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INVESTIGATING THE NON-LINEAR BEHAVIOR OF RC FRAMED STRUCTURES WITH SEMI-RIGID JOINTS UNDER VERTICAL AND LATERAL LOAD EXCITATION

Khair Al-Deen Bsisu[1] and Ibrahim Altarabsheh[2]
[1]Department of Civil Engineering, School of Engineering, The University of Jordan, Amman, Jordan
[2]Structural Engineer, Jordan Pioneer For Metal Industries, Amman, Jordan
E-Mail: k.bsisu@ju.edu.jo

ABSTARCT

This study aims at investigating the nonlinear behavior of adequate reinforced concrete frames with semi-rigid connections under high lateral load and comparing them to the moment resisting systems. These two systems were compared based on their energy dissipation capacity, inter-storey drift ratio, force distribution, ductility, failure mechanism, and self-centering capacity. Also, this study aims at evaluating important parameters for the Reinforced concrete building with semi rigid connections such as over-strength factor, ductility, and response modification factor and compare them with that of moment resisting system. A complete three-dimensional finite element model for the RC connections is developed using ANSYS finite element software to determine the moment rotation curve for the connections based on their size, concrete strength and reinforcement details. SAP2000 finite element analysis model is performed to investigate the impact of semi rigid connections on the nonlinear behavior of RC buildings. The seismic force and displacement demand on the proposed system are determined using nonlinear time history analysis. Moreover, the maximum displacement that the building can withstand is determined using pushover analysis. The study concluded that considering RC beam-column joint as a rigid connection, will significantly overestimate the stiffness of RC buildings and will give erroneous structural responses under earthquake loading.

Keywords: reinforced concrete, frames, non-linear analysis, semi-rigid joints.

1. INTRODUCTION

Reinforced Concrete (RC) buildings must be designed to resists high lateral forces, this can be done by designing the building to have many characteristics such as high energy-dissipation capacity and lateral stability. One example of the lateral resisting systems that are commonly used for RC buildings are moment resisting frames. In order, for this system to behave in ductile manner under strong lateral load, the concept of “strong-column/ weak beam” is used to design this system. The capacity of the system designed using this concept is controlled by the longitudinal beams flexural strength, and the behavior of the beam-column joints will remain elastic. Despite, this behavior of the beam-column joints which has been proven by many researchers and by the current seismic codes, these joints are considered as rigid joints when designing these systems. This results in assuming higher stiffness of the structure than its actual stiffness, and in assuming smaller roof drifts [1]. [2] stated that ‘‘typically, 20% of the inter-storey deflection due to earthquake forces may originate from joint deformations’’. there is still a need to study the nonlinear seismic behavior of RC frame with partially fixed connections including; the ability of the building to dissipate energy, its ability to return to its original position after the application of the lateral force, the ratio of the difference between consecutive stories displacement, the distribution of the force to the structural elements, building ductility, and the distribution of the plastic hinges at failure mechanism compared to the fully fixed joints frame.

This study aims to investigate the nonlinear behavior of reinforced concrete building with semi-rigid connection under high lateral load compared to the moment resisting system. The comparison is based on the systems energy-dissipation capacity, self-centering capacity, inter-storey drift-ratio, force-distribution, ductility and failure mechanism. This will give designers insight into the feasibility of using each system based on its safety under high lateral load. In addition to the above, this study aims to evaluate some parameters such as ductility factor of structure, over-strength factor and response modification factor for the RC building with semi rigid connections and compare them with that of moment resisting system. This study answers the following questions;

Does the RC building with semi rigid connections have high capacity to dissipate energy, and high capacity to return to its original position after the application of the earthquake than that for the moment resisting frame? Does the RC building with semi rigid connections have small difference in the lateral displacement ratio between two successive stories, and are they able to distribute forces among structural elements than that for the moment resisting frame? Does the RC building with semi rigid connections have high structure ductility and flexibility than that for the moment resisting frame? Does the RC building with semi rigid connections have similar distribution of the plastic hinges to that for the moment resisting frame? What are the differences of partially restrained and fully restrained buildings with respect to their ductility factor, over-strength factor and response modification factor.

2. LITERATURE REVIEW

This section review the developed approaches in the literature to model the impact of the connection partial fixity on the behavior of RC-buildings.

First, [3] develops finite element program called SEMIFEM analyze frames with semi-rigid connections. Semi-rigid connections are considered at the column base of a portal frame, joint between beam and column prefabricated structure, the connection of steel brace connection to reinforced concrete (RC) frame and connections between truss members. The study found that in the previous cases moments force, axial force and horizontal displacements can be impacted by considering semi-rigid connection.

Another study was done by [4] who developed a finite element program to determine the reliability indexes and probabilities of failure for the structure. Two four and eight stories steel framed structures were analyzed. The study found that strengthen the portal frames with concentrically steel braces will reduce the inter-story drift and increase the stiffness of the steel frames. In addition, the study found that, the X braced and frame is more reliable than the inverted V braced frame and the inverted K braced frame.

[5] investigated the behavior of RC beam-column connection using finite element analysis. Separate finite element modellings were performed on a cantilever beam-middle column connection to investigate the effect of concrete strength, percentage of reinforcement and bond stress-slip relation on the degree of RC connection fixity.
Based on these models, the characteristics of RC semi-rigid connections are formulated in term of their moments and rotation. The study concluded that, the behavior of RC connections is closer to semi rigid than rigid connections and thus the conventional analysis of RC frames with rigid connections will not lead to accurate results. It's clear that the previous studies did not investigate the behavior of RC semi rigid connection under lateral and vertical loads acting on the connection simultaneously. Also, the previous studies did not investigate how such behavior of the RC semi rigid connection can impact the nonlinear behavior of RC framed structures with semi-rigid connections. In addition, the previous studies did not address the feasibility of using RC framed structures with semi-rigid connections with respect to their safety comparing to other popular RC framed structures such as moment resisting frame and dual system. Finally, the previous studies did not evaluate the behavior parameters for the RC framed structures with semi-rigid connections such as ductility factor of structure, over-strength factor and response modification factor. This study overcome these limitations in the literature.

[6] reviewed the different mechanisms of force and crack development in the RC joints. The authors discussed that due to horizontal and vertical joint shear force, an excessive diagonal tensile stress will be generated in the joint which will results in diagonal tension cracks. The generated diagonal cracked concrete in the joint core can efficiently transfer diagonal compression forces, approximately parallel to the cracks. The authors discussed next the concepts of strut and truss mechanisms. The paper mentioned that strut mechanism results from the combination of both the horizontal and vertical concrete force, together with the major part of the horizontal and vertical steel compression force and the column shear force. On the other hand, the truss mechanism results from the shear force at the face of the joint.

[7] test five samples to derive equations for the relative rotation between beam and column. The study concluded that the concrete compressive strength of the beam and column significantly influenced the moment-rotation curve of the connections. Also, the study found that both concrete tensile strength and the bond-slippage behavior of the longitudinal reinforcement is related to the concrete compressive strength. Finally, the study recommend that the developed model can be improved by including the deformation mechanism associated with slippage of beam longitudinal reinforcements.

3. FINITE ELEMENT ANALYSIS FOR BEAM-COLUMN CONNECTIONS

This study aims to determine the linear stiffness and the moment-rotation curve for the reinforced concrete beam-column connections. To achieve this purpose, a detailed nonlinear finite element model for RC beam column connections was developed using ANSYS finite element software as shown in Figure 2. The modeled connections are the interior and the exterior connections of an eight stories case study building considered in this study. Therefore, a total of 16 connections were modeled in this study (8 interior connections and 8 exterior connections). The case study building was designed by [9] and will be discussed in more details in this paper.

3.1 Geometry and reinforcement details of RC beam-column connections

The general dimension and reinforcement details for the modeled connections is shown in Figure-1, the detailed dimension and reinforcement details for each joint is listed in Table 1 through Table-4.

3.2. Finite element model

The FE model of both the exterior and the interior connections are developed using ANSYS finite element software. Concrete is modeled with Solid65. This is an 8-noded element having three translational degrees of freedom (DOF's) at each node with the ability to simulate cracking and crushing in concrete. The Link180 element is used for modeling of reinforcement. This is a two-node truss-type element having three translational DOF's at each node. The bond strength between the concrete and steel reinforcement should be considered. However, in this study, perfect bond between the materials was assumed. This assumption was based on the recommendation of many researchers such as [1] and [10] where they show that assuming prefect bond between steel and concrete is a good and acceptable estimate for the actual bond between the concrete and steel. To provide the perfect bond, the link element for the steel reinforcing was connected to the

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corresponding nodes of adjacent concrete solid elements, so that the two materials shared the same nodes.

