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REVIEW ON SEISMIC EVALUATION OF REINFORCED CONCRETE BUILDING

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ABSTRACT

In the study of evaluation for retrofitting of any building made up of reinforced concrete is done by using method of inelastic method, capacity curve method etc. Capacity curve which is load deformation plot is output of inelastic method. As this inelastic analysis is non linear static analysis, so load deformation curve is obtained from ANSYS. ANSYS which is based on finite element method is used for performing the non liner static inelastic analysis and cracking pattern can be analyzed in ANSYS. The need of retrofitting of any particular element of any existing building will be obtained by cracking pattern. In this first symmetrical building is analyzed on ANSYS for the procedure development as per ATC-40 then symmetric evaluation is done on unsymmetrical building designed for without considering seismic effect and then same building considering the seismic effect according to IS 1893:2002. These results then compared for suggestion for retrofitting of affected members.

KEYWORDS: elastic-curve, response spectrum, ATC procedure, retrofitting, cracking pattern..

INTRODUCTION

During earthquakes, buildings that appear to be strong enough, crumble like houses of cards and their deficiencies are (may be) exposed. Certain past earthquakes for e.g. earthquakes of Bhuj, 2001, show that most of the buildings collapsed were deficient and did not meet the requirements of the present day codes. Thus, due to the ignorance for earthquakes resistant designing of buildings in our country and also wrong construction practices occurring in India, most of the buildings are vulnerable to earthquakes occurring in future.

Seismic designing, in a simplest case is observed to be a two-step process. Firstly, the most important, is the formation of an effective structural system that needs to be sorted out keeping in mind all-important objectives of seismic performance, ranging from the serviceability of the structure, considering life safety and also keeping in mind the collapse prevention. This step mainly involves the art of seismic engineering as no rigid rules can, or should, be obligatory on the creativity of the engineers. By default, the creation process is dependent on judgment, experience and understanding of the seismic behaviour rather than tedious and rigorous formulations by using mathematics. For an effective structural system, certain point need to be kept in mind-Rules of thumb for stiffness and strength (desired) targets that is based on the fundaments of ground motion and elastic and inelastic dynamic response characteristics. This would help to configure and roughly size an effective structural system.

Secondly (second step), step of design process which should involve demand /capacity/evaluation at all important performance levels, which also requires and involves the identification of all important capacity parameters and also prescription of demands imposed by the ground motions .Suitable capacity parameters and their acceptable values along with a very well suitable methods for demand prediction will depend on the performance level that is to be evaluated. Thus, the above facts shows that it is imperative to seismically evaluate the past/existing buildings with the present day knowledge, so that major quantity of destruction can be avoided in future earthquakes. Thus, buildings found to be seismically deficient should be strengthened/retrofitted is our need. To provide a detailed review of the body of literature associated to seismic evaluation in its total would be too massive to address in this thesis. However, there are good reference that can be used as a basis for research (ATC 40 Manual for Seismic Evaluation and Retrofitting of concrete buildings). This literature review and introduction will focus on recent assistance related to seismic evaluation and precedent efforts most closely related to the desires of the present work.

The goal of seismic evaluation of building is to resolve how buildings will retort to a design of earthquake described by the suggested spectra. In other words, the goal is to find the weak associations and to identify, how their actions will affect the response of the structural system. The position and behavior of a weak connection in a load path of lateral force existing system must be evaluated. The weak associations may function as a base isolator that will restrict the structural response of the lateral force resisting system (NEHRP, Washington, D.C 1992).

The capacity spectrum technique, which is non-linear static procedure, provides a graphical illustration of the global force displacement capacity curve of the structure and compares it to the response spectra illustration of the earthquakes demands. This approach includes consideration of ductility of structure on an element-by-element basis. The inelastic capacity of a building is then a evaluation of its ability to scatter earthquake energy(ATC40).

ATC-40 PROCEDURE FOR SEISMIC EVALUATION

Step-by-Step Procedure to determine capacity:

The most suitable way to plot force displacement curve is by detecting the base shear and roof displacement. The capacity curve is generally made to represent the initial mode response of the structure based on the postulation that the fundamental mode of motion is the major response of the taken structure. This is basically valid for buildings with the fundamental periods of vibration upto about 1 second limit. For more flexible buildings with the fundamental period > 1 second, the analyst should take into account addressing higher mode effects is the done analysis.

1. Creating a computer model of the structure following the modeling rules as given in ATC-40.
2. Classifing each element in model as either primary or secondary.
3. Apply lateral storey forces to the structure in ratio to the product of the mass and fundamental mode shape. This analysis need to also include gravity loads.

