


Fig. C.6 Mode 6, period = 0.2496s.

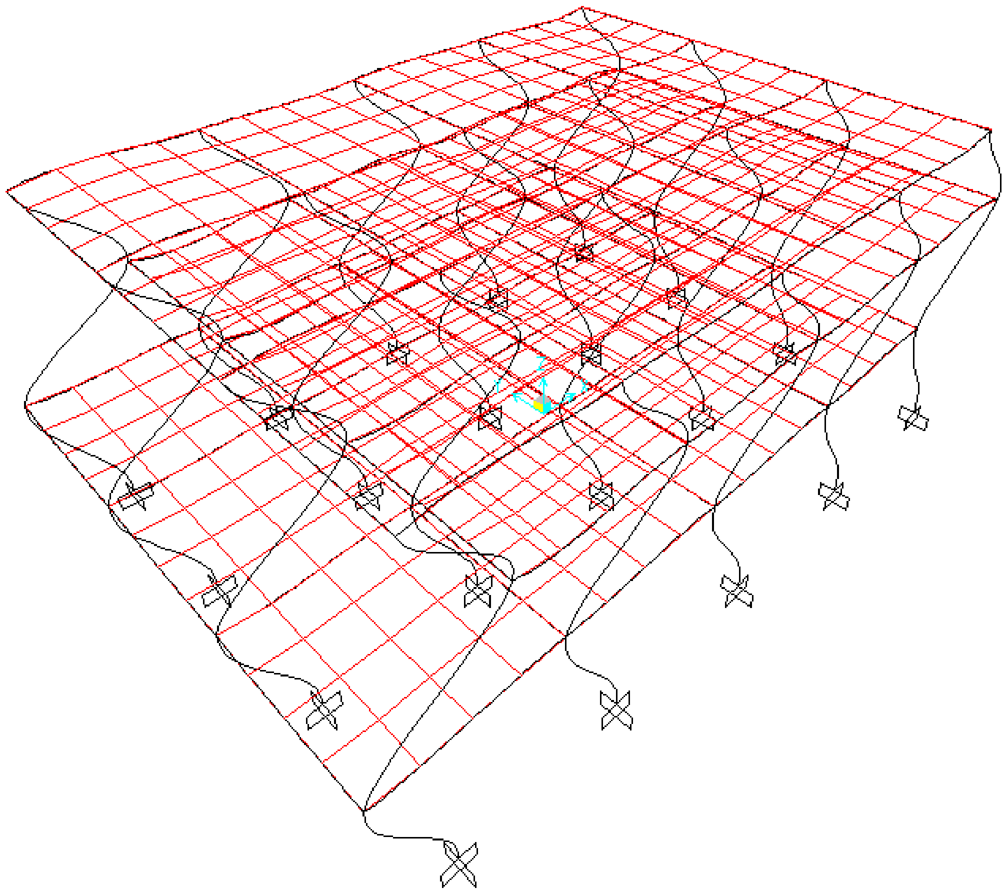


Fig. C.7 Mode 7, period = 0.1988s.

