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SEISMIC EVALUATION AND RETROFITTING OF BUILDINGS AND STRUCTURES

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ABSTRACT

Recent earthquakes in the Indian subcontinent have led to an increase in the seismic zoning factor over many parts of the country. Also, ductility has become an issue for all those buildings that were designed and detailed using earlier versions of the codes. Under such circumstances, seismic qualification of existing buildings has become extremely important. Seismic qualification eventually leads to retrofitting of the deficient structures. Pushover analysis and evaluation of performance of building using Capacity Spectrum Approach or Displacement Coefficient Method are increasingly used for this purpose. There is a need to look at certain important issues for incorporation in the Indian codes before uniformity of approach can be achieved. This, in turn, needs an in-depth understanding on what has gone into ATC-40 (ATC, 1996) or FEMA-356 (FEMA, 2000), and making appropriate modifications to suit the Indian conditions. It is necessary that the debate on this is started and completed early to achieve the desired results.

Pre-disaster preparedness strategies lead to repair/retrofitting of the reinforced concrete structures for ensuring satisfactory performance during earthquakes. Repairs can lead to increased stiffness, strength, and failure-deformation. There is a need to quantify the performance of the structure after the repairs have been carried out. Performance factors have been suggested for such quantification. These are adequate in certain cases but may not be totally satisfactory in others. A large-scale experimental programme undertaken at SERC has shown that if there are inherent weaknesses in the detailing of the original structure, it may not be possible to improve the performance to the desired levels. In these cases, the performance factors may depend on the state of deformation considered for evaluation and may not be unique. There is a need to address this issue, so that suitability of repair measures can be satisfactorily evaluated.

KEYWORDS: Seismic Qualification, Retrofitting, Performance Factors

INTRODUCTION

Concrete has been the most preferred construction material of the twentieth century, and unless a new material with spectacular characteristics is invented, it appears poised to remain this way for another century. This is not to suggest that there has been no progress on concrete and concrete technology over the years. Over the last 50 years, the strengths of various types of concrete have increased from the low levels of 15-20 MPa to values in the range of 40-70 MPa. Strength-based designs are slowly giving way to performance-based designs where strength is only one of the criteria to be satisfied. There is an increased attention being paid to life prediction and maintenance scheduling. Finite element software is extensively used in design offices for the analysis and design of concrete structures. It may be worthwhile at this stage to exactly calibrate the status of present day analysis and design viz., the "realistic estimates" on load effects and deformations. Consider, for example, the design of a multi-storeyed framed structure. The load cases to be considered are the dead load, live load, wind load, seismic load, and their combinations. The input data that is normally fed into the computer software includes modulus of elasticity, Poisson's ratio, density of concrete, areas and moments of inertia of all structural elements, basic wind speed, zoning factor for seismic loading, and so on. Then one goes on to define the load combinations to obtain the worst load effects. Generally the gross section properties are used, and elastic analysis is performed. The design is based on the limit state philosophy. So the elastic load effects that are obtained are multiplied by the load factors to obtain the capacity requirements. Theory of plasticity is then used to proportion the cross-sections for moments and axial forces. Linear variation of strain is assumed across the cross-sections, equilibrium equations for axial forces and moments are written down, and area of the steel reinforcement required is computed. What is done for the design for shear? This is more or less based on the empirical equations derived from the test results. The contribution of concrete is derived based on the strength of concrete and percentage of the tensile reinforcement. The contribution due to the web-steel is based on a 45^0 crack, though everyone realizes that the shear cracks seldom occur at this angle, depend on the M / Vd ratio, and even show change with the loading. The design is then claimed to be based on the limit state design philosophy covering limit states of serviceability and collapse. The limit state of serviceability is deemed to be satisfied if all the recommendations given in IS:456-2000 (BIS, 2000) regarding the detailing are satisfied.

Presently earthquake-resistant design is being discussed in many forums. If one adopts the provisions of IS:13920-1993 (BIS, 1993), the response reduction factor is 5.0 and for normal frames it is 3.0. This has direct impact on the design forces. The method of analysis and design as described above is the most sophisticated procedure adopted in the design offices. This is the present status, irrespective of what the inconsistencies are in elastic analysis, plastic design for moments and axial forces, and empirical approach for design for shear, bond, etc. It is extremely important to realize that the values obtained in the analysis are at best good indicators for the express purpose of “design”. The most essential part, and often the neglected part, is the proper guidance to aid engineers in making the best use of the results available and in providing the reinforcement adequately.

It must be realized at this stage that when one attempts to carry out the seismic evaluation of a building, strictly speaking, the codal provisions at the time of construction, age of the structure, construction practices etc., all become important. It must also be realized that if one is waiting for an exact methodology and rules for the seismic evaluation, one may have to wait for a few more years, if not decades. Presently, there are two nonlinear static analysis procedures available, one termed as the Displacement Coefficient Method (DCM) included in the FEMA-356 document (FEMA, 2000), and the other termed as the Capacity Spectrum Method (CSM) included in the ATC-40 document (ATC, 1996). Both of these methods depend on the lateral load-deformation variation obtained by using the nonlinear static analysis under the gravity loading and idealized lateral loading due to the seismic action. This analysis is generally called as the pushover analysis.

PUSHOVER ANALYSIS

Pushover analysis is a nonlinear static analysis for a reinforced concrete (RC) framed structure subjected to lateral loading. The gravity loads are applied, and then lateral loading is applied – first in *X*-direction starting at the end of the gravity push, and next in *Y*-direction again starting at the end of the gravity push (Valles et al., 1996; CSI, 2000). The concept of plastic hinge is extremely important in the nonlinear analysis.

While a concrete element undergoes large deformations in the post-yield stage, it is assumed that all the deformation takes place at a point called “plastic hinge”, which has approximately a length of the order of the effective depth (also called as plastic hinge length, l_d). The rotation capacity θ of a plastic hinge is taken as $l_d\left(\phi_u - \phi_y\right)$. A similar approach can be used for obtaining the rotation capacity of columns under axial force and bending moment in two directions. Similar plastic hinges with limit capacities on deformation can be defined for all six degrees of freedom, namely, axial force, transverse shear forces in *X*- and *Y*-directions, moments about *Y*- and *Z*-axes, and torsion (moment about *X*-axis). More details on evaluation of ductility, energy absorption, damage modeling, and detailing are available elsewhere (Lakshmanan, 2003a, 2005a). A typical response at a plastic hinge may be as shown in Figure 1. Here, Point *A* is the origin; *B* is the point of yielding; *BC* represents the strain-hardening region; *C* is the point corresponding to the maximum force; and *DE* is the post-failure capacity region. On the frame structure, the analyst identifies the possible locations for plastic hinge formation from his experience. Mathematically, nonlinear static analysis does not lead to a unique solution. Small changes in properties or sequence of loading can lead to large variations in the nonlinear response.

