



## ARTICLE

# Seismic Response Modification Factor of Reinforced Concrete Frames Based on Pushover Analysis

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### ABSTRACT

Response modification factor is an essential factor in seismic analysis to provide economic design of reinforced concrete structures. Base shear force is divided by the response modification factor to consider the ability of the structure to dissipate energy through plastic hinges. The current study investigates the effects of changing some parameters on response modification factor (R-factor). Four groups of reinforced concrete frames were studied with different number of bays, number of stories, load pattern, and fundamental period of vibration. All reinforced concrete frames were analyzed using SAP 2000 then the straining actions results were used at specific excel sheets which are developed to design reinforced concrete members according to the Egyptian code of practice ECP-203 and ECP-201. Frames were analyzed by nonlinear static analysis (pushover analysis) using SAP2000. A sum of thirty two systems of frames was analyzed. According to the results, every frame has its unique value of R-factor. Accordingly, many parameters should be mentioned and considered at code to simulate the actual value of R-factor for each frame. Response modification factor is affected by many factors like stiffness, fundamental period of vibration, number of bays, frame height, geometry of the structure, etc. The given values of R-factor at ECP-201 can be considered conservative; as the accurate values of R-factor is higher than the given values.

## 1. Introduction

Preserving the structure in elastic zone prohibits forming permanent deformations in structural and nonstructural elements which requires massive di-

mensions of structural elements. Due to limited financial and human resources of construction industry; limited damage at structures is acceptable after earthquakes to balance between economic and safe designs. Current codes for seismic design of reinforced concrete structures

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use a factor called response modification factor to reduce the actual lateral load that can be resisted by the structures to consider the permanent deformations at the structures. Egyptian Code uses a list of values for response modification factor “R-factor” for reinforced concrete frames, and there is a different value for R according to lateral resisting system of the structure. The code doesn’t consider differences between frames such as end conditions of columns, stiffness of frame, fundamental period of vibration, number of stories, number of bays, bay width and intensity of loads. Consequently, R-factor values should be re-evaluated in Egyptian Code ECP-201 [1] to enhance seismic design for reinforced concrete structures.

## 2. Definition of Response Modification Factor

Mwafy and Elnashai [2] concluded that R-factor can be defined as the ductility reduction factor ( $R_\mu$ ) multiplied by overstrength factor ( $R_s$ ) as follows:

$$R = R_\mu \cdot R_s \quad (1)$$

## 3. Relation between Ductility Factor and Ductility Ratio

Displacement ductility ratio “ $\mu$ ” (ductility demand) concept is the extent of inelastic deformation experienced by the structural system subjected to a given ground motion or a lateral loading. The ratio of maximum displacement to the yield displacement is the displacement ductility ratio as follows:

$$\mu = \Delta_{max} / \Delta_y \quad (2)$$

The ductility reduction factor ( $R_\mu$ ) represents the ability of a structure to dissipate hysteretic energy by forming of plastic deformations. It can be defined as the reduction in strength demand due to nonlinear hysteretic behavior.

The ductility reduction factor is the ratio between lateral yielding strength required to maintain the system elastic to yielding strength required to maintain the displacement ductility demand  $\mu$  less than or equal to a predetermined target ductility ratio  $m_i$  as follows :

$$R_\mu = F_y (\mu=1) / F_y (\mu=\mu_i) \quad (3)$$

Where:

$\mu$ : displacement ductility demand.

$F_y (\mu=1)$ : lateral yielding strength required to maintain the system elastic.

$F_y (\mu=\mu_i)$ : lateral yielding strength required to maintain the displacement ductility demand  $\mu$  less than or equal to a predetermined target ductility ratio  $m_i$ .