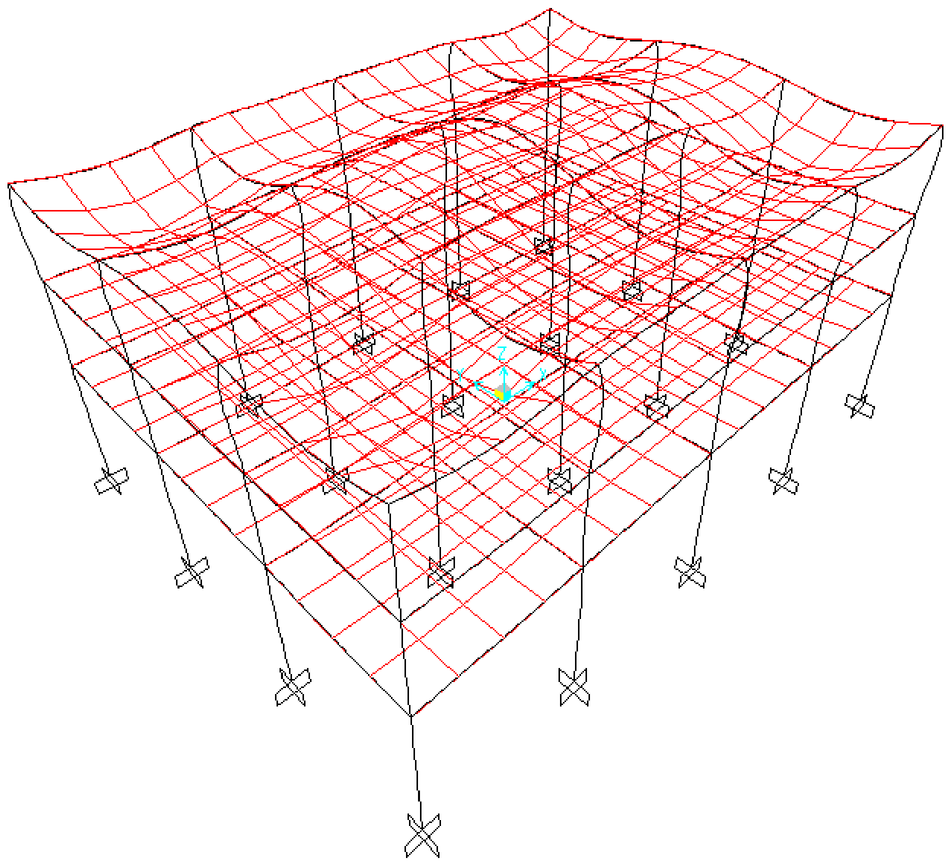


Fig. C.8 Mode 8, period = 0.1233 s.

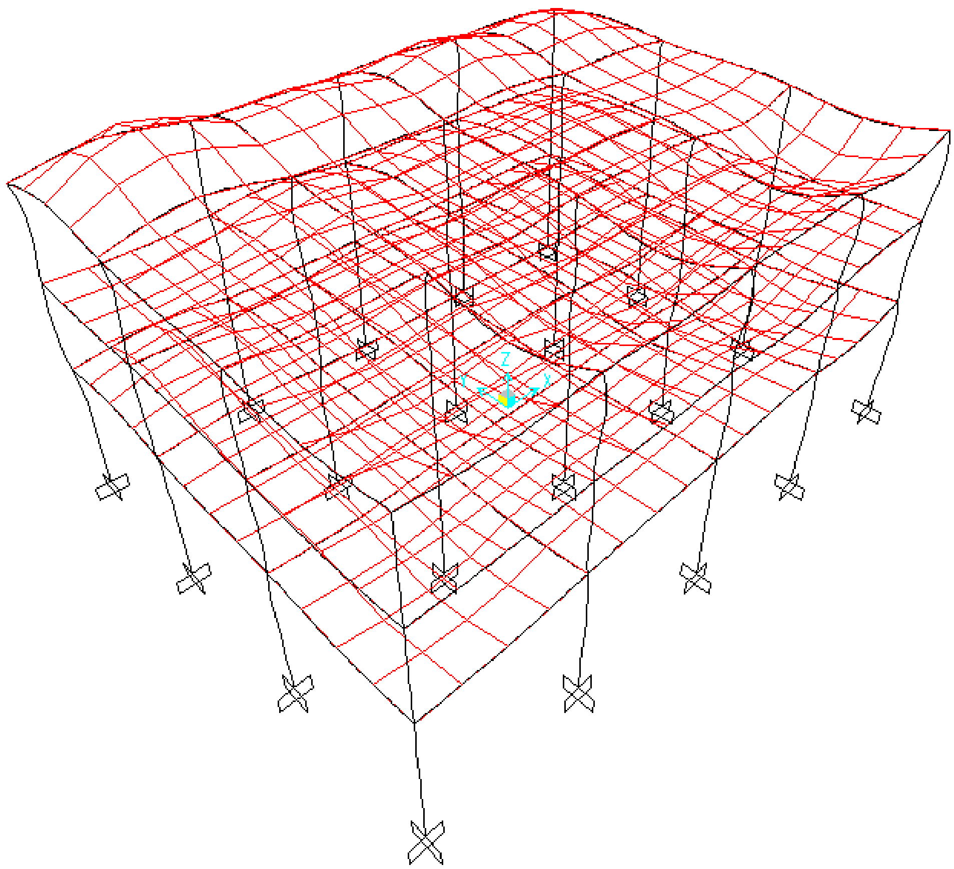


Fig. C.9 Mode 9, period = 0.1190 s.

