Eight storeyed RC building seismic assessment with user defined hinges

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Abstract- *Earthquake causes the random ground motions because of which most of the structures subjected to infrequent loading. In present study is done assigning user defined hinges for beams and columns we given calculated moment curvature relations as input. Nonlinear static pushover analysis is performed in SAP 2000 based on FEMA 365 and ATC 40 guidelines to get the results using user defined hinge properties. The parametric results such as hinge states at performance point and ductility ratio are studied with varying percentage of central openings in brick masonry infill wall.*

Keywords- User defined hinges, infill wall, Pushover analysis, Performance levels, and Ductility ratio.

I. INTRODUCTION

Earthquake causes the random ground motions in all directions, radiating from epicenter. These ground motions causes structure to vibrate and induces inertia forces in them. Since inelastic behavior is intended in most structures subjected to infrequent earthquake loading, the use of nonlinear analyses is essential to capture behavior of structures under seismic effects. Due to its simplicity, the structural engineering profession has been using the nonlinear static procedure (NSP) or pushover analysis, described in FEMA-356 andATC-40. It is widely accepted that, when pushover analysis is used carefully, it provides useful information that cannot be obtained by linear static or dynamic analysis procedures.

The implementation of pushover analysis, modeling is one of the important step. The model must consider nonlinear behavior of structure/elements. Such a model requires the determination of the nonlinear properties of each component in the structure that are quantified by strength and deformation capacities. In practical use, most often the default properties provided in the FEMA-356 and ATC-40 documents are preferred. Due to convenience and simplicity. These default properties can be implemented in well-known linear and nonlinear static and dynamic analysis program such as SAP2000. The use of this implementation is very common among the structural engineering profession and researchers. Although there may not be significant differences in the modeling of steel structures, the use of guidelines requires special care for reinforced concrete (RC) structures. As mentioned above, the deformation capacity of reinforced concrete components depends on the modeling assumptions. FEMA-356 and ATC-40 guidelines are prepared on the basis of some assumptions related to typical reinforced concrete construction in the United States. While the documents provide the hinge properties for several ranges of detailing, the programs (i.e. SAP2000) may implement averaged values .Also, there may be some differences in construction techniques and detailing in other countries. If the user knows the capability of the program and the underlying assumptions, then people can take advantage of the feature provided to avoid an extensive amount of work. In some cases, the default hinge properties are used without any considerations due to simplicity. The definition of user-defined hinge properties requires moment–curvature analysis of each element. For the problem defined, building deformation is assumed to take place only due to moment under the action of laterally applied earthquake loads. Thus user-defined M (moment hinge) hinge for beam and PMM(axial force and moment hinge) hinges for columns are assigned at member ends where flexural yielding is assumed to occur. Moment-curvature relationship was assigned in SAP2000 for both confined and unconfined cases to represent the flexural characteristics of plastic hinges at the ends.

This study aims to the Three-dimensional (3-D) modeling pushover analysis is employed. The SAP2000 program is used for pushover analysis.

II. BUILDING DESCRIPTION

In the present study 3D RC eight storeyed building model isconsidered. The plan and elevations of the building model considered are shown in Fig 1 to Fig 3, the all storey height is 3.5m is kept. The building is considered to be located in the seismic zone III region and intended for office use. In the seismic weight calculations, only 25% of the live load is considered. Slabs loads are applied on the beam. Masonry brick walls are modeled by considering equivalent diagonal strut. M (moment hinge), PM (axial force and moment hinge) hinges with hinge properties are assigned at both ends of beam and column elements by using user defined hinges. Input data given for all the buildings are detailed in below. Two analytical models are considered as below,

A. Plan and Elevation of the Building models

The plan of the building is shown in the Fig.1 and elevation of the building models are shown in Fig 2 and Fig 3.

