Analytical Investigation on Retrofitting of A Multistoried R.C.C. Framed Building Using Pushover Technique

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Abstract - The scope of present study aims at determining the earthquake load carrying of a building and thereby improving its seismic load carrying capacity by providing certain adequate provisions of retrofitting. The performance based seismic engineering technique known has Non-Linear Static Pushover analysis procedure has been effectively used in this regard. The pushover analysis has been carried out using SAP2000, a product of Computers and Structures International. A total of 28 cases for a particular six storey building located in Zone-IV have been analyzed, considering retrofitting of different structural elements, i.e. Beams and Columns, in different combinations as well as at different storey levels. The retrofitting is started from the bottom most storey and subsequently moving towards the top most storey. The response of the building for each case at each storey level is recorded. The results of analysis are compared in terms of base shear and storey displacements.

Keywords: Performance-based seismic engineering (PBSE), Retrofitting, nonlinear static pushover analysis, Performance level, Finite element analysis, Sap 2000.

I. INTRODUCTION TO PUSHOVER ANALYSIS

In Pushover analysis, a static horizontal force profile, usually proportional to the design force profiles specified in the codes, is applied to the structure. The force profile is then incremented in small steps and the structure is analyzed at each step. As the loads are increased, the building undergoes yielding at a few locations. Every time such yielding takes place, the structural properties are modified approximately to reflect the yielding. The analysis is continued till the structure collapses, or the building reaches certain level of lateral displacement.

A. Need for Pushover Analysis

Conventionally, seismic assessment and design has relied on linear or equivalent linear (with reduced stiffness) analysis of structural systems. In this approach, simple models are used for various components of the structure, which is subjected to seismic forces evaluated from elastic or design spectra, and reduced by force reduction (or behavior) factors. The ensuing displacements are amplified to account for the reduction of applied forces.

B. Description of Pushover Analysis

The non-linear static pushover procedure was originally formulated and suggested by two agencies namely, federal emergency management agency (FEMA) and applied technical council (ATC), under their seismic rehabilitation programs and guidelines. This is included in the documents FEMA-273, FEMA-356 and ATC-40.

Methods and design criteria to achieve several different levels and ranges of seismic performance are defined in FEMA 273. The four Building Performance Levels are Collapse Prevention, Life Safety, Immediate Occupancy, and Operational. These levels are discrete points on a continuous
scale describing the building’s expected performance, or alternatively, how much damage, economic loss, and disruption may occur.[4]

The three Structural Performance Levels and two Structural Performance Ranges consist of:

S-1: Immediate Occupancy Performance Level
S-2: Damage Control Performance Range (extends between Life Safety and Immediate Occupancy Performance Levels)
S-3: Life Safety Performance Level
S-4: Limited Safety Performance Range (extends between Life Safety and Collapse Prevention Performance Levels)
S-5: Collapse Prevention Performance Level

In addition, there is the designation of S-6, Structural Performance Not considered, to cover the situation where only nonstructural improvements are made.

The four Nonstructural Performance Levels are:
N-A: Operational Performance Level
N-B: Immediate Occupancy Performance Level
N-C: Life Safety Performance Level
N-D: Hazards Reduced Performance Level

In addition, there is the designation of N-E, Nonstructural Performance Not Considered, to cover the situation where only structural improvements are made.

According to Gajjar R. K. et al (2002), pushover Analysis results from powerful softwares can be transferred to virtual reality platforms in order to make the outputs more user friendly and easy to understand, besides making it very simple to re-analyze and observe the end results any number of times, till the user is able to grasp the full impact of his final decision. Virtual reality platforms provide a fantastic opportunity as add-on modules to complex analysis software which generally need a high degree of decision and understanding of behaviour of the structure under consideration even prior to modeling it on the desktop. Instant graphical outputs in virtual reality, bring into focus the errors in primary configuration details, in modeling or in designing. The user can therefore afford to make mistakes and correct them at the touch of a few strokes on the keyboard. As the concept is still in its infancy, and as 3D graphics have been hitherto limited to the highly sophisticated domain of movie animation, the computer time and effort required in creating real-life images seem extremely daunting, but are worth the pain if the expense and amount of on-site rehabilitation and on-table interpretation from innumerable tables and numbers, is borne in mind. The concept of VR can then be extended to the web where other stake holders too sitting across the globe can interact and give valuable inputs towards an optimum and robust solution [13].