3.3 Material properties

3.3.1 Concrete

To define the concrete material in ANSYS, the following properties are needed (ACI 318-14):

a) The modulus of elasticity for the concrete was considered as $4700\sqrt{f_c^{'}}$ (MPa), in which $f_c^{'}$ is the concrete compressive strength.
b) The ultimate axial tensile strength, or the rupture modulus, can be considered as $0.612\sqrt{f_c^{'}}$.
c) Compressive strength $f_c^{'}$ was considered in this study to equal 25MPa.
d) Poisson's ratio, υ assumed to be equal to 0.2.
e) In this research, the Hognestad equation is used to construct axial stress–strain relation in compression and it can be written as follows [4]:

$$f_c = \begin{Bmatrix} f_c^{''}\left[\dfrac{2\varepsilon_c}{\varepsilon_0} - \left(\dfrac{\varepsilon_c}{\varepsilon_0}\right)^2\right] & \varepsilon_c \le \varepsilon_0 \\ f_c^{''}\left[1 - 0.15 * \left(\dfrac{\varepsilon_c - \varepsilon_0}{\varepsilon_{cu} - \varepsilon_0}\right)\right] & \varepsilon_0 \le \varepsilon_c \le \varepsilon_{cu} \end{Bmatrix} \quad (1)$$

Where: $f_c^{''}$ is the maximum compressive stress in concrete calculated from Equation 2.

$$f_c^{''} = k_s \times f_c^{'} \quad \text{………… (2)}$$

Where k_s is a function of $f_c^{'}$ and its equal to 0.95 for $f_c^{'}$ of 25 MPa. In Equation 1, the strain ε_0 is determined using Equation 3.

$$\varepsilon_0 = 1.8\left(\frac{f_c^{''}}{E_c}\right) \quad \text{……… (3)}$$

The concrete stress–strain curve is illustrated as Figure-3. The corresponding values of the modules of elasticity and modules of rupture modulus are 23.5 GPa and 3 MPa, respectively.

3.3.2 Steel reinforcement

The stress–strain relation of the longitudinal and transverse reinforcement is introduced as multi-linear curves shown in Figure-4. According to reference experimental studies, the yield strength of the longitudinal and transverse reinforcements is taken to be 420 MPa. The Poisson's ratio of steel is assumed to be 0.3.

3.4 Analysis results

This section discusses the analysis results for the 16 RC beam-column connections discussed in the first section of this study.

3.4.1 Failure pattern

The reinforcement details for each connection in addition to its deflection shape and the stress distribution at failure are shown in Figure-5 through Figure-6.

As can be shown in these figures and from the nonlinear FEM analysis, first bending cracks appear in the tensile region of beam and then shear cracks develop in the connection. With increase of load, cracks extend to the mid span of beam. The yielding of the longitudinal rebar of beam at the connection and the development of maximum concrete elements tensile and compressive capacity at the zones adjacent to the connection. Indicates that a plastic hinge forms in the beam at the connection.

3.4.2 Moment-Rotation behavior

The moment rotation curves obtained from the ANSYS model developed in this section are shown in Figure-7 and Figure-8 for exterior and interior connections respectively.

The moment and rotation values shown in the figures are similar to those obtained by other researchers such as [10], [1] and [11] for RC beam-column connections. From the moment-rotation curve for each joint the bending stiffness for a joint can be determined by the slope of the curve where the behavior of the connection still linear. The obtained value for the bending stiffness for the interior and exterior joint is listed in Tables 5 and 6 respectively.

These values are compared with that recommended by both ASCE/SEI 41-06 and FEMA356. First, FEMA356 defines an effective bending stiffness for the frame element as a function of the axial force to be equal to (0.5 - 0.7) EIg (E = modulus of elasticity of concrete, Ig = the gross moment of inertia of member). In contrast, ASCE/SEI 41-06 introduces a smaller stiffness, equal to (0.3 - 0.7) EIg depending on the axial load. The moment rotation curves and the values of the bending stiffness obtained for each joint will be used to evaluate the behavior of RC buildings under seismic forces. In the next section, the results obtained in this section will be validated by conducting experimental study.

4. NONLINEAR ANALYSIS OF PARTIALLY FIXED CONCRETE BUILDINGS

The moment-rotation curves and the bending stiffness values obtained from the analysis in Section three, will be used in this Section to evaluate the nonlinear behavior of partially restrained RC building under high lateral load. SAP2000 finite element software was used for this purpose, and the obtained behavior of the partially restrained RC building is compared with the fully restrained RC building. The comparison is based on many factors including; energy-dissipation capacity, self-centering capacity, inter-storey drift ratio, force distribution, ductility, over-strength factor, response modification factor and failure mechanism.

4.1 SAP2000 model

4.1.1 Case study

An eight stories building is considered in this section to evaluate the impact of considering the RC beam-column connections as semi-rigid connections comparing with rigid connections. The case study building is designed by [9] as reinforced concrete moment frame buildings. The design of the building was based on the concept of strong column/weak beam (SCWB). The SCWB ratio was set by [9] at 1.3 instead of 1.2 as specified in ACI 318-14. The aim of the strong-column weak-beam design criteria is to prevent soft story mechanisms. It should be noted that ACI provision does not fully prevent column hinging but helps to delay column hinging and to spread the damage over more stories of the building.

A typical floor plan for the case study building is shown in Figure-9, and important design parameters are given in Table-7. The second column of Table-7 summarizes the range of values of the design parameters. The column section and reinforcement layout was determined by using PCA-COLUMN. PCA-COLUMN is a software program for design and evaluation of reinforced concrete sections subject to axial and flexural loads. Also, this software can consider the slenderness effects in designing columns, and its design criteria conform to provisions of various codes, such as ACI 318-14.

4.1.2 Material nonlinearity in SAP2000

In this study, the material was assumed to behave in-elastically. This is necessary to obtain the behavior of the building after yielding and finally the expected failure mechanism for the building. SAP2000 software uses concentrated plastic hinge approach to model material nonlinearity. This approach uses zero-length spring to represent the formation of plastic hinges at the end of the beam, while the structure element remains elastic. This approach has been proven to produce satisfactory and good approximation of the real behavior of the beams and columns. Also, this approach requires much less computational time than the distributed plasticity approach.

Plastic hinge in SAP2000 can be defined for each degree of freedom. However, in this study the focus was on defining rotational plastic hinges. SAP2000 uses moment-rotation curve for this purpose. Example of this curve is shown in Figure-10. As can be seen from the figure, the curve gives the yield value and the plastic deformation following yield. Also, the curve is symmetric for positive and negative direction.

The five points on the curve (A-B-C- E - D) represent the following:

a) Point A corresponds to the unloaded condition.
b) Point B corresponds to the material yielding.
c) Point C has resistance equal to the nominal strength.
d) Line CE corresponds to initial failure of the member.
e) Line ED represents the residual strength of the member. It may be non-zero in some cases, or practically zero in others.
f) Point E corresponds to the deformation limit.

The value of the points (A-B-C- E - D) on the plastic hinge can be defined from the moment rotation curves for each joint obtained in section three.

4.1.3 Pushover analysis

The nonlinear properties for structural elements are defined by assigning nonlinear hinges with defined moment rotation behavior at the end of these elements. For the rigid building, typical hinges recommended by FEMA 356 were assigned to each element as shown in Figure-11. On the other hand, nonlinear properties for semi-rigid connections were defined on two stages. Firstly, linear stiffness is assigned for each connection as listed in Tables 5 and 6, then non-linear moment rotation characteristics were assigned for each connection as defined in Figures 7 and 8.

4.1.4 Nonlinear time history analysis

Unlike nonlinear static Pushover, nonlinear time history analysis method is more complicated method and requires more computational time and effort. This analysis was performed using real earthquake record. Specifically, El Centro earthquake was used in this section for this purpose. The record of El Centro earthquake was obtained from the Pacific Earthquake Engineering Research center. This earthquake has epicenter 5 miles north of Calexico, California and occurred at 21:35 Pacific Standard Time on May 18, 1940 in the Imperial Valley in southeastern Southern California near the international border of the United States and Mexico. This earthquake is chosen because it has large magnitude with value of 6.95 on richter scale. Also, this earthquake has peak ground acceleration of 0.33g. The duration of the earthquake was about (89) second, such duration considers long duration comparing to the normal earthquake durations which is between (10-20) seconds. Figure-12 show the acceleration for El-Centro earthquake in term of gravitational acceleration G.

Newmark constant acceleration method is used to carry out the nonlinear time history analysis. Both geometrical and material nonlinearities were considered by the analysis. This is done by including P-delta effect and material yielding (using concentrated plastic hinge approach).

4.2 Analysis results

4.2.1 Pushover analysis results

Figure-13 shows the pushover curves that represents the base shear versus the roof displacement for rigid and semi rigid buildings respectively. The figure shows that the partially rigid building presents much

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higher ductility compared to the rigid building (i.e. larger lateral displacement capacity before loss of lateral strength). This indicates that considering the joint stiffness increase the flexibility of the frame.

This remarkable difference can be attributed to the deformed shapes and location/distribution of plastic hinges as discussed in the following section.

4.2.1.1 Deformed shape and location of yield activity

Figures 14 and 15 show the deformed shape and location of plastic hinges for the rigid and semi-rigid buildings under pushover, respectively. It can be said that there is no formation of plastic hinges in the columns for both buildings. This can be attributed to the fact that both buildings were designed according to the strong columns weak beams concept (SCWB) with ratio of 1.3 instead of 1.2 as required by ACI 318-14.

Finally, these figures show that failure occurs in the fully rigid building, while no failure occurred in the partially fixed building. This confirm that the semi-rigid building is more flexible than the rigid building and thus it can dissipate more energy than the rigid building.

