EFFECTS OF ASPECT RATIO ON NONLINEAR SEISMIC PERFORMANCE OF RC BUILDINGS

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Abstract

Many countries have their own building codes-of-practice for earthquake resistant design of structures. These codes provide conventional approach to earthquake resistant design of buildings with sufficient strength, stiffness and inelastic deformation capacity to withstand a given level of earthquake generated force. Under a given earthquake, the level of damage to a structure is greatly influenced by nonlinear capacity of its members. Even when the nonlinear capacities are same, the overall damage is critically influenced by the structure's shape, size and geometry.

This paper presents the research on the influence of aspect ratio on overall damage of the structure subjected to earthquake loading. In this paper a numerical study is carried out for 3 benchmark structures, with an aspect ratio (height to width ratio) of 3, 1 and 0.3, to define failure pattern in nonlinear state. Nonlinear static pushover analysis is performed to understand the capacity of structure and expended energy based damage assessment is used to estimate the damage. The results are compared in terms of damage, displacement and plastic hinge pattern.

Keywords: *Pushover analysis, total energy, expended energy, damage.*

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INTRODUCTION

In the past earthquakes, many concrete structures have been severely damaged or collapsed, which has raised questions against the seismic adequacy of RC structures. This is generally accomplished through the selection of an appropriate structural configuration and the careful detailing of structural members. In this research, an attempt is made to study the effect of an aspect ratio on damage level of the structure under nonlinear behaviour. Pushover analysis is a simplified procedure to determine the displacement capacity of a building expected to deform inelastically. It is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution, until a target displacement is reached. Pushover analysis methodologies are under continuous development to predict behavior of a structure under real earthquake. Gupta and Kunnath[1] (2000) presented an adaptive pushover method in which external force profile is adjusted in each analysis step to consider structure's current dynamic characteristics. Goel and Chopra[2] (2002) developed a modal pushover analysis (MPA) which accounts for the contribution of higher modes effect. Mohamed Abdel-basset[3] (2012) proved that displacements and curvatures derived from non-linear static response can be used as good damage indicators.

Simplified pushover analysis procedure gives capacity curve (relation between base shear V_B and lateral displacement Δ) and displacement capacity of the structure. Also, at any displacement level, cumulative energy dissipated in the structure (which reflects the inelastic performance of the structure) is estimated as the sum of the inelastic energies expended in individual structural members in each incremental load step of the pushover analysis. Alongside, the area under the pushover curve is estimated; here, $V_B - \Delta$ relation of the frame is obtained using Δ at centre of gravity of external force system instead of using Δ at roof level, because the energy obtained using Δ at centre of gravity of external force system alone gives the balance between the external work done and total energy dissipated by the structure. Using these three quantities, the extent of damage in the structure is estimated at any displacement level.

In the present study, to represent the damage state of a structure in each incremental load step of the pushover analysis, a cumulative dissipated energy is used. Based on the capacity curve of a structure, the damage state of the structure can be seen in four ranges (Figure 1). Point A indicates *elastic state* of the structure, point B a state in between *elastic state* and *ultimate strength state*, point C an *ultimate strength state*, point D a state in between *ultimate strength state* and *collapse state*, point E the *collapse state*. Along range O to C (the *force-control region*), *strength* of the structure increases nonlinearly with displacement, and along C to E (the *displacement-control region*), *displacement* increases but *strength* reduces. Possible damage ranges are shown in Table 1.

In current study, *three methods* are considered for damage estimation of RC structures, and their efficacy examined in capturing the global damage state of the frames at different displacement excursions. Overall damage index is estimated as the ratio of *dissipated energy* to *total energy capacity* of the structure. The three methods considered are:

Method 1: $D_1 = \left(\frac{E - E_{ie}}{E_T - E_{ie}} \right) \times 100$,

(1)

Method 2: $D_2 = \left(\frac{E - E_e}{E_T - E_{ie}} \right) \times 100$, and

(2)

Method 3: $D_3 = \left(\frac{E_L - E_{NL}}{E_{LT} - E_{NLT}} \right) \times 100$,

(3)

where

E = Energy dissipated by structure at displacement level at which damage is being estimated;

E_{ie} = Initial yield energy of structure;

E_T = Total energy absorbed by structure;

E_e = Instantaneous elastic energy at displacement level at which damage is being estimated;

E_L = Linear energy at displacement level at which damage is being estimated;

E_{NL} = Nonlinear energy at displacement level at which damage is being estimated;

E_{LT} = Linear energy at maximum displacement of structure; and

E_{NLT} = Nonlinear energy at maximum displacement of structure.

