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Abstract*:* The aim of seismic evaluation is to assess the seismic capacity of earthquake vulnerable buildings or earthquake damaged buildings for the future use. It has been observed that majority of buildings damaged due to earthquake may be safely reused, if they are converted into seismically resistant structures by employing retrofitting measures. Retrofitting of buildings is generally more economical as compared to demolition & reconstruction even in the case of severe structural damage. The present work emphasize on the seismic evaluation & different retrofitting strategies of R.C. buildings. For this purpose a step by step procedure for seismic analysis is done for a four storey R C building according to IS 1893 (part 1) 2002, the demand capacity procedure is presented. The detailed calculations for demand & capacity of one of the perimeter framed - beam & column are presented systematically.

Key words: *Capacity/Demand method, Basic concepts of retrofitting techniques, Seismic analysis of R C framed building.*

I. INTRODUCTION:

The methods for seismic evaluation of existing buildings are i) qualitative methods, and ii) analytical methods, as shown in figure 1. The qualitative methods are based on the available background information of the structures, past performance of similar structures under severe earthquakes, visual inspection report, some non-destructive test results etc. However, analytical methods are based on considering the capacity and ductility of the buildings, which are based on detailed dynamic analysis of buildings. The methods in this category are capacity/demand method, pushover analysis, inelastic time history analysis etc. Brief discussions on the method of evaluation are as follows.

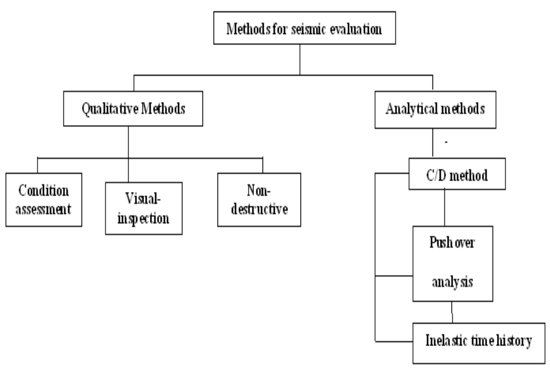

Fig. 1: Methods for Seismic Evaluation

Code-based Seismic Analysis methods:

Equivalent lateral force: Seismic analysis of most of the structures is still carried out on the basis of lateral (horizontal) force assumed to be equivalent to the actual (dynamic) loading. The base shear which is the total horizontal force on the structure is calculated on the basis of structure mass and fundamental period of vibration and corresponding mode shape. The base shear is distributed along the height of structures in terms of lateral forces according to Code formula. This method is usually conservative for low to medium height buildings with a regular conformation.

Response spectrum analysis: This method is applicable for those structures where modes other than the fundamental one affect significantly the response of the structure. In this method the response of Multi-Degree-of-Freedom (MDOF) system is expressed as the superposition of modal response, each modal response being determined form the spectral analysis of single-degree-of-freedom (SDOF) system, which are then combined to compute the total response. Modal analysis leads to the response history of the structure to a specified ground motion; however, the method is usually used in conjunction with a response spectrum.

Capacity/Demand (C/D) method: The method has been initially presented by Applied Technology Council (ATC). The forces and displacements resulting from an elastic analysis for design earthquake are called demand. These are compared with the capacity of different members to resist these forces and displacements. A (C/D) ratio less than one indicate member failure and thus needs retrofitting. When the ductility is considered in the section the demand capacity ratio can be equated to section ductility demand of 2 or 3. The C/D procedures have been subjected to more detailed examination in the light of recent advances in earthquake response studies. The main difficulty encountered in using this method is that there is no relationship between member and structure ductility factor because of non-linear behavior.

Basic concepts of retrofitting techniques are shown in following figures: (a) upgradation of the lateral strength of the structure; (b) increase in the ductility of structure; (c) increase in strength and ductility. These three concepts are schematically shown in following figure.

