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RESEARCH ARTICLE

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DAMAGE ASSESSMENT OF LOW RISE TO MID RISE STRUCTURES SUBJECTED TO REPEATED EARTHQUAKE LOAD

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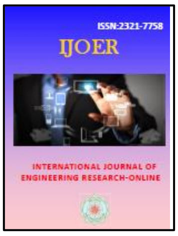


ABSTRACT

Investigation of adequate and accurate assessment of structure during earthquake and after earthquake it is essential to increase life span of structure. This paper assesses the damage formation of an RC frame building using nonlinear static and nonlinear dynamic analysis procedures. To accomplish this objective, five-moment resisting frames with low rise structure as 4 storied and midrise structure with 8 storied, 12 storied, 16 storied, 20 storied buildings were analyzed and designed following the guidelines of the Indian seismic codes IS 1893:2016[1], was subjected to repeated earthquake load. The damage assessment of the building in terms of displacement, inter story drift ratio, hinge formation are assessed. The failure of members of structure and the performance of the structure in terms of displacement and ductility was measured for different repeated earthquake events, these damage assessment result and points of damages in building element are used to modify the sectional dimension of building and to increase the seismic performance of the structure. These factors were used for the design of an earthquake force-resisting system in a new building. In this work, 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. In view of different damage points of the structure when subjected to repeated earthquakes, a new design philosophy was presented

Keywords: ductility, time history analysis maximum considered earthquake, repeated earthquake, nonlinear plastic hinges, damage accumulation. permanent displacement

1. Introduction

In reality earthquake comes in sequence , therefore after every earthquake investigation of performance of structure is essential to increase life span of structure but to find actual performance of existing building it is not easy work. So at the time of initial designing consider all future shocks that need to resist the building in its life span on probabal basis. All current codes gives procedure of designing of structure considering only single earthquake. If this structure is subjected to repeated earthquake then the performance of structure is not better in situation of sequential earthquake. This leads to check the performance of structure by concentrating on repeated earthquake load.

To check the performance of a building subjected to a real earthquake, the designer must go outside the codal method and correctly guess how the structure will behave under an repeated earthquake or often during an extreme event. In reality,earthquakes occur in series, therefore, an actual response or performance study is essential. Repeated earthquakes have given increase to unprecedented injury or destruction to both life and property because of the very short interval of time also it is not possible to repair the building and perform the movement of people.

In real life, the building is shaking many times with repeated earthquakes in a life span of structure and subjected to MCE, Therefore the damage assessment of the structure under these MCE is studied, Sachin Patil et al 2021[3] 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

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. MohdZulhamAffandi 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 damage assessment of the structure in terms of maximum story displacements, interstory drift ratio, base shear of the building.

2.0 Description of Model & Ground Motion Data

2.1 Explanation of Structure and Modeling Criteria

In this paper five different types of structures were considered to represent low-to high-rise structures for analysis and design purposes.

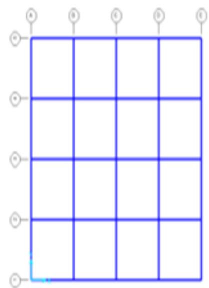


Fig.1 -Plan of a Structure

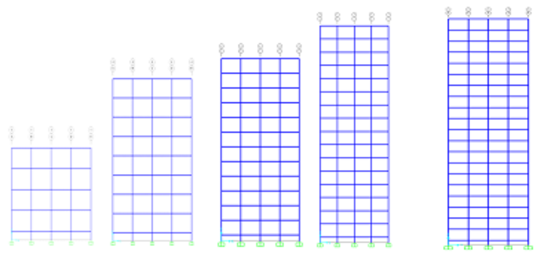


Fig. 2 – 2D Framed Structure

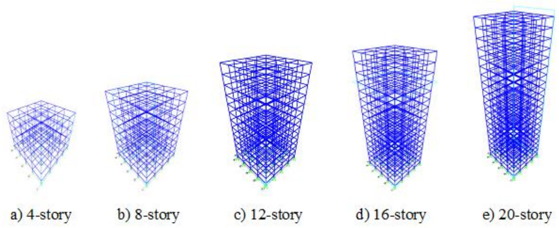


Fig 3-3D Framed Structure

The present paper consider 4 storied building is low rise building and 8-, -, 12-, 16-, and 20 -storied buildings are mid rise buildings because in hige rise structure there is a combination of shear wall or bracing system and moment resisting frames to resist large lateral forces. To perform linear and nonlinear dynamic analysis, the model of the 4-, 8-, 12-,16-,20- storied building structure is taken. The plan dimension of building is 20 m X 20 m with height of each floor is 3.5 m . The footing is rest on hard starta and the hard start is available at 1.5 m from the ground, the building is analysed and designed As per the IS code. Use - M 40 grade of concrete. Fe 415 grade of steel Zone - V, Zone factor = 0.36, Importance factor =1.5, Response reduction factor R= 5, The building is a commercial building, located in a high-seismic zone in India.