Chopra et. al (May 2003), laid down the concept of modal pushover analysis (MPA). They analysis six SAC buildings, each analyzed for 20 ground motions, and their statistical analysis leads to bias and dispersion in the procedure. The results demonstrated that by including a few “modes” (typically two or three), the height-wise distribution of demands estimated by MPA is generally similar to the “exact” results from nonlinear response history analysis. The MPA procedure estimates seismic story-drift demands to a degree of accuracy that should be sufficient for most building design and retrofit applications [15].

Jain et. al (August 2002), carried out pushover analysis for seismic retrofitting of buildings for a flat slab building. The various retrofitting techniques used by them included jacketing of columns only, providing additional beams and providing both columns jacketing and additional beams. They concluded that jacketing or retrofitting of columns result in a much higher drift capacity. The additional beams significantly reduce softening caused by sagging hinges. But they have a comparatively lower drift capacity. However jacketing of both beams and columns result into the best response of the system [12].

II. PREVIOUS STUDIES

According to Jong-Wha Bai (August 2002), Seismic retrofitting is an effective method of reducing the risks for existing seismically deficient structures. Numerous intervention techniques are available for improving the seismic behavior of RC building structures. It is important to obtain accurate as-built information and analytical data to perform a seismic evaluation of the existing structure and to select the appropriate retrofitting strategy. A number of experimental and analytical studies focused on seismic retrofitting techniques and extensive seismic damage control activities in practice have contributed to the present state of development. Further research should be conducted to improve the selection of appropriate retrofit techniques using criteria based on performance, economy and constructability [16].
Finally, certain special combinations of retrofitting have been done from the results obtained from the study, so as to get a highly improved response of structure at a relatively cheaper cost.

The following cases (Table I) have been incorporated in the study:

<table>
<thead>
<tr>
<th>SR. NO.</th>
<th>CASE NO.</th>
<th>DESCRIPTION OF CASES</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>Original structure</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Retrofitting beams of 1st storey only</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>Retrofitting columns of 1st storey only</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>Retrofitting beams &amp; columns of 1st storey only</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>Retrofitting beams of 1st +2nd storey only</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>Retrofitting columns of 1st +2nd storey only</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>Retrofitting beams &amp; columns of 1st +2nd storey only</td>
</tr>
<tr>
<td>8</td>
<td>7</td>
<td>Retrofitting beams of 1st +2nd+3rd storey only</td>
</tr>
<tr>
<td>9</td>
<td>8</td>
<td>Retrofitting columns of 1st +2nd+3rd storey only</td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>Retrofitting beams &amp; columns of 1st +2nd+3rd storey only</td>
</tr>
<tr>
<td>11</td>
<td>10</td>
<td>Retrofitting beams of 1st +2nd+3rd+4th storey only</td>
</tr>
<tr>
<td>12</td>
<td>11</td>
<td>Retrofitting columns of 1st +2nd+3rd+4th storey only</td>
</tr>
<tr>
<td>13</td>
<td>12</td>
<td>Retrofitting beams &amp; columns of 1st +2nd+3rd+4th storey only</td>
</tr>
<tr>
<td>14</td>
<td>13</td>
<td>Retrofitting beams of 1st +2nd+3rd+4th+5th storey only</td>
</tr>
<tr>
<td>15</td>
<td>14</td>
<td>Retrofitting columns of 1st +2nd+3rd+4th+5th storey only</td>
</tr>
<tr>
<td>16</td>
<td>15</td>
<td>Retrofitting beams &amp; columns of 1st +2nd+3rd+4th+5th storey only</td>
</tr>
<tr>
<td>17</td>
<td>16</td>
<td>Retrofitting beams of 1st +2nd+3rd+4th+5th +6th storey only</td>
</tr>
<tr>
<td>18</td>
<td>17</td>
<td>Retrofitting columns of 1st +2nd+3rd+4th+5th +6th storey only</td>
</tr>
<tr>
<td>19</td>
<td>18</td>
<td>Retrofitting beams &amp; columns of 1st +2nd+3rd+4th+5th +6th storey only</td>
</tr>
</tbody>
</table>

A. Description of a Building

In the present work, a six storied reinforced concrete frame building situated in Zone IV, is taken for the purpose of study. The plan area of building is 12 x 12 m with 3.0m as height of each typical storey. It consists of 4 bays of 3m each in X-direction and Z-direction (3 x 4= 12m). Hence, the building is symmetrical about both the axis. The total height of the building is 18m. The building is considered as a Special Moment resisting frame. The retrofitting of frame elements, i.e. Beams and columns is done in various combinations at all the storey levels. The plan of building is shown in fig. 3; the front elevation is shown in fig. 4 and 3d view in fig. 5.
B. Sectional Properties of Elements

The sectional properties of elements in case of the original structure are taken as follows:

- Size of Column = 450 x 450mm
- Size of Beam = 0.230 x 300 mm
- Thickness of Slab = 125mm thick

When the structure was retrofitted, the size of columns was increased to 600x600mm, while that of beam was changed to 300x450mm. A nominal percentage i.e. 1% of the increased area can be provided for the retrofitting purposes.