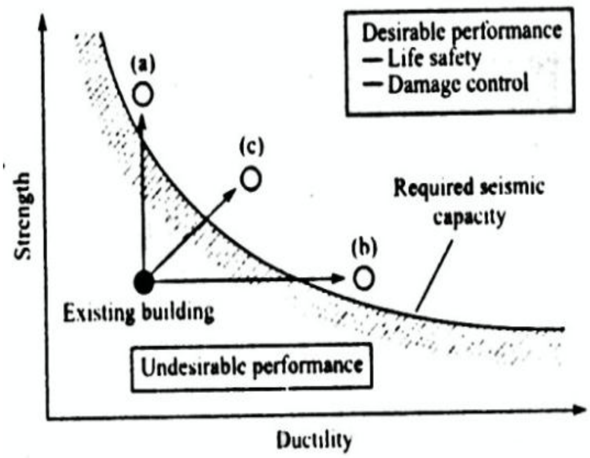

Fig. 2: Basic concepts of retrofitting techniques

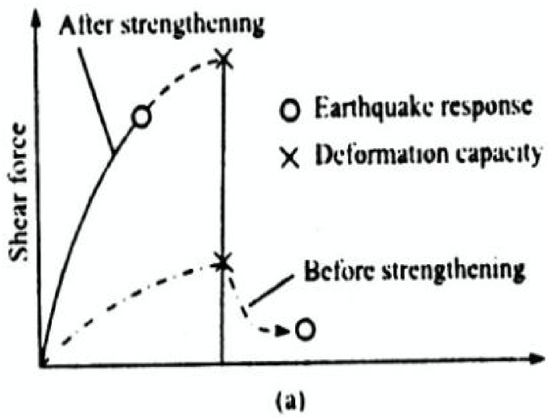

Fig. 3: Up gradation of the lateral strength of the structure

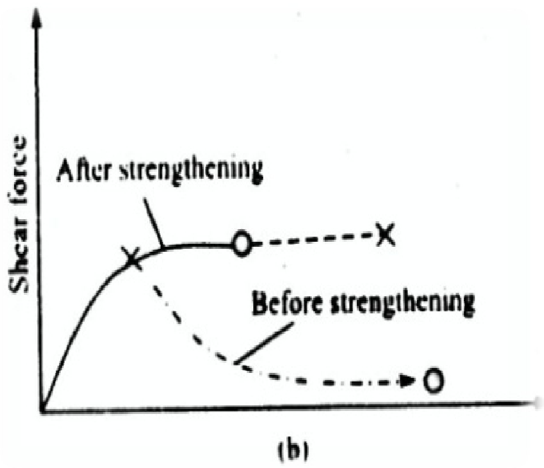

Fig. 4: Increase in the ductility of structure

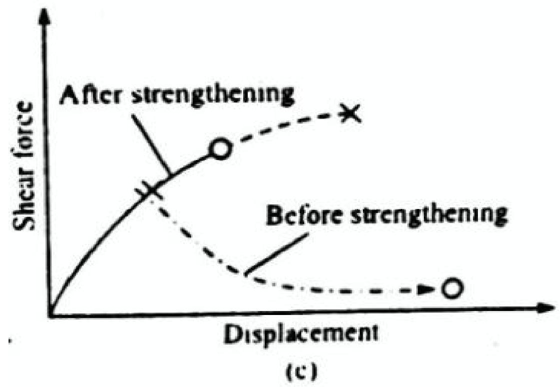

Fig 5: Increase in strength and ductility

II. LITERATURE REVIEW

Sengupta, A. K., Bedari Narayan, V. T. and Asokan, A. (2003) presented seismic retrofit of existing multistoried buildings in India – an overview of the methods and strategies. In this paper the steps in a retrofit programme of a building are given, they (steps) are seismic evaluation of the existing condition, decision to retrofit, selection, design and verification of the retrofit scheme, construction and subsequent monitoring. Studies emphasized a performance based seismic evaluation, as per ATC 40. Studies reviewed the local retrofit strategies of column, beam, beam to column joint; wall and foundation strengthening are reviewed. Studies indicated that under global retrofit strategies, the condition of infill walls, shear walls and steel braces and the reduction of the building irregularities are to be critically assessed. Studies concluded that it is necessary to have seismic evaluation of a building both for the existing and retrofitted conditions. Studies concluded that the performance based evaluation is a rational approach for selecting an effective retrofit scheme and to justify its cost.

Chhatre, A.G., Santosh Kumar, B, Singh, U.P., Ingole, S.M. and Dixit, K.B. (2003) presented seismic reevaluation of the Tarapur atomic power plants 1 and 2. The paper describes the details of the work accomplished during seismic re-evaluation of the two units of boiling water Reactors at Tarapur. Studies of NISA-CIVIL for the analysis and seismic re-evaluation of the civil structures viz. Reactor building, service building, Turbine building, Intake structure and Stack. Authors pointed that the reevaluation of civil structures has been completed as per ACI-349, 2001; these civil structures have been modeled by finite element method. Authors also carried out the time history analysis of these structures by using modal super position technique to arrive at the time histories and the response spectra at the various floors of the structures. Studies further concluded that these response spectra have been used for seismic reevaluation of the equipment and piping supported on various floor of the building.