Fig.1, shows a plan for building, Fig.2 & 3 shows 2D and 3D models of five types of structures.The planned model is a 3D regular model in plan, and elevation and it is 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 paper, 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.

$$\sum \boldsymbol{MRc} \geq 1.3 \sum \boldsymbol{MRb}$$

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. 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]. The lateral stiffness of the structure changed at every three stories. 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.4), as proposed by Zarein and Krawinkler (2009)[11].

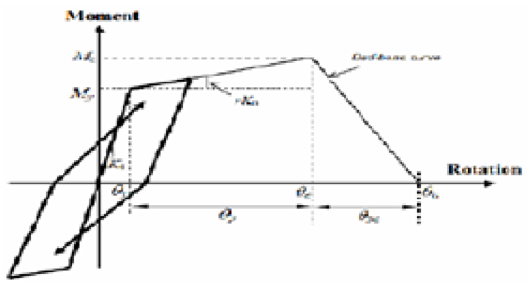


Fig 4- Modified Takeda Hysteresis and backbone curve (Zarein and Krawinkler,2009)

For the linear and nonlinear dynamic analysis, the structure is modeled with default hinges, 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 I

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.5 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.

Table I. 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

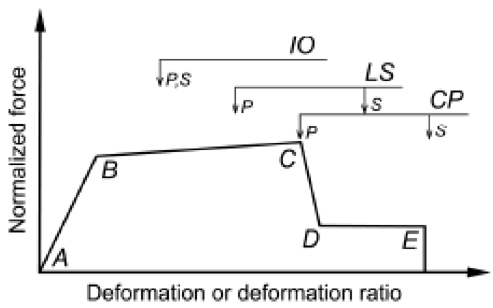


Fig 5- Force –Deformation Relationship of a Plastic Hinge or Element Deformation Acceptance Criteria (FEMA 356).

The total height and yield displacement of a structure as listed in Table II.

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 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.

represent the spatial distribution of mass and stiffness

Table II. Description of structure

Building	Total height (m)	Yield Displ. (mm)
4 storied	15.5	37.728
8 storied	29.5	68.50
12 storied	43.5	73.8477
16 storied	57.5	97.366
20 storied	71.5	84.838

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 damage assesment 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] . 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. The selected earthquake has different distances from the focus, and it is important to study the effects

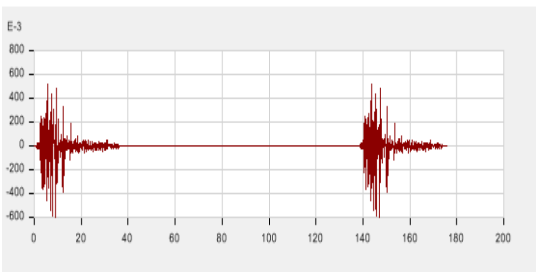


Fig.6a- 2 GM of duzceearthquake with 100 s gap

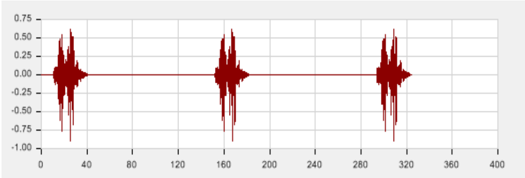


Fig.6b- 3 GM of imperial valley 06 earthquake with 100 s gap

The selected combination of repeated ground motion with a 100 s break is presented in fig 6 a & b. 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 archetype 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. III

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 is defined in terms of the maximum displacement **µmax** and the yield displacement µy.

$$\mu d = \frac{\mu max}{\mu y}$$

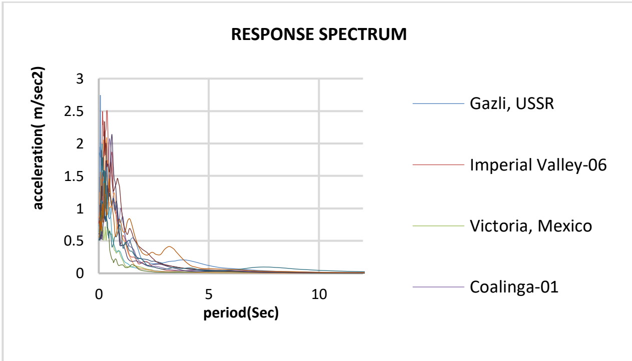


Fig.7- Response spectrum graph for selected earthquake motion.