Newmark and Hall [3] presented a study based on the elastic and the inelastic response spectra of El Centro earthquake and two other recorded ground motions. In their study they concluded that ductility reduction factor can be calculated using the following equations from (4) to (6):

$$R_\mu = 1 \quad \text{when} \quad T < 0.03 \text{ sec} \quad (4)$$

$$R_\mu = \sqrt{2\mu - 1} \quad \text{when} \quad 0.12 \text{ sec} < T < 0.5 \text{ sec} \quad (5)$$

$$R_\mu = \mu \quad \text{when} \quad T > 1 \text{ sec} \quad (6)$$

Ductility reduction factor was presented graphically as follows:

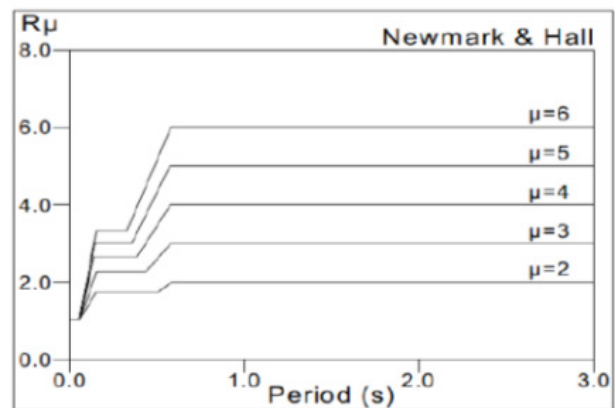


Figure 1. Ductility Reduction Factor versus period (Newmark and Hall) [3]

## 4. Overstrength Factor( $R_s$ )

The overstrength factor ( $R_s$ ) can be defined as the ratio of yield force to its design force as follows:

$$R_s = V_y / V_d \quad (9)$$

Overstrength factor accounts for the fact that the maximum lateral strength of a structure generally exceeds its design strength.

Many researchers have investigated the sources of the overstrength and they concluded that the main sources of the overstrength are: redundancy of the structure, limits of story drift, strain hardening, effect of non-structural elements, higher material strength than used material in the design, member oversize, minimum requirements regarding proportioning and detailing and slab participation. Some of these factors cannot be considered clearly in the design of new buildings because their contribution is probable. Furthermore, other factors are difficult to be calculated because of the complexity of the behavior such

as non-structural elements.

Blume [4] showed the importance of overstrength in the behavior of structures during earthquakes. He investigated many factors that contribute to the actual performance during earthquakes.

Uang [5] and Assaf [6] concluded that as the number of stories increase, the overstrength decreases because that in case of buildings with few numbers of stories, the design is likely to be governed by gravity loads.

Jain and Navin [7] Miranda and Bertero [8] studied low rise buildings in Mexico City. They concluded that the value of the overstrength ranges from 2 to 5. Moreover, this value is significantly higher if the slab contribution and the masonry distribution are considered.

Jain and Navin [7] investigated seismic overstrength of multistory reinforced concrete frames using nonlinear pseudo static analysis on four-bays with three-, six-, and nine-story frames designed for seismic zones I to V as per Indian codes. The study concluded that the overstrength varies with seismic zone, number of stories, and with design gravity loads. The dependence on seismic zone is the strongest. The average overstrength of the studied frames in zones V and I is 2.84 and 12.7, respectively. The overstrength increases as the number of stories decreases; overstrength of the three-story frame is higher than the nine-story frame by 36% in zone V and 49% in zone I. Furthermore, interior frames have 17% (zone V) to 47% (zone I) higher overstrength as compared to the exterior frames of the same building.

Kappos [9] investigated the overstrength factor by both pushover analysis and dynamic time history analysis of a series of low-rise and medium-rise buildings representing different structural systems. These structures were designed according to the provisions of the EC8 [10]. The study concluded that the overstrength was higher in the case of low-rise structures; values up to 2.7 were calculated, compared to about 1.5 for medium and high rise structures.

### 5. Nonlinear Static Pushover Analysis:

Nonlinear static analysis (NSA) or pushover analysis is a popular tool for seismic performance evaluation and can be considered as a reasonable alternative for nonlinear dynamic analysis (NLDA). Pushover analysis has become a familiar procedure in current structural engineering practice because of its reasonable results and simplicity compared to NLDA. Pushover analysis procedure depends on applying monotonically increasing lateral forces to the structure until the structure reaches a target displacement. FEMA-356 [11] recommends using at least two different load patterns and envelope the results.