4.2.1.2 Performance point

As mentioned earlier in this Section, performance point is the intersection point of the capacity curve of the building and demand curve due to expected earthquake ground motion. The performance point indicates the maximum force and deformation a structure can withstand. This point shows whether the building has the capacity to withstand expected earthquake load. The results show that, for semi-rigid building the base-shear force at performance point is about 90% of the base shear in case of the rigid building, and the horizontal displacement for the semi-rigid building is about 2.3 times that for the rigid building. These results agree with the previous conclusion that considering RC connections as semi rigid connections will increase building flexibility and reduce its stiffness.

4.2.1.3 Ductility and force modification factor

Structure ductility defines the ability of the structure to undergo large deformation without significant reduction in its strength. As discussed in Section two, structure ductility can be computed as the ratio of maximum displacement to the yield displacement. In this section ductility is computed by applying nonlinear time history analysis following the procedure explained in Section two. The results show that the partially restrained building has ductility of 1.312, while the fully fixed frame has ductility of 1.118. These results agree with those obtained in the previous section since they confirm that the partially fixed building is more flexible than the fully rigid building.

Next the force modification factor was evaluated for both buildings following the procedure explained in Section 2. The results indicate that the force modification factor for the partially restrained building is 40% higher than that for the fully restrained building (R value for the partially restrained building is 3.4359, and R for the fully restrained building is 2.4427). These results were expected because of the ductility for the partially fixed building is higher than that for the fully fixed building. These results indicate that the partially restrained building will be designed for much less base-shear forces than that for the fully restrained building, which confirm that the partially restrained building is more ductile and have higher energy dissipation capacity than the fully rigid building.

4.2.2 Nonlinear time history analysis

4.2.2.1 Hysteresis behavior and energy dissipation capacity

In this section, the hysteresis behavior of both the fully rigid and the partially rigid building was determined by applying the nonlinear time history analysis. Figure-16 and Figure-17 show the hysteresis loop for the fully fixed and partially fixed frames respectively.

The results from the above figures show that the rigid buildings exhibit less deformation but higher base shear than the partially fixed case study building. For example, the fully restrained building has maximum top storey displacement of 0.18 m and maximum base shear of 1200 kN, while the partially restrained building has maximum top storey displacement of 0.203 mm and maximum base shear of 983 kN. It’s obvious from these results that the partially restrained building is more flexible than the fully rigid building, because it takes more force and required less value of the top displacement to fail. While the partially fixed building requires less force and higher value of the top displacement to fail.

In addition to this, it’s obvious from the previous figures that the area under the hysteresis loop is higher for the partially fixed building than for the fully fixed building, where the area under the hysteresis loop for the partially fixed building was about 1.2 times that for that of the fully fixed building. This indicates that the partially fixed building has more energy dissipation than the fully retrained building.

4.2.2.2 Inter storey drift ratio

Inter storey drift is an important engineering parameter to investigate the performance of the structure under strong earthquake load. The maximum drifts for the partially restrained and the fully restrained buildings are shown in Figure 18. As shown in this figure, the maximum inter storey drift for the partially restrained building is higher than that for the fully restrained building. However, it should be remembered that high values of the inter-storey drift ratio is not desirable since it will result in structural damage. The drift ratio shown in Figure 18 is still within the limitation imposed by FEMA which is 5%. These results are also agree with the results obtained by other researchers such as [a].

Figure-19 show displacement history for the rigid and semi-rigid buildings. This figure shows that, rigid building has smaller lateral displacement at the end of the earthquake than the semi-rigid building. This result was expected since the rigid building will have higher stiffness

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than the semi-rigid building. However, the displacement of both buildings are small and within code limits.

4.2.2.3. Force ratio

Figures 20 and 21 show the ratio of the forces in columns and beams of the semi-rigid building to those of the rigid building. As can be seen from these figures that the axial, shear and moment forces in both beams and columns for the semi-rigid building is smaller than those for the rigid building. This agree with the results obtained previously in this study that the semi-rigid building is more flexible than the rigid building and thus its stiffness is smaller than the rigid building. Therefore, design the beam-column connections as semi-rigid connections will increase the ductility of the building and its energy dissipation capacity. As a result, the base-shear force from earthquake on the building will decrease and the building lateral displacement will increase. Thus, the forces in beams and columns will be less than that for the rigid building.

5. CONCLUSIONS

The following conclusions can be drawn from this study:

a) The bending stiffness obtained for each connection using ANSYS software is within the range suggested by ASCE/SEI 41 and FEMA356.
b) The moment-rotation behavior of each connection using ANSYS software, agree well with the experimental results and agree with the behavior obtained by other researchers. The model captured the modes of failure and the peak load and rotation of the tested specimens.
c) Lateral structural responses of frames were strongly affected by whether joint flexibility was included in modelling. By modelling joint flexibility, the design base shear will decrease and at the same time the maximum lateral top displacement will increase which indicates that the partially fixed building is more flexible than the fully fixed building. This result was confirmed by finding that the partially restrained RC building has higher energy-dissipation capacity and ductility than those found for the fully rigid RC building.
d) Joint flexibility will increase the fundamental period of the structure. It was found that the first period of the building increased by 25% with respect to the building with rigid joint element.
e) Moreover, it was found that considering joint flexibility will increase the inter storey drift ratio of the building, but the drift will be still within the limits suggest by FEMA356.

From the above conclusions, considering the RC beam-column joint region as a rigid element will significantly overestimate the stiffness of building and miss-calculate the structural responses under earthquake loading. Therefore, to understanding Reinforced concrete joint-flexibility is essential to design more economical Reinforced concrete buildings in the future and to retrofit existing RC structures that were constructed in Jordan for more than 50 years now, where most of these old structures have non-seismic joint detailing and conventional analyses. Assuming these joints are rigid may not reflect the realistic responses of those types of RC structures under earthquake loading.

As a future work, it's recommended to do more research to investigate the effect of height on the performance of partially restrained RC buildings. Also, more research is needed on the behavior of RC connections, as the characteristics of the connection change. Such research will help engineers to model the right behavior of RC connections and thus depict the actual behavior and stiffness of RC buildings under lateral load.

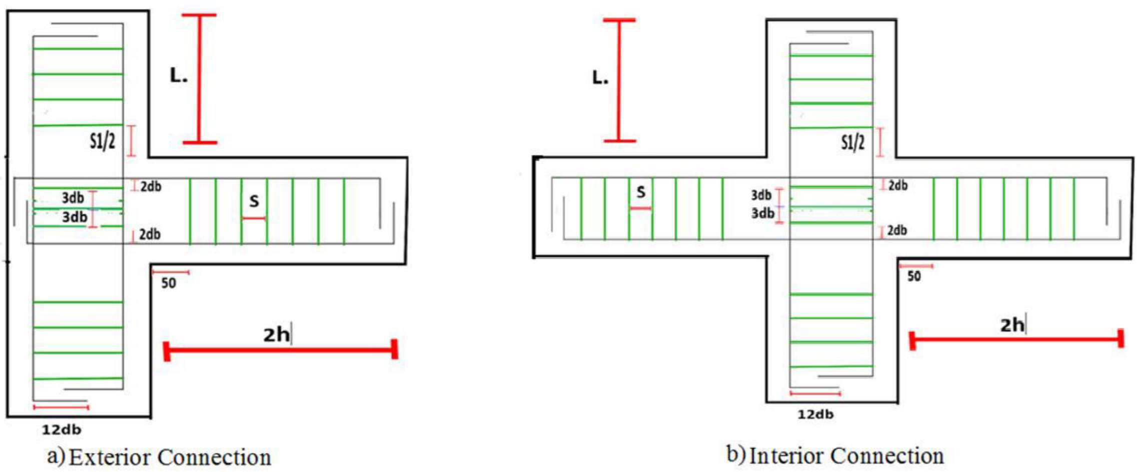

Figure-1. General dimension and reinforcement details for the joints considered in this study.

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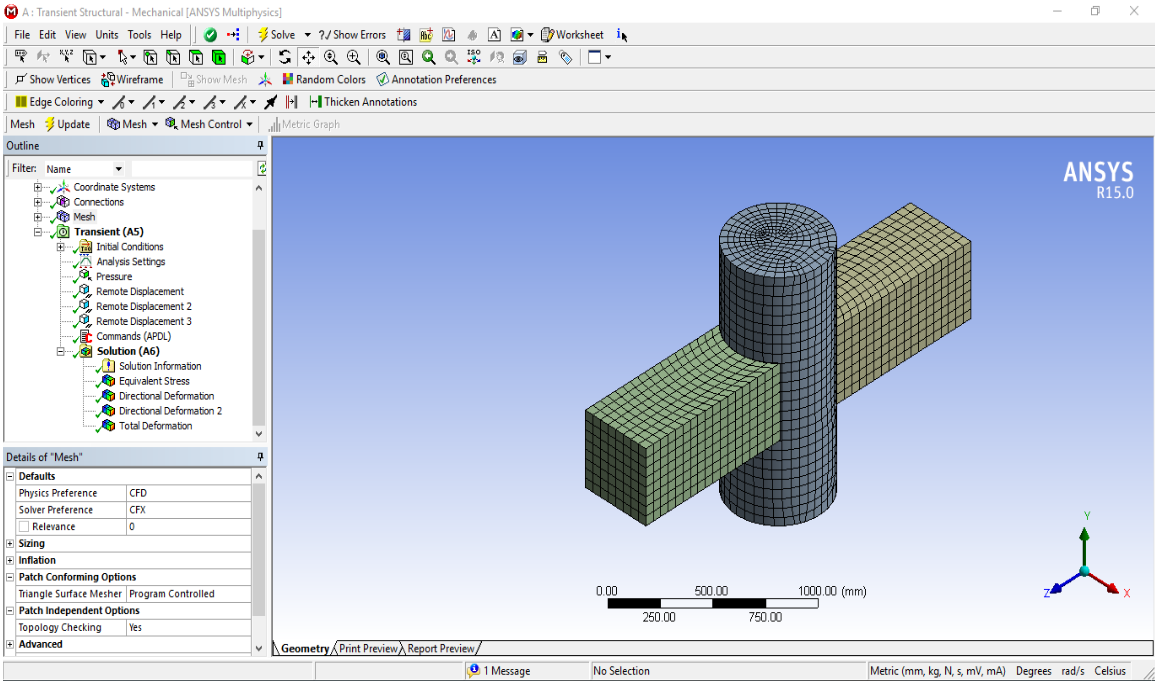

Figure-2. Screen shot of the model developed by ANSYS.