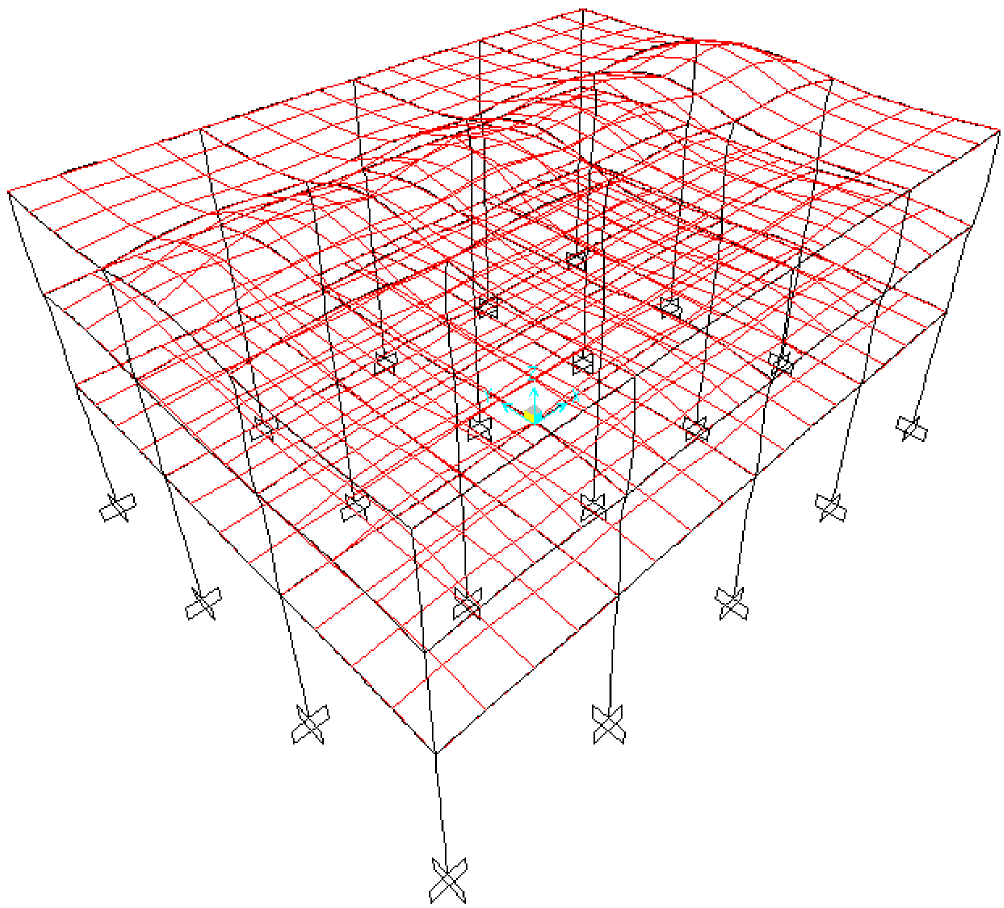


Fig. C.10 Mode 10, period = 0.1162 s.

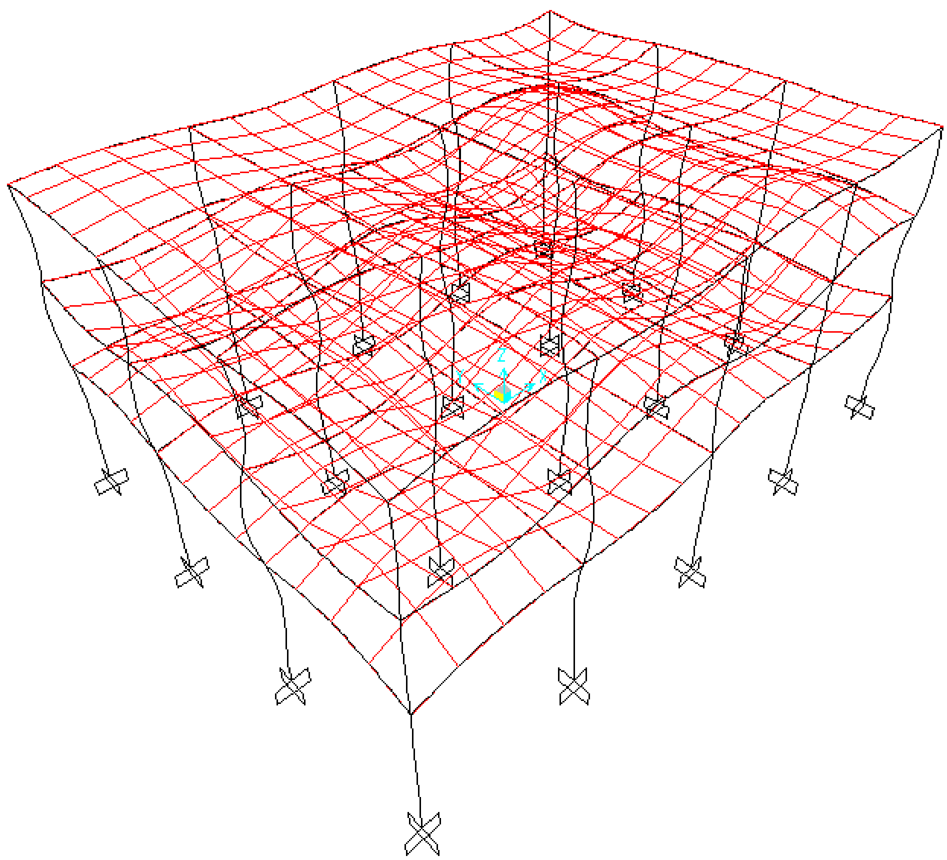


Fig. C.11 Mode 11, period = 0.1138 s.

