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SIMPLE LINEAR ELASTIC STATIC ANALYSIS PROCEDURE TO ATTAIN DESIRED COLLAPSE MECHANISM FOR MOMENT RESISTING FRAMES

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Abstract

Considerable differences exist in current code provisions on the required (flexural) column-to-beam strength ratio (CBSR) at a joint to ensure that ductile flexural hinges occur at beam ends prior to that in columns (and joint). A simple procedure is proposed using results of linear elastic static analysis, adhering to strong-column weak-beam (SCWB) design philosophy, which leads to desired inelastic behavior of moment frame buildings during strong seismic action. The proposed procedure utilizes the ratio of *elastic demand*, determined just prior to the formation of first hinge, and *over strength flexural capacity* at all beam column joints. This procedure guides the designer to arrive at the required relative strengths at each joint of the building.

Keywords: Strong-Column Weak-Beam; Column-Beam Strength Ratio; Soft Storey

1. Introduction

Moment resisting frame (MRF) is the commonly used Lateral Load Resisting System (LLRS) in low-rise multi-storied buildings. But, buildings with MRFs as the sole LLRS have not given satisfactory performance in strong earthquake shaking in the past; partial or total collapse occurred in both steel and reinforced concrete (RC) buildings during past earthquakes. This poor performance was attributed in part to various factors including: (a) unexpected type of ground shaking, (b) smaller earthquake shaking intensities considered in design, (c) amplifications in soft soil, (d) liquefaction of poor soils, (e) insufficient ductility capacity than that was anticipated or assumed in the design process, (f) poor connections in steel buildings, or poor anchorage of beam bars into columns in RC buildings, (f) errors in design and construction, including poor member detailing, and (g) insufficient load paths in the structural system.

Capacity design philosophy is adopted to prevent undesirable seismic performance of buildings; its objective is to ensure that gravity load carrying capacity is not jeopardized at any instant, thereby ensuring no collapse of buildings during earthquakes. Thus, *ultimate limit state* is in focus and not *serviceability limit state*. Two critical aims of this design process are to ensure: (i) a *strength hierarchy* within and between members, which ensures only ductile modes of damage through design and precludes all brittle modes of damage, and (ii) *realisation of a collapse mechanism* of the building, which allows ductile damage only at preferred pre-determined locations to maximize energy dissipation and increase deformability of the building. The ideal collapse mechanism is the *beam sway collapse mechanism* with ductile flexural plastic hinges at the ends of all beam and at columns bases (Fig. 1a) [1]; and the undesirable mechanisms is the *storey mechanism* with flexural hinges at columns (Fig. 1b) [2 to 4].

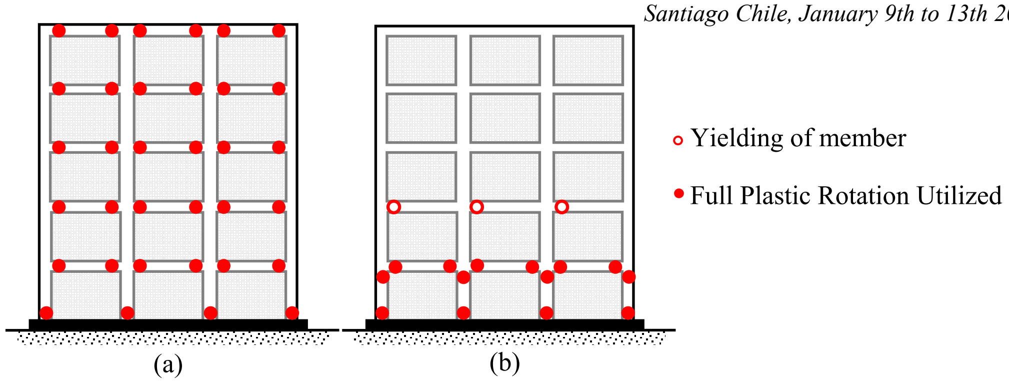

Fig. 1 – Plastic hinges in MRF members: (a) Ideal Collapse Mechanism and (b) Storey Collapse Mechanism