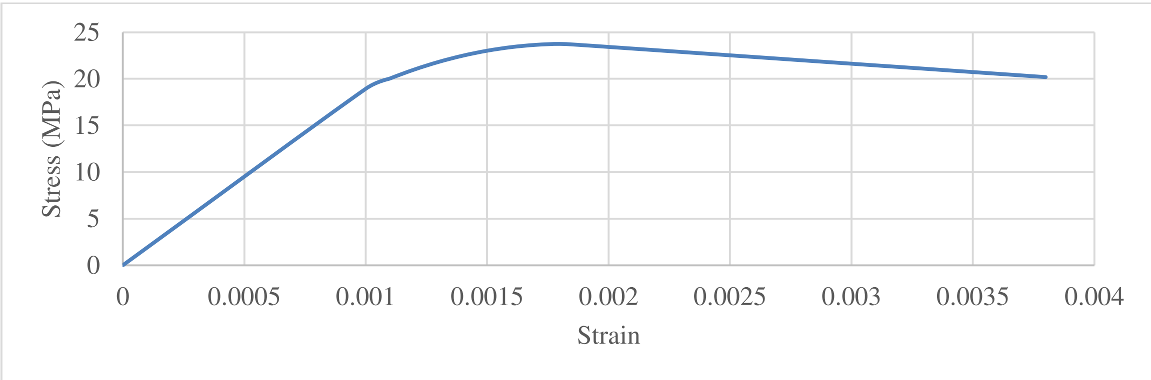

Figure-3. Stress-strain curve for concrete material.

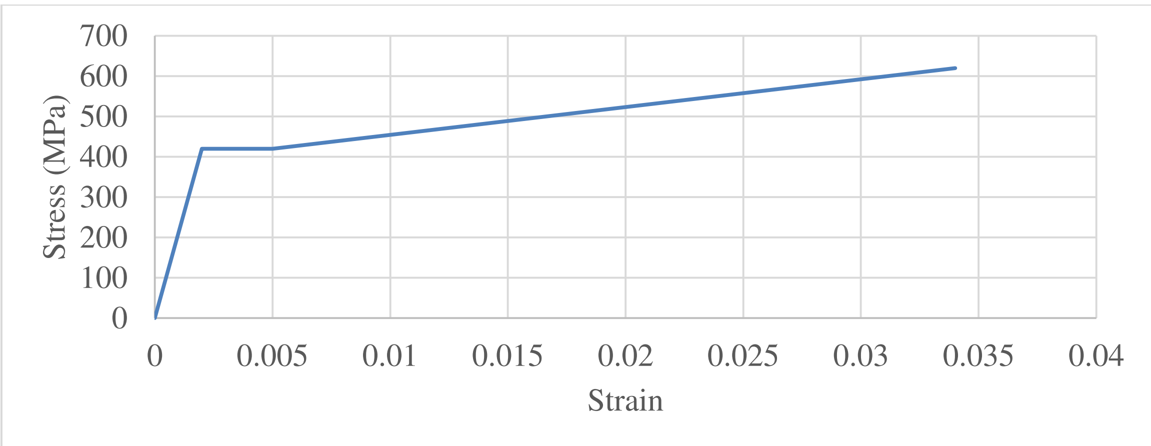

Figure-4. Stress-strain curve for steel material.

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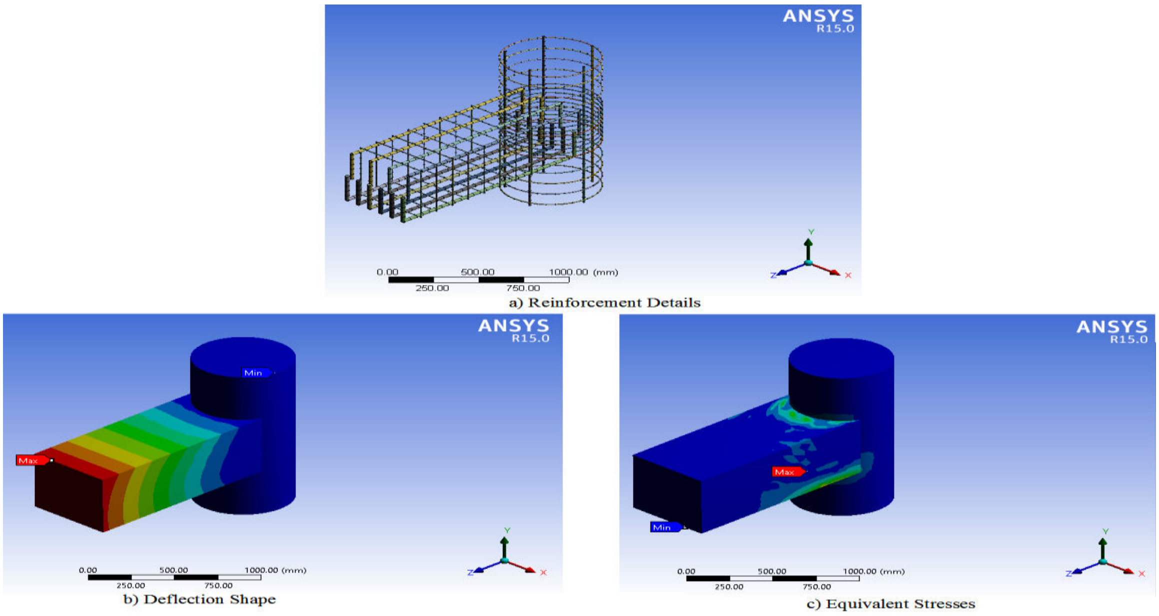

Figure-5. First floor exterior joint model in ANSYS.

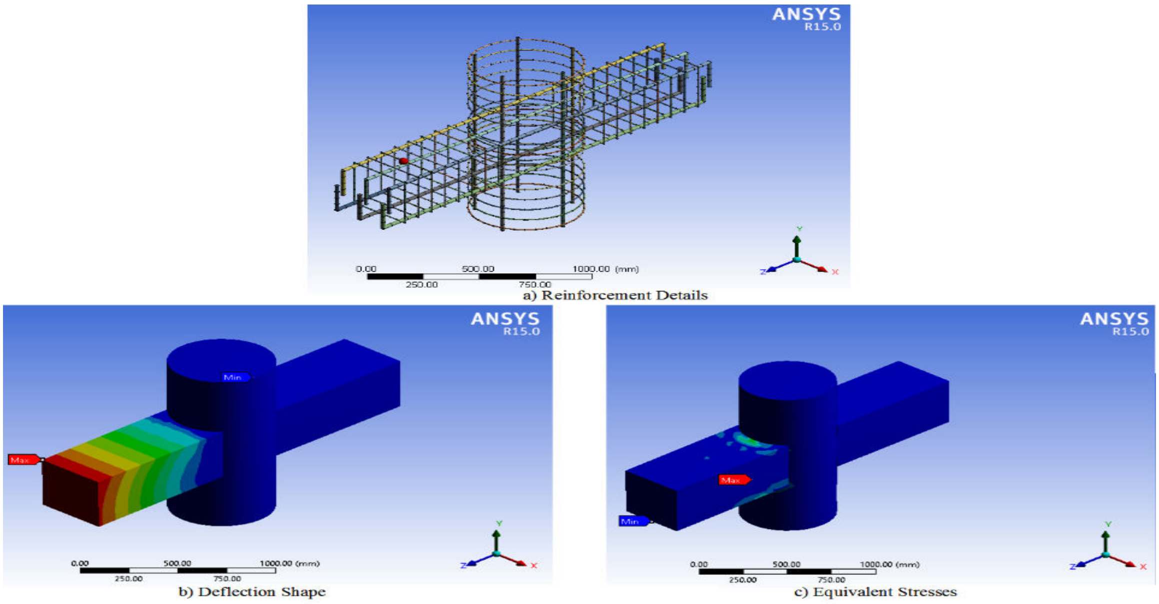

Figure-6. First floor interior joint model in ANSYS.