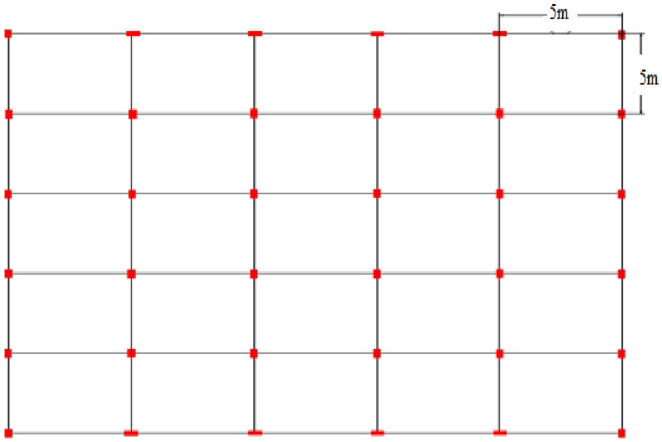

Figure 1. Plan of building

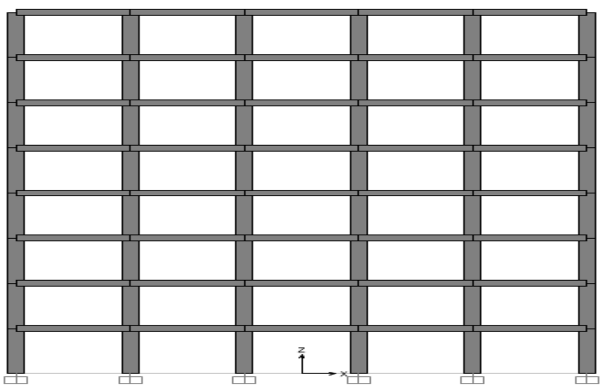

Figure 2. Elevation of the eight storeyed bare frame building model

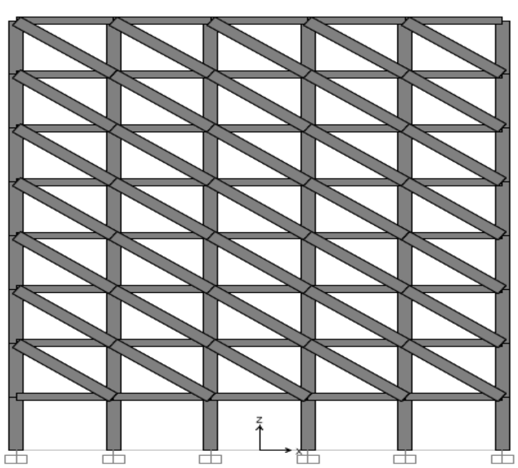

Figure 3. Elevation of the eight storeyed building models with openings (10% to 30%)

B. Modeling infill panel as equivalent diagonal struts (without openings)

It is evident from most of the studies that the infill wall panels fail due to increasing intensity of lateral loads by corner crushing in the infill at least one of its loaded corners associated with strong infill surrounded by a strong frame in which the diagonal compression strut mechanism is fully developed that converts the frame system into the truss, increasing the lateral stiffness of the frame manifold. And the masonry infill wall is modeled as pin-jointed single equivalent diagonal strut (SEDS), carrying axial compressive force only.

Table 1. Width of equivalent diagonal struts given by various researchers

Investigators	**Formula**
Stafford Smith and Hendry (1963)	$\frac{1}{2}\sqrt{\alpha_h^2 + \alpha_L^2}$

$\beta=(E_C A_C)/(G_m A_m)$is a dimensionless parameter, A_C is the gross area of column and

$A_m=(L_m t)$

$$\alpha_h = \frac{\pi}{2}\left[\sqrt[4]{\frac{E_F I_c}{2E_M t \sin 2\theta}}\right]$$

$$\alpha_h = \pi\left[\sqrt[4]{\frac{E_F I_b L}{E_M t \sin 2\theta}}\right]$$

α_h is length of the contact between wall and column and α_l is the length of contact between all and beam where, E_m and E_f are Elastic modulus of the masonry wall and frame material, t, h and L are thickness , height and length of the infill wall .