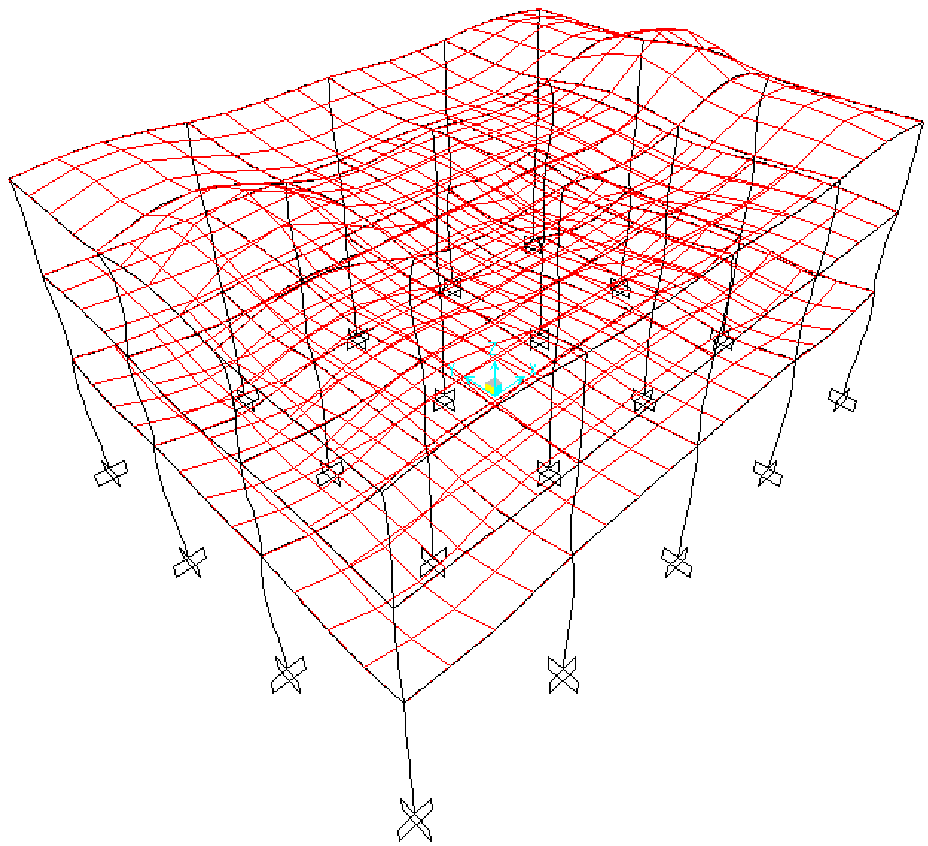


Fig. C.12 Mode 12, period = 0.1045s.

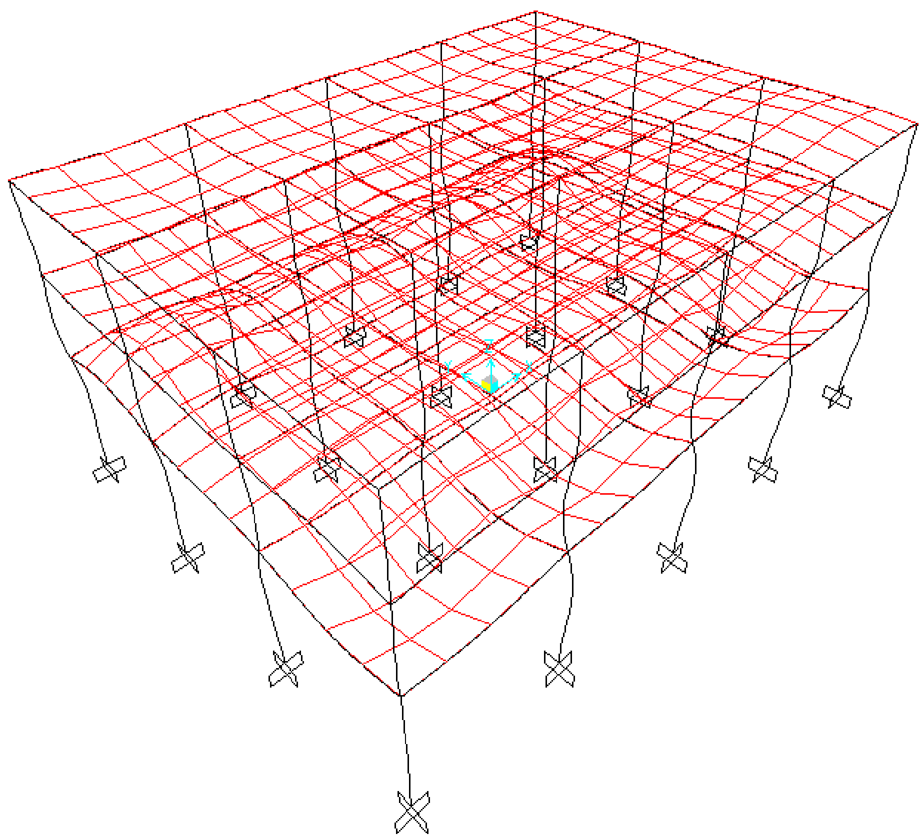


Fig. C.13 Mode 13, period = 0.1021 s.