DETAILS OF STRUCTURES

Both, 11 and 4 story buildings are 15 m by 15 m in plan (Fig. 2(a)). Their typical floor-to-floor height is 4m. The interior frame, as shown in figure 2(a), represents 2-D models of these buildings. The other 4 storey building is 50 m by 15 m in plan (Fig. 2(b)) and its floor to floor height is 4 m. The interior frame, as shown in figure 2(b), represents a 2-D model of the buildings.

Buildings are designed according to Indian codes of practice for plain and reinforced concrete4 (IS: 456) and earthquake resistant design5 (IS: 1893). The buildings are assumed to be situated in seismic zone V of IS: 1893–2002, with an intensity of 0.36g ground acceleration. Material properties are assumed to be 20 MPa for the concrete compressive strength, and 415 MPa for steel yield strength, for both, longitudinal and transverse reinforcements.

ANALYSIS OF STRUCTURES

A two-dimensional model of each structure is created in SAP2000 to carry out non-linear static analysis. Beam and column elements are modeled as non-linear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. Torsion effect in the structure is neglected.

As shown in figure 6, five points, labeled A, B, C, D, and E define the force–deformation behavior of a flexure plastic hinge (ATC40,1996)6. In the present study, failure of each member is considered in two ranges, as shown in figure 6, the first range is load control region in which the member strength carrying capacity increases and it falls in range B to C, the second range is the displacement control range in which the lateral strength carrying capacity decreases drastically, and the member is unreliable in supporting lateral load.

Once the structure is modeled with section properties, steel content and the loads on it, auto hinges are assigned to the elements, PMM hinges for columns and M3 hinges for beams for flexure failure criteria.

The structure is subjected to incremental lateral forces with IS1893 load distribution along the height of the structure and the lateral force at any story is calculated from the following formula:

$$F_i = V_b \frac{w_i h_i^2}{\sum_1^n w_i h_i^2}$$

(6)

Where Vb = Base shear

Wi = Seismic weight of floor i

hi = Height of floor i measured from base

Fi = Lateral force at floor i

External force profile ratio is shown in figure 7. The pushover curve, base shear versus displacement at the centre of gravity of external force profile (Fig.7) are plotted to calculate the energy. At any deformation, the area under the curve represents the total seismic energy absorbed by the structure, which is equal to the work of seismic loads acting on the structure.

BEHAVIOUR OF FRAME-1

The first yielding is observed at a base shear of 189 kN and roof displacement of 0.28 m, as represented in the figure 8 with dotted line. The solid line represents the curve used to calculate the energy. The ultimate base shear is 273 kN at roof displacement of 0.85 m. The structure reached an unstable state at a displacement of 0.92 m i.e at a drift of 2%, where all base columns failed and entered the displacement control region, in which the structure is not reliable enough to support lateral loads any more.

In this case, neither structure reached 4% of the drift or 2/3 of the ultimate strength. So the damage state of the structure was assumed to be 100%, at structural instability. The points shown on the solid line represent the critical points defined on the pushover curve, to know the damage state of the structure. For this structure, D and E points were located at the same point, which depicts that the structure became unstable even before the ultimate base shear reduced by 15%. The damage calculated at all critical points are presented in table 2. Damage method-1, the damage at ultimate base shear is 92%, and method-3 represents is 81%. This deference is because of damage method-1, which considers the linear members energy as non-linear in the non-linear part of the pushover curve. Throughout the pushover curve, some members are in linear state and some are in non-linear state. Damage method-3 considers this effect and gives the exact damage state of the structure and represents the margin left for total damage.