[As the name implies, it is the process of pushing horizontally with a approved loading pattern. Incrementally till the structure reaches a limit state. There are several levels of sophistication that may be used for the pushover analysis]

1. Simply appling a single concentrated horizontal force at the top of the structure (for one storey building)
2. Appling lateral forces to each storey in proportion(ratio) to the standard code procedure excluding the concentrated force F_t at the top
   i.e. $F_x = (W_x h_x / \Sigma PW_x h_x) \times V$ ………… (1)
3. Appling lateral forces in proportion to the product of storey masse and first mode shape of the elastic model of the structure.

i.e. $F_x = (W_x \Phi_x / \Sigma W_x \Phi_x) \times V$. ………..(2)

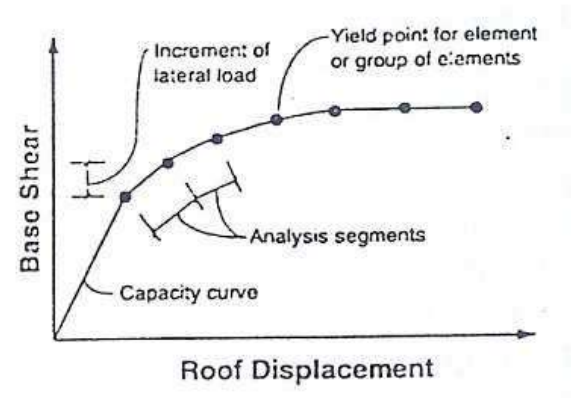
The capacity curve is usually constructed to represent the first mode response of the structure based on assumption that the fundamental mode of vibration is the predominant response of the structure.

4. Same as level three till first yielding. For each increment beyond yielding , regulate the forces to be consistent with changing deflected shape.
5. Similar to (iii) & (iv) above, but include the effects of the higher mode of the vibration in determining yielding in individual structural elements while plotting the capacity curve for the building in terms of first mode lateral forces and displacements. The higher mode effects possibly be determined by doing higher mode pushover analysis. (i.e. Loads may be progressively implied in proportion to a mode shape other than the fundamental mode shape to determine it in elastic behavior) For the higher modes the structure is being both push & pulled concurrently to maintained mode shape.
6. Calculate the member forces for the required combinations of vertical and lateral loads
7. Adjusting the lateral force level so that some elements (on group of elements) are stressed to lie within 10% of its member strength
8. Recording the Base shear and the roof displacement. (It is also helpful to record the member forces & rotations because they will be required for the performance check).
9. Revise the model using zero (or very small) stiffness for yielding elements.
10. Appling a new increment of lateral load to the revised structure such that another element (or group of elements) yields.
    [The actual forces and rotations for elements at the starting of the increment are equal to those at the end of the previous elements. However, each application of an increment of lateral load is a different analysis, which starts from zero initial conditions. Thus, to determine when the next elements yields, it is necessary

to add the forces from the current analysis to the some of those from the previous increments.

11. At the increment of the lateral load and the corresponding increment of roof displacements to the previous total to give the accumulated values of base shear and roof displacement.
12. Repeat steps 7,8 & 9 till the structures reaches an ultimate limit such as: instability from P-effects, distortions considerably beyond the desired performance level, an element attainment a lateral deformation level at which significant strength degradation begins.

Figure:**1**



Capacity Curve

Conversion of Capacity curve to the capacity spectrum:

To use the capacity spectrum method it is essential to convert the capacity curve, which is in terms of base shear and roof displacement to what is called a capacity spectrum, which is a representation of the capacity curve in Acceleration Displacement Response Spectra (ADRS) format i.e. (S_a vs S_d). The required equations to make the transformation are:

$PF_1 = \{\Sigma_{i=1} (w_i \Phi_{i1})/g\} / [\Sigma_{i=1} \{w_i (\Phi_{i1})^2/g\}]$

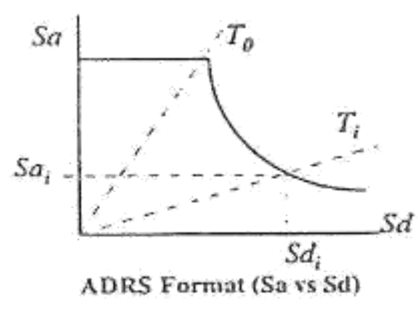
$\alpha_1 = \dfrac{\{\Sigma_{i=1} (w_i \Phi_{i1})/g\}^2 / \{\Sigma_{i=1} (w_i/g\} \ X}{[\Sigma_{i=}\{w_i(\Phi_{i1})^2/g\}]}$

$S_a = (V/W)/\alpha_1$

$S_d = (\ roof) / (PFi\Phi roof.1)$

Where, PF_i = Model participation factor for the first natural mode, α_1 = Model mass coefficient for the first natural mode,PW_i/g = mass assign to level i, Φ_{il} = amplitude of mode one at level i, N = Level N, the level which is the uppermost in the main portion of the structure.