Arya, A. S. (2003) Paper discusses the major earthquakes in India, have clearly indicated the fragility of the building stock in the country in practically all the states to the same extent. Author carried a study based on the building data in the vulnerability Atlas of India 1997, author studied that only in seismic zone 5th of India covering an area of 12% of the total land area of the country, and there are 11.1 million vulnerable housing units as per census of India 1991. Similar vulnerable buildings in seismic zone 4th also, the number is at least 50 million. Author also carried a test at Umerga (Maharashtra) on retrofitting of stone houses, author clearly brought out that if the simple techniques of using seismic belts in horizontal and vertical direction are installed in various masonry buildings, their seismic resistance can be improved to an extent that none of these houses will totally collapse even in one higher intensity of the earthquake occurrence. The study concluded that the cost of such retrofitting measures does not exceed 4% in seismic zone 3rd, 6 to 7% in seismic zone 4th and 8 to 10% in seismic zone 5th of the replacement cost of the building.

Case study

A four storey R.C. moment frame public building (Fig 6) is located in seismic zone five and on medium soil. The building measures 18m in x direction & 27 m in y direction in plan, floor to floor height of building is 4m and slab thickness is 200mm. Columns are placed of size (250 x 600) mm, size of beam is (250 x 700) mm. Material M_{20} and Fe415. Evaluate the building for seismic resistance and provide strengthening options if required for the deficiencies identified.

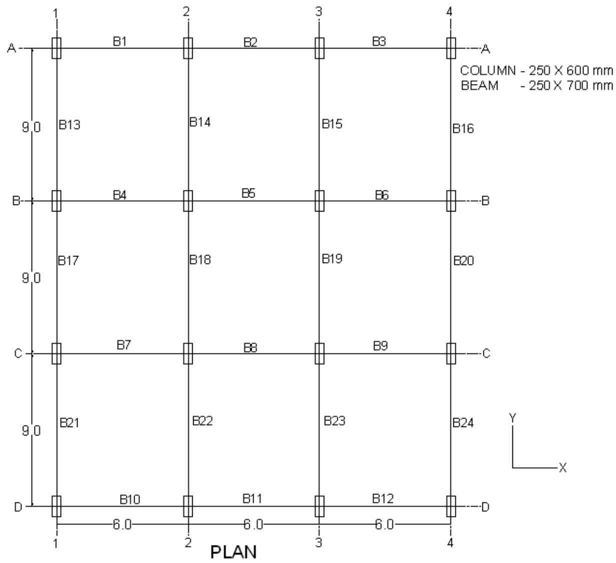

Fig 6: Building plan

Step by step procedure for analysis of a four storeyed reinforced concrete building as per IS 1893 (part 1) 2002

Lateral Load analysis of frame by equivalent static lateral force method.

Vb = AhW Vb = design seismic base shear.

Ah = Design horizontal accelerations spectrum value (7.6.1 Pg. 24 of IS 1893 (I) 2002)

Ah = $\frac{Z.\,I}{2\,R} \cdot \frac{Sa}{g}$ (7.5.3 Pg. 24, IS 1893 (I) 2002)

$Ta = \frac{0.09h}{\sqrt{d}}$ W = seismic weight of building.

$Ta = \frac{0.09\,x16}{\sqrt{18}}$ = 0.34 sec (in x direction),

$\frac{Sa}{g}$ = Spectral acceleration

$Ta = \frac{0.09\,x16}{\sqrt{27}}$ = 0.28 sec (in y direction)

Ta = Fundamental natural period of building [R. C. moment resisting with brick infill panel building].

