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# Performance Evaluation of RC Building Subjected to Repeated Earthquake Load

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## ABSTRACT

This study evaluates the performance of an RC frame structure using nonlinear static and nonlinear dynamic analysis procedures. To achieve this objective, five-moment resisting frames with 4, 8, 12, 16, 20 storied buildings were analyzed and designed following the guidelines of the seismic codes was subjected to single, double, and triple earthquake events, that is, repeated earthquakes. The assessment of the structure in terms of the failure of members and the performance of the structure in terms of displacement and ductility was measured for different earthquake events, which was then converted into a multiplying factor. These seismic performance factors were used to increase the strength and stiffness of the structures at various locations. These factors were used for the design of an earthquake force-resisting system in a new building. In this study, the performance of a building subjected to a maximum considered earthquake (MCE) and for a repeated earthquake is checked and applied to the revised design procedure of the structure. By considering different performance points of the structure when subjected to repeated earthquakes, a new design philosophy was introduced. The building was designed using this new philosophy, and the structural stability of the structure was verified by applying repeated earthquakes.

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## 1. Introduction

To check the performance of a building subjected to a real earthquake, the designer must go outside the codal procedure and accurately predict how the structure will behave under an earthquake or often during an extreme event. In reality, earthquake soccur in series, therefore, an actual response or performance study is essential. Repeated earthquakes have given rise to unprecedented harm or destruction to both life and property because of the short time interval, it is impossible to repair the structure and perform the movement of people. The collapse performance of the building was estimated to be the ground motion intensity related to the maximum considered earthquake (MCE). The multistoried building was designed by considering the design response spectrum taken as two-thirds of the maximum considered earthquake (MCE), which means that the structure was designed for the design response spectra of various earthquakes, and the performance of the structure was checked for MCE.

In real life, the building is shaking with multiple earthquakes in a life span of structure and subjected to MCE, Therefore the performance of the structure under these MCE is studied, Sachin Patil et al. 2021 [1] study the effect of the repeated earthquake and conclude if the building is designed using the defined procedure of different codes i.e. with the application of single earthquake, assessment result of the structure are not correct, so for the realistic, correct assessment of the structure and to find the actual performance of the structure, the structure is analyzed and design with considering repeated earthquake effect. A repeated earthquake means before shock-mainshock- aftershock or mainshock- aftershock or beforeshok- mainshock. A mainshock means larger magnitude earthquakes and beforeshok or aftershock earthquakes are of smaller magnitude. Current practice codes, such as IS1893-2016 [2], EUROCODE 8 [3], FEMA 368 [4], and take only a single design earthquake for the analysis and design of a structure that provides only limited values of displacement, drift, ductility, and performance of the structure; however, in reality, these values will be more for actual earthquake sequences that are applied to multistoried structures.

The performance evaluation process considers the result from nonlinear static (pushover) analysis, that is, the values of the structure over strength factor Ω0, period-based ductility $\mu_{T,}$ and displacement values from nonlinear dynamic (time history ) analysis, that is,. Acceptable values of the response of the structure in terms of displacement, drift, modification factor, ductility, and so on.

Structural assessment is the finding of a major structural or functional deficiency in the structure after the shaking of the earthquake. This finding includes the degradation in structure, stiffness reduction, loss of equilibrium of the structure, or parts of its unacceptable deformations in the structure.

Few researchers have deliberated on the effect of repeated earthquakes on multistoried structures. Amedio et al. (2003) [5] studied the sequential earthquake effect on a structure and confirmed that because of repeated earthquakes the damage level of the structure increases. George Hatzigeorgiou et al. (2009 [6]) showed that an increase in the force reduction factor leads to a rise in the inelastic displacement ratio and vice versa. Mohd Zulham Affandi bin Mohd Zahid et al (2012) [7] studied the effect of a repeated earthquake on near-field multistoried structures and demonstrated that structural response quantities, that is, displacement ductility and story ductility displacement ductility demand, increase. George Hatzigeorgiou et al (2010) [8] showed that repeated earthquakes increase displacement demand in comparison with a single earthquake and that seismic damage is higher in the case of a repeated earthquake than a single effect.

This study investigates the effect of repeated earthquakes on a multistoried structure and determines the performance of the structure in terms of fundamental periods, maximum story displacements, interstory drift ratio, base shear of the building, over strength factor, ductility, and ductility of the structure.

## 2. Description of model & ground motion data

### 2.1. Explanation of structure and modeling criteria

Structural modeling. Here, five different types of structures were considered to represent low-to high-rise structures for analysis and design purposes. The model has 4-storied, eight-, 12-, 16-, and 20 -storied buildings with beam-column RC frames, that is, moment-resisting frames without a shear wall.

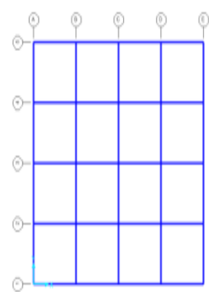
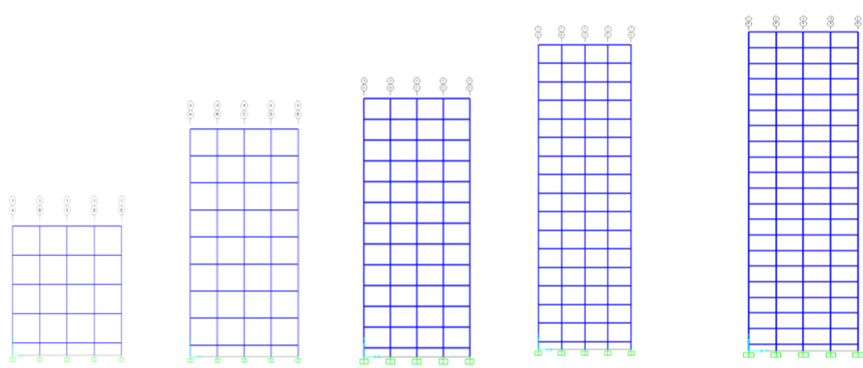
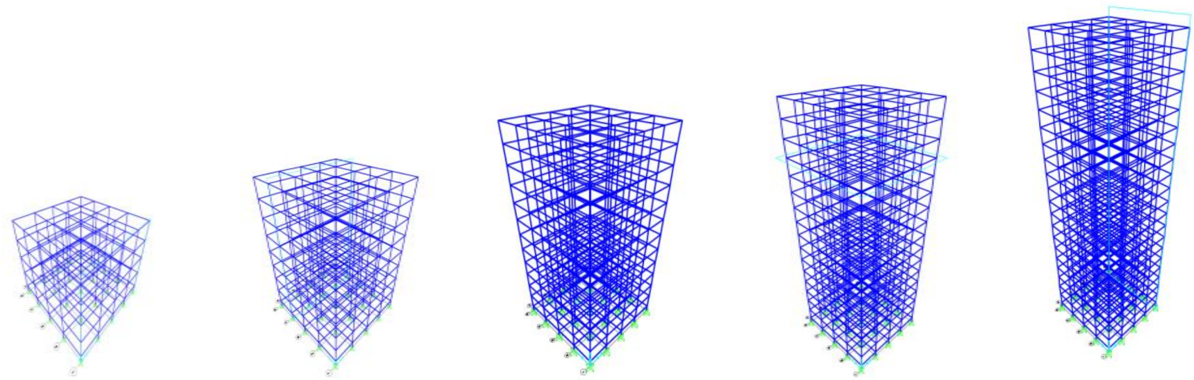