Table III - Earthquake Events

S.N.	EVENT NAME	EVENT STATION	DATE	TIME	MAGNITUDE (MW)	DIST.OF EPICENTER	MECHANISM	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

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.

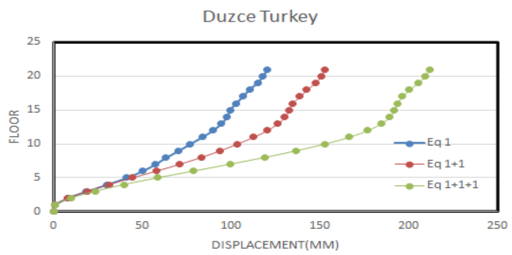
3.0 Result and Discussion

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 III. 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.

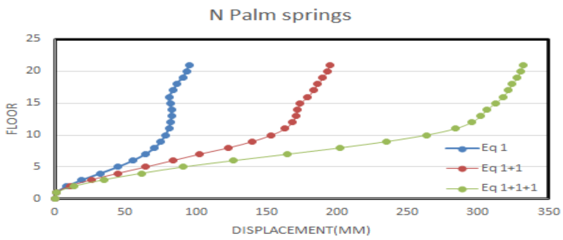
3.1 Maximum Horizontal displacement of structure-

The maximum horizontal displacement for 4-,8-,12-,16-,20 storied buildings and for repeated earthquake events are presented in fig.8 to 12. In the present work, above-ground motion records are applied with a combination of case-I, case-2, case-3 .Between two successive earthquakes, a time gap of 100 s was applied to come to the structure to come in relax condition and to find the permanent displacement and position of the plastic hinges. The effects of these repeated earthquakes for different earthquake events are shown in fig 8–12. Displacement of a 4,8,12,16,20 storied building& the effect of a single, double, and triple earthquake ar presented in graphical format . Fig.8 to 12 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 8 storied 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. 8 to 12, 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. .8 to 12, 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. .8 to 12, shows that if the structure is near the fault, its displacement is large.

