SEISMIC PERFORMANCE EVALUATION AND RETROFITTING OF RC MEMEBERS AND JOINTS

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ABSTRACT: In the present work, structure designed and constructed for only gravity loads is considered for evaluation and retrofitting work. Finite element software ETABS is used to determine the seismic demand of each element. Retrofitting to increase the capacity of elements is suggested for the elements having ratio of Demand to Capacity more than 1. Pushover analysis is used to determine the performance of the structure before and after retrofitting. In the present work, deficient columns are retrofitted and re-analyzed to check performance of the structure in non-linear analysis. Performance of this retrofitted structure is then compared with the existing reinforcement structure and it is found that structure after retrofit have more base shear capacity and displacement capacity, storey drift of the retrofitted structure has decreased thereby ensuring a maximum safety of the structure even to the zone3 level of seismic intensity. From the present study it is brought out that structural elements designed only for gravity loads have less vulnerability to collapse in zone 2 level of seismic intensity, and for zone 3 level of seismic intensity itself structural elements fails to perform both serviceability limit state as well as ultimate-strength limit state.

KEY WORDS: Evaluation, Performance evaluation, Pushover analysis, RC joints, Demand Capacity Ratio, Retrofitting, Non-linear analysis, Performance point,

1. INTRODUCTION

In the conventional limit state design approach, the designer normally takes into account the self-weight of the structure (dead load), imposed loads (live load), and depending on the location of the building, seismic, and climate related loads (wind and snow loads) are considered. While the vast proportion of the existing buildings experience only the types of loads mentioned above during their lifetimes, but building has to be designed to resist seismic load or lateral load which is assumed to occur once its life-time. In these conventional design method only two levels of design is considered, that is, ultimate-strength limit state and service-operational limit state for a building. But performance based design can be viewed as multi level design approach which has definite concern on performance of a building at intermediate limit states related to such issues as occupancy and life-safety standards. Hence we need to adopt a convenient analysis tool to analyze and design for performance-based approach. A structural analysis tool gives a number of analysis methods. For performance based analysis of structures a hierarchy of structural analysis may be made. In which higher level procedure gives more accurate method of the actual performance of building subjected to earthquake loads, but interpretation of the results requires greater efforts and time consuming.

However in this work, existing reinforcement of the building is compared with linear static analysis result obtained as per the IS1893:2002(PART-1), structural elements which ever found deficient will be identified in this process and retrofitting methods are suggested. The performance of the building is checked using Non-Linear static procedure. Pushover analysis is a simplified, static, non linear procedure where a predefined pattern of earthquake loads is applied incrementally to the structure until a collapse mechanism is reached. The use of inelastic analysis procedure is an attempt to understand how structures will behave when subjected to earthquake load; it is assumed that the elastic capacity of the structure will be exceeded.

2. LITREATURE REVIEW

A detailed review has been carried out on the past research work on the behavior of joints both on experimental and analytical sides to focus on recent and past efforts related on seismic evaluation. A few research work done on the above mentioned area's are summarized below.

- ❖ **Umesh Dhargalkar. (2002) [16],** mainly dealt with the seismic assessment for the seismic retrofitting of the structures constructed with or without the seismic effect. The standard and comprehensive assessment involves data collection, compilation of data and assessing possible guidelines. Based on the data collected possible schemes of retrofitting can be checked by modelling an exact replica of the building. The best fit method is selected based on cost and convenience of implementation.
- ❖ **Pradip Sarkar, Rajesh Agarwal, and Devdas Menon (2007) [17],** revised the relevant features of shear design of joints under seismic loads given in international codes of practice (ACI, NZS, EN) highlighting requirements of the various parameters. According to this paper shear transfer mechanism categorized into 2 mechanism viz. diagonal strut mechanism and truss mechanism. Assessment of shear strength, design and detailing of shear reinforcement has been covered. It is seen NZS is very conservative recommendation followed by Euro code and ACI give many practical recommendations. Whereas IS 13920:1993 is silent on many issues related to the design of RC beam column joints under seismic loading. Hence it is necessary to upgrade IS 13920 keeping with international trends.
- ❖ **G.Appa Rao, M.Mahajan, M.Gangaram, and Rolf Eligehausen (2008) [19]**, dealt with the method of strengthening non-seismically designed RC beam-column joints to seismic loading. Typical reinforcement details of joints in pre-seismic design have been explained. Review of strengthening method and features, advantages and test results of FRP in rehabilitation of RC structures have been discussed. Hence in this paper merits and demerits of the strengthening of joints have been highlighted.
- ❖ **S. R. Uma, and Sudhir K. Jain. (2006) [21],** presented critical review of recommendations of well established codes regarding design and detailing aspects of beam column joints. The codes of practice considered are ACI 318M-02, NZS 3101: Part 1:1995 and the Euro code 8 of EN 1998-1:2003. All three codes aim to satisfy the bond and shear requirements within the joint. It is observed that ACI 318M-02 requires smaller column depth as compared to the other two codes based on the anchorage conditions. Significant factors influencing

the design of beam-column joints are identified and the effect of their variations on design parameters is compared. The variation in the requirements of shear reinforcement is substantial among the three codes.