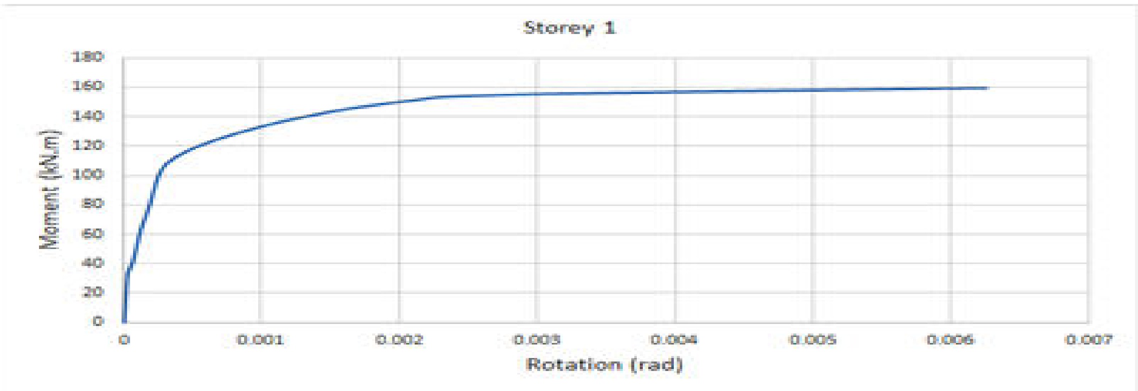

Figure-7. Moment-rotation curve for exterior connections.

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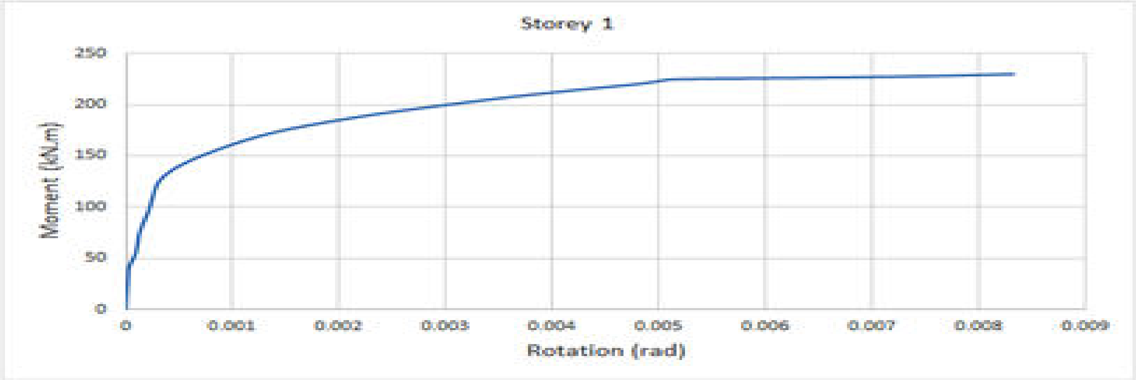

Figure-8. Moment-Rotation curve for interior connections.

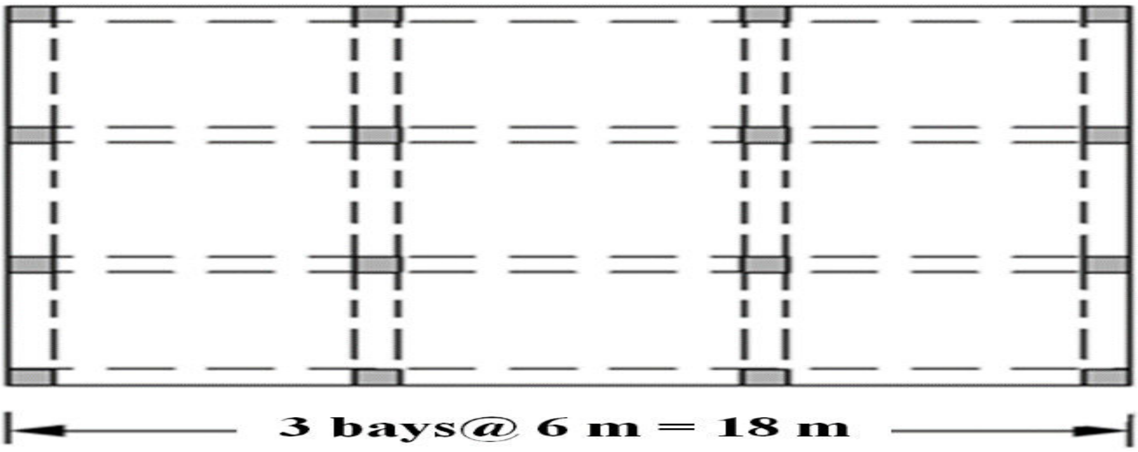

Figure-9. Floor plan of RC space moment frame building.

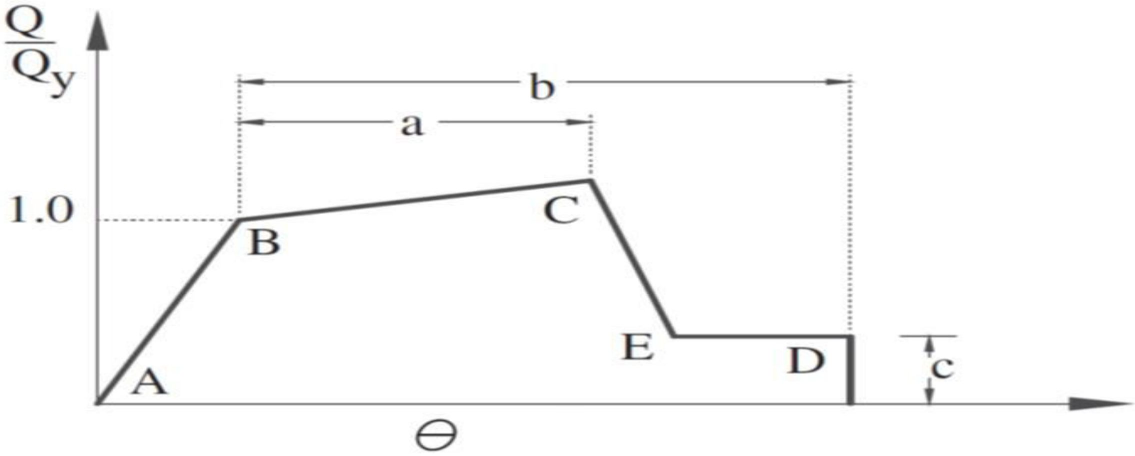

Figure-10. Recommended joint moment-rotation curve for joint modelling (Rathod and Dyavanal, 2014).

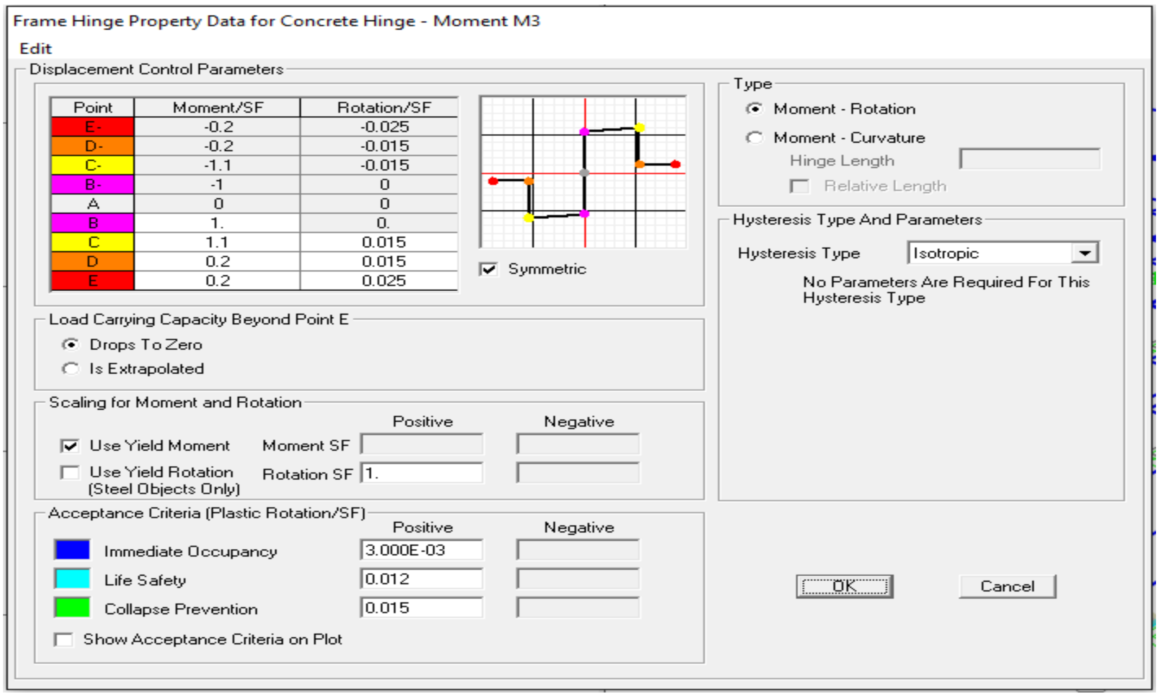

Figure-11. Concrete Hinge properties as defined by FEMA 356.