2. Column-to-Beam Strength Ratio

Column-to-Beam Strength Ratio (CBSR) β is employed to prevent the formation of undesirable storey mechanism [5]; and to guide the formation of beam hinges, thereby *enhancing the likelihood of formation of ideal collapse mechanism* [6]. MRFs have strong-column weak-beam (SCWB) design, if β>1, and weak-column strong-beam (WCSB) design, if β<1. Hence, the damage is limited to the *beam*, if $\beta > 1$, to the *column*, if $\beta < 1$, and to the *joint also*, if β is close to one. SCWB frames are better than WCSB frames from points of view of energy dissipation, ductility capacity, and distribution of damage [3 and 7]. Hence, seismic good design codes impose SCWB requirements on MRFs. The common quantitative definition of β at each joint is given with respect to the flexural strengths of members framing into the joint, but contained in a planar frame oriented in the considered direction of earthquake shaking, as

$$\beta = \frac{\sum M_c}{\sum M_b} > 1 \, ; \tag{1}$$

this inequality is based on static moment equilibrium at the joint. Studies on collapse of buildings revealed β to be the single most important factor governing collapse of buildings [8 and 9]; the probability of collapse is 4-5% of buildings conforming to SCWB criterion and 40-50% of those not conforming. β found its place in code provisions from early 1970s [*e.g.,* ACI 318, 1971], because of advantages of SCWB design. Over the past four decades, the definition of β has been revised considerably (Table 2). Most design codes recommend at least a minimum β, but some design codes place the responsibility on designers to proportion beams and columns, such that buildings achieve the ideal collapse mechanism [*e.g.*, AFPS, 1992].

Considerable differences exist in current provisions related to β in international codes, though the primary intent of providing the code provision is to prevent the formation of storey mechanism [5]. The issues in which they differ include:

(1) Flexural capacity used for evaluating β – *Design*, *Nominal* or *Overstrength* values;
(2) Location where moment equilibrium is considered – *Center of joint* or *face of joint*;
(3) Reduction in column moment capacity due to axial load – *Considered*, or *Not considered*;
(4) Effect of slab on beam moment capacity – *Considered*, or *Not considered*
(5) Moment magnification due to shear – *Considered*, or *Not considered*
(6) Limiting value of β – Varies from 1.0 to 1.4
(7) Conditions when β need not be satisfied – *Provided*, or *Not provided*
(8) Prescription to prevent storey mechanism – *Quantitative*, or *Qualitative*

All effects that increase the demand on the columns framing at the joint, should be considered [1, 6, 27 and 28]. But, uncertainties involved in computing β are many. This explains why buildings have failed in past earthquakes by formation of storey mechanism, even though they satisfied the stated requirements of β [29 and 30]. Hence, adequacy of current provisions is questioned in assisting the formation of ideal collapse mechanism and merely adopting stated β requirement is found not to guarantee that buildings will not form column hinges [31 to 38]. The value of β required for preventing the formation of column hinges is much more than those prescribed in codes [6].

Table 1 – Code provisions on β

Definition	*Remarks*
$\frac{\sum M_{c,design}}{\sum M_{b,design}} \geq 1.0$	Introduced to avoid column hinges [10]
$\frac{\sum M_{c,design}}{\sum M_{b,design}} \geq 1.2$	Sought to reduce likelihood of column yielding [11]
$\frac{\sum M_{c,nominal}}{\sum M_{b,nominal}} \geq 1.4$	Flexural capacity computed without strength reduction factor based on rectangular cross-section neglecting T-beam effect [12]; and later considering T-beam effect [13]
$\frac{\sum M_{c,nominal}}{\sum M_{b,nominal}} \geq 1.2$	Flexural capacity computed without strength reduction factor; consider T-beam strength; moment equilibrium at the face of connection [14]; columns not satisfying the inequality considered as gravity columns [15]
$\frac{\sum M_{c,design}}{\sum M_{b,design}} \geq 1.2$	Moment equilibrium at center of joint [16 to 18]; higher mode effects in tall buildings [19]
$\frac{\sum M_{c,design}}{\sum M_{b,design}} \geq 1.3$	Introduced to avoid column hinges [20]
$\frac{\sum M_{c,design}}{\sum M_{b,design}} \geq 1.1 \text{ to } 1.3$	Varies depending on type of structure [21]
$\frac{\sum M_{c,design}}{\sum M_{b,nominal}} \geq 1.4$	Column design strength to beam nominal strength; moment equilibrium at center of joint [22]
$\frac{\sum M_{c,nominal}}{\sum M_{b,nominal}} \geq 1.0$	Reduced nominal capacity of column due to axial force; nominal flexural capacity of beam including effect of shear amplification from location of plastic hinge to center of joint [23]. β requirement exempted: (1) in single storey buildings and top storey of multi-storey buildings with factored axial load less than 30% of capacity; (2) if sum of design shear strength of all exempted columns in the storey is less than 20% of required storey shear strength; and (3) if the sum of design shear strength of all exempted columns on each column line within that storey is less than 33% of required storey shear strength on that column line; column line defined as a single row of columns located within 10% of plan dimension perpendicular to the row of columns
$\frac{\sum M_{c,nominal}}{\sum M_{b,overstrength}} \geq 1.0$	Nominal moment capacity $M_{c,nominal}$ of column reduced to account for axial force, *i.e.*, $M_{c,nominal} = Z_c\,(F_{yc} - P_{uc}/A_g)$; Overstrength moment capacity $M_{b,overstrength}$ of beam including uncertainty in material strength, strain hardening and shear amplification, *i.e.*, $M_{b,overstrength} = 1.1R_y\,F_{yb}Z_b + M_{shear}$ [24 to 26]