Figure:**2**



Capacity Curve (ADRS Format)

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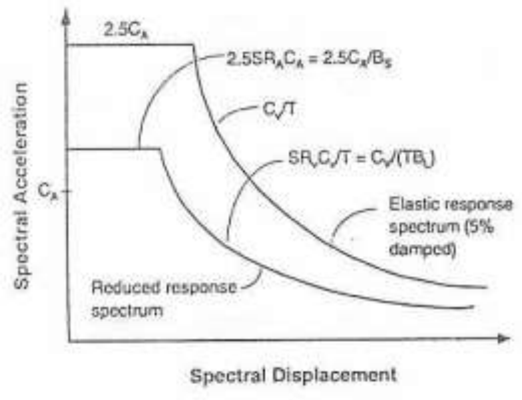
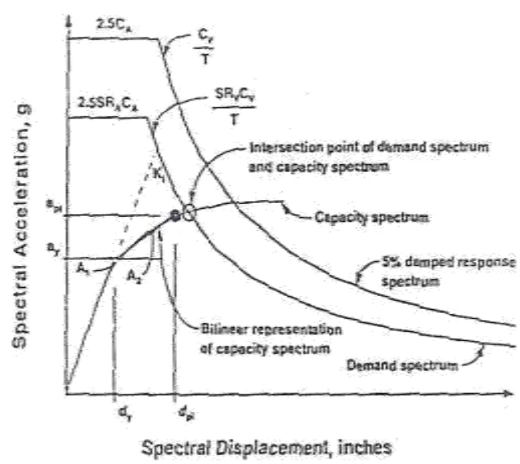
Calculating performance point
Figure:3



Reduced Response Spectrum

Figure:4



Final point of intersection location

The demand spectrum crosses the capacity spectrum within acceptable tolerance than the trial performance points a_{pi}, d_{pi} is the performance point, a_p d_p and the displacement d_p represents the maximum structural displacement probable for the demand earthquake.

A. SEISMIC EVALUATION OF SYMMETRICAL BUILDING FOR PROCEDURE DEVELOPEMENT
Evaluation Based on Elastic Approach
As mentioned in ATC-40, both elastic and in-elastic methods are existing for the analysis of existing concrete buildings. Seismic Evaluation can be performed by Elastic procedures using DCRs (Demand-Capacity Ratios): The work carried out by (Bhardwaj, 2002) is based on elastic approach. This is a linear elastic analysis, which involves the following three stages, namely:

Input data stage.

1. Study of site soil conditions.
2. Measurement of actual geometry of buildings and its element.
3. In-situ NDT to calculate approximately actual strength of concrete in the building components
4. Test to calculate approximately actual strength of steel reinforcement bars in the building components and the amount of corrosion, to carefully estimate their presented diameters.

Analysis stage:

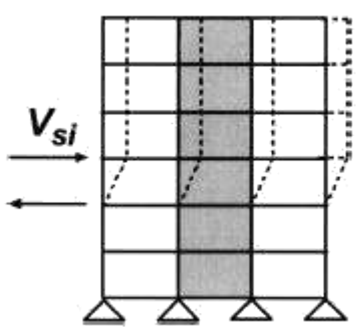
1. Grounding of 3 D model of building frame using measure geometry & material characteristics.
2. Evaluation of design later force on building using IS 1893:2002 for the specified design response spectra.
3. Application of design lateral force on 3D building model to resolve stress resultants (i.e. axial forces, shear forces, bending moments etc.), in the frame members and resolving of inter-story drifts.
4. Determination of RC member limits with original cross-sectional geometry and material properties as per IS 456:2000/IS 13920:1993 and DCR of RC members at critical points.
5. Identification of deficient member or insufficiency in lateral stiffness of the building .

Retrofit and verification stage:

1. Identification of appropriate retrofitting techniques to correct the deficiencies. Evaluation of the new member sizes along with the additional Reinforcement required, and/or the new members need.
2. Reanalysis of buildings to validate the adequacy with then anticipated retrofit techniques.
3. If strength and stiffness necessities are satisfied then the proposed retrofits scheme may be taken, else other more suitable retrofit scheme may be acknowledged.