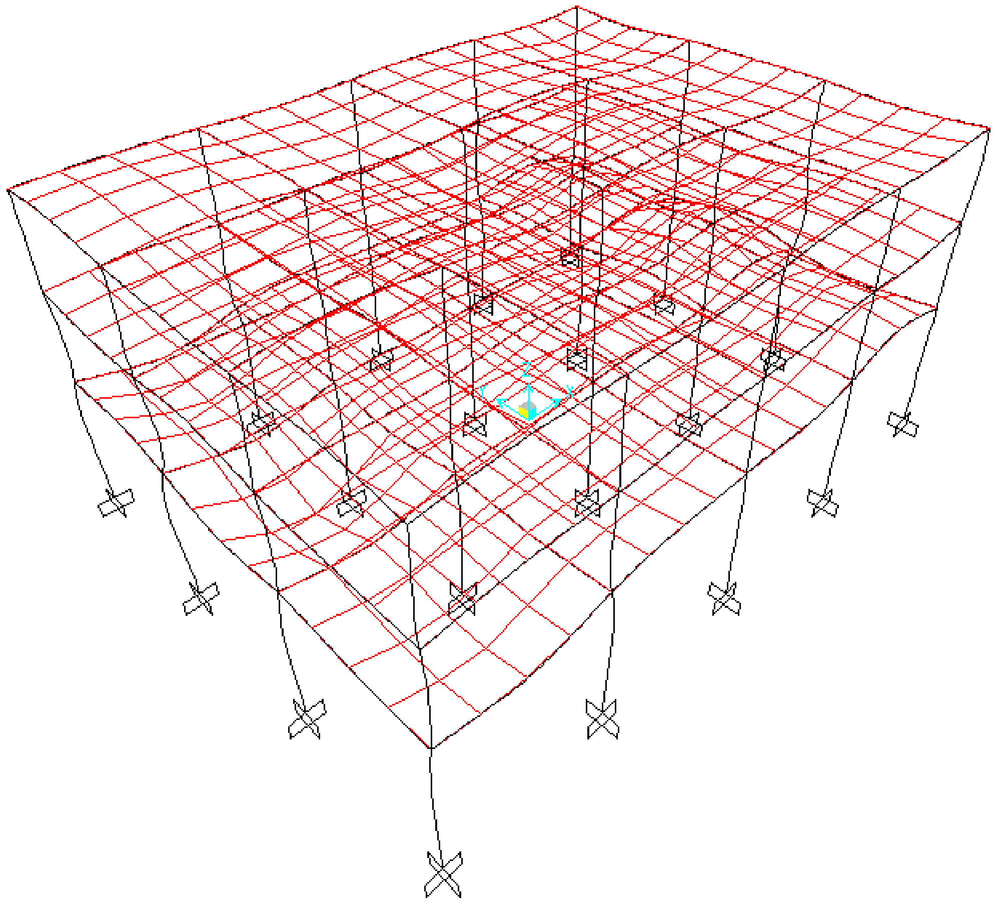


Fig. C.14 Mode 14, period = 0.0940s.

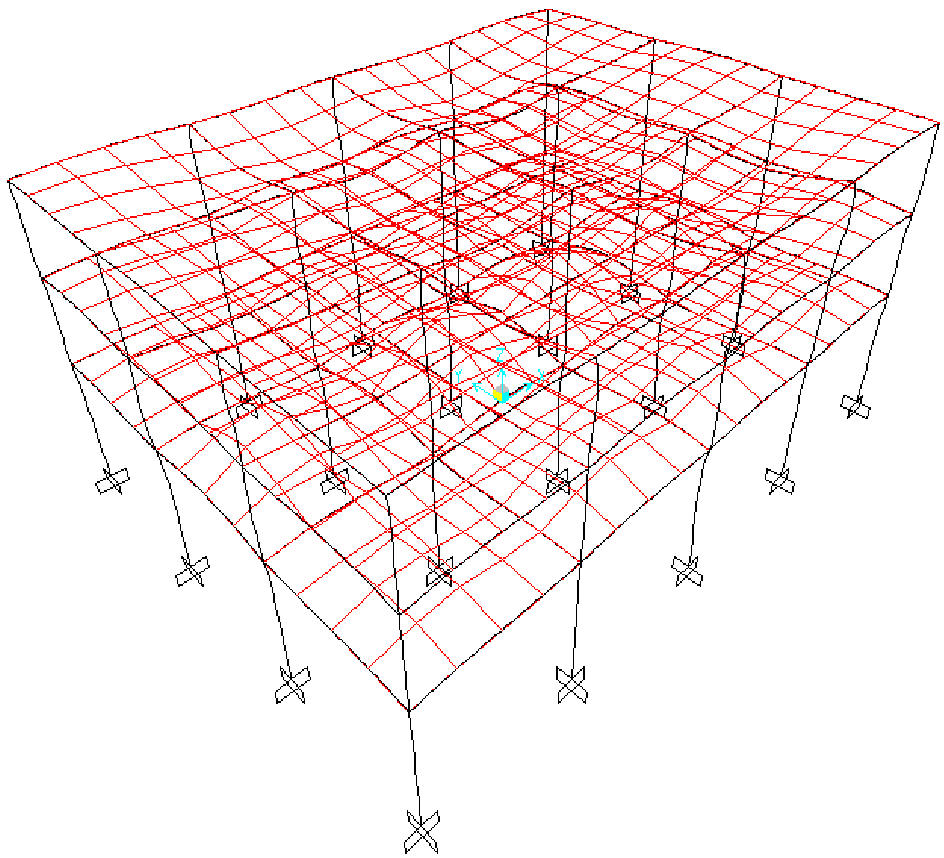


Fig. C.15 Mode 15, period = 0.0824s.