The comparative analysis carried out for the models with wall as a membrane and wall modeled as equivalent diagonal strut shows the similar results, therefore in the present study width of strut is calculated as per the formula obtained by Stafford Smith and Hendry .

Reduction factor for infill walls with central opening

The unreinforced masonry infill walls with central openings are modeled as pin-jointed single equivalent diagonal strut of reduced width, by applying the reduction factor for the width equivalent diagonal strut, modeled for infill wall without opening. And the reduction factor, given in the clause 7.10.2.3 of "Proposed Draft Provision and Commentary on Indian Seismic code IS 1893 (Part 1), [Jain and Murty] is considered and given as below.

$$\rho_w = 1 - 2.6 x a_o .$$

Where,

$\rho_w =$ Reduction factor.

$a_o =$ Percentage of central opening, i.e. the ratio of area of opening to the area of infill.

C. User defined hinges

The definition of user-defined hinge properties requires moment–curvature analysis of each element. For the problem defined, building deformation is assumed to take place only due to moment under the action of laterally applied earthquake loads. Thus user-defined M3 hinge was assigned at member ends where flexural yielding is assumed to occur. Moment-curvature relationship was assigned in SAP2000 for both confined and unconfined cases to represent the flexural characteristics of plastic hinges at the ends.

D. Moment curvature analysis for RC sections

Under flexure forces, internal strain in a member varies along the depth of cross section; the slope of strain with depth is the curvature. For linear materials, the moment of resistance increases linearly with increase in strain in the extreme fiber. However, for nonlinear materials, the moment curvature relationship becomes nonlinear.

Table 2. Moment curvature values for column

Points	Moment	Curvature
A (Origin)	0	0
B (Yielding)	1	0.00123
C (Ultimate)	1.044	0.01623
D (strain hardening)	0.2	0.01623
E (strain hardening)	0.2	0.01845

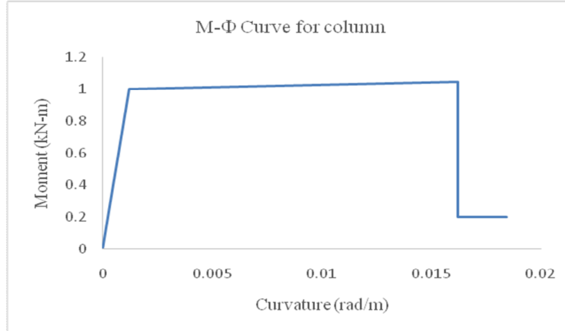

Figure 4. Moment Curvature relations for Column

III. RESULTS AND DISCUSSIONS

A. Performance Evaluation of Building Models.

Performance based seismic evaluation of all the models is carried out by nonlinear static pushover analysis (i.e Response spectrum pushover analysis).

B. Performance Point and Location of Hinges

The base force, displacement and the location of the hinges at the performance point, for various performance levels along longitudinal direction for the building model is presented in the Table 3.

Table 3. Performance point and location of user defined hinges for brick masonry infill for eight storeyed building models by response spectrum pushover analysis

Model No.	Performance Point			Location of Hinges					
	Displacement mm		*Base Force kN*	*A-B*	*B-IO*	*IO - LS*	*LS-CP*	*CP to E*	*Total*
1	Yield	41.618	2568.347	1342	194	0	0	0	1536
	Ultimate	233.1	4852.016	958	62	64	66	386	1536
2	Yield	24.55	15410.14	1451	85	0	0	0	1536
	Ultimate	83.76	36059.73	1375	85	57	2	17	1536
3	Yield	21.64	13910.15	1451	85	0	0	0	1536
	Ultimate	89.366	32551.17	1391	65	38	17	25	1536
4	Yield	23.07	12501.56	1459	77	0	0	0	1536
	Ultimate	95.28	29039.73	1376	79	17	16	48	1536

The base force is more in soft storey building compared to the bare frame building models. As the stiffness of the building decreases with increase in the percentage of central openings for brick masonry infill, the base force at performance point decrease.