The focus of these most current efforts is to seismic evaluation using elastic procedure by scheming Demand-Capacity Ratios (DCRs). Seismic Evaluation by Non-Linear Static Analysis exposes design weaknesses that may remain unidentified in an elastic approach. Such weaknesses include extreme deformation demands, strength irregularities, and overloads on potentially brittle points, such as columns and connections (Krawinkler H., Seneviratna G. 1998).
The research carried out by (Dinh T.V and Ichinose T.) recommended that the deformations demands for the columns, which are critical to the safety limits of the building, are closely associated to the drift in each storey. This is also a case for the deformations demands for the nonstructural elements that are noteworthy to the serviceability limit state. Thus, a designer must calculate the story drift demand, which may change due to uncertainties in the properties of future earthquake motions and the estimation of member strengths.
The FEMA-356 document (FEMA 2000) needs pushover analyses by two types of force distributions, like modal or uniform pattern, to check failure mechanisms and to predict storey drift demands of building. These analyses might discover two kinds of mechanisms. A modal pattern may stimulate a overall mechanism whereas a reliable pattern may result in a storey mechanism at the first storey. However, storey mechanisms at the second and upper stories might be ignored because the storey shear forces of these stories given by similar pattern are smaller than those by modal pattern.
In a building with structural walls, the probability of a storey mechanism decreases as the shear strength of the walls increases, as discussed by Park and Paulay (1975). In a frame building, the probability of a storey mechanism decreases as the column-to-beam strength ratio increases, as discussed by Dooley and Bracci (2001).
To integrate these tendency, a storey-safety factor f_i, is definite by the following equation (Abimanyu et al. 1997): $f_i = V_i / V_{ui}$, for $V_i = V_{si} - V_{ui}$ where, V_{si}= strength under the forces causing a storey mechanism of the (i)th storey as shown in Fig. 2.1 (the sum of the shear strength of the walls and the flexural/shear strength of the columns); and V_{ui}= shear force of the (i)th story when a failure mechanism occurs under overturned triangular forces as

Figure:5



Forces causing a story mechanism

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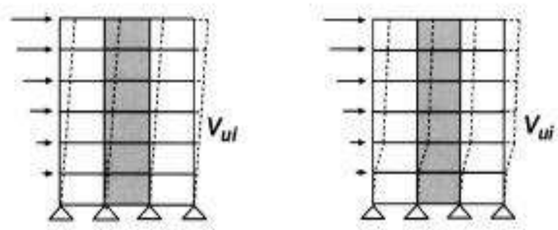
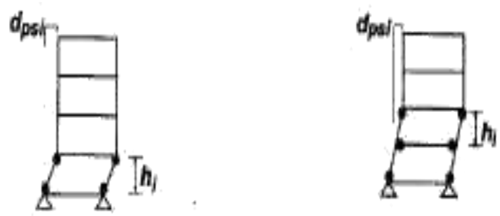


a) Total Mechanism (b) Storey Mechanism

Mechanisms due to inverted triangular forces: (a) total mechanism and (b) storey mechanism

The beginning study by Ichinose and Umeno (2000) showed that the drift responses of structures usually deforms due to the total mechanisms with weak beams Fig. 6(a) and weak columns Fig. 6(b) are almost the same if the primary periods, base shear strengths, and storey-safety factors of the structures are the same even though the column-to-beam strength ratios at the beam-column joints of the left column in Fig. 6(a) are larger than those in Fig. 6(b). Thus, the storey-safety factor enhanced represents the drift reaction than the column-to-beam strength ratio. The storey-safety factor is at present used in the Japanese Standard for the seismic evaluation of pre-existing RC buildings (Ichinose et al. 2002) to fo recast failure mechanisms.

Figure:**6**



(a) Total Mechanism with Weak Beams (b) Total Mechanism with Weak Columns

(c) Story Mechanism (d) Partial Mechanism

Failure mechanisms of frame structure: (a) total (overall) mechanism with weak beams; (b) total mechanism with weak columns; (c) story mechanism; and (d) partial mechanisms.

Further (Bernal D.) investigated that, Gravity loads performing on structural deformations reduce the lateral stiffness of buildings. The reduction is characteristically of minor importance when elastic behavior is taken into account because its magnitude in sensible structures is only a small portion of the first-order elastic stiffness. During response to severe ground movement, however, the lateral stiffness of a building decrease as a outcome of anticipated yielding and the reduction due to gravity can turn out to be a critical consideration. In particular, the potential for instability exists if the structure enters configurations where the effective stiffness linked with any given deformation approach becomes negative.