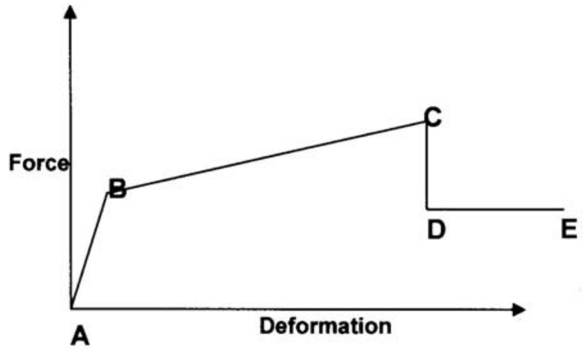


Fig. 1 Idealised force-deformation curve

The pushover analysis may be carried out using force control or deformation control. In the first option, the structure is subjected to an incremental distribution of lateral force, and incremental displacements are calculated. In the second option, the structure is subjected to a deformation profile, and lateral forces needed to generate those displacements are computed. Since the deformation profile is unknown, the first option is commonly used. For the displacement control the user specifies the target maximum displacement at a control point. In certain softwares, displacement control is not the same as applying displacement loading on the structure; displacement control is simply used to measure the displacement that results from the applied loads and to adjust the magnitude of the loading in an attempt to reach certain measured displacement value. The so-called displacement control in this case is essentially a modified form of the force control. The force control strategy can have following options: (i) uniform distribution, (ii) triangular distribution, (iii) generalised power distribution, and (iv) modal adaptive distribution with single or multiple mode participation.

The results of the pushover analysis carried out on a typical reinforced concrete frame, whose isometric view is given in Figure 2, are shown in Figure 3.

IDEALIZED SYSTEM PARAMETERS

The multilinear force-displacement curve obtained after the pushover analysis is idealized as bilinear curve, as shown in Figure 4, with a positive or negative post-yield slope.

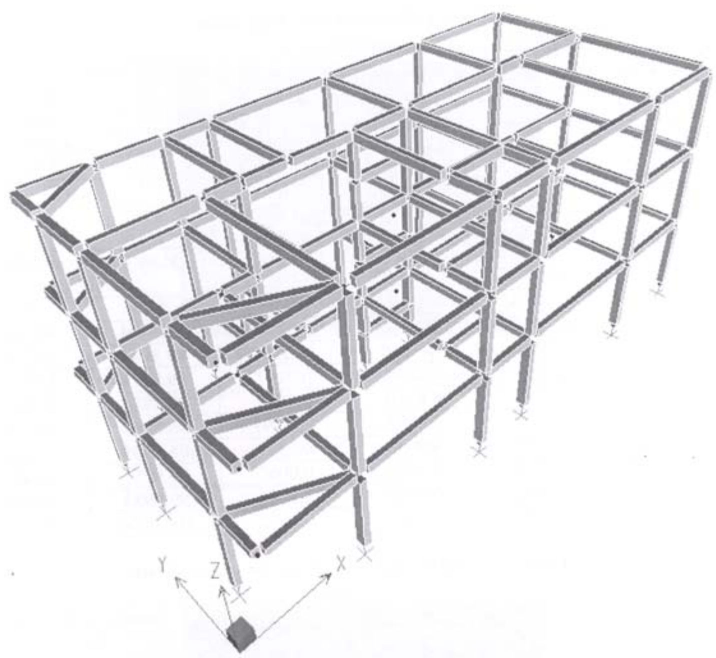
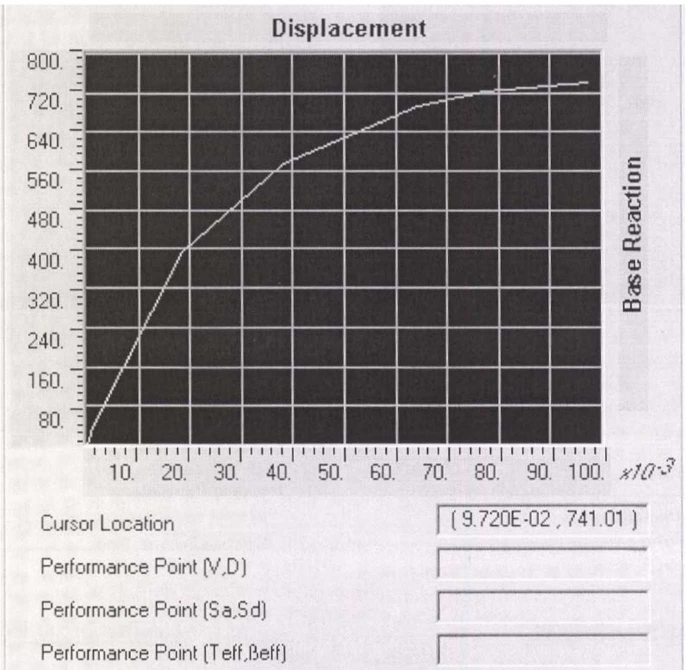
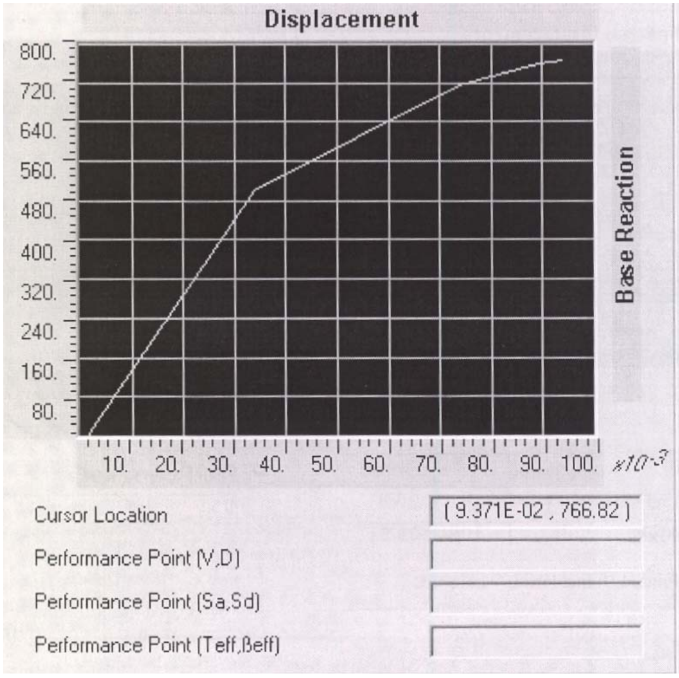