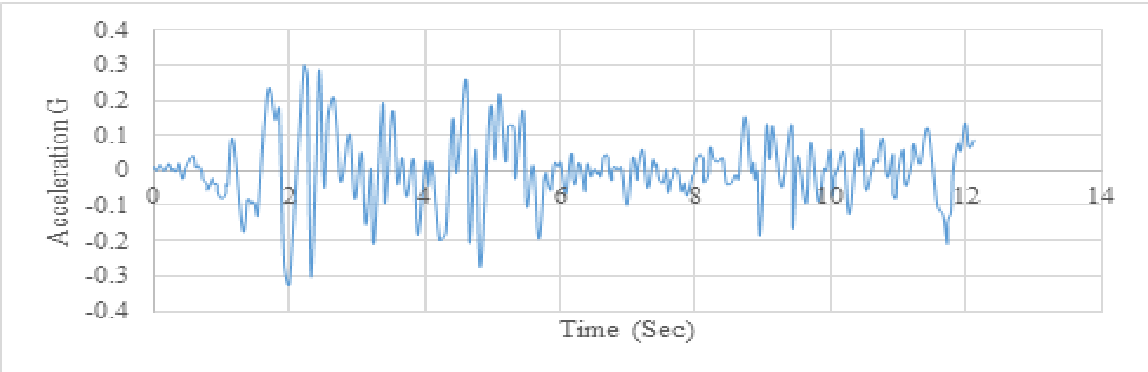

Figure-12. Acceleration of El-Centro earthquake.

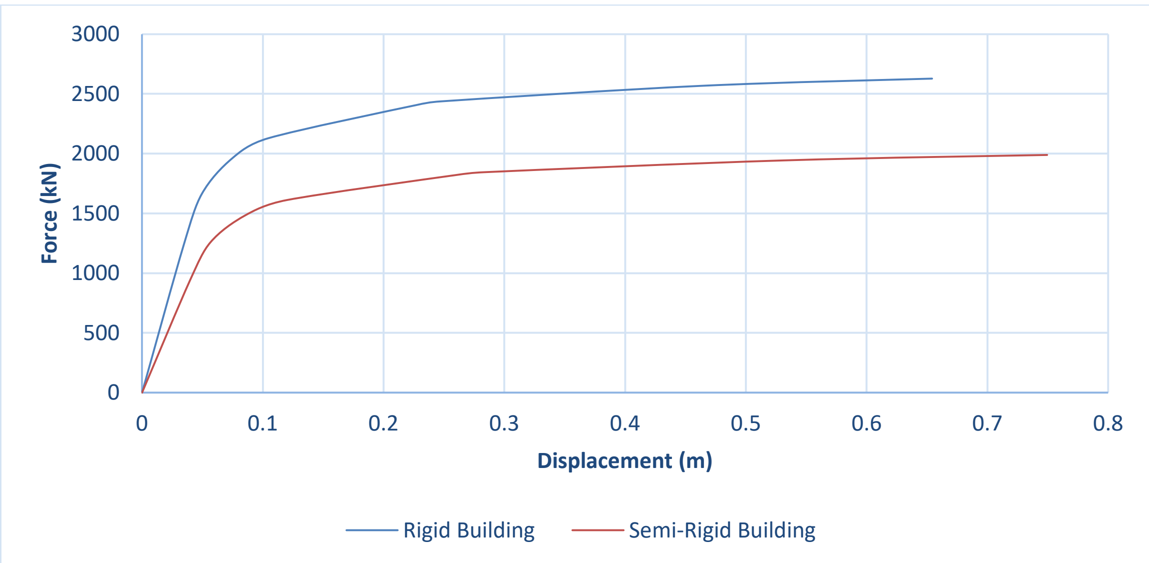

Figure-13. Pushover Curve for both rigid and semi-rigid 8 stories buildings.

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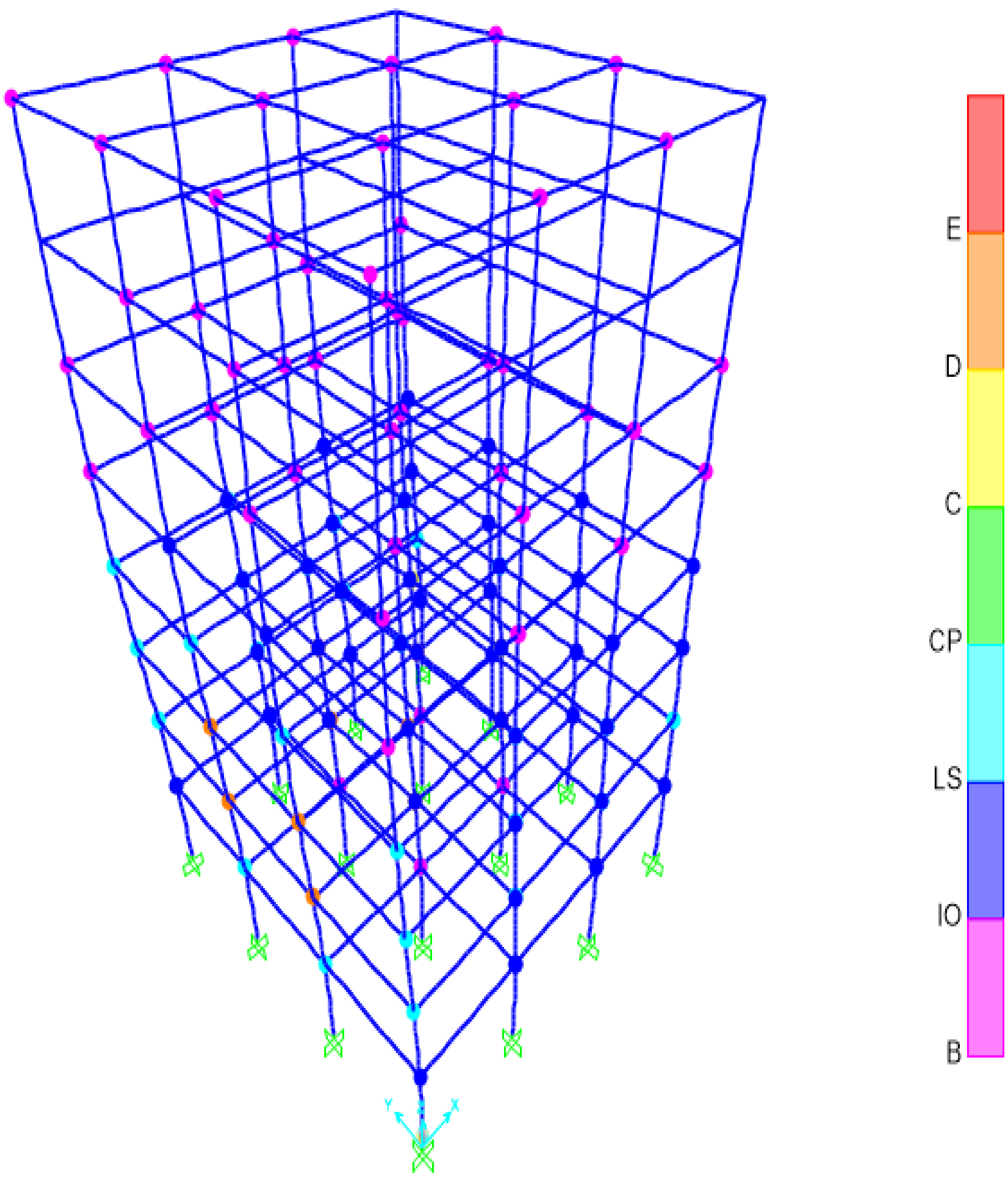

Figure-14. Failure mechanism in the 8 stories fully rigid building.