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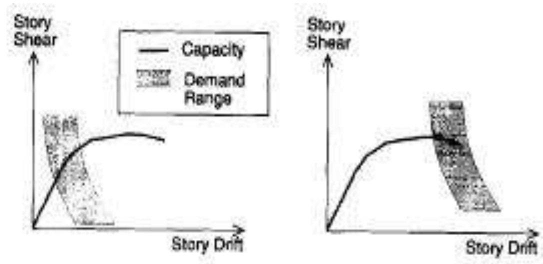
The investigation carried out by (Bracci J.M, Kunnath S.K, Reinhorn A.M) is based upon the evaluation of seismic performance and retrofit of accessibly low to midsize reinforced concrete (RC) buildings. The basis of the anticipated method is to develop a range of site particular demand curves and compares them to a computed pushover capacities at each story level of the structure. The primary differences between the anticipated scheme and capacity spectrum method are 1) the treatment of the applied lateral load during the pushover analysis: 2) the use of story level demand and capacities as opposed to overall base shear V/S top story displacement: and 3) the consideration of different levels in elasticity to generate a range of seismic demand curves.

Figure:**7**



(a) *(b)*

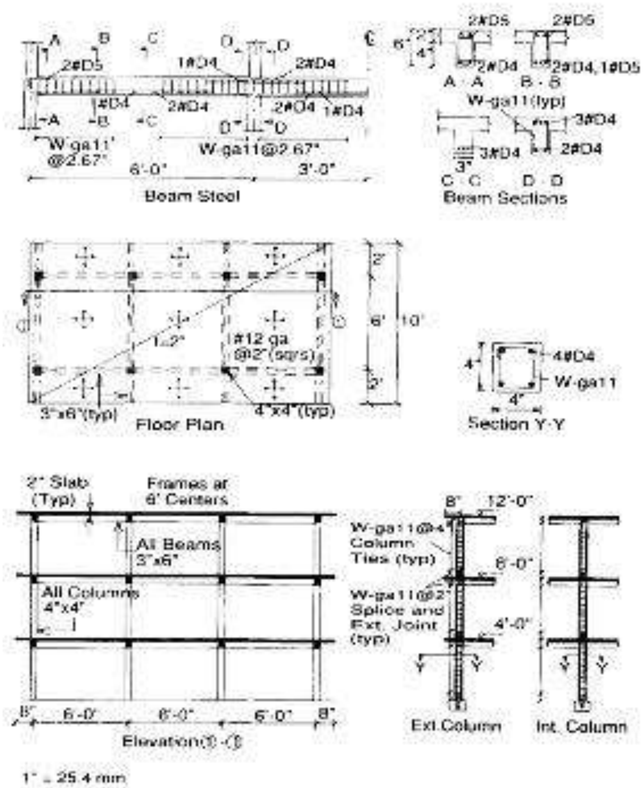
Typical Seismic Story Demand versus Capacity: (a) Safe Design; (b) Unsafe Design

Description of Original frame model Building:

Both current and past proposed practices for low rise RC frame buildings constructed in low to moderate seismic risk areas have generally being for only gravity loads (GLD) according to the non-seismic detailing requirements of the ACI-318-89 code. The seismic response of such structure was evaluated in a dual methodical – experimental shaking table study of 1/3rd scale model building. (Fig 2.5) The seismic deficiencies included in that study, all of which desecrated the seismic provisions of chapter 21 in ACI-318-89

included 1) weak column, strong beam, behavior creating a structure prone to a soft story fall down mechanism: 2) In-adequate transverse reinforcement in columns and joints for shear and confinements: 3) Column loops located in potential hinge zones: 4) irregular positive (bottom) beam strengthening in beam-column joints. The forced shaking table motions on the buildings pretend the 1952 Taft N21E earthquake component with normalized with PGAs of 0.05g, 0.20g and 0.30g. The spring motion test representing that damaging soft story mechanism was rising at the second floor level with potential for structural crash of the entire system.