Fig. 2 Isometric view of the building



Push along the *X*-direction



Push along the *Y*-direction

Fig. 3 Base reaction (kN) versus monitored displacement (m)

First let us consider the equivalent linearization method, which is the basis for the Capacity Spectrum Method. The maximum response of the equivalent system is computed using

$$\ddot{x}_{eq} + 2\xi_{eq}\omega_{eq}\dot{x}_{eq} + \omega_{eq}^2 x_{eq} = -\ddot{x}_g \tag{1}$$

where ξ_{eq} and ω_{eq} are respectively the viscous damping ratio and natural circular frequency of the equivalent linear system. For a bilinear system, the time period T_{eq} of the equivalent system as compared to T_0 of the original system is given by

$$T_{eq} = T_0 \sqrt{\frac{\mu}{(1-\alpha)+\alpha\mu}} \tag{2}$$

where μ is the ductility ratio, and α is the ratio of the positive post-yield stiffness to the original stiffness. The equivalent damping is given by

$$\xi_{eq} = \xi_0 + \frac{2}{\pi}\left[\frac{(1-\alpha)(\mu-1)}{\mu-\alpha\mu+\alpha\mu^2}\right] \tag{3}$$

For an elasto-plastic system,

$$T_{eq} = \sqrt{\mu}T_o \tag{4}$$

and

$$\xi_{eq} = \xi_0 + \frac{2}{\pi}\left(1-\frac{1}{\mu}\right) \tag{5}$$

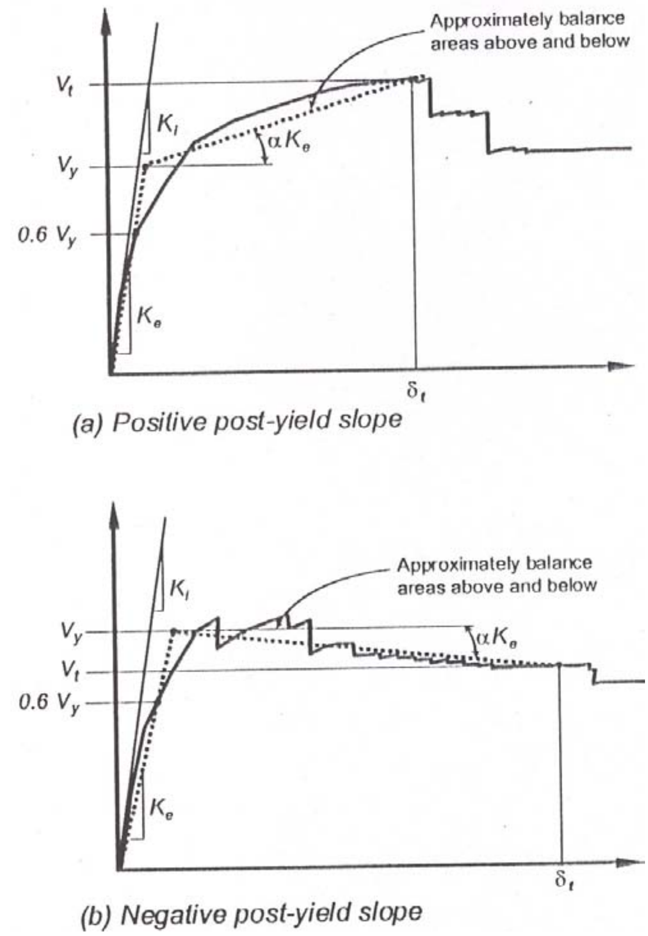


Fig. 5 Idealized force-displacement curves

Using Takeda hysteretic model (Takeda et al., 1970) and experimental investigations on model reinforced concrete frames, an empirical equation for equivalent damping was given by Gulkan and Sozen (1974):

$$\xi_{eq} = \xi_0 + 0.2\left(1-\frac{1}{\sqrt{\mu}}\right) \tag{6}$$

Kowalsky et al. (1994) used the secant stiffness at maximum deformation for defining the period together with hysteretic model given by Takeda et al. (1970) and an unloading stiffness factor of 0.5:

$$\xi_{eq} = \xi_0 + \frac{1}{\pi}\left[1 - \mu^n\left(\frac{1-\alpha}{\mu} + \alpha\right)\right] \tag{7}$$

where *n* is equal to zero for steel structures and 0.5 for concrete structures. For an elasto-plastic system,

$$\xi_{eq} = \xi_0 + \frac{1}{\pi}\left(1 - \frac{1}{\sqrt{\mu}}\right) \tag{8}$$

In ATC-40 (ATC, 1996), ξ_{eq}, given in Equation (3), is used with the proviso that

$$\xi_{eq} = \xi_o + \chi\xi_h \tag{9}$$

where ξ_h is limited to 45%. Further, χ is equal to 1.0 for ξ_h = 16.25% and 0.77 for ξ_h = 45% with linear interpolation for other damping values, in case of structures having stable and full hysteretic behaviour. χ is equal to 0.67 for ξ_h = 25% for reasonably well-behaved systems, and is equal to 0.33 for systems with poor hysteretic behaviour.

In the Displacement Coefficient Method pioneered by Veletsos and Newmark (1960), and Newmark and Hall (1982), the displacement modification factor is shown to depend on the spectral region in which the period of vibration of the single-degree-of-freedom (SDOF) system is located:

$$\begin{aligned} C &= \mu && T < T_a\ (=1/33\,\text{sec}) \\ &= \frac{\mu}{(2\mu-1)^{\beta}} && T_a < T < T_b\ (=0.125\,\text{sec}) \\ &= \frac{\mu}{\sqrt{2\mu-1}} && T_b < T < T_c' \\ &= \frac{T_c}{T} && T_c' < T < T_c\ (=0.57\,\text{sec}) \\ &= 1.0 && T > T_c \end{aligned} \tag{10}$$

where $T_c' = \left(\sqrt{2\mu-1}/\mu\right)T_c$ and $\beta = \log(T/T_a)/2\log(T_b/T_a)$.