**Fig. 1.** Plan of a Structure.



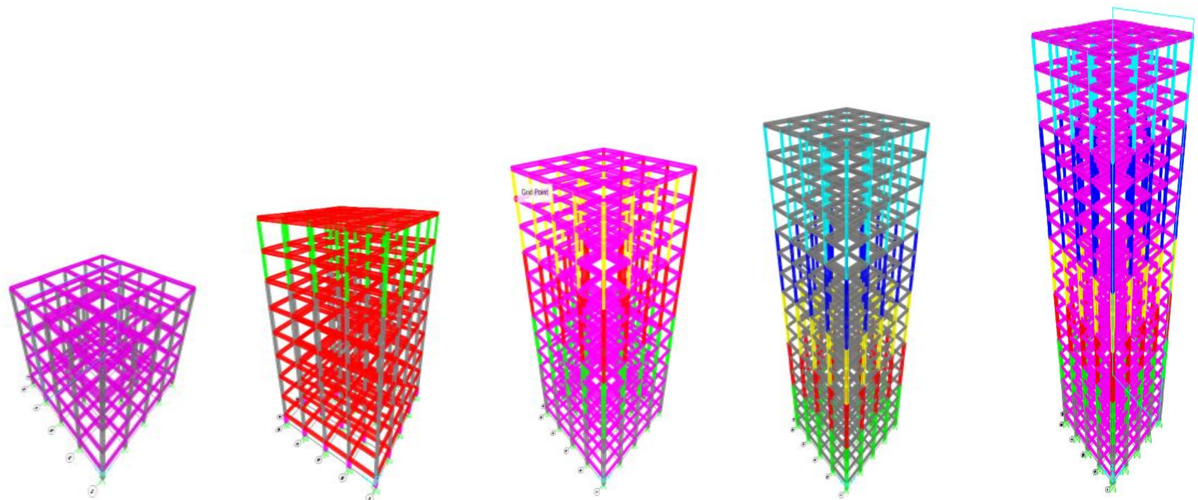
a) 4-Story b) 8-story c) 12 -Story d) 16-Story e) 20-Story

**Fig. 2.** 2D Framed Structure.



a) 4-story b) 8-story c) 12-story d) 16-story e) 20-story

**Fig. 3.** 3D Framed Structure.



a) 4-story b) 8-story c) 12-story d) 16-story e) 20-story

**Fig. 4.** 3D Framed Structure with Section Details.

To perform linear and nonlinear dynamic analysis, the model of the 4-, 8-, 12-, 16-,20- storied building structure is taken and defined as 4 X bays of size 5m X 5m with the same height from the ground floor to the top floor of 3.5 m, and the hard start is available at 1.5 m from the ground, therefore in the model the footing is shown at 1.5 m below ground level. As per the IS code Zone - V, with zone factor = 0.36, importance factor =1.5, and response reduction factor R= 5, the M40 grade of concrete and steel is Fe415. The building is a commercial building, located in a high-seismic zone in India. Fig.1, 2, 3 shows a plan for building, 2D and 3D models of five types of structures. Fig.4 shows the dimensional changes in the building sections.

The planned model is a 3D regular model, a 3-D frame regular in plan, and elevation is considered because RC special moment-resisting frames that are regular in the plan are not sensitive to torsional effects. Therefore, the irregularities in a plan, elevation, mass of the structure, and vertical geometry are not taken into consideration. In this study, a strong column – weak beam concept is assumed; therefore, the design of the structure should satisfy Eurocode 8 [5] ductility state at the joint of the beam and column.

$$\sum \boldsymbol{M}Rc \geq 1.3 \sum \boldsymbol{M}Rb$$

where ∑MRc and ∑MRb are the quantities of the design values of the moments of resistance of the columns and the beam framing the joint, respectively. An over strength factor of (αu/α1) = 1.3 is taken for multi bay and multistory structures, as per Eurocode 8. The stiffness reduces by increasing the height of the structure (F. Dorri et. al 2019) [9] the reduction of stiffness along the height of the building is followed by the method by E. Mirinda and Reyes (2002) [10]. For the correct distribution of lateral stiffness, a reducing stepwise distribution of lateral stiffness, which followed a parabolic stiffness distribution, was used in the study. The lateral stiffness of the structure changed every three stories. Fig. 4 shows a 3D model of the structure that shows the column section of the structure changes at every fourth floor. To simulate the cyclic behavior and to check the stiffness degradation due to repeated earthquakes in RC buildings, this study adopts a modified Takeda hysteresis curve (fig.5), as proposed by Zarein and Krawinkler (2009) [11].

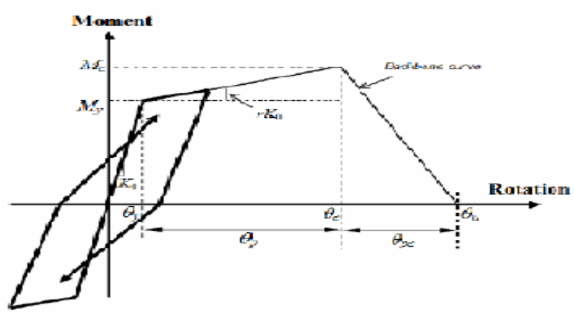



**Fig. 5.** Modified Takeda Hysteresis and backbone curve [11].

For the linear and nonlinear dynamic analysis, the structure is modeled with default hinges, due to assigning of default hinges there is no effect on the total base shear of the structure, yielding state of the structure. The model with default hinges highlights ductile beam behavior in which a strong column weak beam mechanism is followed, and the first failure occurs in a beam. (Mehmet Inel and Hayri) (2006) [12]. Based on a study by Mehmet, the default hinge properties of SAP2000 [13] are suitable for modern code-compliant buildings.

To check the performance of a building, two categories of structures were considered: short-period and long-period structures. In the short-period structure, 4 story building was used for analysis, and 8, 12, 16 20 story buildings were considered for long-period structures. The first, second, and third modal time periods for a structure are shown in Table 1. Above 20 story buildings are not taken into consideration because after 20 stories or in a taller building includes a shear wall or core wall, in addition to a moment-resisting frame, they are required to resist the lateral load.

**Table 1**
Structural Modal time period.

| Building | Total height (m) | Modal time period (Sec) | | |
|---|---|---|---|---|
| | | 1 | 2 | 3 |
| 4 story | 15.5 | 1.287 | 1.287 | 1.18 |
| 8 story | 29.5 | 2.25 | 2.25 | 2.07 |
| 12 story | 43.5 | 2.75 | 2.75 | 2.45 |
| 16 story | 57.5 | 3.50 | 3.50 | 3.13 |
| 20 story | 71.5 | 4.22 | 4.22 | 3.71 |

A 3D model of the structure was prepared in SAP2000 to perform nonlinear static and nonlinear dynamic analyses. The column and beam members were modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the column and beam. For defining the plastic hinges, the default hinge property is used and described in FEMA-356 [14] and shown in fig.6 five-point A, B, C, D, E define the force deformation behavior of plastic hinges and describe the IO limit for a primary and secondary member.LS and CP limit for a member. The default hinges for the columns were assigned as P-M2-M3, and for the beam, M3 hinges were assigned. In addition, the default hinge model was chosen because of its easiness of application.