The damage profile of the three methods is presented in figure 11. The damage profile of method-1 is linear; method-3 profile resembles the deflection profile of the structure. The profile of damage method-3 clearly depicts, the curve becoming almost straight after point C, this means that the damage increases rapidly from point C to D, which depicts the real behavior of the structure. Damage method-2 gave unrealistic values and was not considered for comparison at this point. Hinge formation is shown in figure 14. At point C, where the strength degradation starts, one of the bottom storey columns enter the displacement control region, at which the member cannot be relied upon, to support lateral loads. After crossing point C, the structure attained an unstable state when all bottom storey columns entered into displacement control region.

BEHAVIOUR OF FRAME-2

The first yielding occurred for a base shear level equal to 55 kN and roof displacement of 0.04 m, it is depicted in the figure 9 with a dotted line. The solid line represents the curve used to calculate the energy. The ultimate base shear was 155 kN at roof displacement of 0.32 m. The structure reached an unstable state at a displacement of 0.33 m i.e., at a drift of 2%, where some columns of storey-2 and storey-3 failed, and entered into the displacement control region, in which the structure was not reliable enough to support lateral loads any more.

In this case, neither structure reached 4% of the drift nor 2/3 of the ultimate strength as the structure reached an unstable state before that. So, the damage state of the structure is assumed as 100% at structural instability. The points shown on the solid line represent the critical points defined on the pushover curve, to show the damage state of the structure. For this structure, points D and E were located at the same point, which shows that the structure became unstable even before the ultimate base shear reduced by 15%.

The damage profile of the three methods is presented in figure 12. The structure resisted considerable displacement, to reach point B after yielding. Damage method-3 clearly shows that after point C, the curve becomes almost straight. This means that the damage increased rapidly from point C to D. Damage method-2 shows unrealistic damage values.

Hinge formation is shown in figure 15. At point B, where the stiffness of the structure reduces by 15%, some beam and column members yielded. At point C, where the structure reached ultimate strength, and where the strength degradation started the second and third storey column entered the displacement control region at which the members were not reliable enough to support lateral loads any more. After crossing point C, the structure attained an unstable state by entering a few other columns of the second and third storey into the displacement control region and the structure attained an unstable state, without reaching 4% drift displacement or 2/3 of ultimate strength drop.

BEHAVIOUR OF FRAME-3

The first yield occurred at a base shear of 198 kN and roof displacement of 0.05 m. Pushover curve is represented in figure 10 with dotted line. The solid line represents the curve used to calculate the energy. The ultimate base shear was 471 kN at a roof displacement of 0.28 m. The structure reached unstable state at a displacement of 0.4 m i.e, at a drift of 1.6%, where almost all columns of storey-3 failed and entered the displacement control region, in which the structure was not reliable enough to support lateral loads any more.

In this case, the curve is considered up to 2/3 of the ultimate strength, (Fig 10) which is represented by point E, on the curve. The damage state of the structure is assumed as 100% at point E. The points shown on the solid line represent the critical points defined on the pushover curve, to know the damage state of the structure. Good performance is shown by the structure with good ductility.

The damage calculated at all critical points is presented in table 4. For damage method-1, damage at ultimate base shear is 65% and for method-3 it is shown as 29%. There is a good margin between ultimate strength and total failure. In between, at point B, the damage by method-1 is 75% and by method-3 is 43%, still there is a good margin between point D and total failure.

The damage profile of the three methods is presented in figure 13. The damage profile of method-1 is almost linear, method-3 profile resembles the deflection profile of the structure. Method-3 profile clearly indicates the structure ductility between point D and E, where the profile curve is flattened, not steep, unlike other structures.

Hinge formation is shown in figure 16. At point B, where the stiffness of the structure reduced by 15%, no member reached the displacement control region. At point C, the third storey columns entered the displacement control region. At point D, total instability for storey 3 was attained, which continued till point E. finally, the damage was concentrated at storey-3.

CONCLUSIONS

For a 11 storey structure, with an aspect ratio of about 3, it is observed that the damage is concentrated at the bottom storey columns. Sudden failure observed after ultimate base shear.

For a 4 storey-3 bay structure, with an aspect ratio of about 1, the damage is concentrated at the second and third storey columns and shown good ductile behaviour from B to C.