Miranda (2001) has recently suggested that

$$C = \left[1 + \left(\frac{1}{\mu} - 1\right)e^{-12T\mu'}\right]^{-1} \tag{11}$$

where $\mu' = \mu^{-0.8}$. More recently, Ruiz-García and Miranda (2003), and Chopra and Chintanapakdee (2003) have proposed expressions for C_R and C_μ based on extensive investigations of field data and by using regression analyses.

DISPLACEMENT COEFFICIENT METHOD

The generalized target displacement given in FEMA-356 (FEMA, 2000) includes, in addition to the modification factor C_1 for the inelastic response, modification factor C_0 to relate spectral displacement of the SDOF system to the roof displacement of the multi-degree-of-freedom (MDOF) system, modification factor C_2 for the degraded hysteretic performance, and modification factor C_3 for *P*-Δ effect in a system in which the post-yield stiffness is negative. In general, the target displacement is given as

$$\delta_t = C_0C_1C_2C_3S_a\left(\frac{T^2}{4\pi^2}\right) \tag{12}$$

While C_μ and C_R (which are the counterparts of C_1) have been discussed in detail, the other factors are also under scrutiny (Comartin, 2002).

CAPACITY SPECTRUM APPROACH

The spectral coefficient given in IS:1893-2002 (BIS, 2002) is multiplied by the zoning factor Z to obtain the plot of spectral acceleration versus time period. The Acceleration Displacement Response Spectrum (ADRS), also called demand spectrum, is obtained by plotting S_a versus S_d where $S_d = \left(T_1^2 / 4\pi^2\right) Z \left(S_a / g\right)$. The capacity spectrum is derived from the inelastic shear-roof displacement curve using

$$S_a = \left\{ \left(V / W \right) g \right\} / \alpha_1 \tag{13}$$

and

$$S_d = \Delta_{\text{roof}} / PF_{R,1} \tag{14}$$

where α_1 and $PF_{R,1}$ are the modal mass coefficient and modal participation factor, respectively, for the first mode. Table 1 gives typical values of α_1 and $PF_{R,1}$ for rectangular buildings with uniform mass and straight line mode shape.

Table 1: Effective Mass Coefficient α_1 and Modal Participation Factor $PF_{R,1}$ for Roof

Number of Storeys	α_1	$PF_{R,1}$
1	1.00	1.00
2	0.90	1.20
3	0.86	1.30
5	0.82	1.35
≥ 10	0.78	1.40

Based on the multiplying factors given in IS:1893-2002 (BIS, 2002) for obtaining seismic response acceleration coefficients, the variations of S_a / g with S_d for various damping ratios can be plotted (Figure 5). Performance point will be a point on the capacity spectrum where T_{eq} and ξ_{eq} would be satisfied. To satisfy the life-safety requirement, the roof displacement should be less than 1.2% of height for ordinary frames and 2% for special moment-resisting frames. Life-safety requirement can be considered as corresponding to the design basis earthquake. For collapse prevention, the performance point should exist. In any case, the total drift should not exceed $0.33V / P$, where V is the base shear and P is the gravity load for collapse prevention. Also, in a degrading nonlinear response, the limit of degradation on strength as compared to the peak value is 20%.

The above discussion clearly reveals that pushover analysis needs to be used with lot of caution and expert judgment. In fact, it could become a dangerous tool in the hands of practitioners who have little exposure to nonlinear/dynamic behaviour of concrete structures.

VULNERABILITY INDEX

The vulnerability index is a measure of the damage in a building obtained from the pushover analysis. It is defined as a scaled linear combination (weighted average) of performance measures of the hinges in the components, and is calculated from the performance levels of the components at the performance point or at the point of termination of the pushover analysis (Lakshmanan, 2005b). It has been mentioned earlier that the load-deformation curve for a particular hinge is assumed to be piecewise linear (Figure 1). The plastic plateau (*B*-*C*) in the load-deformation curve is subdivided into the performance ranges, namely, *B*-*IO*, *IO*-*LS*, *LS*-*CP*, *CP*-*C*, *D*-*E*, and > *E*.

After the pushover analysis, performance ranges of the hinges formed in the component can be noted from the deformed shape output. The number of hinges formed in the beams and columns for each performance range are available from the output. A 'weightage factor' (x_i) is assigned to each performance range. The proposed values of x_i are given in Table 2. As columns are more important than beams in the global safety of a building, an 'importance factor' of 1.5 is additionally assigned for columns. The building vulnerability index VI_{bldg} is accordingly given by the following weighted average:

$$VI_{\text{bldg}} = \frac{1.5\sum N_i^c x_i + \sum N_i^h x_i}{\sum N_i^c + \sum N_i^h} \tag{15}$$

Here, N_i^c and N_i^h are the numbers of hinges in columns and beams, respectively, for the ith performance range. The summation sign is intended to cover the performance ranges, i = 1,2,…6.

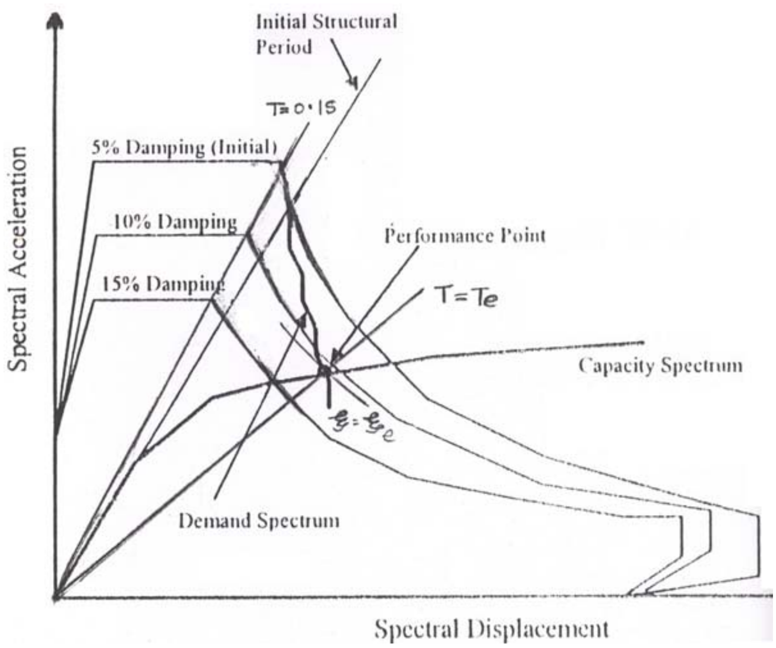


Fig. 5 Demand and capacity spectra

Table 2: Weightage Factors for Performance Range

Serial Number	Performance Range (i)	Weightage Factor (x_i)
1	< *B*	0
2	*B-IO*	0.125
3	*IO-LS*	0.375
4	*LS-CP*	0.625
5	*CP-C*	0.875
6	*C-D*, *D-E*, and > *E*	1.000