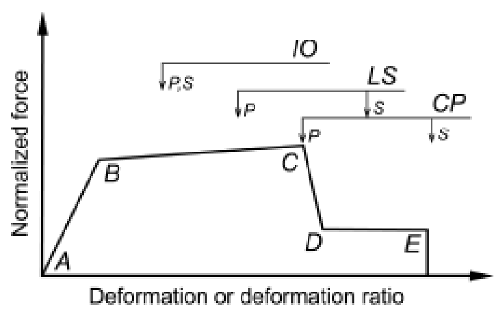



**Fig. 6.** Force -Deformation Relationship of a Plastic Hinge or Element Deformation Acceptance Criteria (FEMA 356).

To calculate the seismic loads, it is appropriate to consider the structure to be fixed at the base, as shown in Fig. 1, 2 & 3.The effective weight W of a structure includes the total dead load, wall load, slab load, and live load applied to the structure, as listed in Table 2.

**Table 2**
Description of structure.

| Building | Total height (m) | Yield Displacement(mm) | Base Shear (kN) at yield displacement | Weight of structure (kN) |
|---|---|---|---|---|
| 4 storied | 15.5 | 37.728 | 1211.319 | 28214.656 |
| 8 storied | 29.5 | 68.50 | 1574.14 | 53466.53 |
| 12 storied | 43.5 | 73.8477 | 1522.63 | 80509.74 |
| 16 storied | 57.5 | 97.366 | 1594.5727 | 107551.78 |
| 20 storied | 71.5 | 84.838 | 1692.85 | 136191.95 |

A mathematical model of the structure is developed to determine member forces and structural displacements resulting from applied loads the model includes the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure.

## 2.2. Selection of ground motion and seismic input method

For the collapse evaluation of the model and to determine the effect of a repeated earthquake based on the Hartzigeorgious (2010) [15] method, the ground motion was selected using the criteria given by Vamvatsikos D, Cornell CA (2002) [16], and Anil Chopra et al. [17] to satisfy the different criteria, as they include a strong motion record (i.e., PGA> 0.2) and a longer duration of shaking. In view of these points, the structure was modeled in SAP 2000, and the list of earthquakes downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) center is shown in Table. III. These records are well suited to hard-rock soils. Every earthquake ground-motion record from the PEER database is a single ground-motion record. The selected earthquake has different distances from the focus, and itis important to study the effects of near- and far-field earthquakes on the structure. In this study, different PGA values were selected for the earthquake. The PGA values range from 0.53 to 0.77 m/sec $^{2}$ are selected to apply on a structure. In this study, for analysis and design, a maximum considered earthquake (MCE) is considered. To analyze the structure for the study of the effect of the repeated earthquake a gap of 100 seconds between two earthquake events is applied. A gap of 100 s with a zero acceleration ordinate was provided. In100s the structures come to relaxation conditions owing to internal inherent damping.

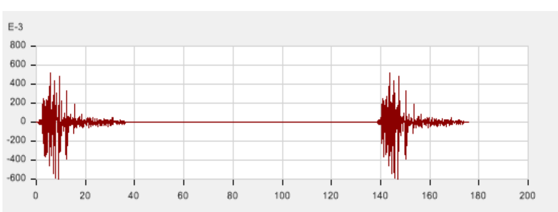



**Fig. 7a.** 2 GM of Duzce earthquake with 100 s gap.

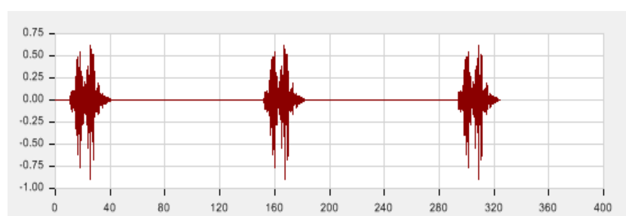



**Fig. 7b.** 3 GM of imperial valley 06 earthquake with 100 s gap.

The selected combination of repeated ground motion with a 100 s break is presented in fig 7. To perform dynamic time-history analysis and dynamic push-over analysis, this study adopts the combination of a real earthquake in two or three events. The same earthquake events are considered two or three times because real earthquakes occur in sequence, and there is a probability of repeating the same intensity or a larger or smaller intensity of an earthquake. To consider repeated earthquakes, a combination of earthquakes is performed. For this study, three cases were taken case-1, case-2, case -3. In case -1 single earthquake was applied to the structure. In case -2, two sequential earthquake shocks are applied to the structure with a time gap of 100 s, as shown in fig.7. Case no-3 has three earthquake shocks with a time gap of 100 s between three ground motions, as shown in Fig.7.In addition, fig.8 shows the response spectrum graph for the selected earthquake motion, which is used to calculate the model time period and horizontal load applied to the structure using the response spectrum method.

The model was analyzed and designed for different load combinations from the IS-1893:2016 Code and EUROCODE 8. The response spectra for the analysis and design of the structure were taken for different earthquakes, as listed in Table.

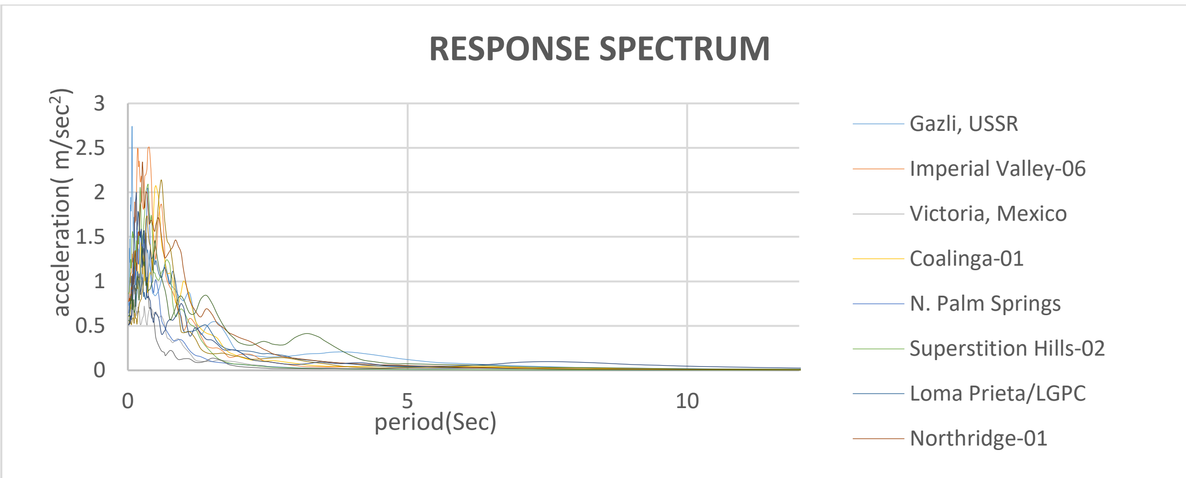



**Fig. 8.** Response spectrum graph for selected earthquake motion.

In addition, the structure has been designed for gravity loads, including a combination of dead load, wall load, and slab load for different load combinations, as per IS 1893:2016 [1]. Before applying the earthquake load, the structural stability was evaluated for different load combinations.