For 4 storey-10 bay structure, with an aspect ratio of 0.3, the damage is concentrated at the third storey columns and frame shown good ductility even after ultimate base shear.

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Table 1: Description of nonlinear behaviour of a typical structure during strong earthquake shaking.

Range of deformation	***Behaviour***	***State***
OA	Elastic	No damage
AB	Strain hardening	Light damage
BC	Ultimate strength	Moderate damage
CD	Strength reduction	Severe damage
DE	Imminent collapse	Extreme damage and collapse

Table 2. Damage in percentage at 5 points on pushover curve for 3 frames

Failure criteria	***Damage Method***	***Damage at Point_A***	***Damage at Point_B***	***Damage at Point_C***	***Damage at Point_D***	***Damage at Point_E***
Frame-1	Method-1	0	19	92	100	100
	Method-2	0	9	75	89	89
	Method-3	0	2	81	100	100
Frame-2	Method-1	0	22	93	100	100
	Method-2	0	6	74	82	82
	Method-3	0	1	85	100	100
Frame-3	Method-1	0	18	65	75	100
	Method-2	0	5	50	66	92
	Method-3	0	0.5	29	43	100

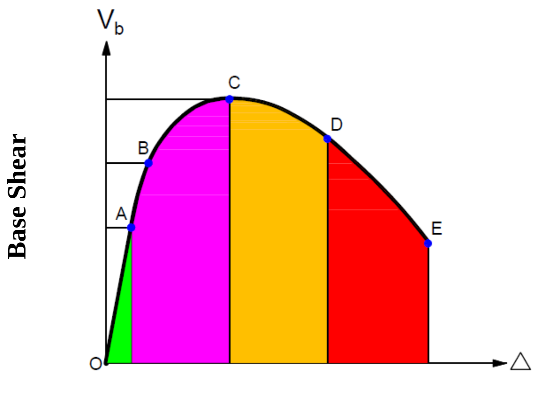
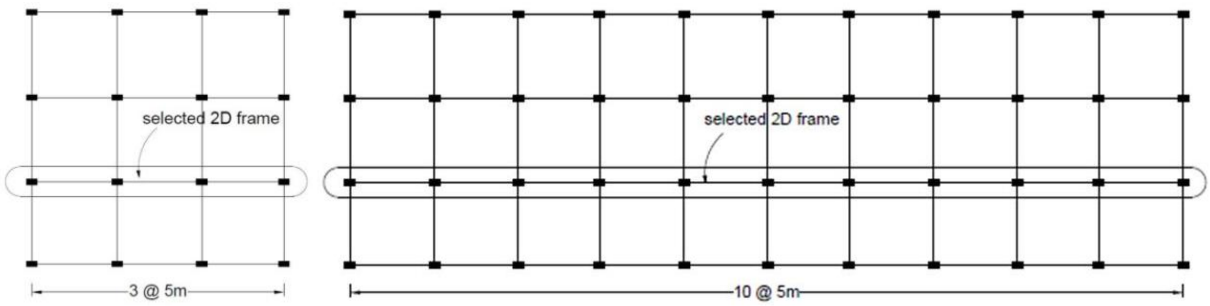
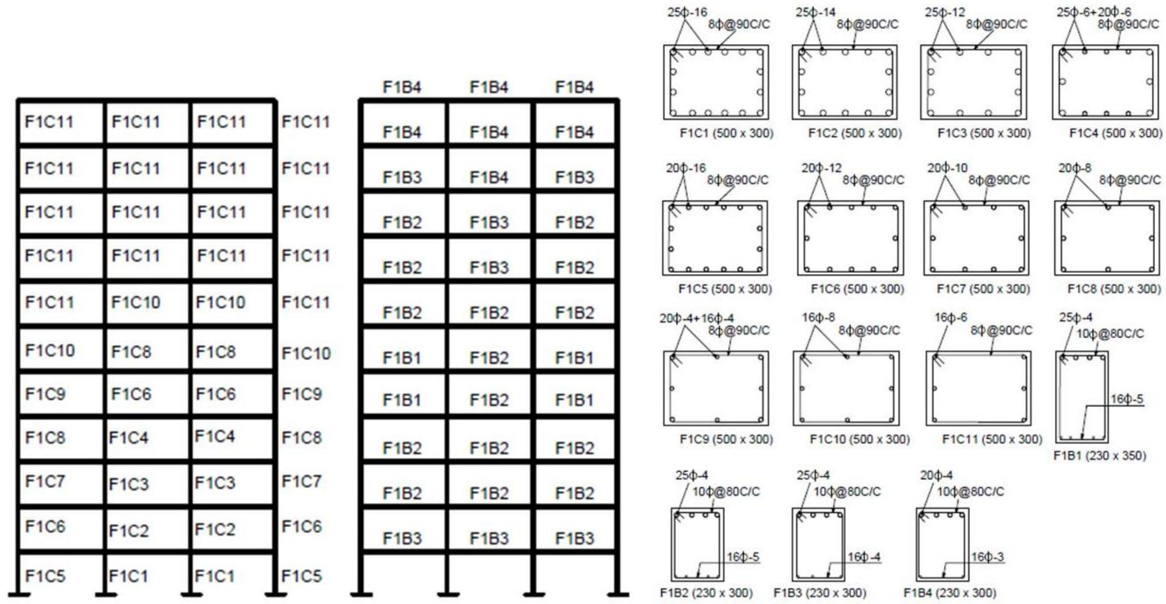