- **Sudhir K. Jain, and T. Srikant (2002) [22],** discussed Pushover analysis for deficient buildings, new buildings or to make existing building perform well in future earthquake. In this work a four storey building with flat slab designed for wind load but not for seismic load is considered for the study. 2D frame of this building is modelled and Pushover analysis is performed in SNAP-2DX. Jacketing of column, providing additional beams and providing both columns jacketing and additional beams are the various retrofit schemes adopted. This scheme is studied at 4 different cases, i.e., at first storey only, first two storey, first three storey and all the four storeys. They found significant increase of strength and drift capacity when both jacketing of column and addition of beam.

3. PUSHOVER ANALYSIS

The Pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and monotonically increasing lateral loads. The equivalent static loads approximately represent earthquake induced forces. A plot of total base shear versus top displacement in a structure is obtained by this analysis (Figure1) that would indicate any premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. On a building frame, load/displacement is applied incrementally. The formation of plastic hinges, stiffness degradation and plastic rotation is monitored, and lateral inelastic force versus displacement response for the complete structure is analytically computed. This type of analysis enables weakness in the structure to be identified. There are different methods followed for pushover analysis. Basically it has been classified into two ways they are Force controlled and displacement controlled. In force control, the structure is subjected to lateral forces and the displacements are calculated. In displacement control, the structure is subjected to a displacement profile and the lateral forces are calculated.

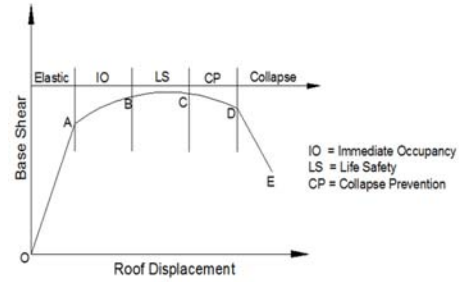

FIGURE.1. IDEALIZED PUSHOVER CURVE

4. PROBLEM STATEMENT

In the present work structural components of the building (Figure.2) are previously designed and constructed without considering the seismic effect. Structure is analyzed using ETABS by considering linear static analysis in x and y direction and Non linear static analysis along x direction only. ETABS design for seismic effect is compared with existing reinforcement and discussed. Capacity of each component with existing reinforcement in this building is compared with demand posed by the analysis results with consideration of lateral force for both Zone 2 and Zone 3 earthquake regions. This comparison is represented in the form of Demand and capacity ratios (DCR). Any structural elements found deficient in this DCR check will be retrofitted. For columns concrete jacketing and for beams Fiber Reinforced Polymer (FRP) wrapping is suggested. These analytical models (Zone2 and Zone3 ETABS designed models, existing reinforcement in Zone2 and Zone3 analysis, column retrofitted models in Zone2 and Zone3) are subjected to PUSHOVER analysis, results obtained from this analysis are (Base shear versus Displacement curve, S_a versus S_d curve, Performance point and Hinge formation at performance point) discussed.

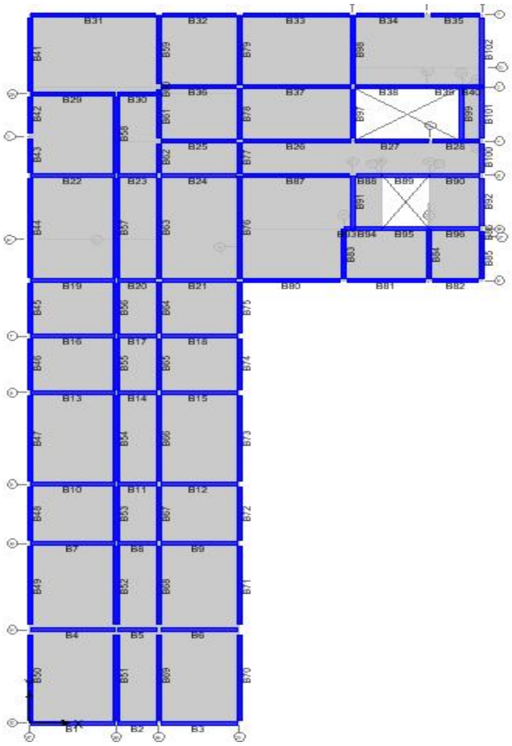

FIGURE.2 PLAN OF THE BUILDING CONSIDERED IN CASE STUDY**5.**

5. RESULTS AND DISCUSSION

I. EVALUATION OF BEAMS.

The capacity (flexural or shear) of the beam is obtained from the following derivation.