The extreme inelastic displacement was obtained by nonlinear time-history analysis, which was carried out on a 3D model excited by 12 types of seismic sequences.

The ductility $\mu_d$ is defined in terms of the maximum displacement $\boldsymbol{\mu_{max}}$ and the yield displacement μy.

$$\mu_d = \frac{\mu_{max}}{\mu_y}$$

The yield displacement was calculated using push-over analysis of the structure with SAP2000, and the maximum displacement of the structure for different single and multiple earthquakes was calculated using the nonlinear dynamic time history analysis method.

**Table 3**
Earthquake Events.

| SR. NO. | EVENT NAME | EVENT STATION | DATE | TIME | MAGNITUDE (MW) | DISTANCE FROM EPICENTER | MECHA NISM | PULSE TYPE | PGA |
|---|---|---|---|---|---|---|---|---|---|
| 1 | El Mayor-Cucapah | El Centro Differential Array | 4/4/2010 | 22:40:00 | 7.2 | 22.83 | Strike-Slip Fault | Non-Pulse | 0.53 |
| 2 | Northridge-01 | Castaic - Old Ridge Route | 17/01/1994 | 12:31:00 | 6.69 | 20.11 | Reverse Fault | Non-Pulse | 0.57 |
| 3 | Victoria, Mexico | Cerro Prieto | 9/6/1980 | 3:28:00 | 6.33 | 13.8 | Strike-Slip Fault | Non-Pulse | 0.63 |
| 4 | Chuetsu-oki | Oguni Nagaoka | 16/07/2007 | 10:13:00 | 6.8 | 10.31 | Reverse Fault | Non-Pulse | 0.57 |
| 5 | Coalinga-01 | Pleasant Valley P.P. - yard | 2/5/1983 | 23:42:00 | 6.36 | 7.69 | Reverse Fault | Non-Pulse | 0.6 |
| 6 | Superstition Hills-02 | Superstition Mtn Camera | 24/11/1987 | 13:16:00 | 6.54 | 5.61 | Strike-Slip Fault | Non-Pulse | 0.73 |
| 7 | Christchurch, New Zealand | Christchurch Botanical Gardens | 21/02/2011 | 23:51:00 | 6.2 | 5.52 | Reverse-Oblique Fault | Non Pulse | 0.55 |
| 8 | Duzce, Turkey | Lamont 375 | 12/11/1999 | - | 7.14 | 3.93 | Strike-Slip Fault | Non-Pulse | 0.7 |
| 9 | Gazli, USSR | Karakyr | 17/05/1976 | - | 6.8 | 3.92 | Reverse Fault | Non-Pulse | 0.7 |
| 10 | Imperial Valley-06 | Bonds Corner | 15/11/1979 | 23:16:00 | 6.53 | 0.44 | Strike-Slip Fault | Non-Pulse | 0.77 |
| 11 | N. Palm Springs | Whitewater Trout Farm | 8/7/1986 | 9:20:00 | 6.06 | 0 | Reverse-Oblique Fault | Non Pulse | 0.63 |
| 12 | Loma Prieta/LGPC | LGPC | 18/10/1989 | 0:05:00 | 6.93 | 0 | Reverse-Oblique Fault | Non Pulse | 0.59 |

## 3. Result and discussion

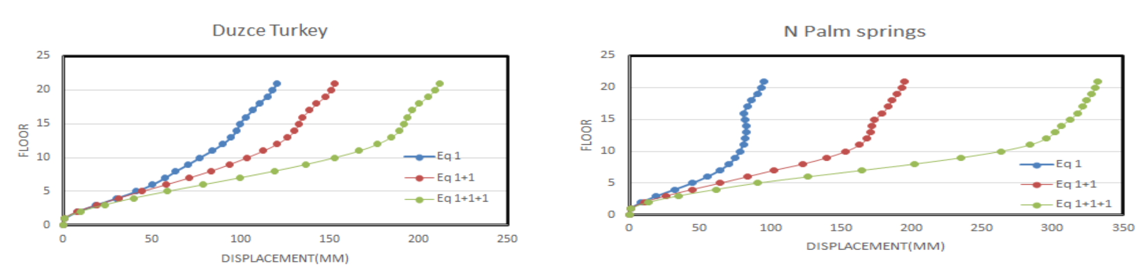
The Indian seismic code-defined peak ground acceleration (PGA) for the maximum considered earthquake (MCE) is **0.36 g** and the design basis earthquake (DBE) is 0.18 g for the service life of a structure; for the highest seismicity area, Zone V is considered. In this study, we considered 12 different earthquakes, as shown in Table 1. The selected PGA ranged from 0.5 to 0.8, and the distance from the epicenter was 0 to 23 km to check the effect of a repeated earthquake on the structure and the failure nature of the structure.

### 3.1. Maximum horizontal displacement of structure

The maximum horizontal displacement for 4-,8-,12-,16-,20 storied buildings and for single, double, and triple events are presented in fig.9 to 13. In the present work, above-ground motion records are applied with a combination of EQ1, that is, a single earthquake ground motion is applied to the structure, and performance is checked.EQ1+0+1 means that after completion of EQ1 ground motion, a gap of 100 s is provided to come to the structure in the rest condition, and then the same earthquake ground motion is applied with continuity, that is, considered as a double earthquake, and performance is checked; similarly, EQ1+0+1+0+1 means that after completion of the second event, a third earthquake is applied to the structure and the structural displacement and failure nature of the structure. Between two successive earthquakes, a time gap of 100 s was applied to come to the structure to relax the condition and find the permanent displacement and position of the plastic hinges. The effects of these repeated earthquakes for different earthquake events are shown in fig 9–13. the fig. 9 to 13 show the displacement of a4,8,12,16,20 storied building & the effect of a single, double, and triple earthquake. Fig. 9 to 13 show that the displacement increases for the second earthquake and for a triple earthquake. At the end of the first earthquake, the building achieved a displacement, and there was also a permanent displacement in the structure. A similar pattern was observed after the end of the second and third earthquake effects. For 20 storied buildings, it is seen that at the end of the third earthquake, the building has a larger displacement and can collapse, which means that the structure cannot sustain the third earthquake; the same pattern is seen in 16 storied and 12 storied buildings. In 8storied and 4 storied buildings, the displacement has a larger value but cannot collapse, which means that small-height buildings can sustain more earthquakes than tall buildings. A comparison of a single double and triple event of earthquakes shows that the displacement can be increased by 20 to 30 % for a double event and 30 to 50% for a triple event, or the structure can be collapsed at the end of a triple event.