### 6. Verification of Response Modification Factor(R) Based on Pushover Analysis

Before starting the required case studies the results of response modification factor was verified by previous published manuscript by Amar Louzai and Ahmed Abed [12] to ensure that the current study procedure is correct regardless the difference at used codes . Three stories of reinforced concrete frame is studied according to Algerian seismic code RPA 99/Version 2003 [13]. Non-linear static pushover analysis using inverted triangular loading pattern was carried out to compute the R factor components, such as ductility and over strength factors, with the consideration of failure criteria at both member and structural levels.

A two-dimensional model of each frame structure is created in SAP2000 to carry out pushover analysis. Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns, where bilinear moment-rotation relationships are assigned. Plan and elevation of studied frame is shown in following figures from 2 to 3.

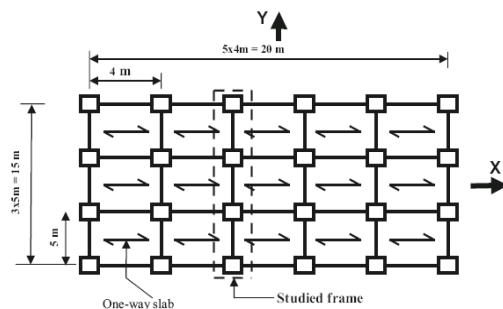


Figure 2. Plan of building containing 3 stories frame

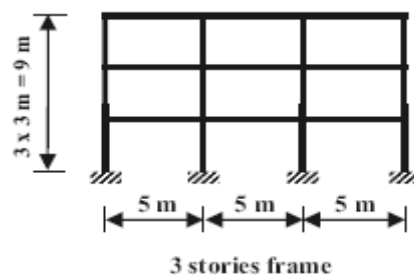


Figure 3. Elevation of 3 stories studied frame

The assigned loads on the frame are as following:

- (1) Dead load on the roof = 0.51 t/m<sup>2</sup>
- (2) Live load on the roof = 0.25 t/m<sup>2</sup>
- (3) Dead load on the typical story = 0.58 t/m<sup>2</sup>
- (4) Live load on the typical story = 0.10 t/m<sup>2</sup>

The frames were designed according to Algerian seismic code RPA 99/version2003 [13] with the following parameters: zone of high seismicity, zone III, importance class group 2, soil type S3 (soft soil), quality factor Q = 1

and viscous damping ratio  $\xi = 6\%$ . The analysis will be performed for the zone acceleration factor  $A = 0.25$ .

A Seismic behavior factor of  $R = 5$  was taken into account for reinforced concrete frames without masonry infill.

The member cross-section sizes and steel bars are given in the following table:

**Table 1.** cross section dimensions and reinforcement

| Storey level              | Beams           |  | Columns         |  |
|---------------------------|-----------------|--|-----------------|--|
|                           | Cross section   |  | Cross section   |  |
|                           | b × h (cm × cm) |  | a × a (cm × cm) |  |
| <i>Three-storey frame</i> |                 |  |                 |  |
| 1                         | 30 × 40         |  | 40 × 40         |  |
| 2                         | 30 × 40         |  | 40 × 40         |  |
| 3                         | 30 × 40         |  | 40 × 40         |  |

**6.1 Failure Criteria**

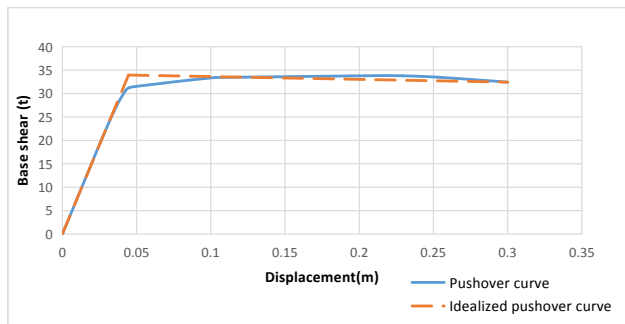
The adopted global failure criterion is:

An upper limit of the inter-story drift ( $\Delta$ ) equals to 3% of the story height ( $h_c$ ). This limit is also specified in Mwafy and Elnashai<sup>[2]</sup>, and close to those adopted by seismic design code UBC 97<sup>[14]</sup>.