1. Moment of resistance of the beam (M_{ur}): For a given cross section of the beam and for the existing reinforcement, Moment of resistance is calculated by finding Neutral axis (X_u) and determining stress and strain at the level of X_u. This is derived as follows.

STEP1: Stress at the level of neutral axis (X_u) is

$$\boldsymbol{\varepsilon_{se}}=\left(\frac{0.0035\times(X_u-d')}{X_u}\right)$$

For the calculated stress, strain is obtained using stress-strain curve given in SP16.

STEP2: Based on the ideology, Compression (C) and Tension (T) are equal at X_u. Then C and T are calculated by;

$$\mathbf{C} = (0.36\times f_{ck}\times b\times X_u)+Asc\times(f_{sc}-0.45f_{ck})\ ;\ \mathbf{T}=(f_{sc}\times A_{st});$$

Where

b = breadth of the beam,

f_{ck}= flexural strength of concrete.

STEP3: After determining C & T moment of resistance is determined by

$$\mathbf{M_{ur}}= (0.36f_{ck}bX_u)\times(d-0.42X_u)+A_{sc}(d-d')\times(f_{sc}-0.45f_{ck}).$$

STEP4: This Moment of resistance M_{ur} must be greater than Flexural demand M.

After determining the flexural capacity of the beam elements, it is compared with the demand obtained by ETABS linear static analysis with Zone2 and Zone3 seismic intensity and following results are obtained. And typical graphical representation is shown in Figure 3, 4 & 5.

1. 39 elements in left end of the beam, 82 elements in mid-span of the beam, 34 elements in right end of the beam have DCR value 1to2 in seismic Zone2. Where as in seismic zone 3 it is found that 83 elements in left end, 88 elements in mid-span, 89 elements in right end are deficient.
2. 12 elements in left end of the beam, 14 elements in mid-span of the beam, 6 elements in right end of the beam have DCR value 2to3 in seismic Zone2. Where as in seismic zone 3 it is found that 27 elements in left end, 16 elements in mid-span, 17 elements in right end are deficient.
3. 2 elements in left end of the beam, 1 element in mid-span of the beam, 3

elements in right end of the beam have DCR value 3to4 in seismic Zone2. Where as in seismic zone 3 it is found that 7 elements in left end, 10 elements in right end are deficient.

4. 4 elements in left end of the beam, 4 elements in mid-span of the beam have DCR value above 4 in seismic Zone2. Where as in seismic zone 3 it is found that 5 elements in left end, 5 elements in mid-span, 5 elements in right end are deficient.