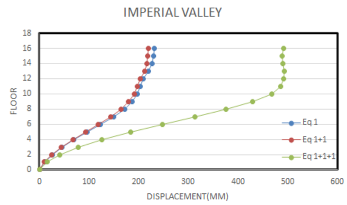
The above result shows that the displacement increases with the number of shocks and the Fig. 9, 10, 11, 12, 13 are identical for a single, double, and triple event. Earthquakes that occur in fields close to a fault are called near-field earthquakes. (UBC-97 Code) [18] Considered a distance of less than 15 km from the earthquake epicenter as the near-field range. Also from Fig. 9, 10, 11, 12, 13, it is clear that if the structure is near the fault, that is, the rupture distance is minimal, the structure cannot resist the third earthquake. In addition, if the structure is near the fault, it has a large displacement and drift compared to a far-field earthquake. The Fig. 9, 10, 11, 12, 13 shows that if the structure is near the fault, its displacement is large.

**Maximum Horizontal Displacement Graphs**

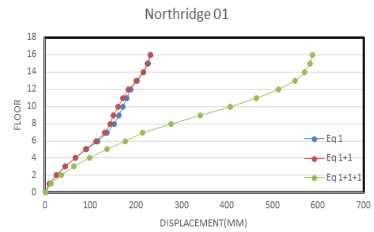



a) Displacement of Duzce Turkey earthquake b) Displacement of N Palm spring earthquake

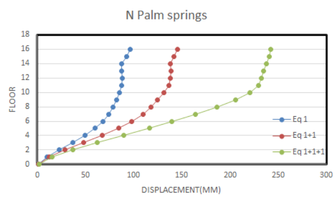
**Fig. 9.** Maximum Horizontal Displacement for 20 storied building.



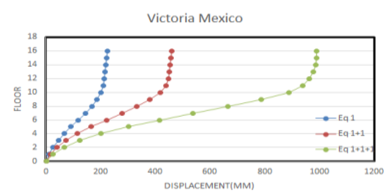

a) Displacement of Imperial Valley earthquake




b) Displacement of Northridge 01 earthquake

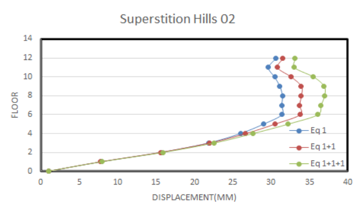



c) Displacement of N Palm springs earthquake

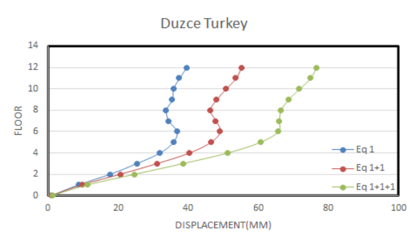



d) Displacement of Victoria Mexico earthquake

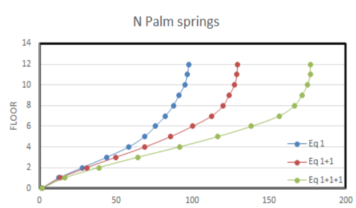
**Fig. 10**. Maximum Horizontal Displacement for 16 storied building.



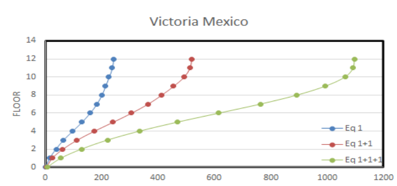

a) Displacement of Superstition Hills earthquake




b) Displacement of Duzce Turkey earthquake

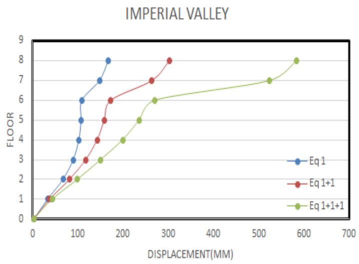



c) Displacement of N Palm springs earthquake

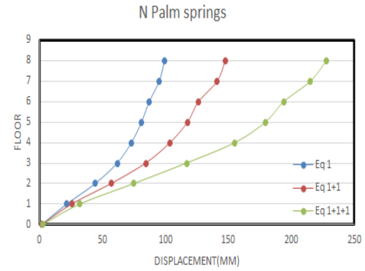



d) Displacement of Victoria Mexico earthquake

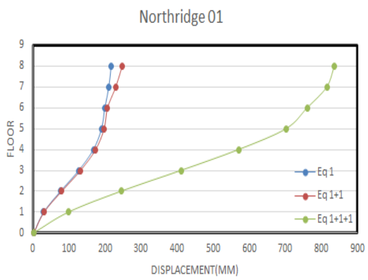
**Fig. 11.** Maximum Horizontal Displacement for 12 storied building



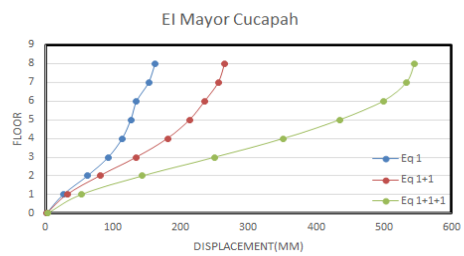

a) Displacement of Imperial Valley earthquake




b) Displacement of N Palm springs earthquake

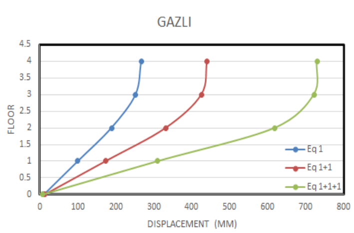



c) Displacement of Northridge 01 earthquake

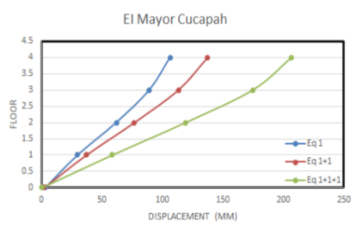



d) Displacement of Mayor Cucapah earthquake

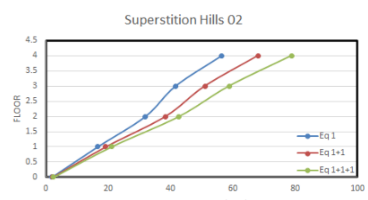
**Fig. 12.** Maximum Horizontal Displacement for 8 storied building.