Figure 1: Critical points in the damage index estimation methods



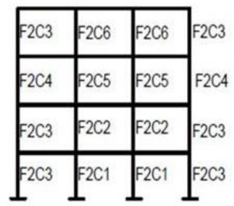
(a) (b)

Figure 2. (a) Plan view of 11 and 4 storey buildings. (b) Plan view of 4 storeys 10 bay building.

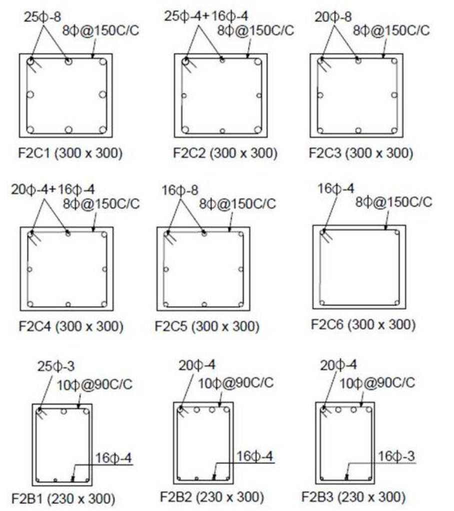


(a) Column labeling (b) beam labeling (c) Beam-Column reinforcement detail

Figure 3. Properties of frame-1 (11 storey-3 bay).

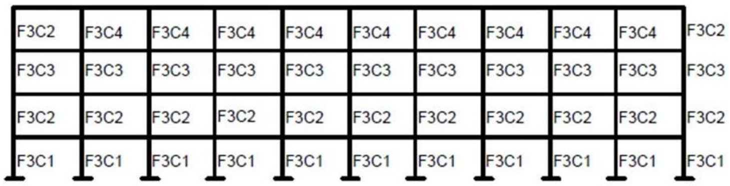


(a) Column labeling

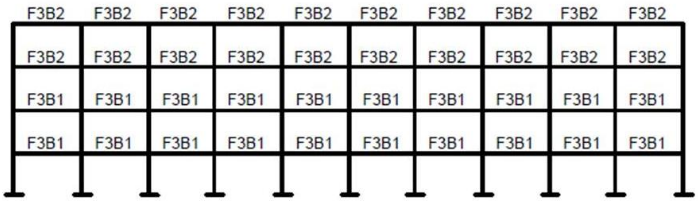


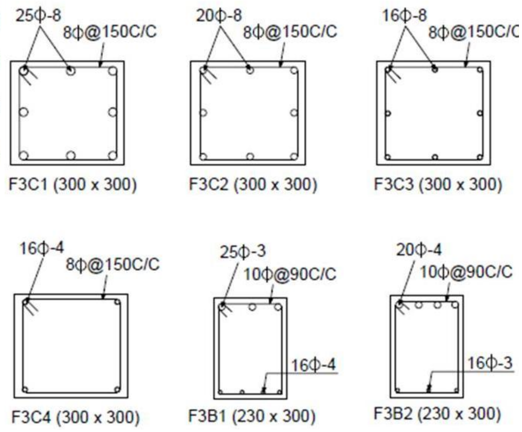
(b) Beam labeling (c) Beam-Column reinforcement detail

Figure 4. Properties of frame-2 (4 storey-3 bay).