Figure:**8**



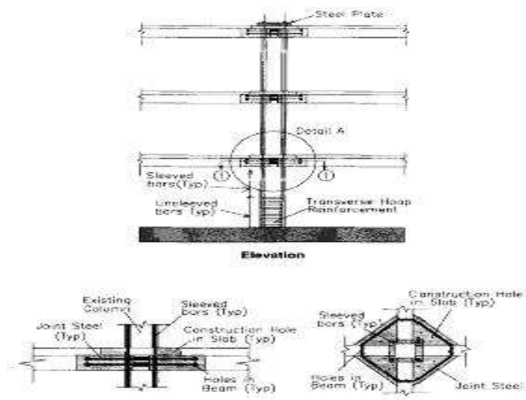
Details of Original Model

<u>Description of Retrofitted Frame Model Building:</u>

Following the series of damaged shaking table excitations on the original model structure, numerous seismic retrofit alternatives for GLD RC frame structure were investigated for improving their local and global response performance during low-to-moderate type earthquakes by comparing structural behavior from nonlinear dynamic analysis. These retrofit schemes were designed to distribute damage throughout the building with suitable control of story deformations by averting a terrible soft-story failure and enforcing a more ductile beam-sideway mechanism. A least seismic retrofit was performed on the interior columns of the beforehand damaged one-third scale model building using a prestressed concrete jacketing technique by (1) encasing selected columns in a concrete jacket with additional longitudinal and transverse reinforcement;(2) providing a reinforced concrete fillet about the unreinforced beam-column joints; and (3) post tensioning the additional longitudinal column reinforcement. By providing strength only to the internal columns of the model building, the retrofit scheme was measured to provide a minimum seismic resistance satisfactory for low-to-moderate seismicity zones and was considered insufficient for high seismic zones.

The experimental testing of the retrofitted model building was then done on the shaking table with the same excitations as in the original model testing with PGAs of 0.20g and 0.30g. When compared to pre-retrofit performance, the new experimented results of the retrofitted model structure showed that the total behavior and damage can be more efficiently controlled using only minimum column and strengthening of joint techniques, which may be appropriate for structure in low-to-moderate seismic risk zones. The retrofit scheme was successful in altering the failure mode from an undesirable column sides sway mechanism to a more ductile beam sides sway mechanism.

Figure:**9**



Detail A: Reinforced Fillet   Section 1-1: Retrofitted Prestressed Concrete Jacketing Retrofit

Nonlinear pushover analyses were performed on both the original and retrofitted model buildings.

For the 0.05g PGA excitation, the demand region intersect the story capacity envelopes near the elastic portion of the response, implying elastic behavior. For the 0.20g and 0.30g PGA excitation, the demand curves intersect the capacity covering in the region at initial and full failure mechanism, respectively.

From the point of view of estimating the margin of safety of the structure adjacent to the imposed loading, the following observations can be made from Figs.10-11 for the original building: the structure performed in the elastic limit during the 0.05g shaking: for the 0.20g intensity shaking, the structure responded in the inelastic range with likely elastic in some members, but had a sufficient margin of safety against structural fall down as seen from the strength and deformation capacity beyond the connection of the demand end capacity lines; however,for the 0.30g shaking, the demands interconnect the capacities with little strength and deformation set aside. Therefore, it can be concluded that the margin of safety against fall down for the original building was small at this power of shaking.

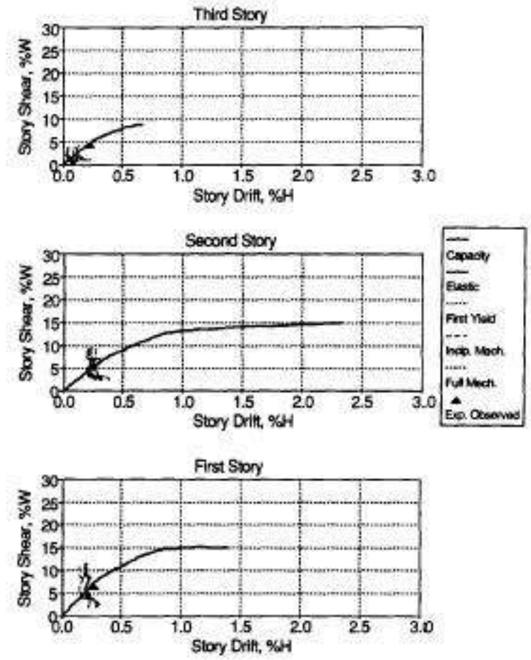
For the retrofitted model building under the similar earthquake ground motions (0.20g and 0.30g), it can be seen from Figs. 2.10 and 2.11 that most of the inelastic response occurs on the first and second stories.

The assumed maximum response for each storey co-relates well with the experimentally witnessed maximum behavior.