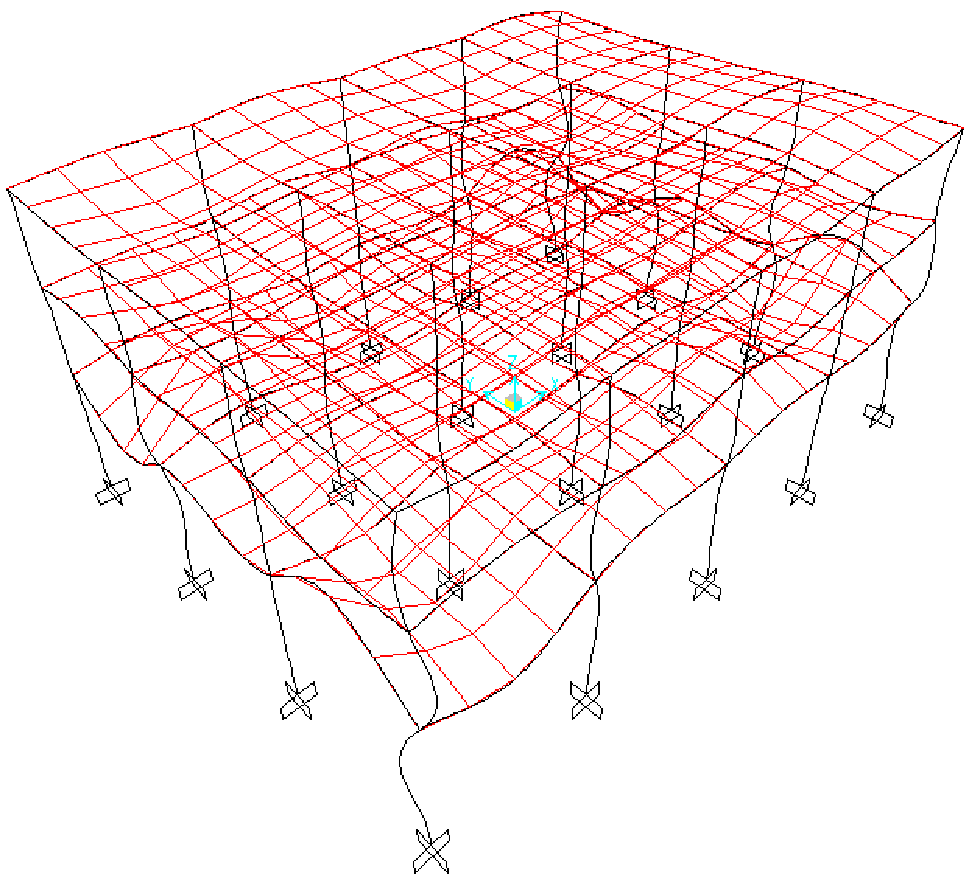


Fig. C.16 Mode 16, period = 0.0670s.