Maximum Horizontal Displacement Graphs–

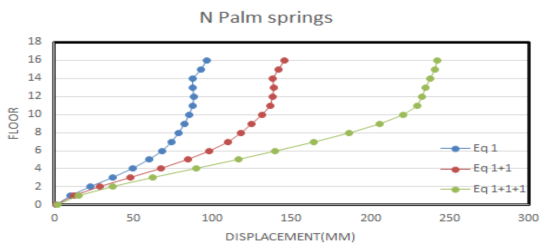


a) Displacement of Duzce Turkey earthquake

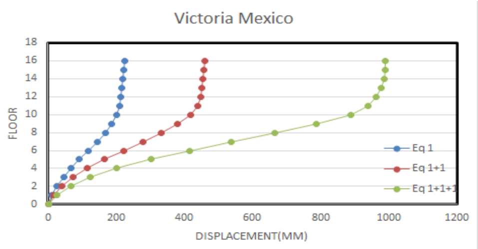


b) Displacement of N Palm spring earthquake

Fig. 8 - Maximum Horizontal Displacement for 20 storied building

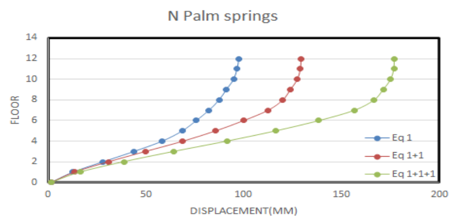


a) Displacement of N Palm springs earthquake

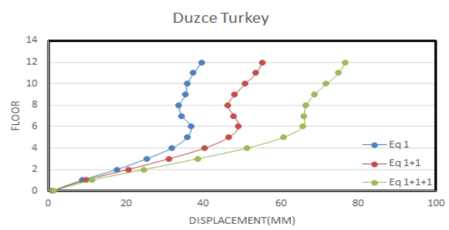


b) Displacement of Victoria Mexicoearthquake

Fig. 9 - Maximum Horizontal Displacement for 16 storied building

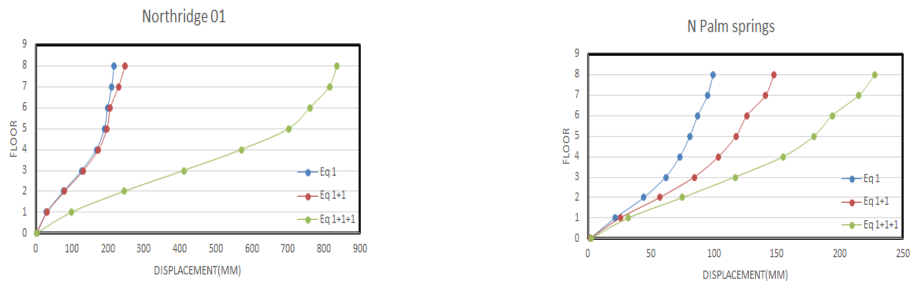


a) Displacement of N Palm springs earthquake



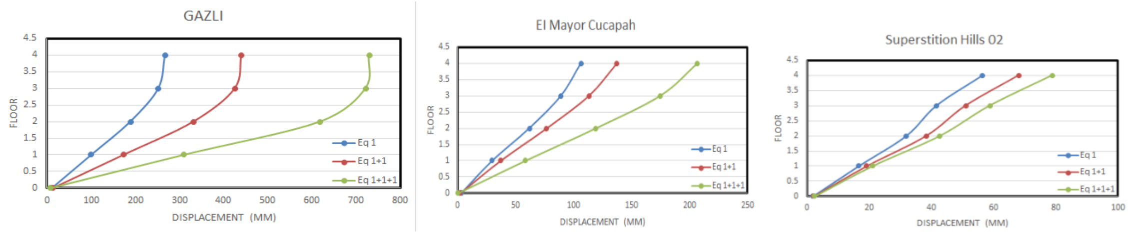
b) Displacement of Duzce Turkey earthquake

Fig. 10 - Maximum Horizontal Displacement for 12 storied building



a) Displacement of Northridge 01 earthquake b) Displacement of N Palm springs earthquake

Fig. 11 - Maximum Horizontal Displacement for 8 storied building



a) Displacement of Gazali earthquake b) Displacement of Mayor Cucapah earthquake c) Displacement of Superstition Hills earthquake

Fig. 12 - Maximum Horizontal Displacement for 4 storied building

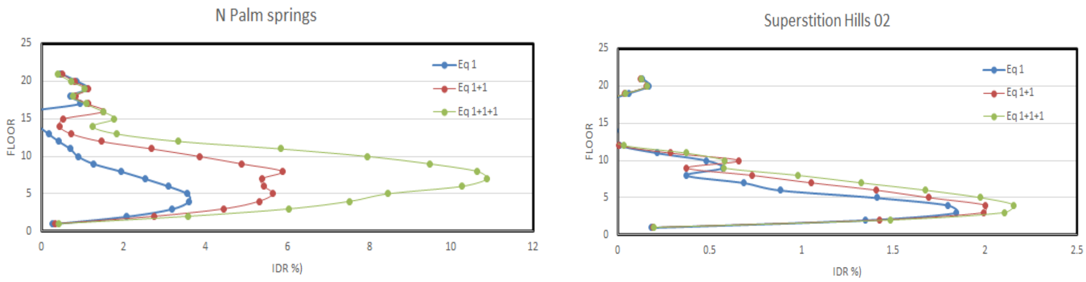
3.2 Interstorydrift ratio (IDR) -

The interstory 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 interstorey drift (dr) (EN 1998) using equation – $dr/h \leq \alpha/v$

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 .

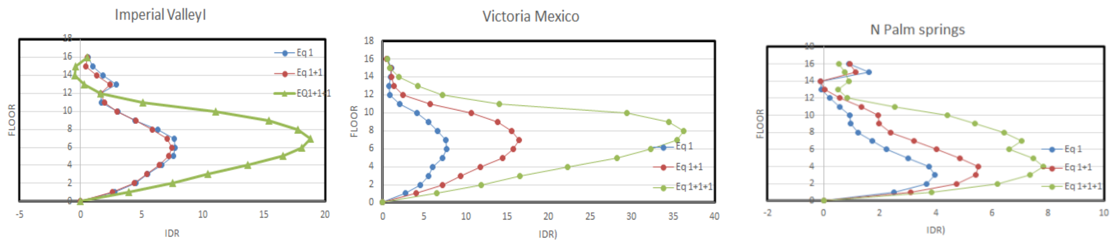
⍰ is the reduction factor that considers the lower return period of the seismic action associated with the damage-limitation requirement. This depends on the class of buildings.