From the point of view of evaluation of the capability of the retrofit behavior, it can be witnessed that the story demands overlap the capacity envelopes with adequate strength and displacement reserves.

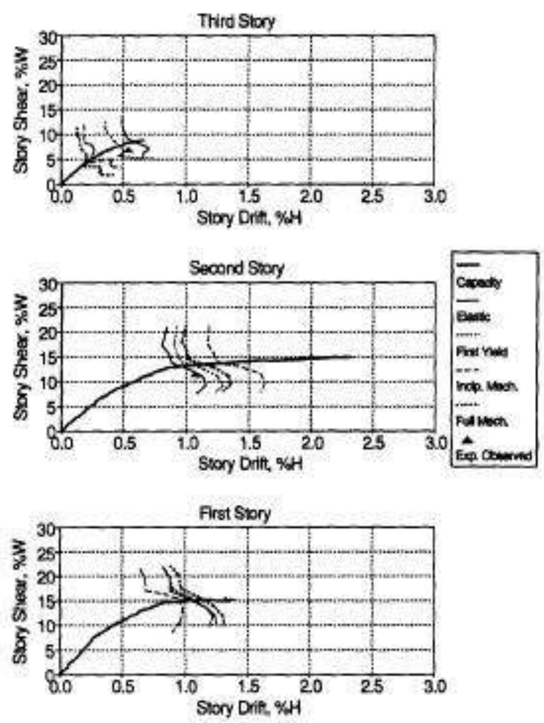
Although inelastic response is obvious during these base motions, collapse of the retrofitted structure is not forthcoming for these levels of ground motion excitation.

Figure:**10**



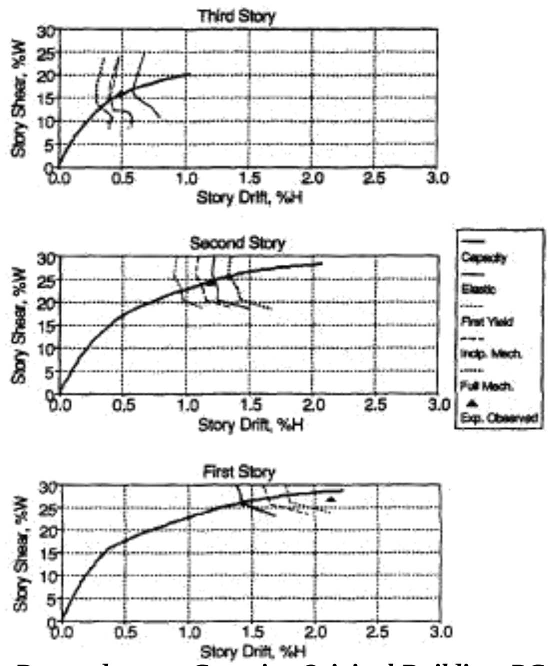
Seismic Demand versus Capacity, Original Building, PGA 0.05g

Figure:**11**



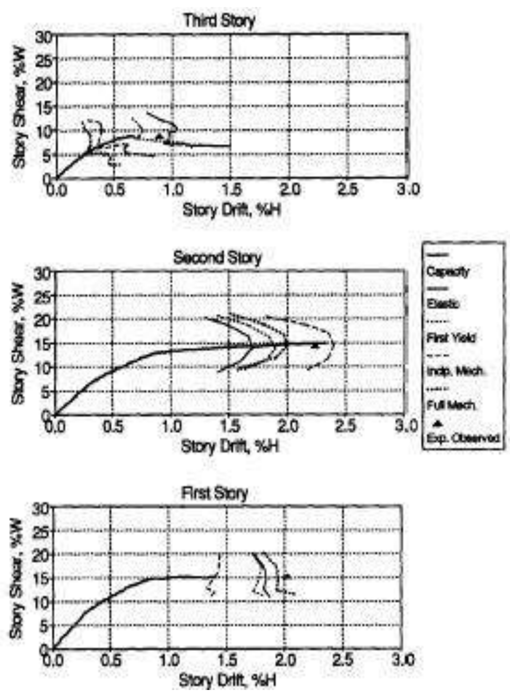
Seismic Demand versus Capacity, Original Building, PGA 0.2g