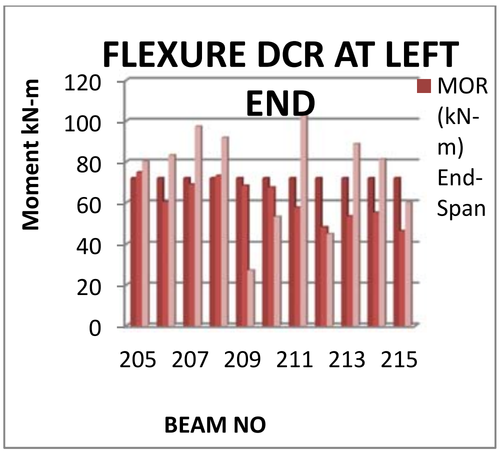

FIGURE.3. COMPARISON OF FLEXURAL DEMAND AND CAPACITY OF BEAM AT LEFT END

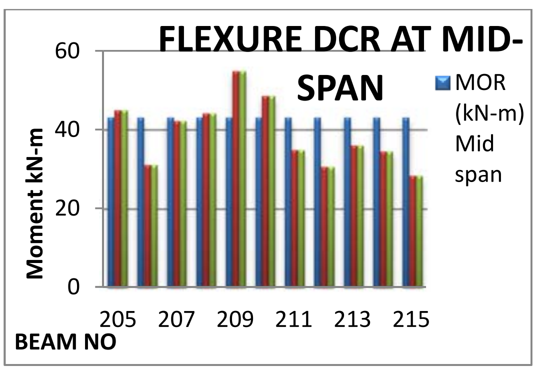

FIGURE.4. COMPARISON OF FLEXURAL DEMAND AND CAPACITY OF BEAM AT MID-SPAN

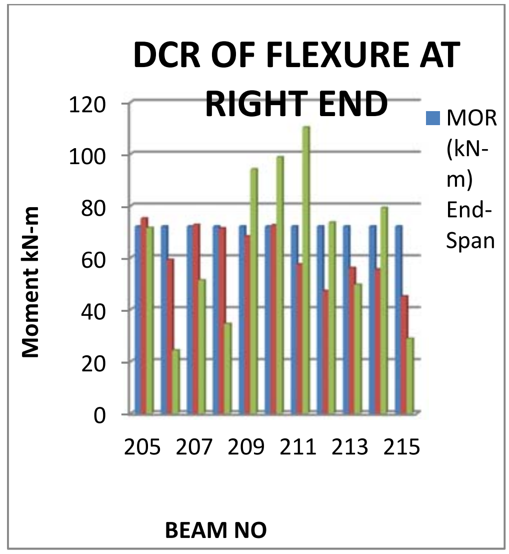

FIGURE.5 COMPARISON OF FLEXURAL DEMAND AND CAPACITY OF BEAM AT RIGHT END

From the above results it is observed that number of beam elements having DCR value greater than 1 are more in case of seismic zone3 and the structure is more vulnerable to seismic intensity zone3 itself.

2. Shear Capacity calculation (V_u): Shear strength concrete (V_{uc}) and Shear is strength of steel (V_{us}) is calculated and summation of this is compared with the Shear demand (Vu). The calculation of the shear demand is given by;

STEP1: For the existing $A_{st,}$ Shear strength in concrete τ_c is found by utilizing IS456:2000 Table19. Where $\tau_c < \tau_{max.}$

STEP2: Shear resistance of concrete $V_{uc} = \tau_c bd$

STEP3: Shear resistance of the steel $V_{us} = \left(\frac{0.87 A_{sv} f_y d}{S_v}\right)$

STEP4: Ultimate Shear resistance is given by $Vu = V_{us} + V_{uc}$,

where Vu>Shear demand (V)

STEP5: According to IS13920:1993, Shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span.

For sway to right: $V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M^{As}{}_{u,lim} + M^{Bh}{}_{u,lim}}{L_{AB}}\right]$

And $V_{u,b} = V_b^{D+L} + 1.4\left[\frac{M^{As}{}_{u,lim} + M^{Bh}{}_{u,lim}}{L_{AB}}\right]$

Where

$M^{As}{}_{u,lim}$ & $M^{Bh}{}_{u,lim}$ are sagging and hogging moments of resistance of the beam section at ends A and B.

V_a^{D+L} and V_b^{D+L} are the shears at ends A and B due to vertical loads with 1.2 partial safety factor on loads.

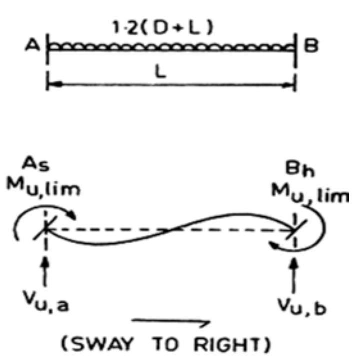

FIGURE.6. CALCULATION OF DESIGN SHEAR FORCE FOR BEAM

In determination of Shear demand of the beam we have taken only the gravity load combination (i.e. DL+LL) hence we get only one shear demand and following results are obtained in terms of Demand Capacity Ratio DCR.

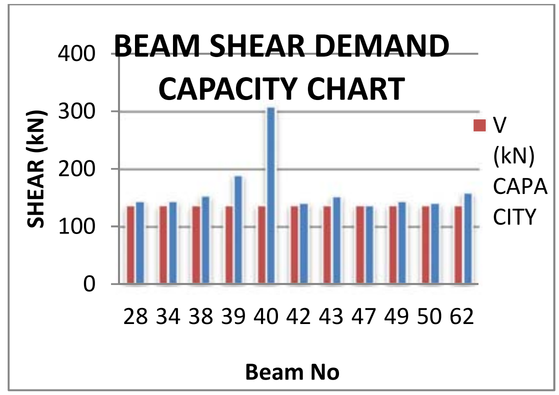

FIGURE.7.COMPARISON OF SHEAR DEMAND AND CAPACITY OF BEAM AT ENDS

1. 183 beam elements in the structure have DCR value 1 to 2.
2. 6 beam elements have DCR value 2 to 3.
3. 5 beam elements have DCR value 7 to 8 and
4. 5 beam elements have DCR value greater than 9.

II. EVALUATION OF COLUMN

In evaluation of the column the Factored axial load and moments (uni-axial or bi-axial) are compared with the ultimate moment carrying capacity of the column with the existing reinforcement and DCR is calculated. Similarly shear demand also calculated but compared with the capacity as given in IS13920:1993.given by

1. MOMENT CAPACITY OF THE COLUMN SECTION:

STEP1: For the given column section, existing reinforcement Pt, known d`/D ration, p_t/f_{ck} is determined.

STEP2: By referring the interaction curve given in the SP16 for the actual [P_u / $f_{ck}bD$], p_t/f_{ck}, determine [M_u / $f_{ck}bD^2$] and calculate M_u, then compare it with the moment from the analysis.