Thus, literature clarifies that the minimum value of β of 1.2 to 1.4 adopted in codes is highly insufficient to preempt plastic moment hinges in beams in place of columns; value of β of at least 2.4 to 2.8 is required [39]. But, there is lack of clarity on the limits of applicability of this range of β. Time and again factors influencing β have been reported in the literature, with an objective of recommending a particular value of β with which the desired behavior of the structure can be attained [28, 33, 36 and 40 to 49]. Ideally, if column should not form the plastic hinge, the *overstrength* moment *demand* from the *beam* should be less than the *design* moment *capacity* of the *column*. The overstrength demand on the beam is limited to the overstrength moment capacity of beam. And, if the ratio of *overstrength moment capacity* and *design moment capacity* of the *beam* is in the range of 1.5 to 2.0, the *design moment capacity* of the *column* should be at least 50% more than this (to account for dynamic effects and to be reasonably away from β=1); the design moment capacity of the column should be at least in the range (1.5×1.5=) 2.25 to (2.0×1.5) = 3.0 times *design moment capacity* of the beam. Thus, the suggestion made in literature seems reasonable that the design moment capacity of the column should be 2.4 to 2.8 times the design moment capacity of beam. Clearly, the values of β currently adopted in codes of 1.2 to 1.4 are only about a half or even less of that required based on the above simple mechanistic argument.

Thus, it is evident that lack of clarity exists in the definition and use of β to attain the intended inelastic behavior of building during earthquakes. To overcome this, an alternate procedure is presented for symmetric building, using ratio of linear elastic demand estimated just prior to the formation of first hinge to the overstrength capacity of member (*eDoCR*) at all possible hinge locations, to prevent formation of undesirable hinges prior to formation of desirable hinges, and to ensure the formation of ideal beam sway collapse mechanism.

3. Proposed Linear Elastic Static Procedure

A member remains elastic, if the imposed demand is less than its overstrength flexural capacity. On the contrary, a member forms a plastic hinge, if the imposed demand reaches the overstrength capacity. Thus, the propensity of hinge formation may be assessed using the ratio of imposed demand to the overstrength capacity of the member. In the present paper the terms yielding and non-yielding members refer to members that yield and that remain elastic, respectively, when the ideal beam-sway collapse mechanism is formed. Thus, when buildings attain ideal beam-sway collapse mechanism, the ratios of imposed demand to overstrength capacity, for all yielding members reach a value of 1 and in non-yielding members less than 1, respectively (Fig. 2a). Although, overstrength capacities are known, during design stage, demands imposed on all members are not know, when the ideal collapse mechanism is formed, unless non-linear static analysis is performed. Consequently, using results of linear elastic analysis, it is not feasible to estimate the ratio of imposed demand to overstrength capacity of all members. But, using linear elastic analysis, demand imposed on members, just prior to the formation of first hinge, can be estimated. Therefore, it is feasible to estimate the ratio of linear elastic demand, estimated (just prior to the formation first hinge) and the overstrength capacity of member. Henceforth, for brevity, the acronym *eDoCR* refers to ratio of the linear *elastic Demand* (estimated just prior to the formation of first hinge) to the *overstrength Capacity* of the member.