Figure:12



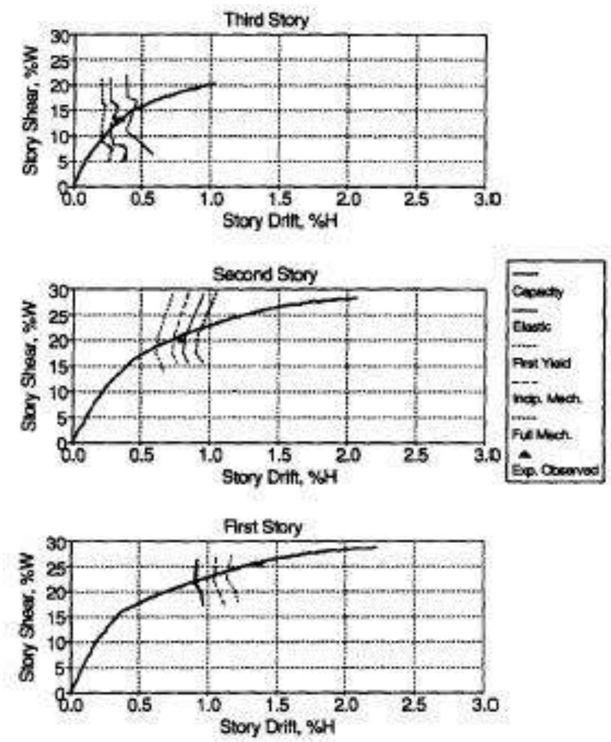
Seismic Demand versus Capacity, Original Building, PGA 0.3g

Figure:13



Seismic Demand versus Capacity, Retrofitted Building, PGA 0.20g

Figure:**14**



Seismic Demand versus Capacity, Retrofitted Building, PGA 0.30g

Further, the research done by (Kappos A.J, Manafpour A.) suggested that, along with numerous other seismic codes currently in action, the American UBC was eventually based until 1994 on elastic analysis for equal static forces or a design spectrum. The 1997 publication of this code introduced for the earliest time a few detailed provisions for the application of both elastic and inelastic time–history analysis, mainly in regard to the selection and scaling of ground motions. A alike development took place in the latest NEHRP Provisions [3] but in this case time–history study is undoubtedly limited to structures with seismic isolation. The seismic Euro-code, EC8 [4], recognizes that inelastic time–history analysis may be used in the design procedures, but supervision is only given in regard to the selection of input accelerograms and the manner they need to be scaled to match the design spectrums. No clear suggestion is given in EC8 as to what model(s) should be used, or which response quantities should be sought, if such an investigation is utilized. On the other hand, the New Zealand Code [5] is clear in specifying that the reason of using inelastic time–history analysis may be either used to calculate strength necessities in yielding members, or assess inelastic demands and/or capacity actions.

(Krawinkler H., Seneviratna G.D.P.K), suggested that a cautiously performed pushover analysis will provide insight into structural aspects that control performance during high intensity earthquakes. For structures that shake primarily in the fundamental mode, the pushover analysis will very likely give good estimates of global, as well as local inelastic, deformation demands. This analysis will also expose design weaknesses that may remain concealed in an elastic analysis. Such weaknesses include storey mechanisms, extreme deformation demands, strength irregularities and overloads on potentially brittle elements like columns and connections.

On the negative side, deformation estimates obtained from a pushover analysis may be very inaccurate (on the high or low side) for structures in which high mode effects are significant and in which the storey shear force versus storey drift relationships are sensitive to the applied(provided) load pattern. This problem can be mitigated, but usually not removed, by applying more than one load pattern, including load patterns that report for elastic higher mode effects (e.g. SRSS load patterns). Perhaps most critical is the concern that the pushover analysis may find out only the first local mechanism that will form in an earthquake and may not find out other weaknesses that will be generated when the structure's dynamic characteristics changes after formation of the first local mechanism.

CONCLUSION

The literature review suggested that Seismic Evaluation of R.C Buildings using non-linear approach was undoubtedly feasible as it exposes design weaknesses that may remain hidden in an elastic approach. Such weaknesses involve excessive deformation demands, strength irregularities, and overloads on potentially brittle points, such as columns and connections (Krawinkler et al, 1998). It was decided to use ANSYS as the FE modeling package. As capacity curve is the output of Pushover analysis, Load Deformation Plot (capacity curve plot) can be obtained from ANSYS non-linear static analysis. Seismic Evaluation of pre-existing R.C buildings is carried out. Firstly, analysis was done on symmetrical building for procedure development as given in ATC 40 guidelines. Then, analysis is done on the asymmetrical building (L-shape). In first case, evaluation is carried out on the building designed non-seismically and its results (outcomes) have been compared with the analysis of seismically designed building (as per IS 1893:2002). And the affected members have been recommended for strengthening.

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