The building described in this paper is classified as importance class II (EN 1998-1) and the corresponding reduction factor ⍰ amounts 0.5, as per the IDR limitation is 2 %. From fig. 13 to 16, 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. 13 to 16 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; this leads to increase the middle floor sectional dimension .



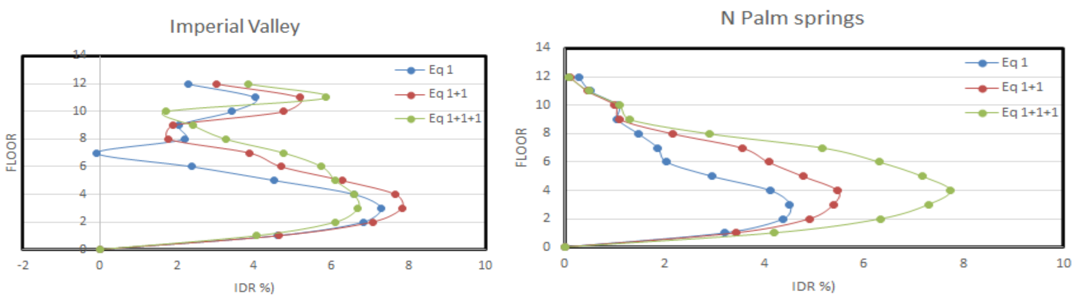
a) IDR curve for N Palm Springes earthquake b) IDR curve for Superstition Hills earthquake

Fig.13 –IDR for 20 storied building



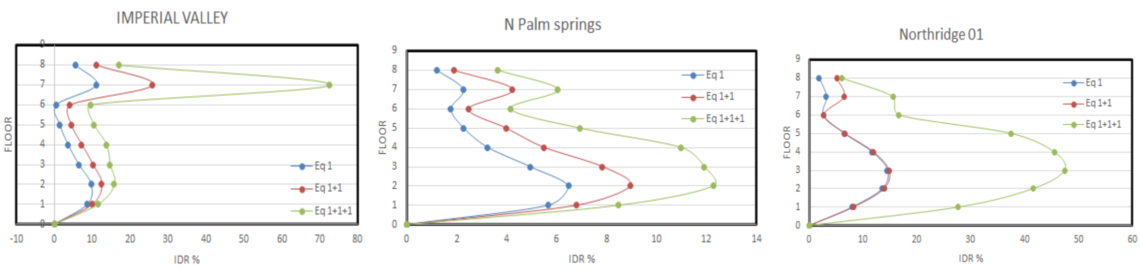
a) IDR curve for Imperial Valley earthquake b) IDR curve for Victoria Mexico earthquake c) IDR curve for N Palm Springes earthquake

Fig. 14 – IDR for 16 storied building



a) IDR curve for Imperial Valley earthquake b) IDR curve for N Palm Springes earthquake

Fig. 15 – IDR for 12 storied building



a) IDR curve for Imperial Valley earthquake b) IDR curve for N Palm Springes earthquake c) IDR curve for Northridge earthquake

Fig. 16 – IDR for 8 storied building

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3.3 Yielding of structure-

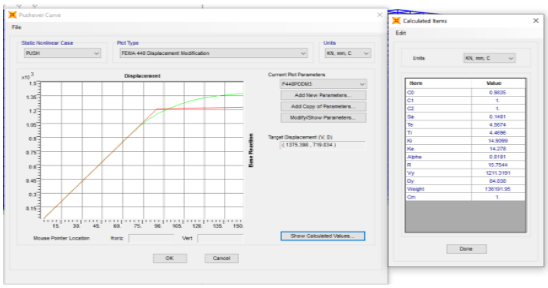


Fig. 17 – yielding for 20 storied buildings

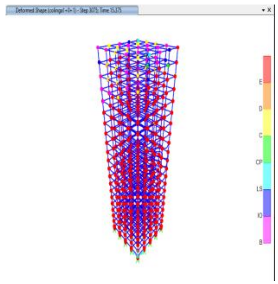
Fig 17 show the displacement versus base reaction curve for the structure, and Table No. II shows the base shear, yield displacement, and total weight of the structure for the different types of structures considered. This table is important for checking the structural behavior and performance. After 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.