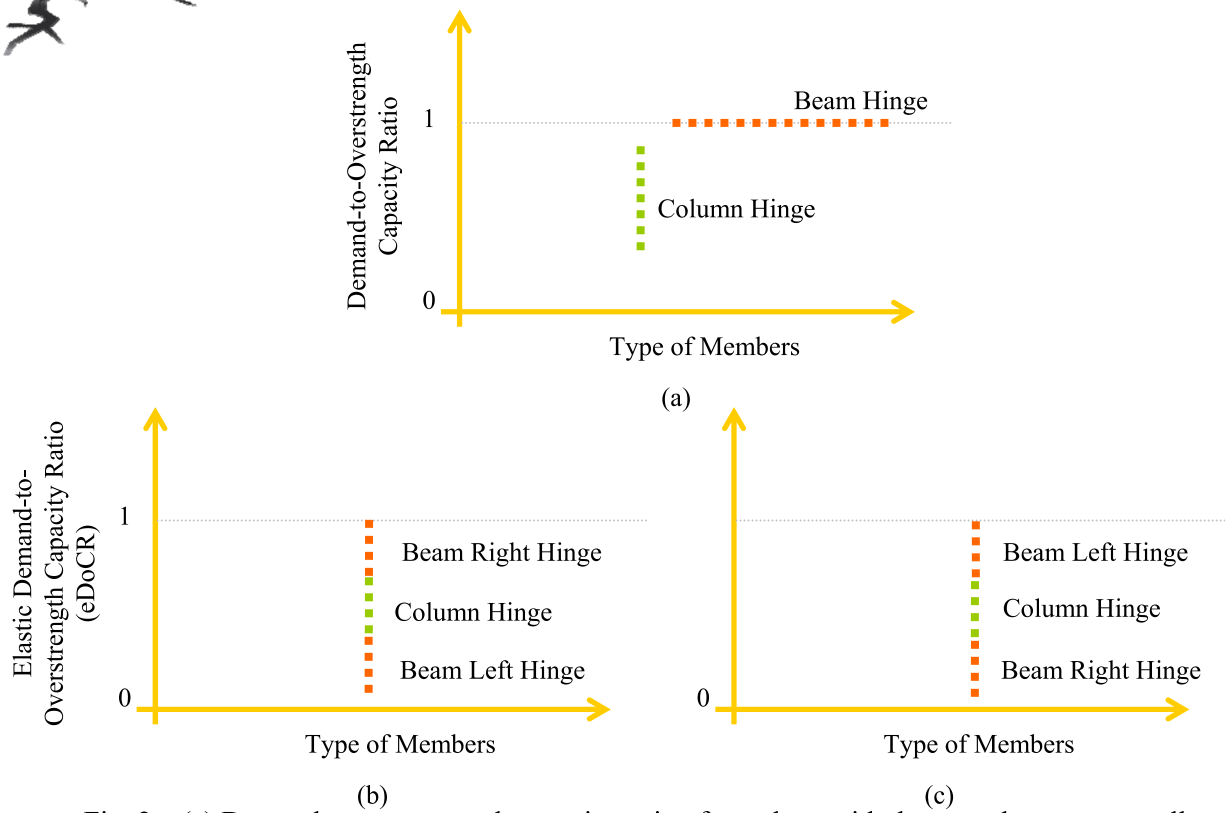

Fig. 2 – (a) Demand-to-overstrength capacity ratio of members with the onset beam-sway collapse mechanism, (b) and (c) Ratio of elastic demand, estimated prior to formation of first hinge, to overstrength capacity of member when the building sway to the right and left, respectively

Fig. 2b and 2c shows the *eDoCR* of a building, when it is pushed to the right and left, respectively. Terms '*beam right hinge*' and '*beam left hinge*' refer to hinges present in right and left ends of the beam, respectively. During earthquake shaking, a building sways in both directions. Consequently, location and lateral deformation at which the first hinge forms during the left sway of the frame may differ from those that form during the right sway of the frame. But, for frames with symmetric configuration, symmetric strength capacities of member and symmetric distribution of gravity loads, the lateral deformation at the formation of the first hinge during the right and left sway would be same, and the locations of the hinges would be a mirror image of each other. In such frames, *eDoCR* of all members are estimated as the maximum values obtained when the frame sways either to the right or left. Fig. 3a and 3b show two possible distributions of maximum *eDoCR* of all members; (1) Case A: *eDoCR* of all yielding members is larger than those of non-yielding members, and (2) Case B: *eDoCR* of some yielding members are lower than that for non-yielding members. To quantify the extent of overlap between *eDoCR* of yielding and non-yielding members, the term "*percentage overlap*" is defined as the ratio of number of hinges having *eDoCR* between maximum *eDoCR* of non-yielding member and minimum *eDoCR* of yielding member to the total number of possible hinges. "*Percentage overlap*" in Case A (Fig. 3a) tends to zero and in case B (Fig. 3b) would be a finite number 0 and 100%.

The likelihood of formation of desired collapse mechanism is higher in Case A than in Case B, because, in case A, all designated yielding members have higher *eDoCR* than all designated non-yielding members. On the contrary, in Case B, *eDoCR* of some designated yielding members is lower than that of designated non-yielding members; this may lead to formation of undesirable hinge and/or undesirable collapse mechanism. But, numerical results (discussed in Section 4.1) indicate that Case B does not always lead to undesirable hinge formation and/or undesirable collapse mechanism. It is observed that an increase in percentage overlap leads to increase in undesirable effects. Also, by increasing the capacity of non-yielding members, percentage overlap can be reduced, thereby leading to better performance of the frame. Thus, the proposed procedure ensures the formation of ideal collapse mechanism, in symmetric buildings, by proportioning strength of non-yielding member such that the percentage overlap tends to zero.