VI_{bldg} is a measure of the overall vulnerability of the building. A high value of VI_{bldg} reflects poor performance of the building components (i.e., high risk) as obtained from the pushover analysis. However, this index may not reflect a soft storey mechanism, in which a performance point may not be achieved. Table 3 gives the vulnerability index of a typical framed building analysed using the pushover analysis. The damage index for *X*-push is computed as 0.34 and *Y*-push as 0.41.

A storey vulnerability index VI_{storey} can be defined to quantify the possibility of a soft/weak storey with the formation of flexural hinges. For each storey VI_{storey} is defined as

$$VI_{\text{storey}} = \frac{\sum N_i^c x_i}{\sum N_i^c} \tag{16}$$

where N_i^c is the number of column hinges in the storey under investigation for a particular performance range. In a given building, the presence of soft/weak storey is reflected by a relatively high value of VI_{storey} for that storey, in relation to the other storeys. If the analysis is terminated due to the formation of shear hinges, then the above definition is not applicable.

Table 3: Vulnerability Index Based on Pushover

Serial Number	Direction	Element	Weight, W_i	Coefficient, C_i	Number of Hinges	$W_iC_iN_i$
1	X	Beam	1.0	0.125	316	39.38
				0.375	268	100.50
				0.625	28	23.75
		Column	1.5	0.125	301	56.44
				0.375	292	164.25
				0.625	43	40.31
				1.000	2	3.00
2	Y	Beam	1.0	0.125	87	10.88
				0.375	71	26.63
				0.625	203	126.88
				1.000	1	1.00
		Column	1.5	0.125	97	18.19
				0.375	27	15.19
				0.625	1	0.94

DAMAGE INDICES AND DAMAGE MODELLING

Even though the performance-based evaluation approach *per se* is based on damage classification, it is clearly evident that damage indices are used only for the quantification of the conclusion of an analysis. Damage indices are not used to influence the way structural response evolves. The response of a damaged system not only depends on the previous history but also on the rate at which damage accumulates. A system with same initial conditions can reach failure at different stages due to characteristics of the earthquake and damage accumulated over the response time.

The response-based damage indices are ductility ratio δ_u / δ_y, inter-storey drift as a percentage of storey height, slope ratio defined by the ratio of the slope of loading branch to the slope of unloading branch, stiffness ratio defined as the ratio of initial stiffness to the secant stiffness at the maximum displacement, maximum permanent drift, and so on. All these fail to recognise the cyclic nature of response.

Cumulative damage law based on low-cycle fatigue leads to an expression for damage as (Cosenza et al., 1993)

$$D = \sum_{i=1}^{n} \left[\frac{\mu_i - 1}{\mu_u - 1} \right]^b \tag{17}$$

where b typically varies from 1.5 to 1.8. The most popular Park and Ang model is a combined model based on deformation and accumulated damage:

$$D = \frac{\mu_c}{\mu_{st}} + \frac{\beta}{F_y \delta_u} \int dE \tag{18}$$

The basic assumption in the above damage model is that the damage due to response and the damage due to cumulative energy can be superposed linearly, and β is to be obtained through laboratory tests or field data. Williamson and Kaewkulchai (2003) have suggested:

$$D = \alpha U(\delta) + \beta W(\delta) \tag{19}$$

where α and β are constants; $U(\delta)$ is a function of maximum deformation; and $W(\delta)$ is a function of the accumulated plastic energy. α and β can be adjusted to account for different rates of damage leading to a variety of response models. Another variation of the Park and Ang rule is given by Lowes et al. (2004):

$$D = \alpha_1 \left(\tilde{d}_{\max}\right)^{\alpha_3} + \alpha_2 \left(\frac{E_i}{E_u}\right)^{\alpha_4} \tag{20}$$

where $\left(\tilde{d}_{\max}\right) = \max\left[\frac{d_{\max,i}}{de_{f\max}}, \frac{d_{\min,i}}{de_{f\min}}\right]$ and $E_i = \int dE.$

Often the monotonic test results show that the strength is capped and is followed by a negative tangent stiffness. In general, cyclic response indicates that the strength decreases with number and amplitude of cycles even if the displacement associated with the peak strength is not reached. Strength deterioration occurs at a faster rate in the post-capping region, and the unloading stiffness may also deteriorate. Also, it is possible that the reloading stiffness may deteriorate at a faster rate. A holistic damage model individually accounting for all these has been proposed by Ibarra et al. (2005).

SEISMIC STRENGTHENING

Repair and retrofitting of concrete structures have been attracting the attention of researchers over the last two decades. Various repair/retrofit options available today include crack injection, shortcreting, steel jacketing, steel plate bonding, CFRP/GFRP jacketing, RC jacketing, addition of new structural elements (braces, walls, etc.), incorporation of passive energy dissipation devices, and provision of base isolation. Retrofitting can be at the system level or at the local level. Introduction of additional shear walls, braces, base isolation etc., to enhance the performance of a structure belongs to the former category, while repair of a beam or column element using various jacketing techniques, such as jacketing using micro-concrete, steel, carbon fibre reinforced plastics (CFRP), and glass fibre reinforced plastics (GFRP), essentially falls under the category of local retrofitting. Repair and retrofit techniques can be used for enhancing the stiffness, strength, and/or ductility. In general, repair techniques may affect more than one of the above parameters. Studies have been conducted at Structural Engineering Research Centre (SERC), Chennai on beams, columns, and beam-column joints to evaluate the performance of bonded steel plate jacketing and FRP wrapping techniques using CFRP and GFRP (Lakshmanan et al., 2004; SERC, 2004a, 2004b, 2004C).