**6.2 Results**

The results obtained by me and the results obtained by Amar louzai and Ahmed Abed<sup>[12]</sup> is illustrated in table 2 and pushover curve by me is shown in figure 4:

**Pushover curve:**



**Figure 4.** Pushover curve of 3 stories frame under study

**Table 2.** comparison between A.louzai results and results done by me to ensure that my used procedure is correct

| Parameter                       | Amar Louzai and Ahmed Abed study | Results by me |
|---------------------------------|----------------------------------|---------------|
| Ductility factor ( $\mu$ )      | 1.72                             | 1.70          |
| Overstrength factor( $\Omega$ ) | 2.53                             | 2.83          |
| Response modification factor    | 4.35                             | 4.81          |

**7. Description of The Case Study**

Ordinary reinforced concrete frames with different conditions of no. of bays, no. of stories; lateral load pattern and fundamental period of vibration are divided into four groups:

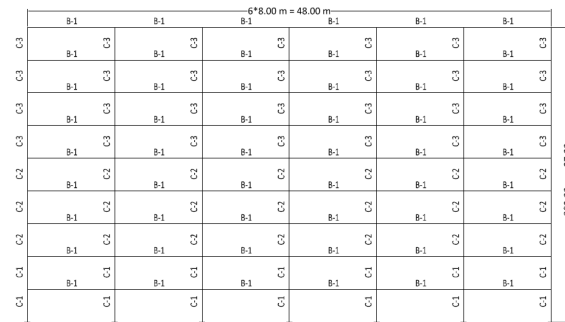
(1) 3, 5, 7, 9 stories frames with six bays, 8 m bay width, 3 m story height and fundamental period of vibration is calculated according to Egyptian code of practice empirical formula.

(2) 3, 5, 7, 9 stories frames with six bays, 8 m bay width, 3 m story height and fundamental period of vibration is considered according to SAP 2000<sup>[15]</sup> results.

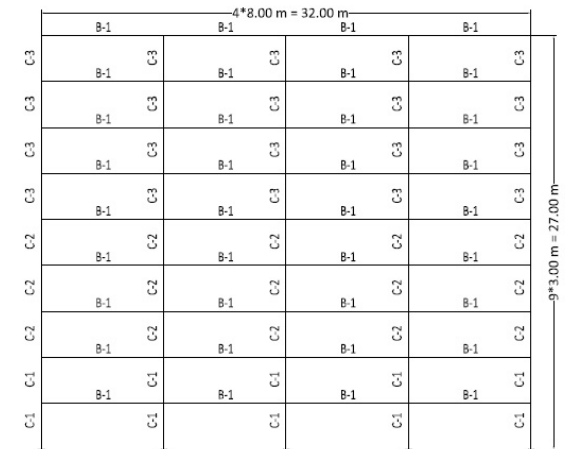
(3) 3, 5, 7, 9 stories frames with four bays, 8 m bay width, 3 m story height and fundamental period of vibration is calculated according to Egyptian code of practice empirical formula.

(4) 3, 5, 7, 9 stories frames with four bays, 8 m bay width, 3 m story height and fundamental period of vibration is calculated according to SAP 2000 results.

For example the following figure 2 and 3 shows the layout of 6 bays frame with 9 stories and 4 bays frame with 9 stories. It worth to mention that 4 bays frames have the same geometry of 6 bays frames but the only difference is the number of bays.



**Figure 5.** Layout of 9-story frame-6 bays



**Figure 6.** Layout of 9-story frame-4 bays

General plan for the frame under study is shown in figure 3:

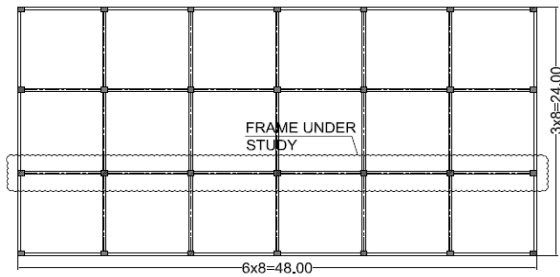


Figure7. General plan for the frame

Columns details shall be shown in the following table 3:

Table 3. Column reinforcement of frames

| Frame type      | Story no. | Column type | Column dimensions |
|-----------------|-----------|-------------|-------------------|
|                 |           |             | (m)               |
| 3 stories frame | 1,2       | C-1         | 0.50x0.50         |
|                 | 3         | C-2         | 0.45x0.45         |
| 5 stories frame | 1,2       | C-1         | 0.50x0.50         |
|                 | 3,4,5     | C-2         | 0.45x0.45         |
| 7 stories frame | 1,2       | C-1         | 0.60X0.60         |
|                 | 3,4,5     | C-2         | 0.50x0.50         |
|                 | 6,7       | C-3         | 0.45x0.45         |
| 9 stories frame | 1,2       | C-1         | 0.60X0.60         |
|                 | 3,4,5     | C-2         | 0.50x0.50         |
|                 | 6,7,8,9   | C-3         | 0.45x0.45         |

All beams type is (B-1) and have dimensions 0.30x0.70 m

### 8. Design Codes

All the frames considered in this study are designed according to the provisions of the Egyptian code ECP-203 [16]. Besides, Gravity and lateral loads are calculated according to the Egyptian code for calculation of loads for structures ECP-201 [1]. The following load values are assigned to the structures: Own weight of 14 cm slab = 0.35 t/m<sup>2</sup>, Flooring cover = 0.15 t/m<sup>2</sup>, Walls including plaster = 0.20 t/m<sup>2</sup>, Live load = 0.20 t/m<sup>2</sup>

### 9. Pushover Analysis Procedure

As cited by Amira [17], Procedure of pushover analysis can be summarized as following:

- (1) Create the model of the structure to represent all important elements of the building.
- (2) Assign gravity loads to the structure to represent all dead and live loads.
- (3) Assign appropriate lateral load pattern to represent earthquake load on the building.
- (4) Assign plastic hinges positions and properties along beams and columns according to FEMA 356.

- (5) Choose target displacement or force control to be maintained.
- (6) Push the structure to the predetermined target displacement or force control.
- (7) Plot base shear versus top displacement to get push-over curve.

### 10. Failure Criterion

The chosen failure criterion at this study is according to UBC 97 [14]. Maximum story drift shall be 0.025 times the story height for structures having a fundamental period of vibration less than 0.70 second. For structures having a fundamental period of vibration 0.70 second or greater, the maximum story drift shall be 0.020 times the story height.

### 11. Results and Discussion

The results of pushover curve of triangular pattern for 6 bays frames only are shown at the following figures from figure (5) to (8), but both of uniform pattern and triangular pattern results are listed in table of results of each group:

#### 11.1 6 Bays Frame (Fundamental Period of Vibration is Calculated Using Empirical Formula of ECP-201)

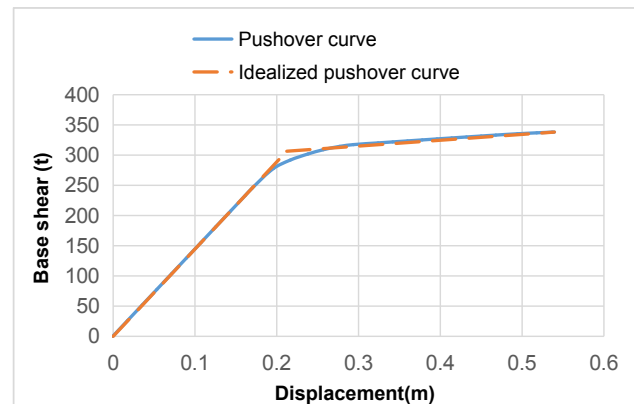


Figure 8. Pushover curve for 9 stories-6 bays frame using triangular pattern-T according to ECP-201

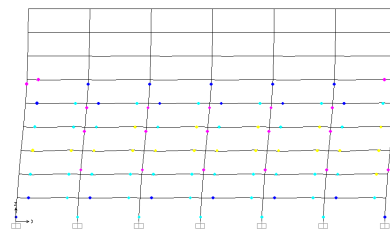


Figure 9. Plastic hinge formation at failure for 9 stories-6 bays frame using triangular pattern-T according to ECP-201

Where :

(1) Point B represents the yield. Before this point, all deformations are linear and occur in the frame element, not the hinge. Only plastic deformation after this point is displayed by the hinge.