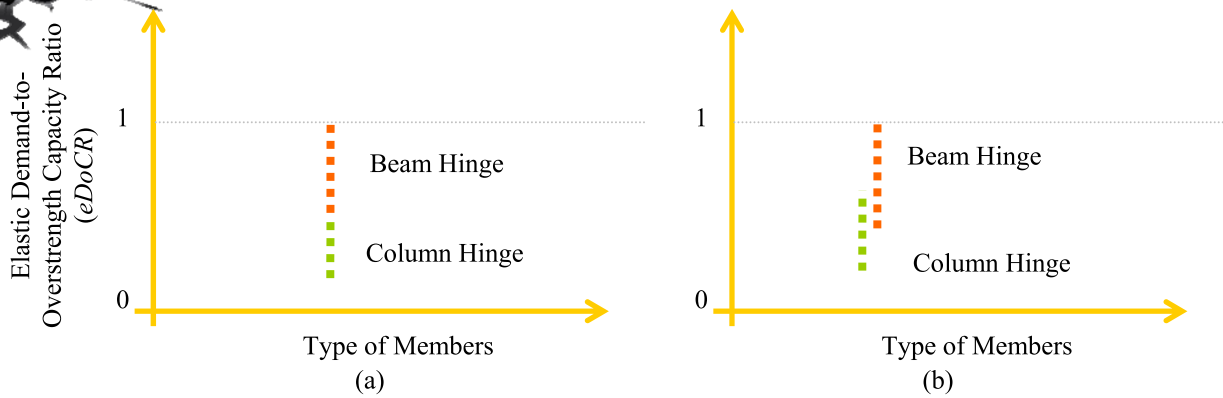

Fig. 3 – Maximum ratio of elastic demand, prior to formation of first hinge, to overstrength capacity of member: (a) NO Overlap observed, and (b) SOME Overlap observed between yielding and non-yielding hinge

The following step-wise procedure is proposed to achieve the desired collapse mechanism in a symmetric building:

Step 1 : Determine elastic flexural demand of all hinge locations;
Step 2 : Design and determine the flexural design capacity of beams;
Step 3 : Design shear reinforcement based on overstrength flexural capacity of beams;
Step 4 : Design columns considering loads from load combinations and minimum β required by the code;
Step 5 : Compute elastic demand to overstrength capacity ratio (*eDoCR)*, when the building is subjected to gravity and design lateral force; capacities of columns are determined using overstrength PM interaction;
Step 6 : Scale lateral force such that the at least one possible high has reached its capacity, *i.e.*, *eDoCR* of at least one member reaches the value of 1.0;
Step 7 : Determine *eDoCR* at all possible hinge location using the gravity and scaled lateral force as obtained in Step 6;
Step 8 : Reverse the direction of lateral load and determine *eDoCR* in the considered direction using Steps 5 to 7;
Step 9 : Determine the maximum *eDoCR* at all possible hinge locations by comparing *eDoCR* obtained in Step 7 and 8;
Step 10 : Check whether minimum *eDoCR* of a possible yielding member is smaller than maximum *eDoCR* of non-yielding member. If so, proceed to Step 11, and if not proceed to Step 12;
Step 11 : Increase the capacity of the non-yielding member such that there is no overlap between the *eDoCR* of yielding and non-yielding member;
Step 12 : Design shear reinforcement using overstrength flexural capacity of the column.

The proposed procedure does not explicitly account for the higher flexural demand on non-yielding members resulting from *dynamic amplification*. But, it implicitly accounts for the higher demand by increasing the capacity of non-yielding members to an extent that there is no overlap between the *eDoCR* of yielding and non-yielding members. Further, restricting axial demand on columns (while estimating *eDoCR*) to a value below the balance point of axial-flexural interaction increases the likelihood of attaining desired collapse mechanism. Alternatively, the effect of increased demand, due to dynamic amplification, can be addressed by outlining a maximum allowable value of *eDoCR* (=α) of non-yielding members, and thus, apportioning (1- α) capacity of the non-yielding members to account for possible increase in the demand. But, quantification of α requires a comprehensive study with multiple buildings subjected to a suit of ground motions, which is beyond the scope of this paper.

4. Numerical Study

Results of nonlinear static analyses are presented to ascertain whether the proposed procedure indeed leads to the formation of desired collapse mechanism. For this purpose, a 5-storey RC building is considered. To demonstrate the implication of percentage overlap (defined in Section 3) on formation of difference collapse mechanisms, the two buildings are with all properties identical, except that the flexural capacity of column set C2 is significantly larger than that of set C1.