Since seismic zone considered is five,
Zone factor Z = 0.36 (table 2 Pg. 16 of IS code).
I = Importance factor (Pg 18, Table 6), I = 1.5
R = Response reduction factor, R = 5.0 (SMRF)
Ta = 0.34 sec, corresponding to Ta = 0.34 sec, medium soil and 5% damping,

$\frac{Sa}{g}$ = 2.5 [Fig. 2, Pg. 16],

$\therefore$ Ah = 0.135.
Calculations of seismic weight on frame (D – D)
Storey 1. (Weight = Unit weight x Length x Breadth x Thickness).
Dead Load of Slab = 405.0 KN
D.L. of Beam = 78.75 KN
D.L. of Column = 49.5 KN
D.L of Wall = 273.24 KN
Live Load (50%) = 162 KN
$\therefore$ (W_1) = 968.49 KN , Storey W_2 = Storey, W_3 = 968.49 KN
Live load = 0 KN, $\therefore$ (W_4) = 659.79 KN
hence seismic weight of building = $W_1 + W_2 + W_3 + W_4$ = 3565.26 KN,
Since Ah = 0.135
V_B = base shear
= Ah x W = 481.31 KN.
V_{Bm} = modified base shear = 0.67 x V_B
= 322.48 KN.

Distribution of base shear along the height of building.

$$Q_i = V_{Bm} \ x \ \frac{w_i h_i^2}{\sum_{j=1}^{n} w_j h_j^2}$$

Table 1: Base shear calculations.

Storey No. (1)	W_i (kN) (2)	h_i (m) (3)	$W_i h_i^2$ (4)	$\frac{w_i h_i^2}{\sum w_i h_i^2}$ (5)	Lateral force at i^{th} level for EL in direction. Vbm x col.5 X- direction in 'KN'
1	968.49	4.0	15495.84	0.0401604	12.951
2	968.49	8.0	61983.36	0.1606419	51.804
3	968.49	12.0	139462.56	0.3614443	116.559
4	659.79	16.0	168906.24	0.4377533	141.167

$\Sigma = 385848$

Fig 7: Seismic forces in X-X direction

Demand Capacity Calculations for Beam.

Beam forces **-** [Load coming on each external beam]

Total dead load = $\frac{28.56}{2}$ = 14.28 kN/m, Total live load = $\frac{6}{2}$ = 3 kN/m.

Consider the combination 1.2[D.L. + L.L. + E. L.]

$\therefore$ 1.2 (DL + LL) = 20.736 kN/m.

1.2 E. L. = 1.2 x 141.167 = 169.400 kN.

Analysis for lateral load by portal method

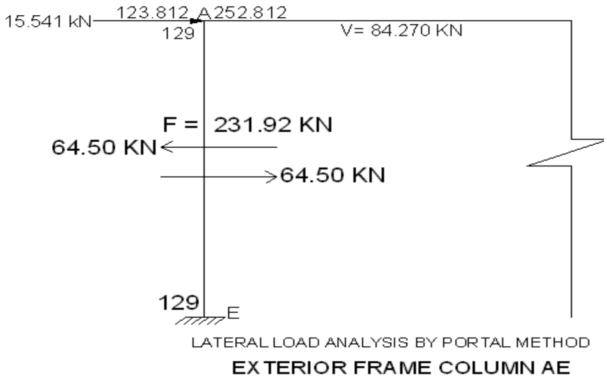

Fig 8: Forces on ground storey column

Moment due to gravity load

$$= \frac{wl^2}{8} = \frac{20.736 \times (6)^2}{8} = 93.312 knm$$

Moment due to lateral load = 252.812 KNM (at 1^{st} floor)

(1) Capacity of section

Considering span AB at 1^{st} floor of exterior frame D-D.

Calculation of Moment of Resistance in hogging.

C = force in compression, T = force in tension

C = 0.36 fck b.x_u, T = 0.87 fy Ast

Ast = 1388.58 mm^2 at support A.

$\therefore$ C = 0.36 x 20 x 250 x Xu ,
T = 0.87 x 415 x 1388.58

Equating C = T for balance section.

$\therefore$ 0.36 x 20 x 250 x Xu = 0.87 x 415 x 1388.58

$\therefore$ Xu = 278.52 mm.

Xu lim = 0.48 x d = 0.48 x 650 = 312 mm.

As Xu < Xu lim i.e. 278.52 mm < 312 mm.

i.e under R/F section.

$\therefore$ Moment of Resistant = T x Z = 0.87 x 415 x 1388.58 x (650 - 0.42 x 278.52)

M. R. = 267.23 KNM > 252.812 KNM.

D.C.R. = 252.812 / 267.23 = 0.94 < 1.0

Hence O.K. or safe. i.e. beam is not deficient.

Calculations of Moment of Resistance in Sagging.