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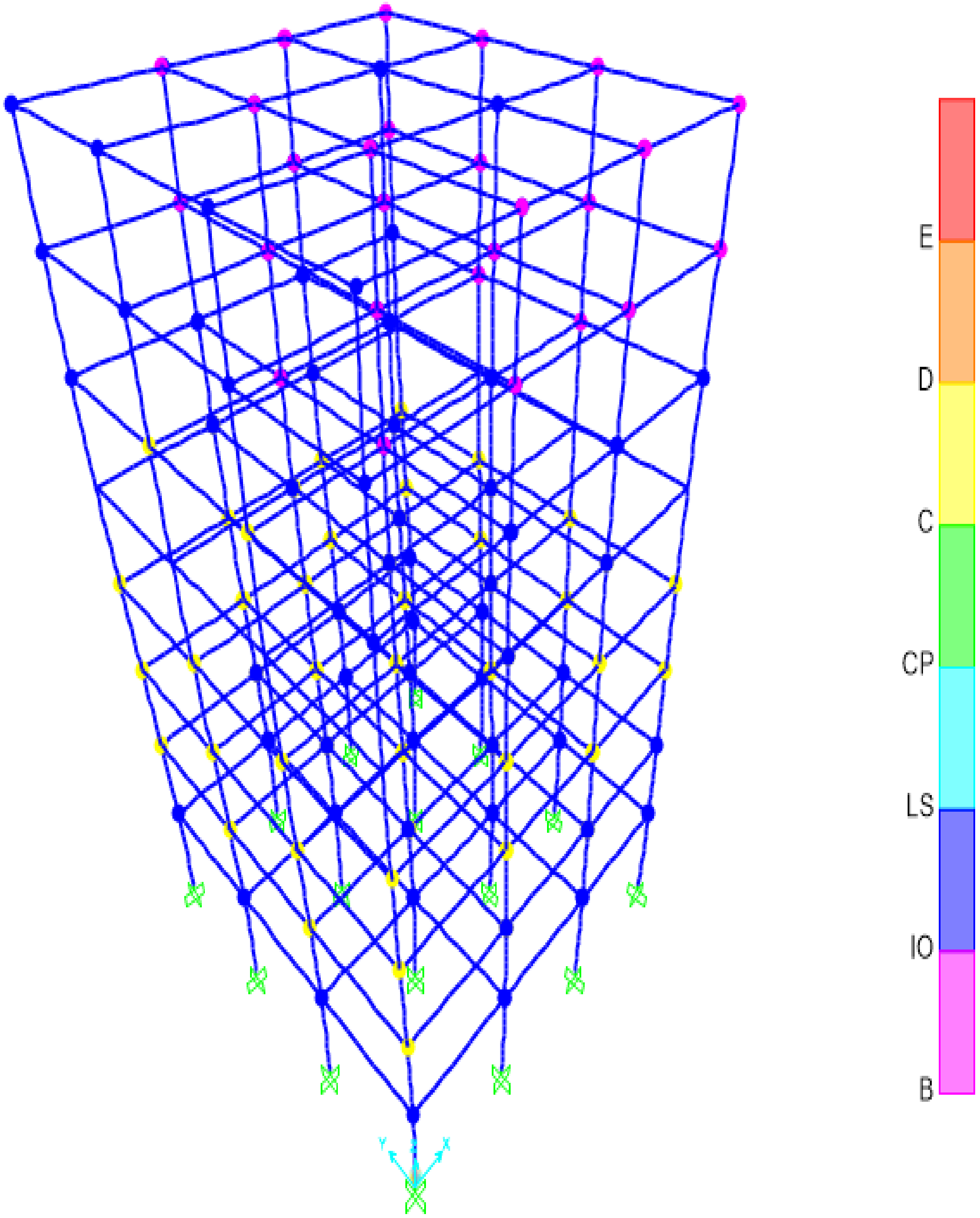

Figure-15. Failure mechanism in the 8 stories partially rigid building.

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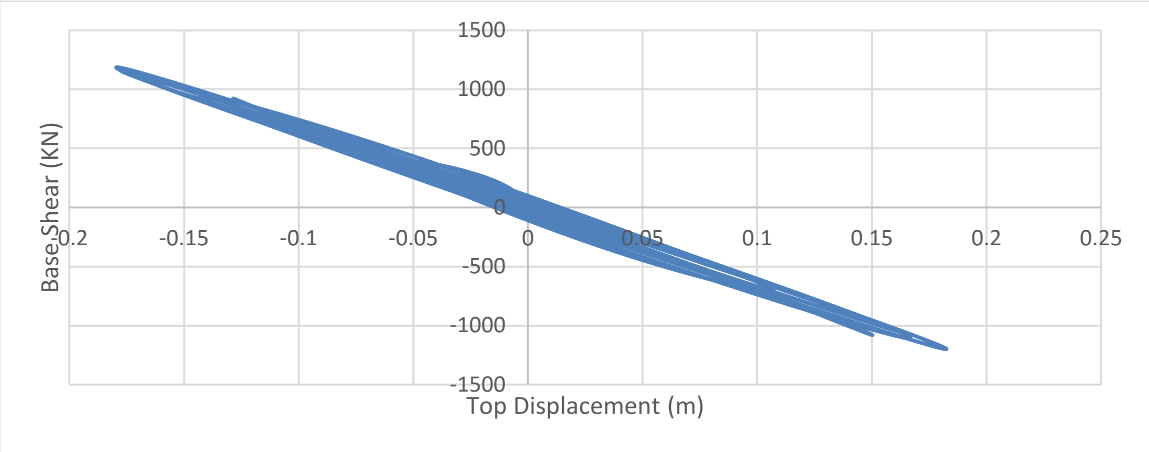

Figure-16. Hysteresis loop for the 8 stories fully rigid case study building.

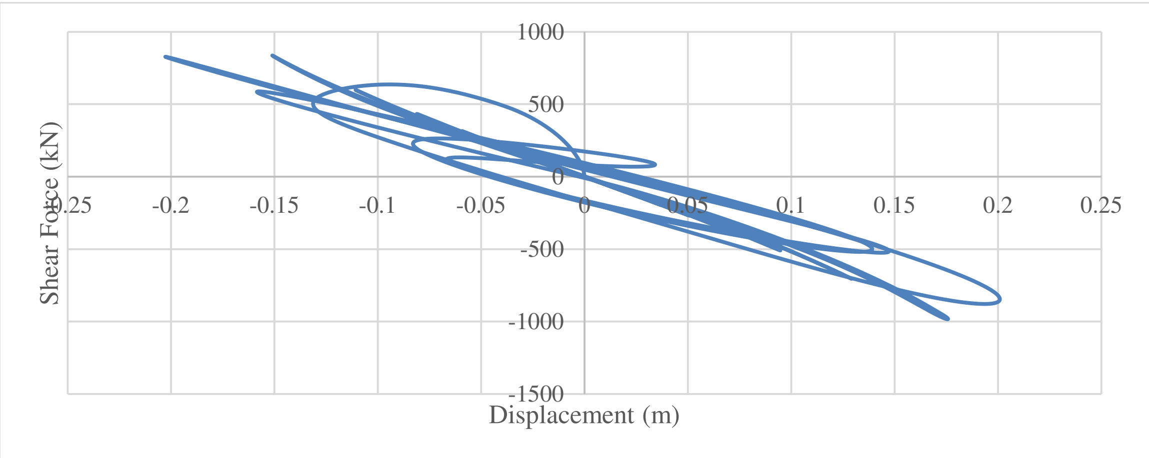

Figure-17. Hysteresis loop for the 8 stories partially rigid case study building.

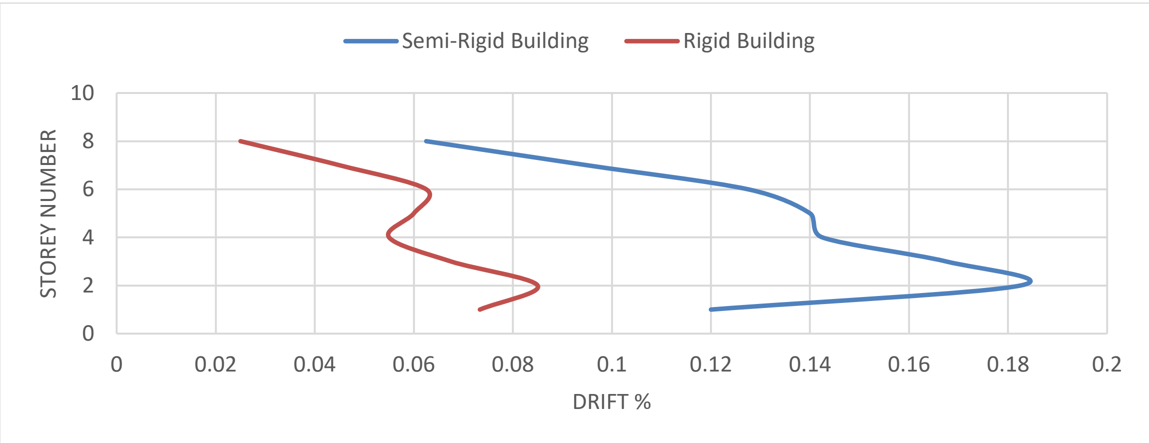

Figure-18. Inter-storey drift ratio.

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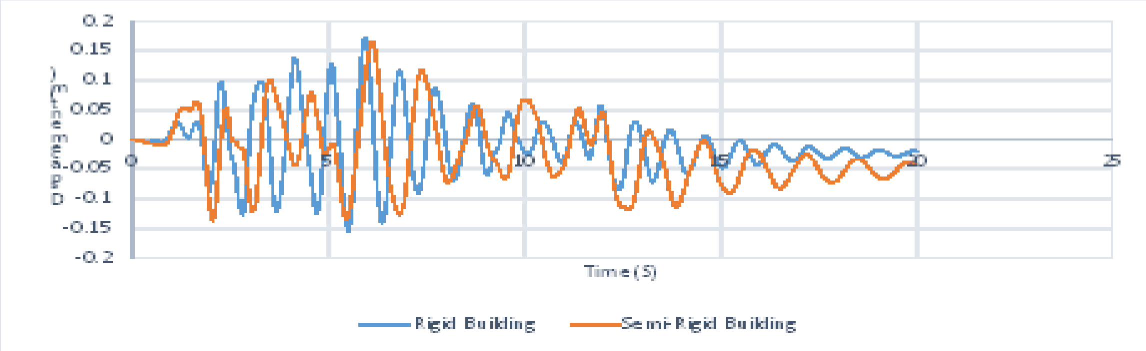

Figure-19. Displacement history for rigid and semi-rigid buildings.