The 5-storey RC building considered has a moment resisting frame as its LLRS. The building has 4 bays (of 4m each) and 6 bays (of 3m each) along the two plan directions. Ground storey columns are considered pinned at their bases and the typical storey height is taken as 3m (Fig. 5). The building is considered to be located in Seismic Zone V of India and resting on soft soil stratum. Apart from the self weight, gravity load considered includes Live Load of 2 kN/m^2 and Super-imposed Dead Load of 1 kN/m^2. In addition, the mass of 250mm thick masonry infills is considered, but its effect on lateral stiffness is not. Geometric details of RC members in the building are as follows: all beams are 300 × 400 mm, all columns are 400 × 400 mm, all slabs are150 mm thick. Unit weight of masonry and RC are 20kN/m^3 and 25kN/m^3, respectively. For the 5-storey building, equivalent design earthquake lateral load and load combinations are computed as per IS 1893 (Part 1) - 2002. Beams and columns are designed for shear; it is assumed shear failure is precluded prior to formation of plastic hinges. A typical interior frame is considered in this analytical study (Fig. 4); it is modeled with lineal frame members with lumped plastic hinges, using commercial software, SAP 2000 [50]. For the analysis and design of beams and columns, effective moment of inertia of $0.4I_{gross}$ and $0.6I_{gross}$, are considered, respectively, to account for cracked stiffness properties of the members [5]. The joints in MRFs are modeled as fully-rigid end-zones. Idealized elasto-plastic lumped plastic hinges are located at a distance *d/2* in beams/columns (of overall depth *d*) from the face of column/beam.

Although for the present study, linear elastic analysis is sufficient, pushover analysis was performed which validate the proposed procedure to attain the desired objective. Therefore, details pertaining to inelasticity in beams and columns are presented in this section. Inelasticity in beams is represented by pure flexural hinge and that in columns by axial-flexural hinge (*P-M* hinge). Details of reinforcement and moment-rotation characteristic of beams adopted for the study is as shown in Table 2 (Fig. 5a). Contribution of slab is ignored in the calculation of stiffness and flexural capacity of the beam. Tension side reinforcement and compression side reinforcement were assumed to be same. For the calculation of overstrength capacity of both beams and columns effective confined concrete properties are used (Concrete compressive strength f_c=28MPa; strain corresponding to peak stress ε_{co}=0.003; ultimate strain ε_{cu}=0.005).

Details of column sizes, reinforcement and flexural capacities used in the study building, are as shown in Table 3. P-M interaction varied depending on the percentage reinforcement in the column (Fig. 5b). *P-M* envelope for all sections with percentage reinforcement between 2.43% and 1.22% was observed to be encompassed between the curves shown in Fig. 5b. In the *M-θ* curve of the column, yield rotations θ_y are calculated for different levels of axial load as per FEMA 356 [51] using

$$\theta_y = \frac{ML}{6EI}, \tag{2}$$

where M, L, E and I refer to the moment capacity corresponding to each axial load, effective length of the member, modulus of elasticity of the material, and moment of inertia of the section. Plastic rotations θ_p, θ_1, and θ_2 (Fig. 5a) are largest corresponding to balance point and are taken as 0.015, 0.018 and 0.025, respectively, as per FEMA 356. Plastic rotations θ_p, θ_1, and θ_2 are taken as zero corresponding to point with pure compression failure of the column. For intermediate points, θ_p, θ_1, and θ_2 are linearly interpolated between the said maximum and minimum values.

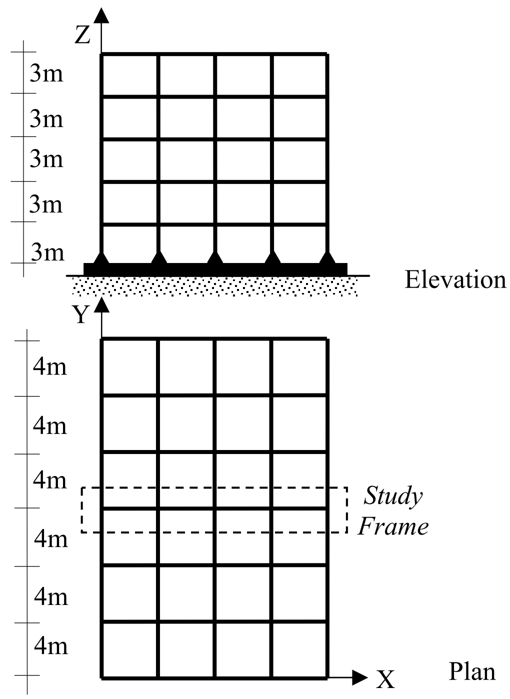

Fig. 4 – Elevation and Plan of the considered building

Table 2 – Details of beam reinforcement and force-deformation characteristics

Beam Label	*Floors*	*Reinforcement Detail*	*$M_{b,design}$ (kNm)*	*$M_{b\Omega}$ (kNm)*	*θ_y (rads)*	*As a multiple of θ_y*		
						θ_p	*θ_1*	*θ_2*
B1	All	2ϕ 22 + 1ϕ 20	108	132	0.00550	4.30	*6.45*	*8.60*