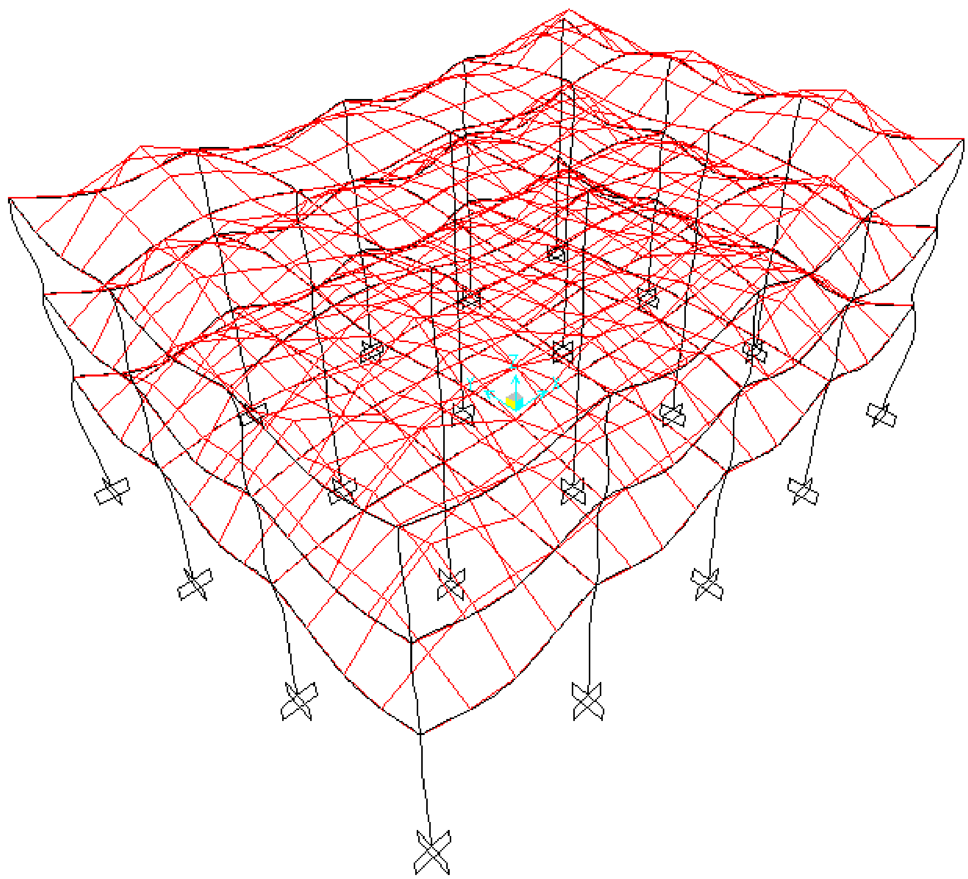


Fig. C.17 Mode 17, period = 0.0582s.

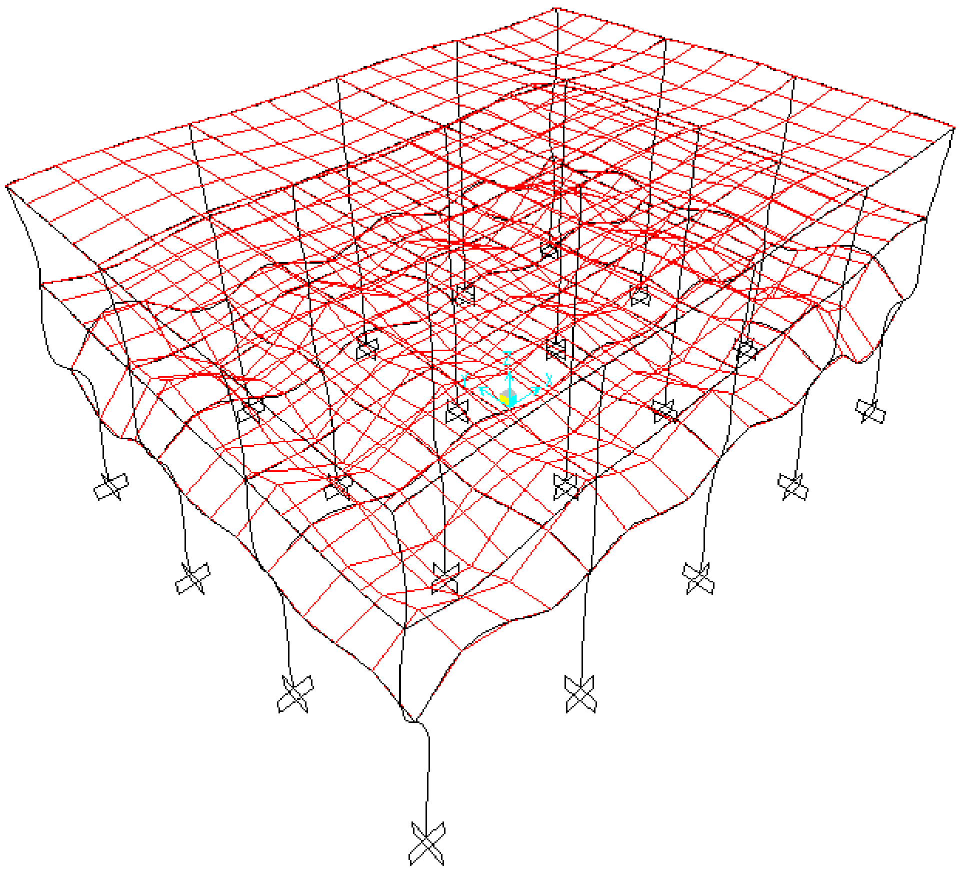


Fig. C.18 Mode 18, period = 0.0376s.