STEP3: Determine the DCR of bending moment, member is safe if DCR<1.

After determining the DCR of flexure following results are obtained.

1. 29 column elements have DCR value 1-2, 3 column elements have DCR value 2 to 3, 3 column elements have DCR value greater than 3 in zone2 seismic intensity. And
2. In zone 3 seismic intensity 46 column elements have DCR value 1 to 2, 8 elements have DCR 2 to 3, 3 elements have DCR 3 to 4 and 3 elements have DCR above 4.

In the case study structure 61 column elements were present out of which 60 columns have very less capacity in Zone 3 seismic intensity.

2. SHEAR CAPACITY OF THE COLUMN: Calculation of shear capacity in column requires an assumption that one face of the steel reinforcement is completely in tension. Method of calculation of the shear capacity is explained with Figure.8.

STEP1: Area of reinforcement (A_s) of that face is calculated, for which τ_c is calculated from IS456:2000 Table 19.

STEP2: Since the shear reinforcement is known, Design shear strength is determined by;

$$V_{us} = \left(\frac{0.87 f_y A_{sv} d}{S_v}\right)$$

STEP3: Total shear strength (V_u) of the section is calculated by summing up the concrete shear strength (V_c) and shear strength of the stirrups (V_{us}), which is termed as shear capacity of the column.

STEP4: From the IS13920 the design shear force for column shall be the maximum of;

a) Calculated factored shear force from the analysis, and

b) A factored shear force given by Vu= 1.4 $\left[\frac{M_{u,Lim}{}^{bL} + M_{u,Lim}{}^{bR}}{h_{st}}\right]$

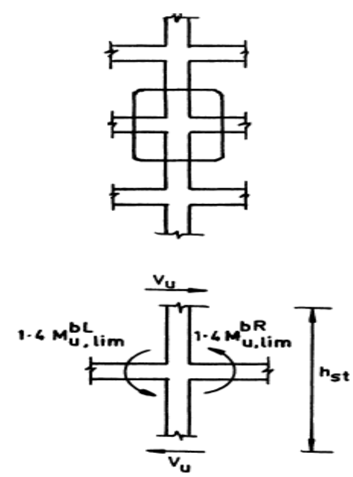

FIGURE.8. CALCULATION OF DESIGN SHEAR FORCE FOR COLUMN

Where,

$M_{u,Lim}{}^{bL} + M_{u,Lim}{}^{bR}$ Are moment of resistance, of opposite sign, of beams framing into the column from opposite faces and h_{st} is the storey height.

After a detailed evaluation of column it is found that all columns have higher shear strength capacity.

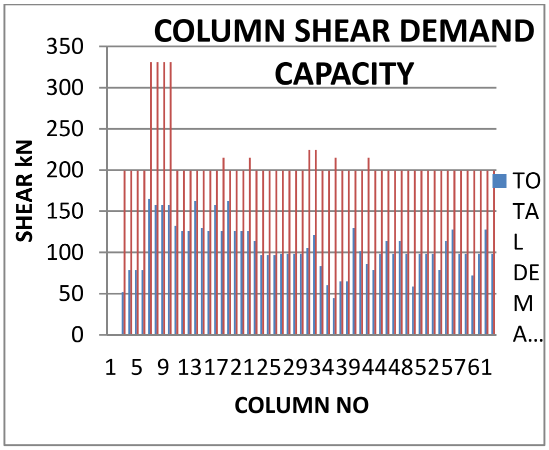

FIGURE.9.COMPARISON OF COLUMN SHEAR DEMAND AND CAPACITY

III. STRONG COLUMN WEAK BEAM

The current approach to the design of earthquake resistant RC rigid (i.e., moment resistant) frame is to have most of the significant inelastic action or plastic hinging occur in the beams rather than in columns. This is referred to as the “STRONG COLUMN-WEAK BEAM” concept and is intended to help ensure the stability of the frame while undergoing large lateral displacement under earthquake excitation. IS15988:2013 gives following equation to determine the strong column-weak beam

$$\sum M_c \geq 1.1 \sum M_B$$

Since interior column consists of beams running in two perpendicular direction, for the simplification it is divided into Major and Minor axis. Number of Columns which do not holds good with Strong Column Weak Beam philosophy are tabulated in Table.1

TABLE.1. NUMBER OF COLUMNS WITH STRONG BEAM AND WEAK COLUMN

AXIS	STOREY 1	STOREY 2	STOREY3	STOREY 4	STOREY 5
MAJOR AXIS	37	48	38	34	39
MINOR AXIS	25	20	11	9	12

Since too many columns in both major and minor axis are weak compared to its adjacent beams, retrofitting has to be adopted in all the storey level and performance is rechecked. Beam-Column joints of the case study 1 building are checked, and it is found that all joints are safe.