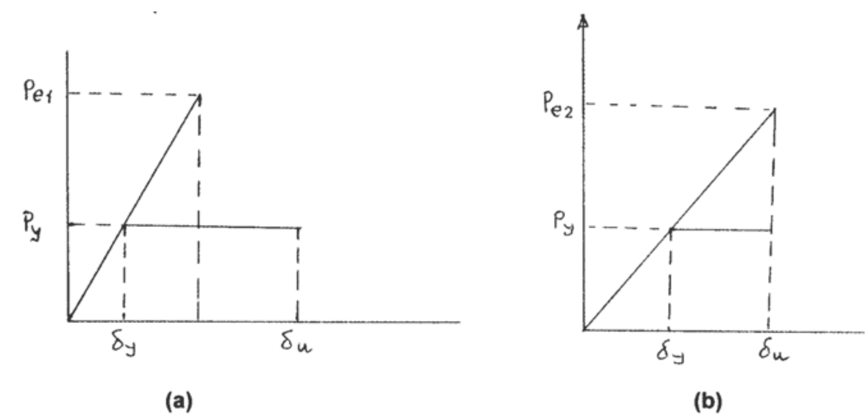


Fig. 6 Computation of equivalent elastic forces for ductile structures

Ductility-based design concepts use equivalence of failure deformation (Figure 6(a)) or equivalence of failure energy (Figure 6(b)) between the elastic and elasto-plastic systems, or modifications to damping factor based on the ductility ratio. These three approaches have been used for a comparative evaluation of the performance of jacketed specimen viz., conventional specimen.

The equivalent elastic forces P_{e1} and P_{e2} are computed using

$$P_{e1} = \left[\frac{2AP_y}{d_y} \right]^{1/2} \tag{21}$$

$$P_{e2} = P_y \left(\frac{\delta u}{\delta y} \right) \tag{22}$$

Hence it is felt that to evaluate the effectiveness of any repair measure, the following effectiveness factors may be used. The same procedure may be adopted for evaluating new materials against conventional reinforced concrete. The effectiveness factors may be defined as

$$F_1 = \frac{P_{e1}\,(\text{retrofit})}{P_{e1}\,(\text{control})} \tag{23}$$

and

$$F_2 = \frac{P_{e2}\,(\text{retrofit})}{P_{e2}\,(\text{control})} \tag{24}$$

IS:1893-2002 (BIS, 2002) gives the multiplying factors for obtaining seismic forces in case of different damping factors (Table 3 of IS:1893-2002). Based on these, the efficiency, particularly of a ductility-based repair strategy, can be evaluated as

$$F_3 = \frac{m_{\text{com}}}{m_{\text{repair}}} \tag{25}$$

where m_{com} is the multiplying factor for damping in the original structure, and m_{repair} is the multiplying factor for damping in the repaired structure.

The above performance factors have been discussed in a number of publications (Lakshmanan, 2003b, 2005c, 2005d).

BEAMS WITH BONDED LAMINATES TESTED UNDER PURE FLEXURE

Table 4 shows typical details of RC beams tested with bonded laminates. More details on the size of specimen, test procedure etc., are available in Lakshmanan (2006). A typical load-deflection diagram of CFRP-bonded specimen is shown in Figure 7. Typical failures of jacketed beams are shown in Figure 8. The performance factors for the beams tested are given in Table 5.

Table 4: Details of RC Beams

Serial Number	Beam Number	Beam Details	Types of Wrapping	Number of Layers
1	S1	Control	-	-
2	S2	Retrofitted	CFRP	Single Layer (Parallel)
3	S3	Retrofitted	CFRP	Single Layer (Perpendicular)
4	S4	Retrofitted	CFRP	Double Layer (One Parallel and One Perpendicular)
5	S5	Retrofitted	CFRP	Double Layer (Both Layers Parallel)
6	S6	Retrofitted	CFRP	Single Layer (Continuous)
7	S7	Retrofitted	CFRP	Single Layer (Two Overlaps)
8	S8	Retrofitted	CFRP	Double Layer
9	S9	Retrofitted	CFRP	Double Layer

10	S10	Retrofitted	Steel Plate	-
11	S11	Retrofitted	Steel Plate	-
12	S12	Retrofitted	Steel Plate	-

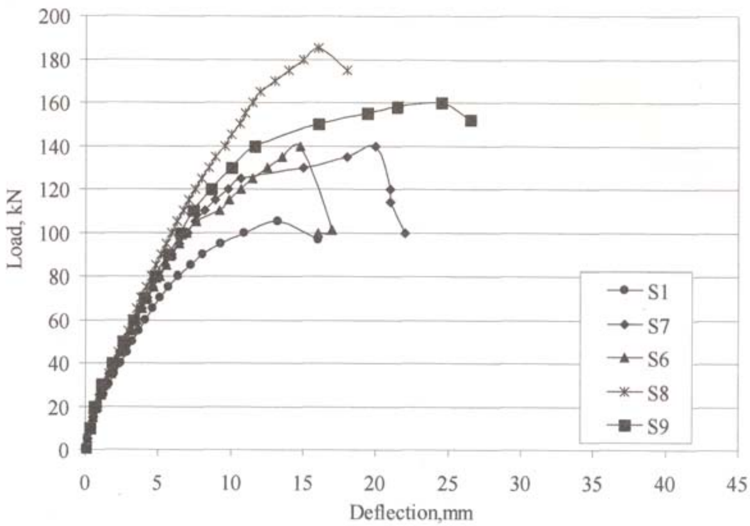


Fig. 7 Load-deflection diagram of CFRP-bonded beams

CFRP specimen

GFRP specimen

Bonded steel plate

Fig. 8 Typical failures in bonded beams

Table 5: Performance and Multiplication Factors of the Retrofitted RC Beams

Serial Number	Beam Number	P_{e1} (kN)	P_{e2} (kN)	F_1	F_2	ξ (%)	F_3
1	S1	166.8	180.4	1.00	1.00	7.93	1.00
2	S2	259.0	302.7	1.55	1.68	11.14	1.12
3	S3	162.2	172.3	0.97	0.96	7.60	1.00
4	S4	206.3	223.3	1.24	1.24	9.35	1.04
5	S5	266.0	308.1	1.59	1.71	11.23	1.13
6	S6	222.7	275.0	1.34	1.52	10.60	1.10
7	S7	263.2	327.7	1.58	1.82	11.56	1.13
8	S8	276.2	311.7	1.66	1.73	11.29	1.13
9	S9	325.7	428.6	1.95	2.38	12.87	1.18
10	S10	311.3	499.2	1.87	2.77	13.54	1.19
11	S11	297.2	434.9	1.78	2.41	12.94	1.18
12	S12	394.0	749.2	2.36	4.15	15.09	1.24

BEAM-COLUMN JOINTS

A number of beam-column joints have been tested using the CFRP and GFRP wrappings. A schematic diagram of loading on a beam-column joint is shown in Figure 9, and the reinforcement details are given in Figure 10. The load-deflection diagrams of various beam-column joints tested are shown in Figures 11 and 12. Figure 13 shows the failure of a typical beam-column joint strengthened with the CFRP wrap. The details of the beam-column joints tested are given in Table 6.