(2) Point C represents the ultimate capacity of the hinge.

(3) Point D represents the residual strength of the hinge.

(4) Point E represents total failure of the hinge. After this point the hinge will drop the load to zero.

(5) The point IO on the above curve refers to immediate occupancy, which means that the post-earthquake damage is very limited and the building is safe to occupy.

(6) The point LS refers to life safety, which means that the post-earthquake damage is significant but it is possible to repair the structure. However, the repair is not practical for economic reasons.

(7) The point CP refers to collapse prevention, which means that the building is very close to partial or total collapse and is not safe for re-occupancy.

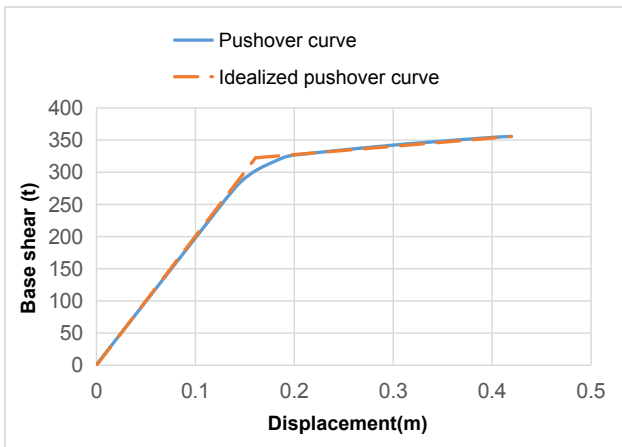


Figure 10. Pushover curve for 7 stories-6 bays frame using triangular pattern-T according to ECP-201

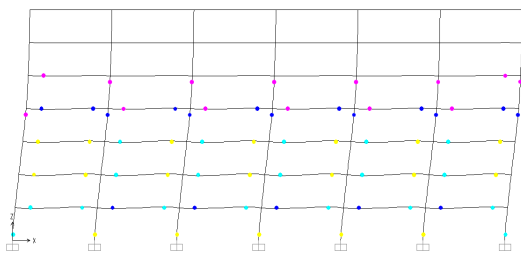


Figure 11. Plastic hinge formation at failure for 7 stories-6 bays frame using triangular pattern-T according to ECP-201

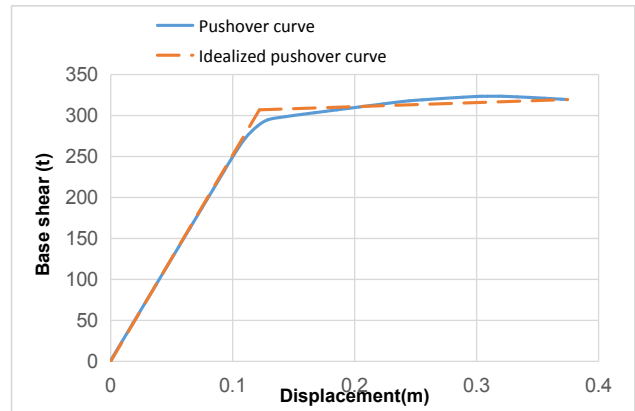


Figure 12. Pushover curve for 5 stories-6 bays frame using triangular pattern-T according to ECP-201

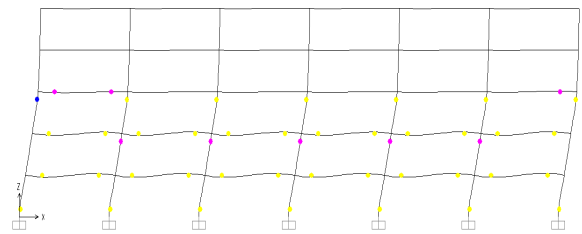


Figure 13. Plastic hinge formation at failure for 5 stories-6 bays frame using triangular pattern-T according to ECP-201