IV. RETROFITTING OF COLUMN

Evaluated columns after comparison with demand it is found that all columns are against the philosophy of “Strong Column Weak Beam”. Hence depending upon the DCR ration columns are categorized and retrofitted. A simplified analysis for the flexural strength of a retrofitted column can be done by the traditional method of interaction curves (SP 16: 1980, “Design Aids for Reinforced Concrete to IS 456: 1978, published by the Bureau of Indian Standards). The retrofitted columns and dimension are shown in Table.2.

TABLE.2. Details Of Retrofitted Columns

SL.NO	Existing Colum	Revised	Increased Ast mm^2	Total Columns

	n Size (mm)	Section s mm		
1	200X380	300X480	1561	40
2	200X685	300X785	3000	4
3	200X380	300X480	1273	3

V. STOREY DRIFT

TABLE.3. PERCENTAGE DECREASE IN STOREY DRIFTS OF THE RETROFITTED BUILDING

ZONES	DRIFT X	DRIFT Y
ZONE 2	45%	40%
ZONE 3	45%	42%

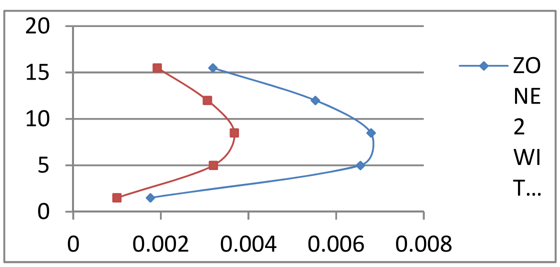

FIGURE.10.STOREY DRIFT X IN ZONE 2

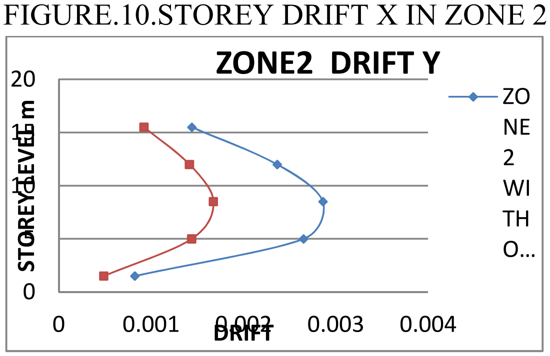

FIGURE.11.STOREY DRIFT Y IN ZONE3

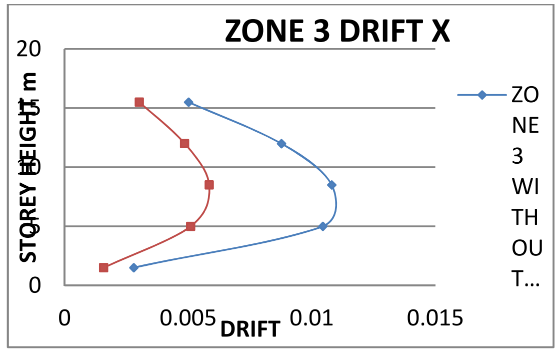

FIGURE.12.STOREY DRIFT X IN ZONE3

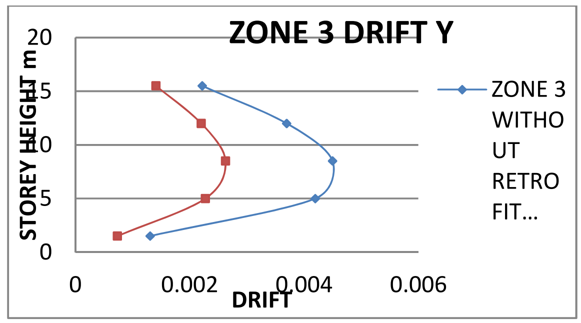

FIGURE.13.STOREY DRIFT Y IN ZONE 3

The actual storey drift of the case study building is displayed in Figure 10, 11, 12 and 13. In these figure's displacement of building in X and Y direction, with existing reinforcement and retrofitted members are compared and the percentage of decreased drift in the retrofitted model are shown in the Table.3. From the results of the storey drift we can conclude that, after retrofitting the building storey drift is reduced by 45% in X direction in both Zone2 and Zone3 seismic region and a 40% reduction in storey drift in Y direction in both Zone2 and Zone3 seismic region. Hence the building is safe after retrofitting under serviceability limit state.

6. RESULTS AND DISCUSSION OF PUSHOVER ANALYSIS

The result obtained from the Pushover analysis that is Base shear versus Displacement, Spectral acceleration versus Spectral Displacement and Hinge formation at performance point are discussed.