As the percentage opening increases from 10% to 30% the base force decreases at yield and ultimate state. For eight storeyed building models, there is decrement in the base force at the ultimate state from model 2 to model 4 is 2.15% and 2.11% for brick masonry infill by response spectrum pushover analysis.

The locations of the hinges formed at the performance point, displacement and base force at ultimate state are shown in the Table 3. In most of the buildings, plastic hinges are formed in the first storey because of open ground storey. The plastic hinges are formed in the beams and columns.

For eight storeyed building models, from the above Table 3 it can be observed that, the hinges are formed within the life safety range at the ultimate state is 74.86%, 98.89%, 98.37%, and 96.87% for infill as brick masonry by response spectrum pushover analysis method. We can also observed that, the hinges are formed beyond the CP range at the ultimate state is 25.13%, 1.1%, 1.62%, and 3.12% for as brick masonry infill by response spectrum pushover analysis method.

From the above results it can be concluded that, as the stiffness of infill wall is considered in the soft storey buildings, base force is more than that of the bare frame building. The stiffness of the building decreases with the increase in percentage of central openings from 10% to 30%. The performance of all the building models is within the life safety range at the ultimate state for response spectrum pushover analysis method. These results reveal that, seismically designed multi storeyed RC buildings are safe to earthquakes. And as the collapse hinges are few, retrofitting can be completed quickly and economically without disturbing the incumbents and functioning of the buildings.

C. Ductility Ratio (DR)

Ductility ratio means it is the ratio of collapsed yield (CY) to the initial yield (IY) [23]. Ductility ratio (DR) of eight storeyed building models are tabulated in the below Table 4.

The ductility of a structure is a one of the most important factors affecting its earthquake performance. One of the primary tasks of an engineer designing a building to be earthquake resistant is to ensure that the building will possess enough ductility to withstand the size and types of earthquakes it is likely to experience during its lifetime. In present study the ductility parameter is studied in order to know the behavior of the building under seismic loading.

Reinforced concrete structures for earthquake resistance must be designed, detailed and constructed in such a way that ductility factor will be at least 4 up to the point beginning of visible damage and even greater , to point of beginning of structural damage and limitation.
The structures can be classified depending on the different design ductility levels

- Elastically responding structure, μ=1
- Structures responding in ductile manner, μ>1

They can be further divided as,

- Fully ductile structures with 4<μ<8
- Structures with restricted ductility with 1.5 <μ< 4

Table 4. Ductility ratio for eight storeyed building models by response spectrum pushover

Model No.	Brick masonry infill		
	IY	CY	DR
1	41.62	233.1	5.6
2	24.55	83.76	3.41
3	21.69	89.36	4.10
4	23.07	95.28	4.13

The lateral stiffness of the building increases the lateral strength, but reduces the energy absorption capacity of the building, hence ductility ratio decreases. From above result it is clear that the ductility ratio of the bare frame is larger than that of the soft storey building models.

For eight storeyed building models, the ductility ratio is found more in bare frame building (model 1) compare to soft storey building (model 2) by 39% for brick masonry infill by response spectrum method. As the percentage of openings increases from 10% to 30% the ductility ratio increases from model 2 to model 4 by 17.43% for brick masonry infill by response spectrum method.

The ductility ratio is more in bare frame compare to the soft storey building models. And also from the above results reveal that, increase in openings increases the DR nearer or slightly more than the target value.

IV. CONCLUSION

Based on the results obtained from different analysis for the various building models, the following conclusions are,

1. As the stiffness of the building decreases with the increase in the percentage of central opening varies from 10% to 30% from model 2 to model 4, the base shear decreases.
2. As the percentage of central opening increases, the lateral displacement increases.
3. For the response spectrum method, the storey drift is found to be within the limit for all building models.
4. The base force at performance point decreases with increases in the percentage of central openings from 10% to 30%.
5. Most of hinges are found within the life safety range at the ultimate state by response spectrum pushover analysis.
6. Ductility ratio are found more in the bare frame compare to the soft storey building models by response spectrum pushover analysis.

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