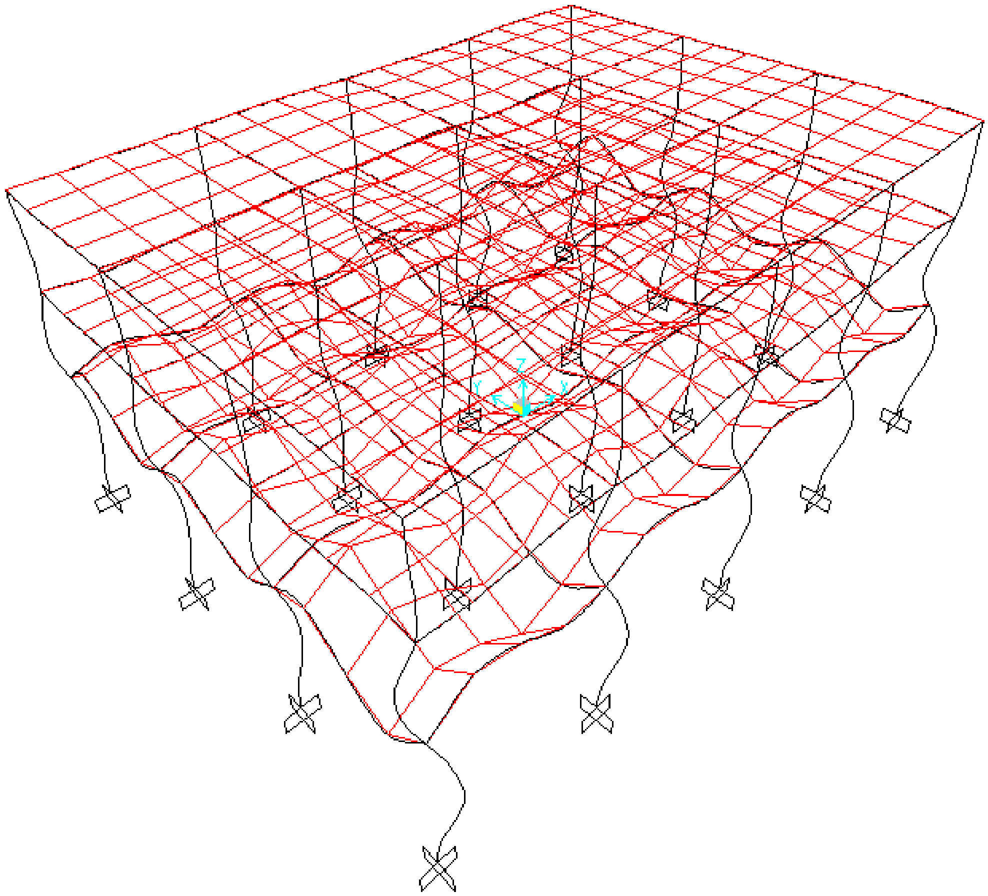


Fig. C.19 Mode 19, period = 0.0337s.

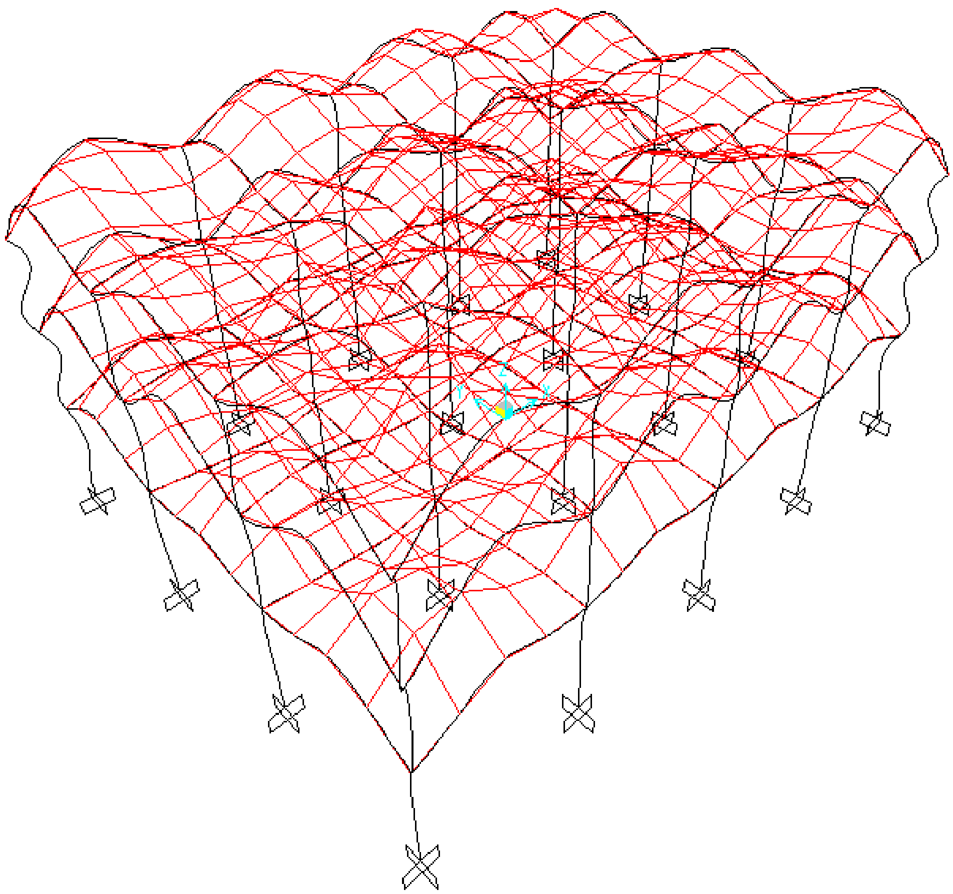


Fig. C20 Mode 20, period = 0.0336 s.