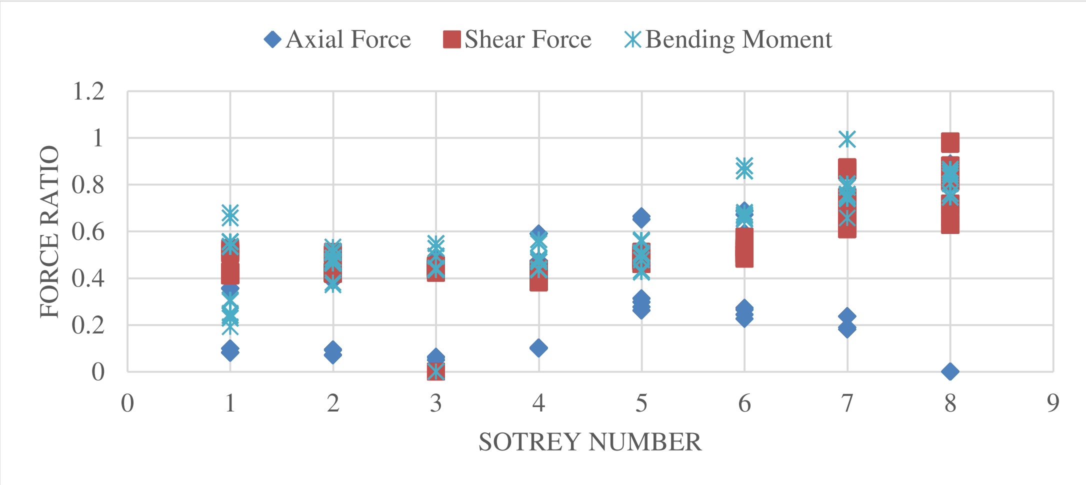

Figure-20. Forces ratio in columns for the semi-rigid to rigid buildings.

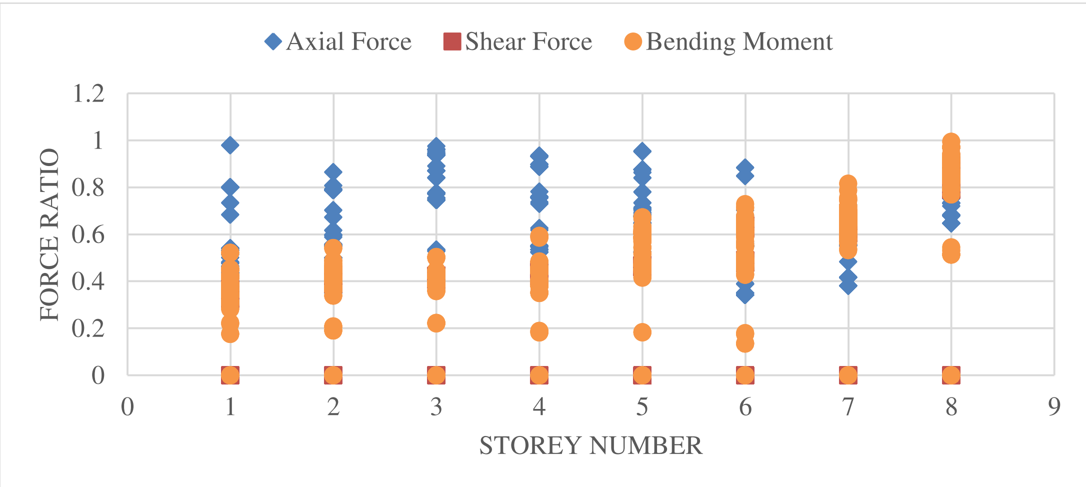

Figure-21. Forces ratio in beams for the semi-rigid to rigid buildings.

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Table-1. Reinforcement details for the interior RC connection.

Beam square section (mm)	Number of Tension Reinforcement	Selected Bar Diameter (mm)	Number of Compression Reinforcement	Selected Bar Diameter (mm)	column diameter (mm)	Number of Longitudinal reinforcement	Longitudinal reinforcement diameter (mm)
406.4	3	20	3	22	558.8	5	16
406.4	3	22	5	22	558.8	6	20
406.4	4	22	5	25	558.8	6	20
406.4	4	22	6	25	558.8	11	35
558.8	3	22	5	25	609.6	10	32
558.8	4	22	5	25	609.6	9	28
558.8	4	22	6	25	609.6	9	28
558.8	4	22	6	25	609.6	9	28

Table-2. Geometry details for the interior RC connections.

S	2h	3db	0.5S1	Lo	2db	12db
86.6	812.8	48	48	450	32	192
86.35	812.8	60	50	450	40	240
86.35	812.8	60	50	450	40	240
86.35	812.8	105	50	450	70	420
124.45	1117.6	96	50	450	64	384
124.45	1117.6	84	50	450	56	336
124.45	1117.6	84	50	450	56	336
124.45	1117.6	84	50	450	56	336

Table-3. Reinforcement details for the exterior RC connection.

Beam square section (mm)	Number of Tension Reinforcement	Selected Bar Diameter (mm)	Number of Compression Reinforcement	Selected Bar Diameter (mm)	column diameter (mm)	Number of Longitudinal reinforcement	Longitudinal reinforcement diameter (mm)
406.4	3	20	3	22	558.8	5	16
406.4	3	22	5	22	558.8	6	20
406.4	4	22	5	25	558.8	6	20
406.4	4	22	6	25	558.8	10	32
558.8	3	22	5	25	609.6	9	28
558.8	4	22	5	25	609.6	9	28
558.8	4	22	6	25	609.6	9	28
558.8	4	22	6	25	609.6	5	16

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Table-4. Geometry details for the exterior RC connections.

S	2h	3db	0.5S1	Lo	2db	12db
86.6	812.8	48	48	450	32	192
86.35	812.8	60	50	450	40	240
86.35	812.8	60	50	450	40	240
86.35	812.8	96	50	450	64	384
124.45	1117.6	84	50	450	56	336
124.45	1117.6	84	50	450	56	336
124.45	1117.6	84	50	450	56	336
124.45	1117.6	48	48	450	32	192

Table-5. Bending stiffness of exterior RC Connections.

Joint Floor	Stiffness from ANSYS Model (kN.m)	Stiffness range recommended by ASCE (kN.m)	Stiffness recommended by Fema (kN.m)
8	30,812	16,949 - 39,547	16,949 - 28,248
7	31,598	16,949 - 39,547	16,949 - 28,248
6	35,413	16,949 - 39,547	16,949 - 28,248
5	37,115	16,949 - 39,547	16,949 - 28,248
4	64,000	60,585 -141,366	60,585 -100,975
3	74,904	60,585 -141,366	60,585 -100,975
2	75,000	60,585 -141,366	60,585 -100,975
1	75,547	60,585 -141,366	60,585 -100,975

Table-6. Bending stiffness of interior RC Connections.

Joint Floor	Stiffness from ANSYS Model (kN.m)	Stiffness range recommended by ASCE (kN.m)	Stiffness recommended by Fema (kN.m)
8	31,379	16,949 - 39,547	16,949 - 28,248
7	35,087	16,949 - 39,547	16,949 - 28,248
6	37,974	16,949 - 39,547	16,949 - 28,248
5	39,413	16,949 - 39,547	16,949 - 28,248
4	67,000	60,585 -141,366	60,585 -100,975
3	75,680	60,585 -141,366	60,585 -100,975
2	77,523	60,585 -141,366	60,585 -100,975
1	77,523	60,585 -141,366	60,585 -100,975

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Table-7. Ranges of design parameters for the 8 stories model.

Design Parameters	Range Considered in Archetype Design
Structural System	
Reinforced Concrete Moment Frame (as per 2003 IBC, ACI 318-05)	All designs meet code requirements
Seismic design level	Design Category D
Seismic framing system	Space frames
Configuration	
Building height	8 Stories
Bay width	6 m
First story and upper story heights	4.5/4 m
Element Design	
Confinement ratio (ρs) and stirrup spacing (s)	Conforming to ACI 318-05.
Concrete compressive strength	25 MPa
Loading	
Design floor dead load	8.5 KN/m2
Design floor live load	Constant 2.5 KN/m2

Table-8. Section design details of beams for 8 stories building.

Floor	Section Depth (mm)	Section width (mm)	Tension reinforcement ratio (%)	Compression reinforcement ratio (%)
8	405	405	0.42	0.66
7	405	405	0.54	1.13
6	405	405	0.71	1.45
5	405	405	0.82	1.68
4	560	560	0.37	0.75
3	560	560	0.4	0.8
2	560	560	0.42	0.83
1	560	560	0.43	0.87

Table-9. Section design details of Columns for 8 stories building.

	Floor	dc (mm)	Longitudinal reinforcementsize (#)	Longitudinal reinforcement Numbers	Reinforcement ratio (%)
Exterior Column	Roof	560	5	16	1.025
	8	560	6	16	1.455
	7	560	6	20	1.818
	6	560	10	8	2.099
	5	610	9	8	1.389
	4	610	9	8	1.389
	3	610	9	8	1.389
	2	610	5	24	1.292
Interior Column	Roof	560	5	16	1.025
	8	560	6	20	1.818
	7	560	6	24	2.182
	6	560	11	8	2.579
	5	610	10	8	1.764
	4	610	9	12	2.083
	3	610	9	12	2.083
	2	610	9	12	2.083

Table-10. Fundamental Period for 8 stories rigid and semi-rigid building.

Mode Number	Semi-Rigid Frame	Rigid Frame
1	1.31487	0.88794
2	1.31487	0.88794
3	1.133	0.80873
4	0.52096	0.37717

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