Equating C & T, Ast = 1256.63 mm^2

0.36 x 20 x 250 x Xu = 0.87 x 415 x 1256.63

$\therefore$ Xu = 252.05 mm

Xu lim = 0.48 x d = 0.48 x 650 = 312 mm.

Xu < Xulim i.e. Under R/F section.

$\therefore$ Moment of Resistant = T x Z

= 0.87 x 415 x 1256.63 x (650 – 0.42 x 252.05)

$\therefore$ M.R = 246.87 KNM > 93.312 KNM (required moment at centre)

Demand capacity ratio = 93.312 (demand) / 246.87 (capacity)

D. C. R. = 0.38 < 1.0

Hence safe or O. K. i.e. beam is not deficient.

Check for Shear capacity of Beam.

The shear reinforcement provided in the existing beam at support is 2 legged, 8 mm dia. Fe 415 @ 120 mm c/c.

Asv = 2 x $\frac{\pi}{4} x (8)^2$ = 100.53 mm^2

Pt = 100 $\frac{Ast}{bd}$ = $100\, x \frac{100.53}{250\, x\, 650}$ = 0.061 %

τ_c = 0.28 Mpa. IS 456: 2000 table 19, pg.73

$V_u = V_{us} + \tau_{c\,bd}$

$V_{us} = \frac{0.87\ fy\, A_{sv}\, d}{S_v} + \tau_c$ bd

$V_{us} = \frac{0.87 \times 415 \times \left[\frac{\pi}{4} \times (8)^2 \times 2\right] \times 650}{120}$ + 0.28 x 250 x 650

= 242.10 KN (capacity)

Shear demand in beam.

Design shear force as per analysis = $\frac{wl}{2}$

= shear in beam due to gravity load

i.e 1.2 (D.L + L.L) + shear in beam due to lateral load moment.

= $\frac{20.736 \times 6}{2}$ + 84.27 = 146.48 KN,

$\therefore$ Shear Demand = 146.48 KN ----------------- (i)

Moment capacity of beam

M.R.^{H} = 267.23 KNM , M.R.^{S} = 246.87 KNM

L_c = clear span = 0.9 L = 0.9 x 6, L_c = 5.4m

Design shear force $Va^{D+L} = Vb^{D+L}$ = 62.208 KN

Vu from capacity design

Vu = 62.208 + 1.4 $\frac{M.R.Hogging + M.R.Sagging}{Lc}$ IS 13920 pg.5, fig.4

= 62.208 + 1.4 $\frac{267.23 + 246.87}{5.4}$

=195.49 KN------------------------------------(ii)

$\therefore$ final shear demand is greater of (i) & (ii)

i.e.195.49 KN

i.e. 242.10 KN > 195.49 KN

D. C. R. = $\frac{195.49}{242.10}$ = 0.807 < 1.0 $\therefore$ safe or O.K.

Thus, the shear capacity is greater than shear demand on the beam indicating the non deficiency of the beam (i.e. stronger) in shear under seismic loads.

Demand capacity calculations for column.

Calculating the column bending capacity for ground storey column. The column demand by load combination is

Pu = 231.92 KN (portal method), Mu = 129 KNM (Demand)

$\frac{d'}{D} = \frac{60}{600} = 0.1$ Ast = $16 x \frac{\pi}{4} x (12^2)$ = 1809.55 mm^2

Pt = 100 $\frac{Ast}{bd}$ = $100x \frac{1809.55}{250x540}$ = 1.34 %

$\frac{p}{fck} = \frac{1.34}{20} = 0.067$

$\frac{Pu}{fck\, x\, b\, x\, D} = \frac{231.92\, x 10^3}{20\, x\, 250\, x\, 600} = 0.077$

(referring chart 21, pg. 329, S Ramamrutham)

$\therefore \frac{Mu'}{fck\, x\, b\, x\, D^2} = 0.09$

$\therefore$ Mu' = 0.09 x 20 x 250 x 600^2 = 162 KNM (capacity)

Mu'= 162 KNM. > 129 KNM, $\therefore$D.C.R = $\frac{129}{162}$

= 0.79 < 1.0, $\therefore$O.K.

Since the bending moment capacity is larger than the demand, the column is found to be stronger in bending under seismic loads.

Column shear capacity.

Considering that the steel in one face will be in tension.