(a) Column labeling





(b) Beam labeling (c) Beam-Column reinforcement detail

Figure 5. Properties of frame-3 (4 storey-10 bay).

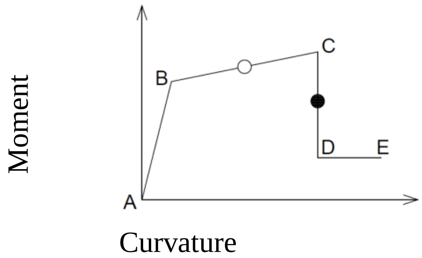
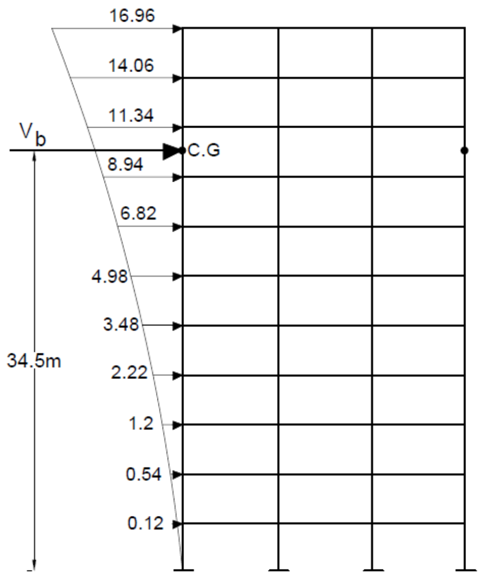
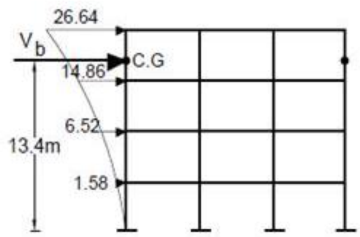
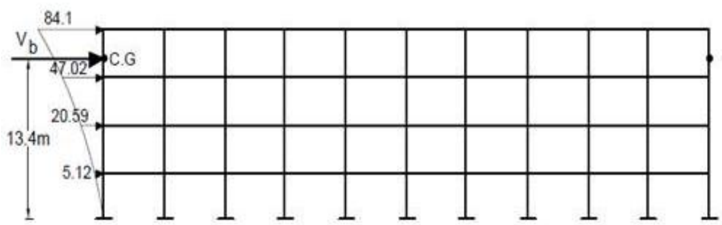


Figure 6. Force–deformation relationship of a typical plastic hinge.





(b) Frame-2



(a) Frame-1 (c) Frame-3

Figure 7. C.G of external force profile for 3 frames.

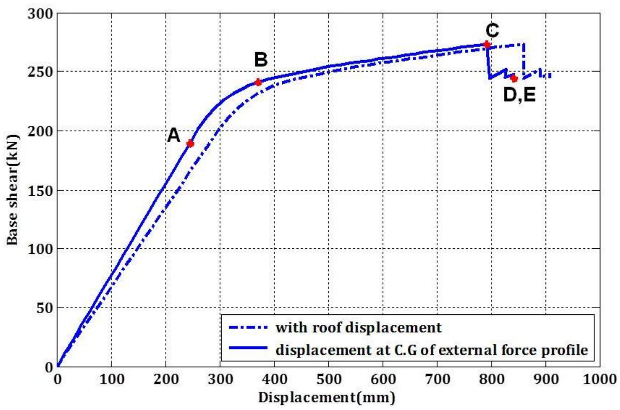


Figure 8. Pushover curves for frame-1.