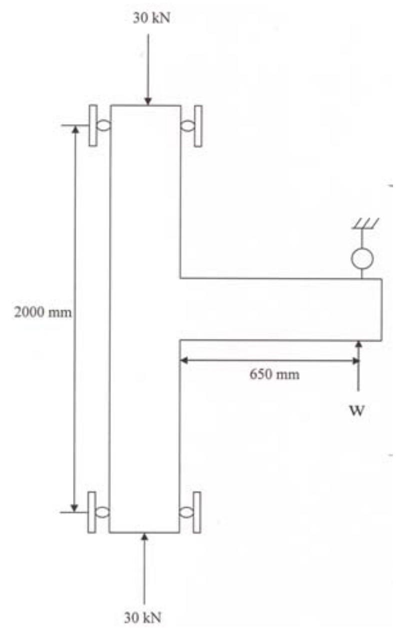


Fig. 9 Schematic diagram of loading on a beam-column joint

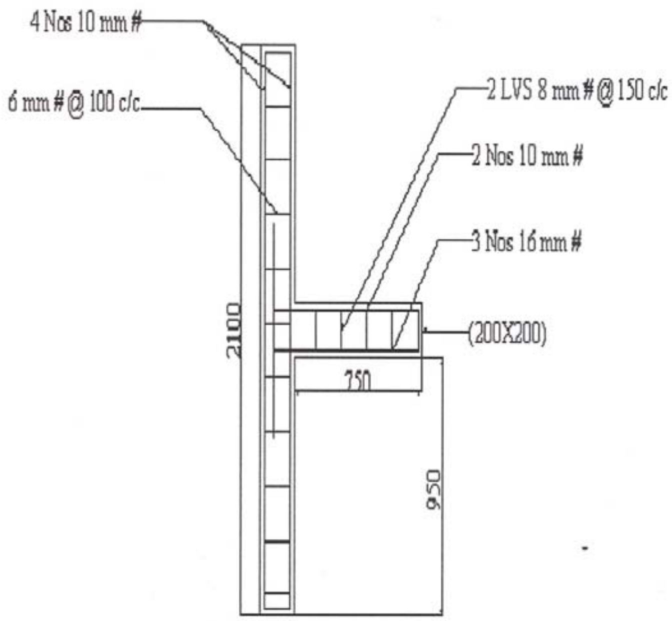


Fig. 10 Reinforcement details of a beam-column joint

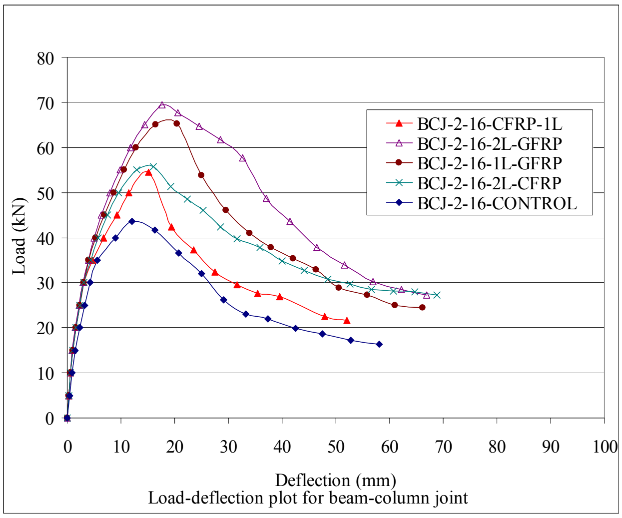


Fig. 11 Load-deflection diagram of beam-column joints

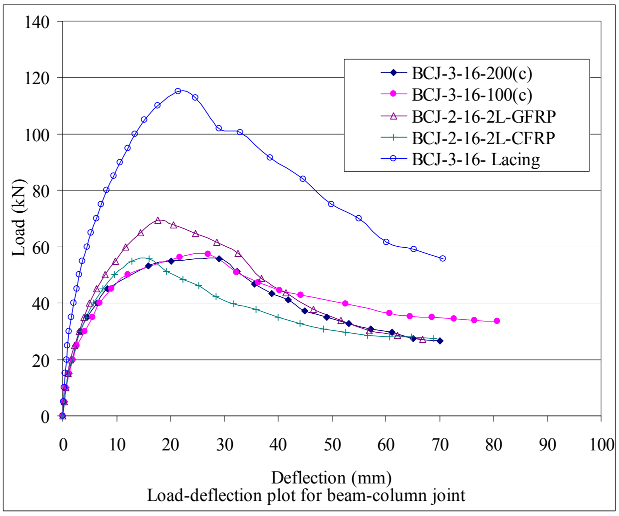


Fig. 12 Load-deflection diagram of beam-column joints

Fig. 13 Typical failure of beam-column joint with CFRP

Table 6: Details of the Reinforced Concrete Beam-Column Joint Specimens

Specimen ID	Strengthening Method	Spacing of Stirrups in the Column
BCJC1	Control Specimen	200 mm c/c (with 3 # 16 mm Bars as Tension Reinforcement in the Beam)
BCJC2	Control Specimen	200 mm c/c (with 2 # 16 mm Bars as Tension Reinforcement in the Beam)
BCJR1	CFRP (Single Layer)	200 mm c/c (with 2 # 16 mm Bars as Tension Reinforcement in the Beam)
BCJR2	CFRP (Double Layer)	
BCJR3	GFRP (Single Layer)	
BCJR4	GFRP (Double Layer)	
BCJR5	CFRP (Double Layer)	200 mm c/c (with 3 # 16 mm Bars as Tension Reinforcement in the Beam)
BCJR6	Microconcrete with Lacing	200 mm c/c (with 3 # 16 mm Bars as Tension Reinforcement in the Beam)

The load-deformation characteristics clearly reveal

- that there is no significant improvement to stiffness because of GFRP or CFRP wrappings;
- it is feasible to enhance the capacity of the beam-column joint by GFRP or CFRP wrappings;
- the post-peak response of control, as well as strengthened beams, show a high rate of load-drop which is nearly constant in all the beam-column joints tested;
- GFRP gives better overall performance as compared to CFRP; it gives higher strength, and the load deflection curve envelops the load deflection curve of CFRP at all stages.