3.4 Hinge formation–

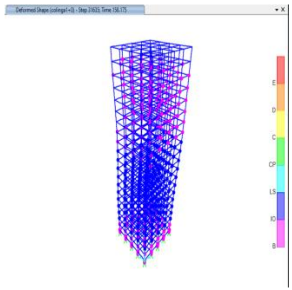
The yielding of reinforced concrete (RC) members or structures is the main cause of collapse of RC buildings during earthquake excitation. A hinge means having no ability to resist a moment, and the idea of hinge formation is important in understanding structural failure. The patterns of plastic hinge 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 18 c 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 18.

This study also investigates how the hinge formation is changed at a middle story (Fig.18b) 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.

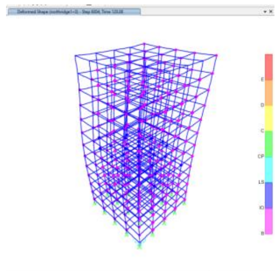
Hinge Formation in Element-



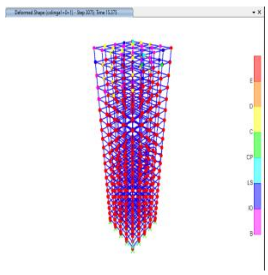
a) C – Collapse type of hinge formation



b) IO to LS type of hinge formation



c) IO type hinge formation



d) Total Collapse of structure hinge formation

Fig 18 – Hinge formation in the 3D structure

3.5 Permanent displacement: Owing to repeated earthquakes, the structure has a permanent displacement at the end of every earthquake shock. And due to permanent displacement the displacement values and interstory drift values are increase after every shock this generate more P-delta load on structure. Fig. 19 shows the permanent displacement of the structure. At the termination of an earthquake, 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. Therefore, the P-delta load on the structure was applied up to the lifespan of the structure. In this study, a break of 100 sec is put between two earthquakes to check whether the structure is in a resting or moving condition. The fie.20 shows that at the fishing stage of an first earthquake, the structure comes in rest condition. To provide a 100 sec gap between earthquake a zero acceleration is provided for a time span of 100 sec that indiacates zero acceleraton force applied on structure. Fig.21 shows the collapse nature of the joint as well as the structure.

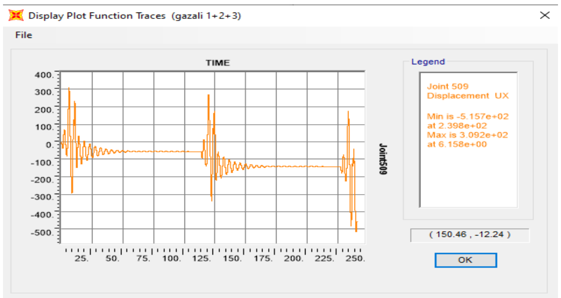


Fig 19 –Permanent displacement

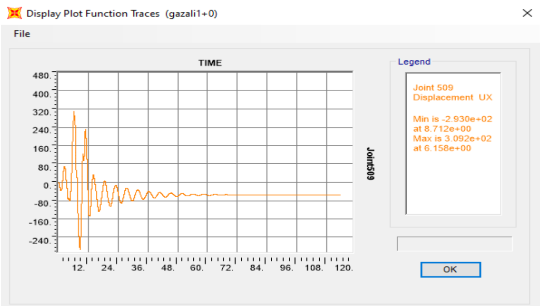


Fig 20 –Structure Rest Condition at the end of earthquake.

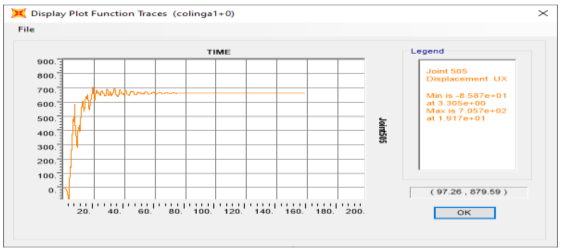


Fig 21 –Collapse Nature of Joint

4.0 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. Due to double event of earthquake displacement of structure is increased by 10 to 20 % and due to triple event of earthquake displacementis increased by 10 to 50 % as compaire to single earthquake displacement value.
2. The IDR values are increase at middle floor due to repeated earthquake and the structure is collapsed at middle floor .To overcome this middle sectional dimension are increased by 10 to 20 % . Use multiplying factor as a 1.10 to 1.25 to increase sectional dimension of column.
3. To reduce the displacement, IDR values and to increase the performance of a structure subjected to repeated earthquakes, the structure is analysised and designed for a maximum considered earthquake in that area or earthquake-prone area, not for a design-based earthquake.
4. The permanent displacement in the structure at the end of every earthquake event and P-Delta effect is controlled in the structure by increasing the 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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