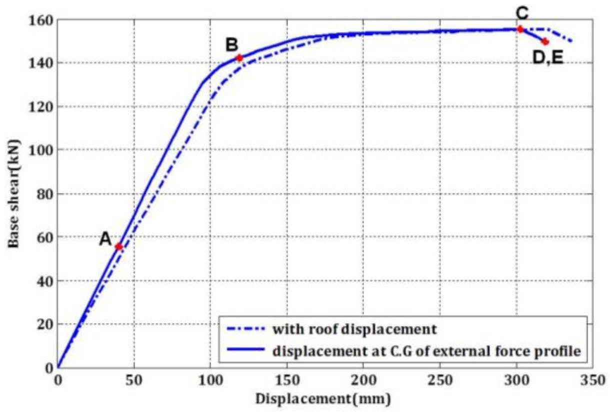


Figure 9. Pushover curves for frame-2.

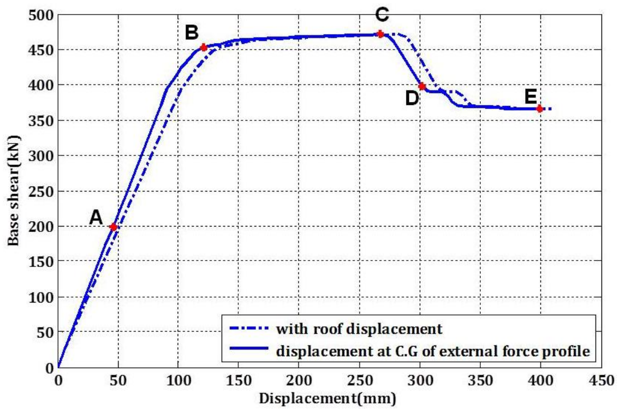
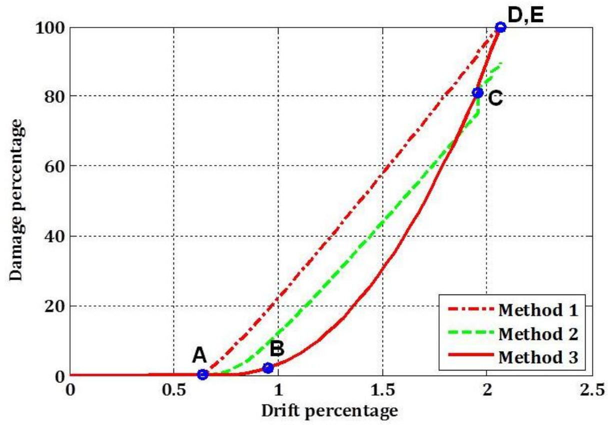


Figure 10. Pushover curves for frame-3.



F**igure11**. Damage estimation for frame-1.

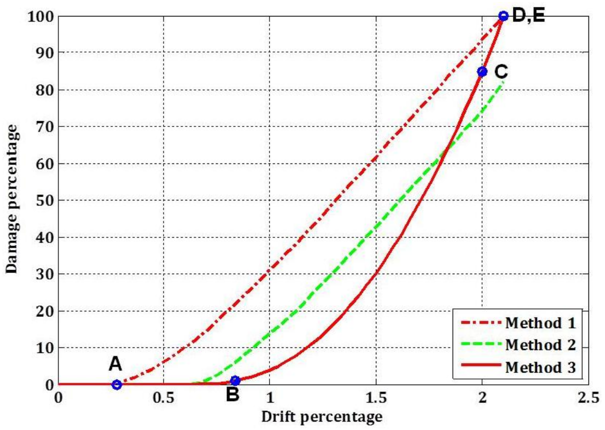


Figure 12. Damage estimation for frame-2.