1. Procedure-1

As can be seen from the load-deformation behaviour of the beam-column joints tested, there is a significant load drop in the post-peak load behaviour, possibly due to an inherent detailing deficiency in the beam-column junction. All repair measures carried out could not rectify this inherent weakness, though they have registered higher load and deformation levels. As a first approximation, the maximum permissible load drop has been assumed as 25% and an analytical modeling has been done. With reference to Figure 14, Point *C* is chosen such that it has a value of load equal to $0.75P_{\max}$ (at Point *B*). *D* is deflection corresponding to the above load.

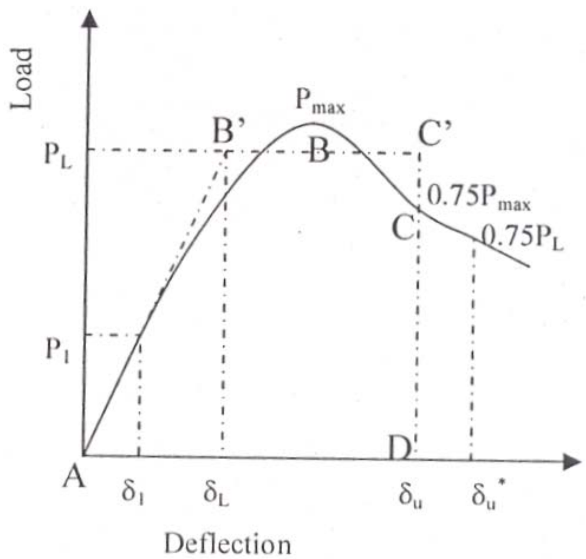


Fig. 14 Idealised load-deformation diagram of beam-column joint

Approximating area under the experimental load-deflection curve *ABCD* by the area under the idealized curve *AB'C'D*, we obtain

$$A_e = P_L\delta_u - P_L\delta_L / 2 \tag{26}$$

Substituting $\delta_L = \left(P_L / P_1\right)\delta_1$, this becomes

$$A_e = P_L\delta_u - P_L^2\left(\delta_1 / P_1\right)/2 \tag{27}$$

Since A_e, δ_u, δ_1, and P_1 are known, P_L can be obtained. Corresponding to a 25% drop in P_L, the deformation δ_u^* and effective area A_e^* can be computed. Using P_L, δ_L, δ_u^* and A_e^*, the performance factors F_1 and F_2 are computed as given in Table 7.

Table 7: Performance Factors for Beam-Column Joint Specimens

Beam ID	P_L **(kN)**	$\frac{P_{L,\text{repaired}}}{P_{L,\text{companion}}}$	δ_L **(mm)**	δ_u **(mm)**	P_{e1} **(kN)**	F_1	P_{e2} **(kN)**	F_2
BCJC2	39.30	1.00	5.50	26.85	162.87	1.00	191.79	1.00
BCJR1	35.51	0.90	4.45	40.11	215.94	1.33	319.93	1.67
BCJR2	50.04	1.27	6.15	36.28	227.20	1.39	295.42	1.54
BCJR3	58.22	1.48	6.85	31.69	232.73	1.43	269.37	1.40
BCJR4	61.63	1.57	7.15	39.18	272.03	1.67	337.69	1.76
BCJC1	50.82	1.00	5.42	44.19	277.08	1.00	414.28	1.00
BCJR5	63.25	1.24	6.23	48.19	332.55	1.20	489.43	1.18
BCJR6	96.57	1.90	4.33	52.52	647.07	2.34	1170.83	2.83

From the above table, it is seen that the performance factors F_1 and F_2 are improved for the RC beam-column joint specimens provided with 2 nos. of 16 mm diameter bars and retrofitted with carbon as well as glass fibres compared to the control specimens. The performance factors F_1 and F_2 are also improved for the RC beam-column joints provided with 3 nos. of 16 mm diameter bars and retrofitted with two layers of CFRP and microconcrete. Further, those are much higher for the RC beam-column joint specimens retrofitted with microconcrete and lacing compared to the specimens with FRP wrapping.

2. Procedure-2

The basic hypothesis made based on the experimental results is that the repair does not alter the slope of the falling branch. Also, the failure deflection in all these cases has been observed to be between 55 to 70 mm, and allowing for experimental scatter, this can be taken as a constant at 60 mm. This would straight away lead to the same value for P_{e2}, and F_2 would be constant at 1.0. Hence, for long-period structures with drooping post-peak responses and constant failure deformations, the repairs are ineffective unless the stiffness can be significantly altered. This has not happened in these cases. The value of P_{e1} can be shown to be

$$P_{e1} = \left[\frac{2\left(A_e - A_0\right)P_y}{\delta_y}\right]^{\frac{1}{2}} \tag{28}$$

where A_e is the area of the elasto-plastic system, and A_0 is the reduction due to the post-peak response. This leads to the values of F_1 equal to 1.07, 1.16, 1.23, and 1.33 for the Beams BCJR1 to BCJR4, which are significantly lower than the values given in Table 7. Further studies are, however, needed for a more realistic evaluation.

CONCLUSIONS

This paper attempts to gather the available information particularly on the nonlinear behaviour, and the various approaches available to evaluate the seismic safety of buildings. It is emphasized that the existing procedure is grossly approximate, and hence improving sections of the approach to high levels of accuracy would not necessarily lead to a better result. The need of the present hour is to see what needs to be done in the Indian context. The basic inputs are earthquake spectrum, nonlinear load-deformation behaviour under monotonic cyclic and random loadings, acceptable levels of damage under various performance levels etc., and these have a lot of grey areas. There needs to be a wider discussion among the researchers to evolve a good standard for use by the profession.

The need for evaluating the various repair strategies for use in the improvement of the seismic performance of reinforced concrete structures has been highlighted. The behaviour of repaired beams and beam-column joints has been discussed. It is observed that inherent deficiencies in the detailing of the beam-column joints get reflected even after repair, though the performance factors indicate significant improvement. There is a need to evolve suitable performance factors when the system shows a negative stiffness. Two of the logical extensions show that the repair would not be as effective in these cases.

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