C. Loads Considered

The following loads were considered for the analysis of the building. The loads were taken in accordance with IS:875[1][2].

D. Gravity Loads

The intensity of dead load and live load at various floor levels and roof levels considered in the study are listed below [9].

- **Dead Load**

  At all Floor Levels
  - Weight of Slab: 0.125 x 25 = 3.125 kN/m²
  - Weight of Screed: 0.050 x 20 = 1.000 kN/m²
  - Weight of Floor Finish: 0.025 x 24 = 0.600 kN/m²
  - Weight of partition Wall = 1.000 kN/m²
  - Total Dead Load = 5.725 kN/m²
  - Total Dead Load Taken = 6.0 kN/m²

  A wall load of 12kN/m has been applied to all the outer beams at all the floor levels

- **Live Load**

  - Live load at all floor levels = 3.0 kN/m²

  This live load is reduced by 25% for calculating the seismic weight of the structure as per provisions of IS1893:2002(PART 1).

E. Seismic Loads

The design lateral force due to earthquake is calculated [11] as follows:

- **Seismic Base Shear**

  The design horizontal seismic coefficient Ah for a structure shall be determined by the following expressions:-

  \[ Ah = Z I S a / g \]

  Where:
  - Z = Zone factor
  - I = Importance factor depending upon the functional use of the structure.
  - R = Response reduction factor, depending upon the perceived seismic damage performance of the structure.

  \[ S a / g = \text{Average response acceleration coefficient for rock or soil sites.} \]

  \[ Vh = AhW \]

  Where W is the seismic weight of the building.

Fundamental Natural Time Period

The approximate fundamental natural time period of vibration (Ts) in seconds of a moment resisting frame building without brick infill panels may be estimated by the following empirical expressions:

- **RC framed building**

  \[ TS = 0.075h^{0.75} \]

- **Steel framed building**

  \[ TS = 0.085h^{0.75} \]

  Where h=Height of the building in meters

  For all other buildings, it is given by:

  \[ Tn = 0.09h/\sqrt{d} \]

  Where h=Height of the building in meters
  - d= base dimension of the building at the plinth level, in meters, along the considered direction of the lateral force.

  \[ ]
The design base shear ($V_h$) computed is distributed along the height of the building as below:

$$Q_i = V_h \frac{W_i h_i^2}{\sum W_i h_i^2}$$

Where,

- $Q_i$ = design lateral force at each floor level $i$
- $W_i$ = seismic weight of floor $i$.
- $i$ = height of floor $i$ measured from the base.

Design lateral force

The design lateral force shall first be computed for the building as a whole. The design lateral force shall then be distributed to the various floor levels. The design seismic force thus obtained at each floor level, shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

IV. PUSHOVER ANALYSIS USING SAP2000

The following steps are included in the pushover analysis. Steps 1 to 4 are to create the computer model, step 5 runs the analysis, and steps 6 to 10 review the pushover analysis results.

1. Create the basic computer model (without the pushover data) as shown in Figure 6. The graphical interface of SAP2000 makes this quick and easy task. Assigned sectional properties & applies all the gravity loads i.e. Dead load and Live load on the structure [5].

2. Define properties and acceptance criteria for the pushover hinges as shown in Figure 7. The program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members and average values from FEMA-273 for steel members. In this analysis, PMM hinges have been defined at both the column ends and M3 hinges have been defined at both the ends of all the beams.

3. Locate the pushover hinges on the model by selecting all the frame members and assigning them one or more hinge properties and hinge locations as shown in Figure 8.

4. Define the pushover load cases, figure 9(a) and (b). In SAP2000 more than one pushover load case can be run in the same analysis. Also a pushover load case can start from the final conditions of another pushover load case that was previously run in the same analysis. Typically the first pushover load case was used to apply gravity load and then subsequent lateral pushover load cases were specified to start from the final conditions of the gravity pushover. Pushover load cases can be force controlled, that is, pushed to a certain defined force level, or they can be displacement controlled, that is, pushed to a specified displacement. Typically a gravity load pushover is force controlled and lateral pushovers are displacement controlled. In this case a Gravity load
combination of DL+0.25LL has been used. This combination has been defined as GRAV. The lateral loads have been applied to a case called PUSHPAT.