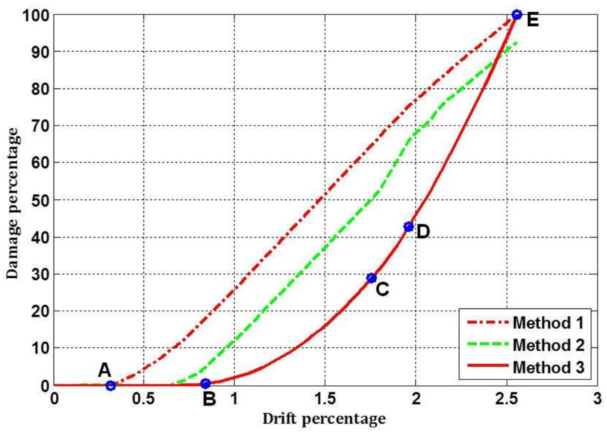
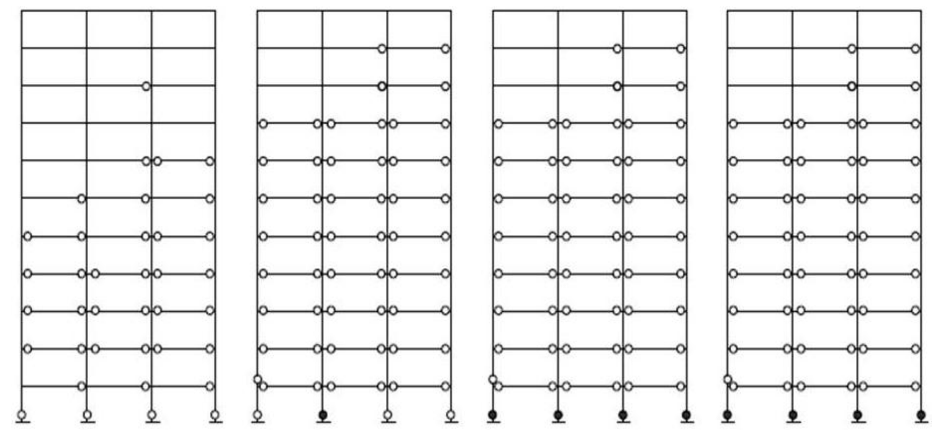
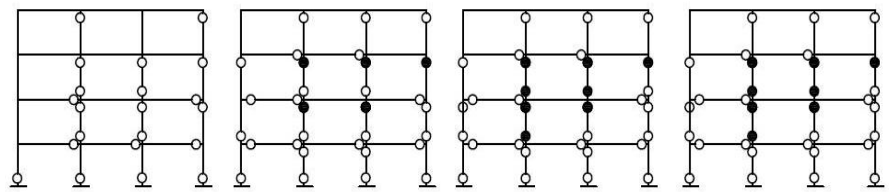


Figure 13. Damage estimation for frame-3.



(a) at B (b) at C (c) at D (d) at E

Figure 14. Hinge pattern at defined points on pushover curve for frame-1.



(a) at B (b) at C (c) at D (d) at E

Figure 15. Hinge pattern at defined points on the pushover curve for frame-2.

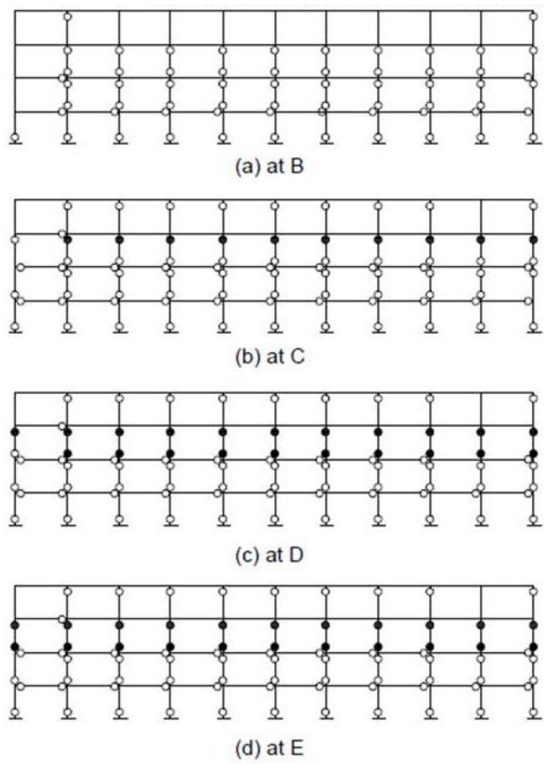


Figure 16. Hinge pattern at defined points on the pushover curve for frame-3.