Table 3 – Column size considered to reduce DCRs of non-yielding members

Column Label	*Column size (mm×mm)*	*ρ (%)*	*Reinforcement, Number of layers of reinforcement*	*Moment capacity (kNm)*		
				$M_{c,\ design,\ P=0}$	*$M_{c,\ \Omega,\ P=0}$*	*$M_{c,\ \Omega,max}$*
C1	400×400	1.90	8 ϕ22, 3layers	150	178	205
C2	600×600	1.37	8 ϕ 28, 3layers	402	469	672

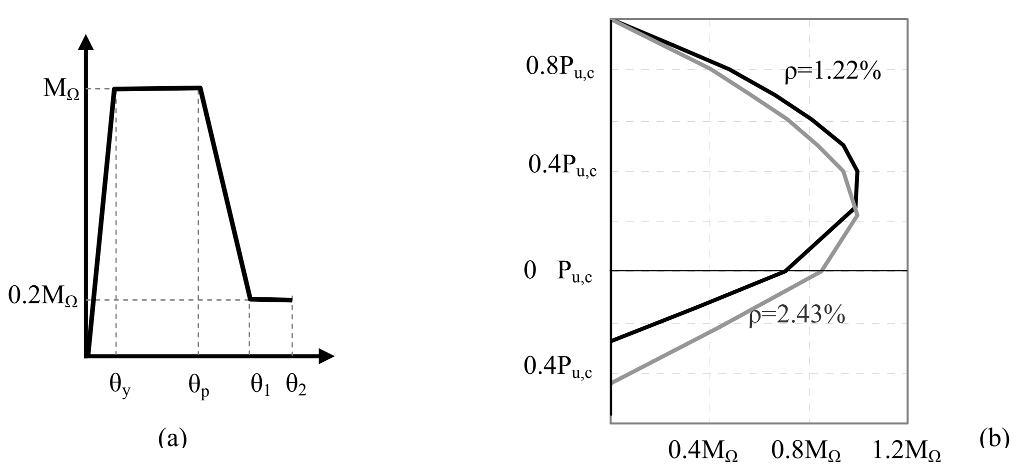

Fig. 5 – (a) Normalised Moment-rotation for all beams and columns;

and (b) Normalized P-M hinge for columns;

4.1 Results

For the considered frame, Column C1 (Table 3) and Beam B1 (Table 2) satisfy both load, including the minimum requirement of β (>1.2), and deformation requirements stipulated by the code. Thus, Column C1 and Beam B1 are considered the starting point for the proposed procedure (Steps 1 to 4). Maximum *eDoCR* at all possible locations of hinge are determined, from Steps 5 to 9. Using the maximum values *eDoCR,* the percentage overlap is estimated as 85%, which is significantly higher than the ideal value of zero, and as expected, pushover analysis of the frame indicates storey mechanism as its collapse mechanism (Fig. 6a). Thus, it is evident that the code prescribed value for β is not sufficient to prevent formation of undesirable collapse mechanism. To pre-empt the formation of the ideal collapse mechanism, the percentage overlap needs to be reduced to zero (or to a value close to zero). To achieve this, columns C1 are replaced with columns C2, which have significantly higher flexural capacity than columns C1. Thus, the *eDoCR* of columns decreased and led to a percentage overlap value of 13%, which is close to zero, and as expected, pushover analysis indicates the formation of ideal collapse mechanism (Fig. 6b). Although, the percentage overlap could still be reduced to a value lower than 13%, providing columns with higher flexural capacity than that of C2 would not make any difference in the collapse mechanism. And, any further increase in the percentage overlap, more than that estimated using column C2, does not lead to the formation of ideal collapse mechanism.

Although, several columns with flexural capacity, in between that estimated for C1 and C2, were considered, none of them led to the formation of ideal collapse mechanism; consequently, for brevity, results of the same are not reported. But, as expected, percentage overlap decreased and a gradual transition from undesirable behavior to desirable behavior was observed with increase in flexural strength of columns.

From the example, it is inferred that, zero percentage overlap would definitely lead to formation of ideal collapse mechanism. Additionally, values of percentage overlap near zero also lead to the formation of ideal collapse mechanism. But, in the latter case, the formation of ideal collapse mechanism needs to be verified using nonlinear pushover analysis. Thus, it can be concluded that a decrease in the percentage overlap accentuates the formation of ideal collapse mechanism (Fig. 7).