5. Run the basic static analysis. Then ran the static nonlinear pushover analysis.

6. The Pushover curve was made for control nodes at each storey level. This was done by defining a number of pushover cases in the same analysis, and displacement was monitored for a different node in each case.

7. The pushover curve was obtained as shown in Figure 10. A table was also obtained which gives the coordinates of each step of the pushover curve and summarizes the number of hinges in each state (for example, between IO and LS, or between D and E). This table is shown in Figure 11.

8. The capacity spectrum curve obtained is shown in Figure 12. The magnitude of the earthquake and the damping information on this form can be modified and the new capacity spectrum plot can be obtained immediately. The performance point for a given set of values is defined by the intersection of the capacity curve and the single demand spectrum curve. Also, a table was generated which shows the coordinates of the capacity curve and the demand curve as well as other information used to convert the pushover curve to Acceleration-Displacement Response Spectrum format (also known as ADRS format). See Figure 13.
9. The pushover displaced shape and sequence of hinge information on a step-by-step basis was obtained.

10. Output for the pushover analysis can be printed in a tabular form for the entire model or for selected elements of the model. The types of output available in this form include joint displacements at each step of the pushover, frame member forces at each step of the pushover, and hinge force, displacement and state at each step of the pushover [5].

V. RESULTS AND DISCUSSIONS

A. Base Force

The base force for the six-storey building with different combination of element retrofitting at various floor levels is presented in Table 2. The variation of base force for various cases of retrofitting of building is shown in Figure 2.

It is observed that with retrofitting of beams only, there is a very minimal percentage increase in the base force varying from 11.9% to 26.93%, which the structure can carry. However, with the retrofitting of storey columns, there is quite an appreciable gain in the base force carrying capacity of the structure. The percentage change varies from 15.64% to 98.25%. Further it is observed that, retrofitting of columns at 2nd storey there is a decline in the base force capacity, but after 2nd storey, there is predominant increase in base force due to retrofitting of columns only. The combination of retrofitting of beams and columns both, show a consistent increase in base force capacity but it becomes more predominant from 3rd Storey onwards.

TABLE II. COMPARISON OF BASE SHEAR

<table>
<thead>
<tr>
<th>RETROFITTING LEVEL</th>
<th>CASES</th>
<th>INCREASE IN NO. OF ITERATIONS</th>
<th>BASE SHEAR (KN)</th>
<th>PERCENTAGE INCREASE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original structure</td>
<td>4</td>
<td>3049.4314</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RETROFITTING UPTO 1st STOREY</td>
<td>CASE 1</td>
<td>5</td>
<td>3415.1372</td>
<td>11.9</td>
</tr>
<tr>
<td></td>
<td>CASE 2</td>
<td>6</td>
<td>3722.8994</td>
<td>22.08</td>
</tr>
<tr>
<td></td>
<td>CASE 3</td>
<td>5</td>
<td>3763.8350</td>
<td>23.42</td>
</tr>
<tr>
<td>RETROFITTING UPTO 2nd STOREY</td>
<td>CASE 4</td>
<td>7</td>
<td>3800.3967</td>
<td>24.62</td>
</tr>
<tr>
<td></td>
<td>CASE 5</td>
<td>4</td>
<td>3526.5369</td>
<td>15.64</td>
</tr>
<tr>
<td></td>
<td>CASE 6</td>
<td>6</td>
<td>3543.8384</td>
<td>16.21</td>
</tr>
<tr>
<td>RETROFITTING UPTO 3rd STOREY</td>
<td>CASE 7</td>
<td>5</td>
<td>3588.6655</td>
<td>17.68</td>
</tr>
<tr>
<td></td>
<td>CASE 8</td>
<td>4</td>
<td>3689.0637</td>
<td>20.97</td>
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<td></td>
<td>CASE 9</td>
<td>4</td>
<td>3679.8408</td>
<td>20.67</td>
</tr>
<tr>
<td>RETROFITTING UPTO 4th STOREY</td>
<td>CASE 10</td>
<td>5</td>
<td>3646.4312</td>
<td>19.57</td>
</tr>
<tr>
<td></td>
<td>CASE 11</td>
<td>8</td>
<td>3848.0723</td>
<td>26.18</td>
</tr>
<tr>
<td></td>
<td>CASE 12</td>
<td>8</td>
<td>5204.1719</td>
<td>70.66</td>
</tr>
<tr>
<td>RETROFITTING UPTO 5th STOREY</td>
<td>CASE 13</td>
<td>6</td>
<td>3870.7119</td>
<td>26.93</td>
</tr>
<tr>
<td></td>
<td>CASE 14</td>
<td>6</td>
<td>5983.6665</td>
<td>96.22</td>
</tr>
<tr>
<td></td>
<td>CASE 15</td>
<td>8</td>
<td>6493.8042</td>
<td>112.95</td>
</tr>
<tr>
<td>RETROFITTING UPTO 6th STOREY</td>
<td>CASE 16</td>
<td>6</td>
<td>3835.5139</td>
<td>25.77</td>
</tr>
<tr>
<td></td>
<td>CASE 17</td>
<td>6</td>
<td>6045.7153</td>
<td>98.25</td>
</tr>
<tr>
<td></td>
<td>CASE 18</td>
<td>8</td>
<td>6533.5293</td>
<td>114.25</td>
</tr>
</tbody>
</table>
B. Roof Displacement