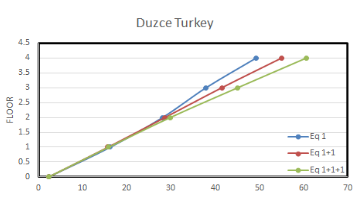

a) Displacement of Gazali earthquake




b) Displacement of Mayor Cucapah earthquake

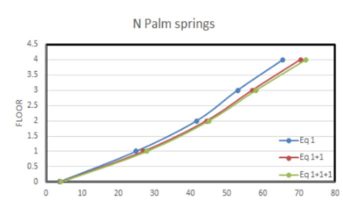



c) Displacement ofSuperstition Hills earthquake




d) Displacement of Duzce Turkey earthquake


e) Displacement of Chustsu-oki earthquake




f) Displacement of N Palm springs earthquake

**Fig. 13.** Maximum Horizontal Displacement for 4 storied building.

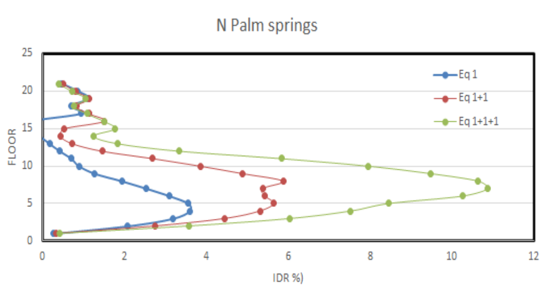
## 3.2. Interstory drift ratio (IDR)

IDR is computed as the difference in the deflections at the centers of mass at the top and bottom of the story under consideration normalized to the story height. The inter story drift ratio is a significant engineering parameter and an indicator of the structural performance. Ade Faisal et. al. [19] Different codes provide different criteria to check the performance of the structure depending on the IDR. As per FEMA 356 [14] IDR ≥ 4%, the structure is considered as collapsed. The damage limitation requirement should be verified in terms of the inter story drift (dr) (EN 1998) using equation – dr/h ≤α/v

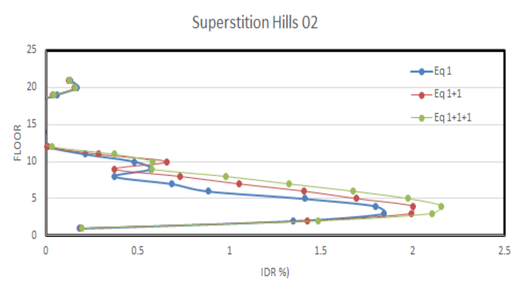
where dr is the story drift, h is the story height, and α is a factor that considers the type of non-structural elements and their arrangements in the structure. It amounts to 0.005, 0.0075 and 0.01.

ν is the reduction factor that considers the lower return period of the seismic action associated with the damage-limitation requirement. This depends on the class of buildings.

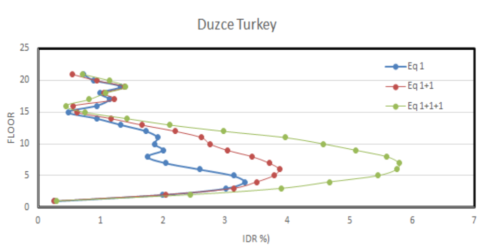
The building described in this paper is classified as importance class II (EN 1998-1) and the corresponding reduction factor ν amounts 0.5, as per the IDR limitation is 2 %. From fig. 13 to 17, it is clear that if the structure is subjected to MCE and also for repeated earthquakes, the IDR value is greater than 2 %, investigation from the analysis results and fig. 14 to 18 show the IDR due to repeated earthquakes is increases. The IDR values for single, double, and triple events or sequential earthquakes indicate that the inter-story drift ratio increases as the number of shocks increases, leading to larger IDR values compared to a single earthquake event. This leads to an increase in displacement and story drift with repeated earthquakes. In addition, it is important to note that IDR values are maximum in the middle story; from this, it is clear that the dimension of the section in the middle story is the same as that of the bottom story.



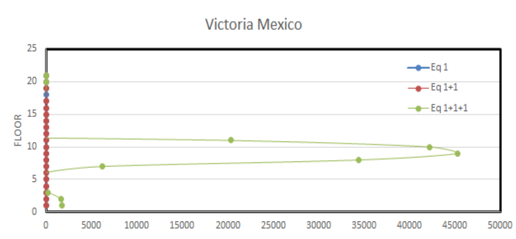

a) IDR curve for N Palm Springes earthquake




b) IDR curve for Superstition Hills earthquake

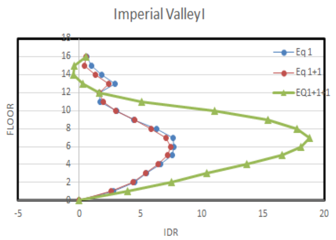



c) IDR curve for Duzce Turkey earthquake

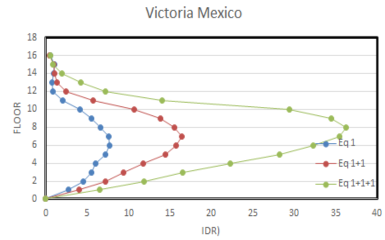



d) IDR curve for Victoria Mexicoearthquake

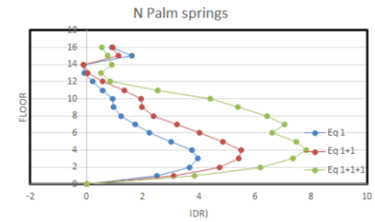
**Fig. 14.** IDR for 20 storied building



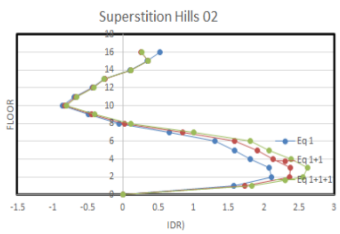

a) IDR curve for Imperial Valley earthquake



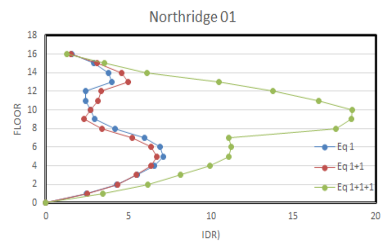

b) IDR curve for Victoria Mexico earthquake



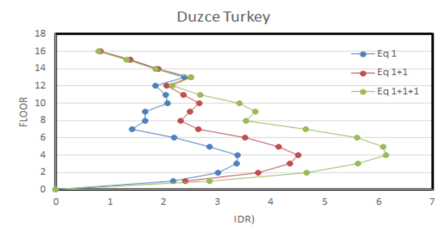

c) IDR curve for N Palm Springes earthquake




d) IDR curve for Superstition Hills earthquake

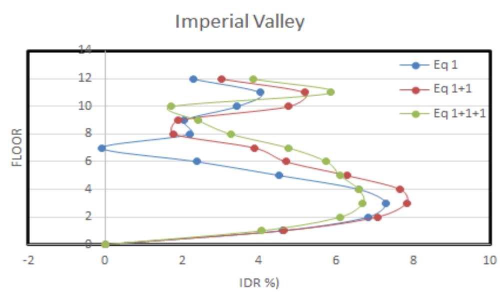



e) IDR curve for Northridge earthquake

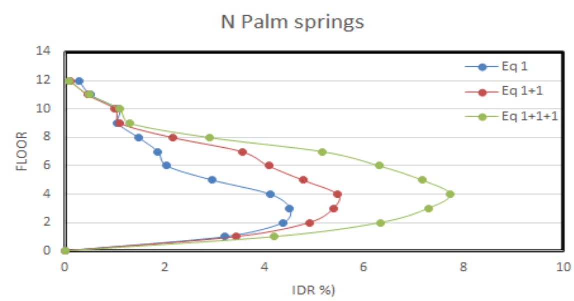



f) IDR curve for Duzce Turkey earthquake

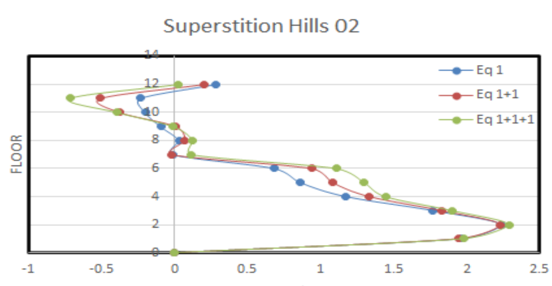
**Fig. 15.** IDR for 16 storied building.