I. COMPARISON OF PUSHOVER CURVES

The case study building is designed in ETABS for ZONE 2 and ZONE 3 seismic loading. The same building is analyzed with the existing reinforcement without altering the member dimensions. During the evaluation part of the case study1 structure, it is observed that the columns are deficient in load carrying capacity. Hence retrofit is carried out to all columns. This retrofitted column is provided as such in ETABS and analyzed. Pushover analysis is carried out to all the above cases and 4 different PUHOVER curves are obtained and displayed in Figure.14.

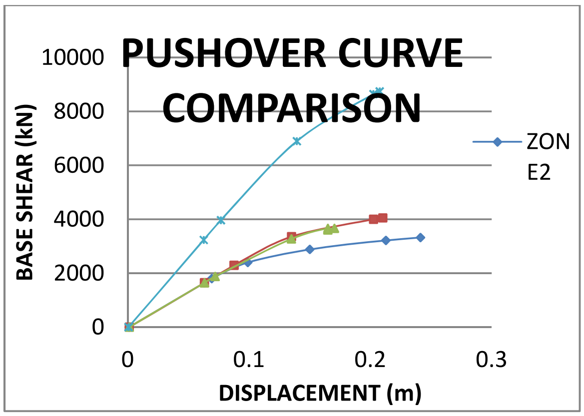

FIGURE.14. PUSHOVER CURVE COMPARISON

From the comparison of the PUSHOVER curves following conclusions are drawn

i. Structure with existing reinforcement has lesser base shear capacity and displacement capacity compared to the same structure designed for Zone2 and Zone3 level of earthquake. Hence, the structure with existing reinforcement is more vulnerable compared to those designed for earthquake loads.

ii. Structure designed for Zone 2 has lesser base shear capacity compared to the structure designed for Zone3. Hence it can be concluded that structures designed for higher zones of earthquake have better seismic capacity.

iii. The pushover curve for retrofitted building shows very high base shear capacity and displacement capacity compared to all other structure. Hence this structure is less vulnerable compared to all other building.

iv. Retrofitting of the existing deficient buildings as detailed in the present study can be an efficient way of improving the seismic performance of vulnerable buildings.

II. COMPARISON OF PERFORMANCE POINT

TABLE.4. COMPARISION OF HINGES AT PERFORMANCE POINT

	Displacement	Base	Force	A-B		B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E TOTAL
ZONE 2	0.099	2398.3	1358	134		118	0	0	0	0	0	1610
ZONE 3	0.1351	3356.4	1332	137		122	19	0	0	0	0	1610
EXISTING	0.135	3274.9	1469	44		54	42	0	1	0	0	1610
RETROFI	0.0768	3965.4	1542	56		12	0	0	0	0	0	1610

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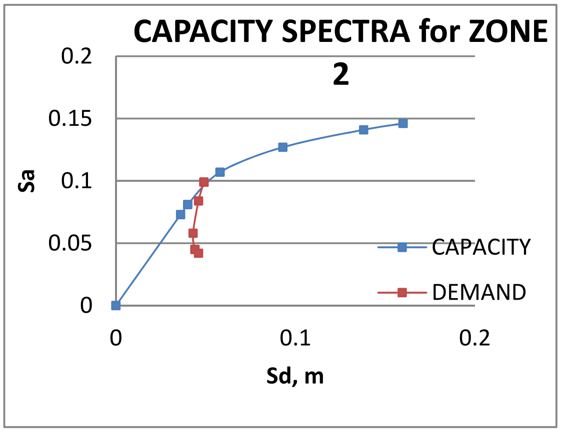