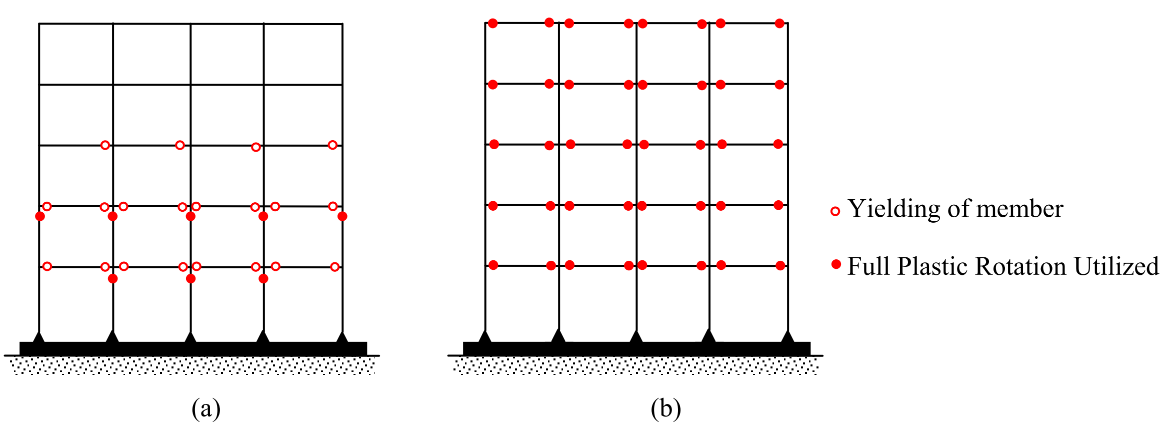

Fig. 6 – Collapse mechanism observed for the frame with: (a) Column C1, and (b) Column C2.

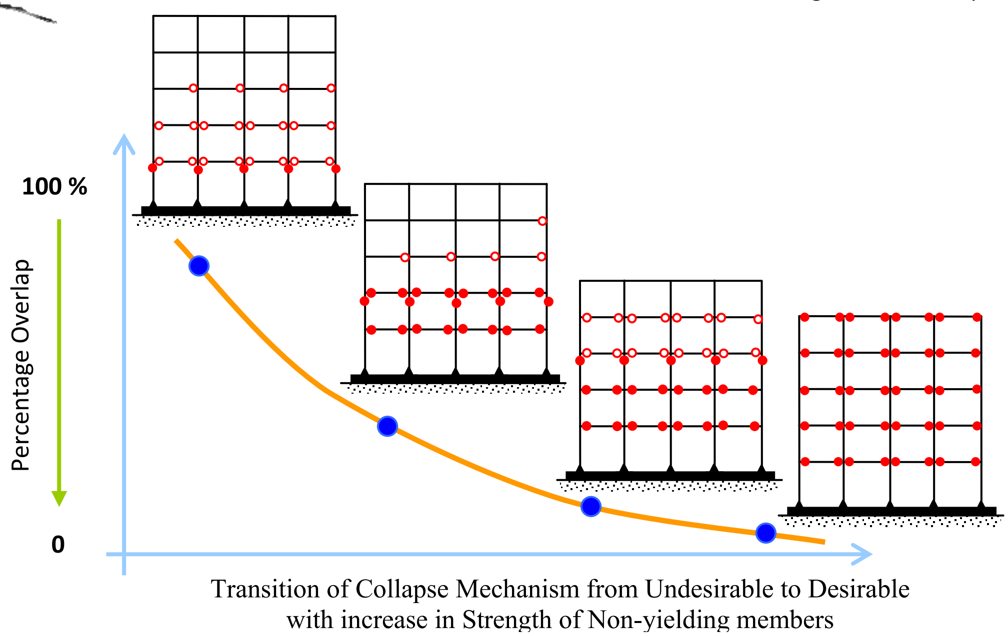

Fig. 7 – Decrease in percentage overlap leads to formation of ideal collapse mechanism

5. Summary

A lack of clarity exists in the definition and use of β to attain the intended inelastic behavior of building during earthquakes. Hence, a simple linear elastic static analysis procedure is proposed, which utilizes the ratio of elastic demand, determined just prior to the formation of first hinge, to the overstrength capacity at all possible hinge locations (*eDoCR*) and strength proportioning of members to attain desired inelastic behavior of building during earthquakes. The proposed procedure attains the said objective by ensuring minimum *eDoCR* of yielding members larger than the maximum *eDoCR* of non-yielding member. This method stands a simple tool for designers to use to learn the possible collapse mechanism of a building.

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