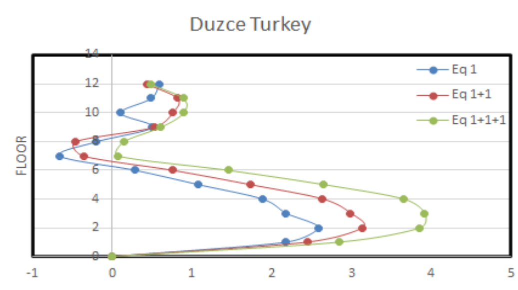

a) IDR curve for Imperial Valley earthquake




b) IDR curve for N Palm Springes earthquake

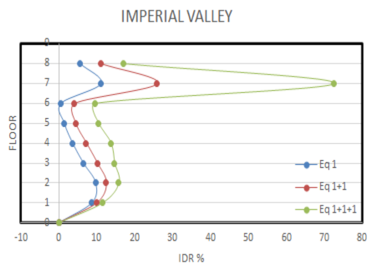



c) IDR curve for Superstition Hills earthquake

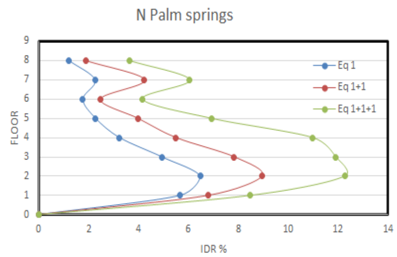



d ) IDR curve for Duzce Turkey earthquake

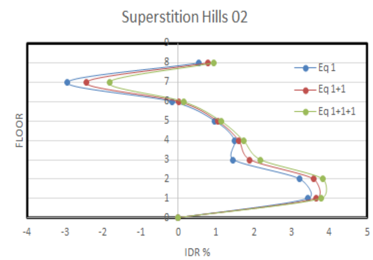
**Fig. 16.** IDR for 12 storied building.



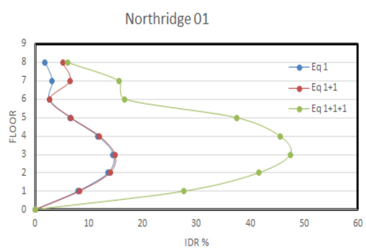

a) IDR curve for Imperial Valley earthquake



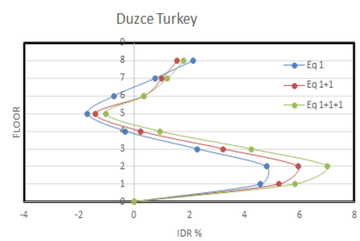

b) IDR curve for N Palm Springes earthquake



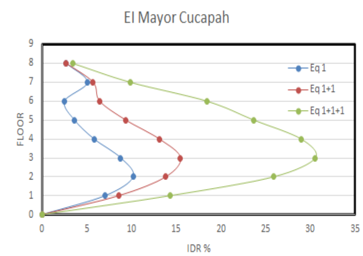

c) IDR curve for Superstition Hills earthquake




d) IDR curve for Northridge earthquake




e ) IDR curve for Duzce Turkey earthquake




f) IDR curve for EI Mayor Cucapah earthquake

**Fig. 17.** IDR for 8 storied building.

## 3.3 Yielding of structure

Fig 19 and 20 show the displacement versus base reaction curve for the structure, and Table No. II shows the base shear, yield displacement, and total weight of the structure for the different types of structures considered. This table is important for checking the structural behavior and performance. After the yield displacement, the structure enters the non-linear stage; therefore, the values of the yield displacement are useful to check whether the structure enters the nonlinear or linear stage.

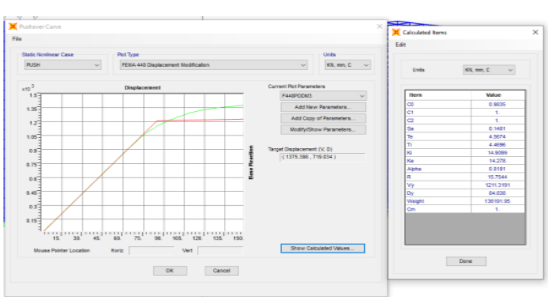


**Fig. 19.** Yielding for 20 storied buildings.

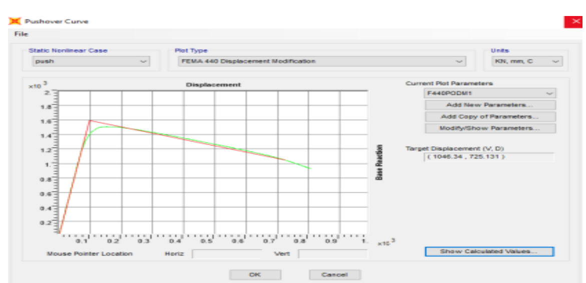


**Fig. 20.** Yielding for 16 storied building.

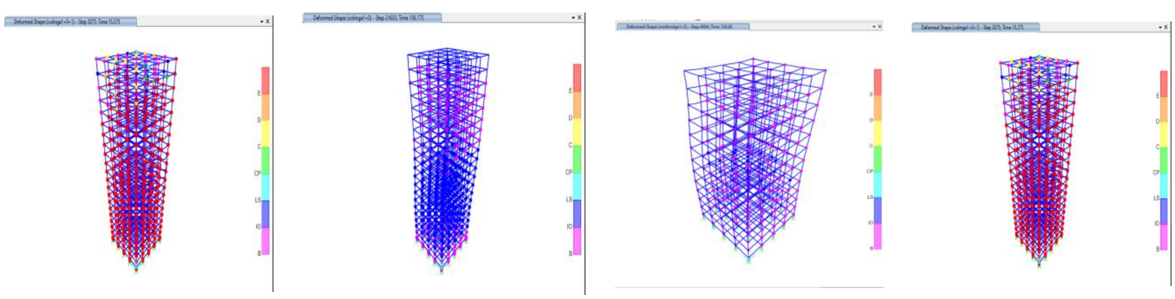
## 3.4 Hinge formation

The yielding of reinforced concrete (RC) members or structures is the main cause of collapse of RC buildings during earthquake excitation. A hinge means having no ability to resist a moment, and the idea of hinge formation is important in understanding structural failure.

The patterns of plastic hinge formation under a single earthquake and the formation of plastic hinges under repeated earthquakes are different. From the study of a structure subjected to repeated earthquakes, it should be noted that the repeated earthquake distribution of plastic hinges differs from that of a single major earthquake. Owing to repeated earthquakes, the formation of plastic hinges is increased and the state of the hinges is changed from B- Io- LS –C. Fig 21c shows that at the end of the first earthquake, the structure has IO-type hinges, which means that at the end of the first earthquake, the structure is in the nonlinear stage, but immediate occupancy is possible. If the structure is subjected to a double or triple earthquake, the structure is in a nonlinear stage and hinges change their nature from IO to C, that is, in the collapse stage; therefore, the structure can be fully collapsed, as shown in Fig 21a,21b & 21 d.