Ast = $3 x \frac{\pi}{4} x (12^2)$ = 339.29 mm^2 $\therefore$Pt = 100 $\frac{Ast}{bd}$

= $100\, x \frac{339.29}{250\, x (600 - 60)}$ = 0.25 %

Pt = 0.25 %, τ_c = 0.36 Mpa. IS 456: 2000 table 19, pg.73

Stirrups, 8mm dia, 4 legged @ 180 mm c/c.

$V_{us} = \frac{0.87\ fy\, A_{sv}\, d}{S_v} + \tau_c$ bd

$V_{us} = \frac{0.87 \times 415 \times \left[\frac{\pi}{4} \times (8)^2 \times 4\right] \times 540}{180}$ + 0.36 x 250 x 540

$\therefore$ V_u = 266 KN. (capacity).

Shear demand in column.

V as per analysis = 64.50 KN. -------------------- (i)

Moment capacity of Beam.

M R^H = 267.23 KNM , M R^S = 246.87 KNM

Hc= height. of column = 4.0 m

V_u from capacity design

Vu = 1.4 x $\frac{M.R.Hogging + M.R.Sagging}{Hc}$ IS 13920 pg.5, fig.4

Vu = 1.4 x $\frac{267.23 + 246.87}{4.0}$

Vu = 179.93 KN ----------------------------------- (ii)

So final shear demand is greater of
(i) & (ii) = 179.93 KN.
Capacity = 266 KN,

$$\therefore \text{ D.C.R} = \frac{179.93}{266} = 0.67 < 1.0 \therefore \text{O.K.}$$

Thus, the shear capacity of column is greater than the shear demand on the column, which indicates that the column is stronger in shear under seismic loads.

Results & Discussions

Table 2: Design moments

Sr. No.	Method	Span moment KNM		Support moment KNM	
		AB & CD	BC	Intermediate	End
1	Moment distribution	204.30	74.08	260.45	0 (simple support)
2	IS coefficient	223.0	167.265	267.62	0 (simple support)
3	Substitute frame	176.83	108.67	225.39	220.0 (fixed support)

Maximum & Minimum span moment is 223 KNM & 74.08 KNM respectively. The difference between maximum & minimum span moment is 148.92 KNM. If only the Moment distribution method or substitute frame method is considered for design of beam then the section may be deficient in flexure.

Table 3: Design Shear forces

Sr No	Method	A 'KN'	B 'KN'	C 'KN'	D 'KN'
1	Moment distribution	266.43	223.02/179.61	179.61/223.02	266.43
2	IS coefficient	178.416	245.32/267.624	267.624/245.32	178.416
3	Substitute frame	221.45	222.96/222.07	222.07/222.96	221.45

The difference between maximum & minimum shear force is 89.208 KN. If only the IS coefficient method & substitute frame method is considered for design of shear reinforcement of beam then the beam may be deficient in shear.

Table 4: The lateral (seismic) forces on the perimeter frame are as below

Storey	Forces on external frame 'KN'
4	141.167
3	116.559
2	51.804
1	12.951

If the demand capacity ratio is more than 1.0 then following strengthening options are suggested.

A) Increasing the ductility & capacity of the frames by encasing the existing beams & columns in a reinforced concrete jacket.
B) Adding new shear walls infilled in the existing perimeter concrete frames so that seismic stresses in the existing frames can be reduced.
C) Constructing parallel concrete moment frames in the exterior of the building so that seismic stresses in the existing frames can be reduced.
D) Modification & or limited replacement of the existing perimeter concrete to improve their strength & ductility.

CONCLUSION

1. The result of the elastic analysis & design of four storey R.C.building indicates that –
   he span moments & intermediate support moments calculated by using IS (456-2000) coefficients are more than the moments obtained by moment distribution & substitute frame method.
   The substitute frame method gives maximum end moments.
   The moment distribution method gives maximum shear forces at end supports where as I S coefficient gives maximum shear forces at intermediate supports.
2. Detailed evaluation of the beam & column element of one of the perimeter frame is as below.

Table 5: Demand capacity ratios

Sr. No.	Check	D.C.R	Remarks
1	Moment of resistance of beam in hogging.	0.94	Less than 1.0, Check satisfied.
2	Moment of resistance of beam in sagging.	0.38	Less than 1.0,Check satisfied
3	Shear capacity of beam.	0.807	Less than 1.0,Check satisfied
4	Column flexural capacity	0.79	Less than 1.0,Check satisfied
5	Column shear capacity	0.67	Less than 1.0,Check satisfied

Thus, the above evaluation suggests that the beam & column need not to be strengthened & retrofitted.

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