FIGURE.15.CAPACITY SPECTRA FOR STRUCTURE WITH ZONE2 DESIGN

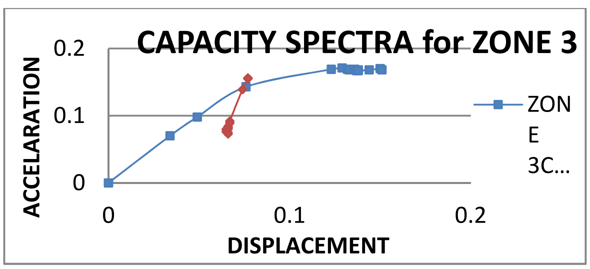

FIGURE.16.CAPACITY SPECTRA FOR STRUCTURE WITH ZONE3 DESIGN

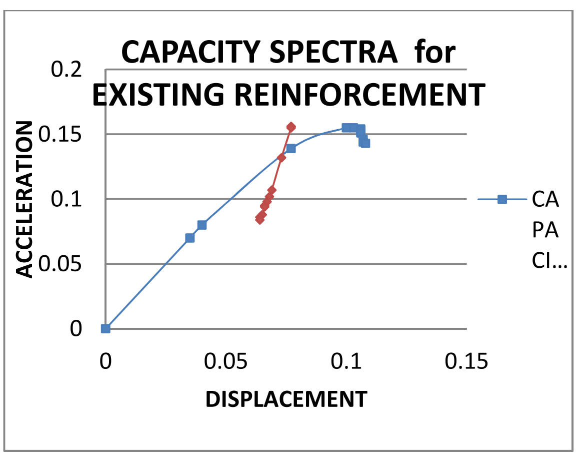

FIGURE.17.CAPACITY SPECTRA FOR STRUCTURE WITH EXIXTING REINFORCEMENT

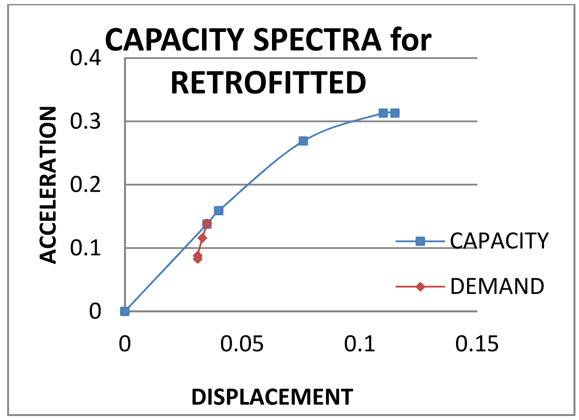

FIGURE.18.CAPACITY SPECTRA FOR STRUCTURE WITH ZONE2 DESIGN

The representation of the two curves in one graph is termed as the Acceleration versus Displacement Response Spectrum (ADRS) format as in Figure 15, 16, 17 and 18. The performance point is the point where the capacity spectrum crosses the demand spectrum. If the performance point exists and the damage state at this point is acceptable, then the building is considered to be adequate for the design earthquake. In the present case study, 4 different capacity spectrums are generated for the same structure (i.e., those designed for Zone2 and Zone3, existing reinforcement, retrofitted structural elements). Hence the capacity spectra of all the 4 models are represented and compared. The status of hinges formed at the performance point is depicted in Table.4. With the performance point in all the 4 cases, we can conclude that in those designed for Zone2 and Zone3 cases, number of hinges in vulnerable damage states formed at performance point are more compared to the other two cases. In structure with existing reinforcement at performance point itself hinges have reached Collapse prevention level, where in other 3 cases no hinges are in collapse prevention level. Very few hinges are formed at immediate occupancy to life safety level in case of structure with Zone3 design and existing reinforcement cases, where no hinges formed in structure with Zone2 design and retrofitted case. In retrofitted structure very less hinges are formed and more hinges are there in elastic region itself. This concludes that retrofitted structure has very few elements that are vulnerable.

7. CONCLUDING REMARKS

- The data obtained in the form of results of analysis for structural elements (Beams and Column) by ETABS is huge. This has to be sorted out systematically so that evaluation of members becomes easier.
- Results of analysis for gravity and earthquake loading obtained in the ETABS are considered as Demand posed on the structure. This demand is compared with the capacity of the elements
- During the linear static analysis of the structure, it is observed that seismic demand of the structural elements increase with the change in the seismic zone and soil type. But the capacity remains unchanged. Hence this demand and capacity of the elements is compared.
- DCR of beam and column in flexure and shear in zone 2 exceeding 1 is less than that in seismic zone 3. This states that elements are more vulnerable to seismic zone3. Hence such column elements are identified and retrofitted using concrete jacketing.
- Before retrofitting almost all columns were failed in ETABS design check. But after retrofitting, all the column elements became safe to the ETABS design check.
- The pushover analysis is a relatively simple way to explore the non linear behavior of the buildings. The results obtained in terms of demand, capacity gave an insight into real behavior of the structure.
- Pushover analysis of casestudy1 building is carried out in 4 different cases. By the comparison of the pushover curve at all incidences we can say that existing structure, which was originally designed for gravity loads only, is more vulnerable to the lateral loads. Hence retrofit is recommended.
- Retrofitted structure is then analyzed using Pushover analysis. It is observed that with the increase in the column section and reinforcement, the base shear capacity and displacement capacity is increased tremendously.
- Pushover analysis also gives status of hinge formation at different level of displacement/base shear. After comparison of the hinge formation at the level of performance point in existing structure, it is found that more hinges have crossed elastic limit than the retrofitted structure. Also some hinges have been observed at collapse level in existing reinforcement structure.
- Comparison of Storey drift of the existing and retrofitted structure shows that structure after retrofit have about 50% less storey drift.
- Hence with all these information it can be concluded that structure after retrofitting the columns only, have shown increased performance for both linear static and non linear static analysis.
- Performance based evaluation of structures gives true picture of element level and global level states of buildings. Pushover analysis can be effectively used in assessing the seismic performance evaluation of buildings.

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