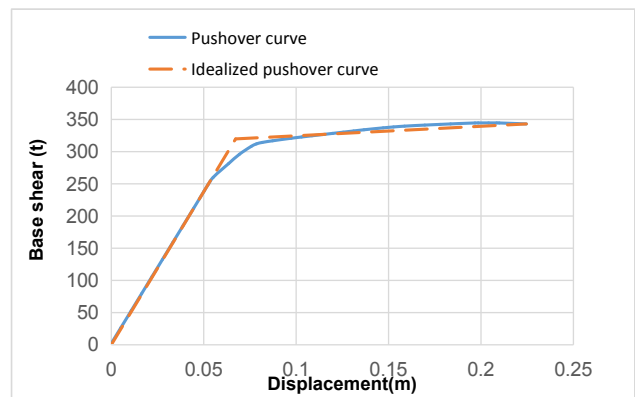


Figure 14. Pushover curve for 3 stories-6 bays frame using triangular pattern-T according to ECP-201

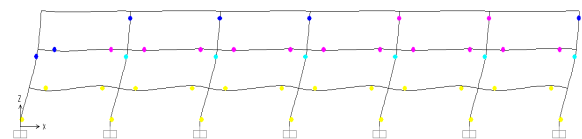


Figure 15. Plastic hinge formation at failure for 3 stories-6 bays frame using triangular pattern-T according to ECP-201

Values of R factor for 6 bays frames (fundamental period of vibration is calculated using empirical formula of ECP-201) are shown in the following table :

**11.2 6 Bays Frames (Fundamental Period of Vibration is Calculated Using Modal Analysis of SAP 2000)**

Values of R factor for 6 bays frames (fundamental period of vibration is calculated using modal analysis of SAP 2000) are shown in the following table 5:

**11.3 4 Bays Frames (Fundamental Period of Vibration is Calculated Using Empirical Formula of ECP-201)**

Values of R factor for 4 bays frames (fundamental period of vibration is calculated using empirical formula of ECP-201) are shown in table 6:

**11.4 4 Bays Frames (Fundamental Period of Vibration is Calculated Using SAP 2000)**

Values of R factor for 4 bays frames (fundamental pe-

**Table 4.** R factor- 6 bays- T according to ECP 201

| No. of stories | Type of load pattern | T (Second) | $\mu$ | $\Delta_y$ (m) | $V_y$ (t) | $R_\mu$ | $R_s$ | R     | Chosen R | (R) according to code |
|----------------|----------------------|------------|-------|----------------|-----------|---------|-------|-------|----------|-----------------------|
| 3              | Triangular           | 0.39       | 3.34  | 0.067          | 319.77    | 2.17    | 5.34  | 11.60 | 11.60    | 5                     |
|                | Uniform              | 0.39       | 3.69  | 0.061          | 348.32    | 2.27    | 5.81  | 13.20 |          |                       |
| 5              | Triangular           | 0.57       | 3.125 | 0.12           | 306.96    | 2.38    | 3.84  | 9.14  | 9.14     |                       |
|                | Uniform              | 0.57       | 3.41  | 0.11           | 348.87    | 2.55    | 4.37  | 11.14 |          |                       |
| 7              | Triangular           | 0.74       | 2.625 | 0.16           | 322.48    | 2.32    | 3.68  | 8.54  | 8.54     |                       |
|                | Uniform              | 0.74       | 2.80  | 0.15           | 385.78    | 2.51    | 4.41  | 11.04 |          |                       |
| 9              | Triangular           | 0.89       | 2.54  | 0.21           | 306.45    | 2.43    | 3.29  | 7.99  | 7.99     |                       |
|                | Uniform              | 0.89       | 2.80  | 0.19           | 374.93    | 2.80    | 4.03  | 10.71 |          |                       |