In addition, as shown in fig 22, hinge formation starts from a beam and then goes to a column. This formation shows that the strong column- weak beam concept is followed by the structure. This study also investigates how the hinge formation is changed at a middle story (Fig.21b) and a middle story of the structure can collapse first, so we have to focus on the middle story sectional dimension of a column to resist the structural collapse of the structure, and these stories can be strengthened.

**Hinge Formation in Element**



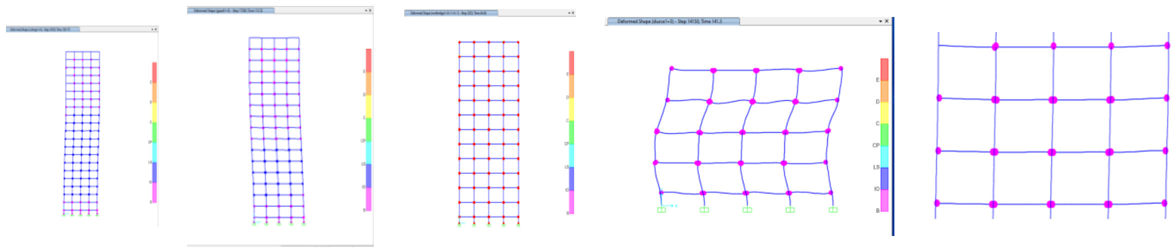
a) C – Collapse type of hinge formation

b) IO to LS type of hinge formation

c) IO type hinge formation

d) Total Collapse of structure hinge formation

**Fig. 21.** Hinge formation in the 3D structure.



a) hinge formation in 20 storied structure

b) hinge formation in 16 storied structure

c) hinge formation in 12 storied structure

d) hinge formation in 4 storied structure

e) hinge formation in beam & column

**Fig. 22.** Hinge formation in the structure and pattern of hinges in 2D structure.

## 3.5. Permanent displacement

Owing to repeated earthquakes, the structure experiences a permanent displacement at the end of every earthquake shock. Fig. 23 shows the permanent displacement of the structure. At the end of an earthquake, there is permanent displacement can take place, which is the plastic range of the structure. In the plastic range, the structure could not undergo the original shape and size of the structure at the end of the earthquake. Therefore, the P-delta load on the structure was applied up to the lifespan of the structure. In this study, a gap of 100 s is given between two earthquakes to check whether the structure is in a resting or moving condition. The Fig. 24 shows that at the end of an earthquake, the structure is in rest condition owing to a zero acceleration of a 100-sec gap provided between two earthquakes, and there is a permanent displacement at the end. Fig. 25 shows the collapse nature of the joint as well as the structure.

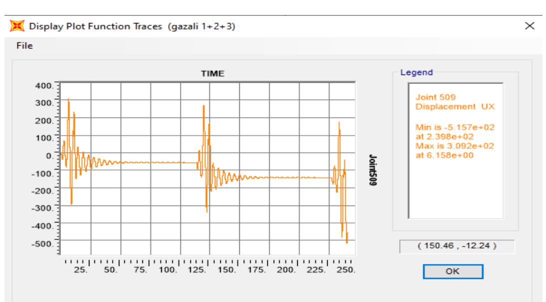



**Fig. 23.** Permanent displacement.

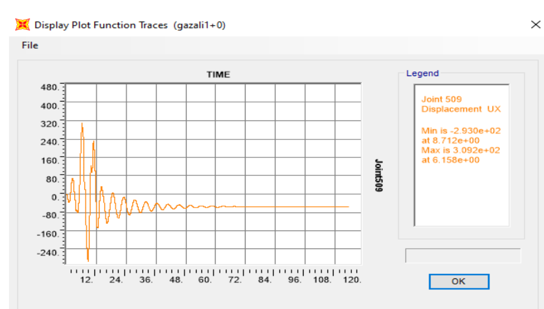
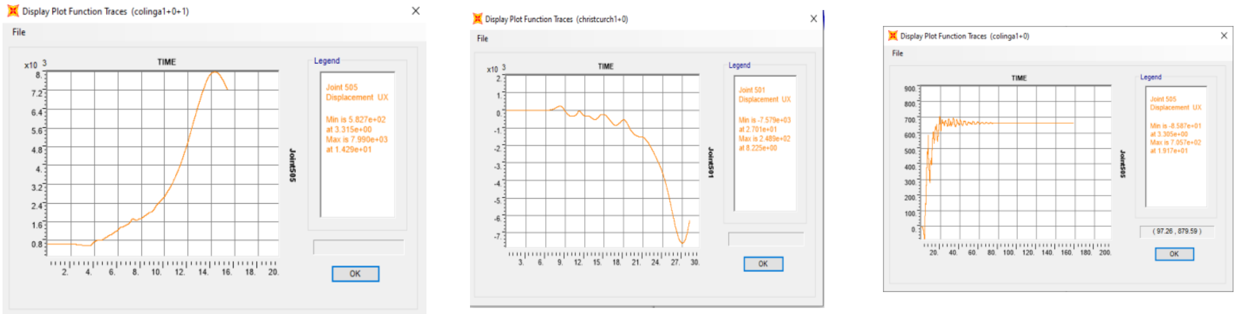



**Fig. 24.** Structure Rest Condition at the end of Earthquake.




a b c

**Fig. 25.** Collapse nature of joint.

## 4. Conclusions

From the study of the displacement curve, IDR curve, hinge formation in the column and beam, axial force in a column, BM of the beam member, permanent displacement of the structure, and the detailed study of different types of low-rise to high-rise structures, the following conclusions can be drawn:

1) A comparison of the single earthquake effect and multiple earthquake effect leads to shows that the maximum horizontal displacement increased to 20 to 50% of the single earthquake displacement values.

2) The IDR curve shows that there is a maximum IDR value for the middle floors, and these values focus on increasing the sectional dimensions of the middle floor. The analysis results show that the bottom story dimensions are continued to the middle floor to reduce the IDR value and reduce the displacement of the structure.

3) To increase the performance of a structure subjected to repeated earthquakes, the structure is designed for a maximum considered earthquake in that locality or earthquake-prone area, not for a design-based earthquake.

4) The permanent displacement and P-Delta effect can be controlled in the structure by increasing the middle floor section dimension of the column.

5) To increase the performance capacity of the structure the beam-column joint can be well modeled.

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## Conflict of Interest Statement

We certify that they have no affiliations with or involvement in any organization or entity with any financial interest (such as honoraria; educational grants; participation in speakers' bureaus; membership, employment, consultancies, stock ownership, or other equity interest; and expert testimony or patent-licensing arrangements), or non-financial interest (such as personal or professional relationships, affiliations, knowledge, or beliefs) in the subject matter or materials discussed in this manuscript. There are no conflicts of interest to declare.

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