The Roof displacement for the six-storey building with different combination of element retrofitting at various floor levels is presented in Table 3. The variation of Roof displacement for various cases of retrofitting of building is shown in Figure 3.

It is observed that with retrofitting of beams only, there is a decrease in the roof displacement upto 4th storey and after 4th storey it got little increased upto 5th storey (31.16% to 8.05%) and after 5th storey it again decreases to (14.23%). This percentage varies from 31.16% to 8.05%. However, the trends shown by retrofitting of columns only is there is a decrease in the roof displacement upto 2nd storey and after 2nd storey it predominantly increases upto 5th storey and again decreases slightly at 6th storey. The percentage change varies from -8.59% to 102.61%. The combination of retrofitting of beams and columns both, show a consistent decrease in the roof displacement upto 3rd storey and after 3rd storey it predominantly increases upto 5th storey and again decreases slightly at 6th storey.

**TABLE III. COMPARISON OF ROOF DISPLACEMENT**

<table>
<thead>
<tr>
<th>RETROFITTING LEVEL</th>
<th>CASE</th>
<th>INCREASE IN NO. OF ITERATIONS</th>
<th>ROOF DISPLACEMENT (mm)</th>
<th>PERCENTAGE INCREASE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original structure</td>
<td>4</td>
<td>148.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RETROFITTING UPTO 1st STOREY</td>
<td>CASE 1</td>
<td>5</td>
<td>169.6</td>
<td>13.90</td>
</tr>
<tr>
<td></td>
<td>CASE 2</td>
<td>6</td>
<td>168.0</td>
<td>12.82</td>
</tr>
<tr>
<td></td>
<td>CASE 3</td>
<td>5</td>
<td>169.6</td>
<td>13.90</td>
</tr>
<tr>
<td>RETROFITTING UPTO 2nd STOREY</td>
<td>CASE 4</td>
<td>7</td>
<td>195.3</td>
<td>31.16</td>
</tr>
<tr>
<td></td>
<td>CASE 5</td>
<td>4</td>
<td>136.1</td>
<td>-8.59</td>
</tr>
<tr>
<td></td>
<td>CASE 6</td>
<td>6</td>
<td>124.4</td>
<td>-16.45</td>
</tr>
<tr>
<td>RETROFITTING UPTO 3rd STOREY</td>
<td>CASE 7</td>
<td>5</td>
<td>163.3</td>
<td>9.67</td>
</tr>
</tbody>
</table>

VI. CONCLUSIONS

Based on the present study, the following conclusions can be drawn:

1. There is a minimal increase in the base shear due to retrofitting of beams only. An increase of only 11.9% to 26.93% is observed when the beams are retrofitted.
2. The retrofitting of columns results into an appreciable gain in base shear. This increase varies from 15.64% to 98.25%. The maximum increase is for the case when all the columns are retrofitted upto 6th storey only.
3. The retrofitting of both beams and columns gives an appreciable increase in the base shear of the structure. This range varies from 16.21% to 114.25%. The maximum value of 114.25% increase is obtained when all the beams as well as columns are retrofitted upto the 6th storey.
4. The retrofitting of beams results into a decrease in the roof displacement up to 4th storey and again got increased at 5th storey and again further decreases at 6th storey of the structure. This decrease varies from 31.16% to 8.05%.

5. The retrofitting of columns results into an appreciable decrease in the maximum roof displacement up to 2nd storey and suddenly appreciable increase is seen at 5th storey and again further slightly decreases at 6th storey which the structure can carry without failure. This decrease varies from 102.61% to -8.59%. The maximum roof displacement is observed for the case, when all the columns have been retrofitted up to 5th storey.

6. The retrofitting of both beams and columns in different combinations cause a decrease of 13.90% to -22.16% in roof displacement up to 3rd storey and further increases up to 5th storey from -22.16% to 96.17% in roof displacement and again slightly decreases at 6th storey.

REFERENCES