**Table 5.** R factor- 6 bays- T according to SAP 2000

| No. of stories | Type of Load pattern | T (Second) | $\mu$ | $\Delta_y$ (m) | $V_y$ (t) | $R_\mu$ | $R_s$ | R     | Chosen R | (R) according to Code |
|----------------|----------------------|------------|-------|----------------|-----------|---------|-------|-------|----------|-----------------------|
| 3              | Triangular           | 0.24       | 3.34  | 0.067          | 319.77    | 1.89    | 5.34  | 10.07 | 10.07    | 5                     |
|                | Uniform              | 0.24       | 3.69  | 0.061          | 348.32    | 1.92    | 3.73  | 11.17 |          |                       |
| 5              | Triangular           | 0.42       | 3.04  | 0.123          | 308.95    | 2.13    | 3.34  | 7.12  | 7.12     |                       |
|                | Uniform              | 0.42       | 3.41  | 0.109          | 348.87    | 2.25    | 3.77  | 8.50  |          |                       |
| 7              | Triangular           | 0.55       | 3.12  | 0.17           | 335.72    | 2.37    | 2.87  | 6.80  | 6.80     |                       |
|                | Uniform              | 0.55       | 3.48  | 0.15           | 400.42    | 2.55    | 3.42  | 8.70  |          |                       |
| 9              | Triangular           | 0.72       | 2.54  | 0.213          | 307.11    | 2.25    | 2.68  | 6.01  | 6.01     |                       |
|                | Uniform              | 0.72       | 2.56  | 0.193          | 374.93    | 2.33    | 3.27  | 7.60  |          |                       |

**Table 6.** R factor- 4 bays- T according to ECP 201

| No. of stories | Type of Load pattern | T (Second) | $\mu$ | $\Delta_y$ (m) | $V_y$ (t) | $R_\mu$ | $R_s$ | R     | Chosen R | (R) according to code |
|----------------|----------------------|------------|-------|----------------|-----------|---------|-------|-------|----------|-----------------------|
| 3              | Triangular           | 0.39       | 3.18  | 0.071          | 214.31    | 2.12    | 5.35  | 11.37 | 11.37    | 5                     |
|                | Uniform              | 0.39       | 3.48  | 0.065          | 237.31    | 2.21    | 5.93  | 13.11 |          |                       |
| 5              | Triangular           | 0.57       | 2.97  | 0.126          | 208.59    | 2.33    | 3.83  | 8.93  | 8.93     |                       |
|                | Uniform              | 0.57       | 3.37  | 0.111          | 234.72    | 2.53    | 4.40  | 11.13 |          |                       |
| 7              | Triangular           | 0.74       | 2.53  | 0.166          | 217.14    | 2.26    | 3.71  | 8.38  | 8.38     |                       |
|                | Uniform              | 0.74       | 2.83  | 0.148          | 260.55    | 2.48    | 4.45  | 11.04 |          |                       |
| 9              | Triangular           | 0.89       | 2.55  | 0.21           | 206.15    | 2.43    | 3.31  | 8.07  | 8.07     |                       |
|                | Uniform              | 0.89       | 2.80  | 0.19           | 253.02    | 2.66    | 4.07  | 10.82 |          |                       |

**Table 7.** R factor- 4 bays- T according to SAP 2000

| No. of stories | Type of Load pattern | T (Second) | $\mu$ | $\Delta_y$ (m) | $V_y$ (t) | $R_p$ | $R_s$ | $R$  | Chosen R | (R) according to code |
|----------------|----------------------|------------|-------|----------------|-----------|-------|-------|------|----------|-----------------------|
| 3              | Triangular           | 0.26       | 3.12  | 0.08           | 214.48    | 2.17  | 3.57  | 7.75 | 7.75     | 5                     |
|                | Uniform              | 0.26       | 3.18  | 0.06           | 237.30    | 2.44  | 3.95  | 9.65 |          |                       |
| 5              | Triangular           | 0.45       | 2.59  | 0.14           | 205.45    | 1.99  | 3.56  | 7.09 | 7.09     |                       |
|                | Uniform              | 0.45       | 2.81  | 0.13           | 247.63    | 2.08  | 4.29  | 8.94 |          |                       |
| 7              | Triangular           | 0.55       | 3.11  | 0.17           | 225.64    | 2.37  | 2.88  | 6.82 | 6.82     |                       |
|                | Uniform              | 0.55       | 3.44  | 0.15           | 270.53    | 2.53  | 3.45  | 8.73 |          |                       |
| 9              | Triangular           | 0.72       | 2.55  | 0.21           | 206.54    | 2.25  | 2.69  | 6.06 | 6.06     |                       |
|                | Uniform              | 0.72       | 3.06  | 0.18           | 254.38    | 2.61  | 3.31  | 8.66 |          |                       |

riod of vibration is calculated using SAP 2000) are shown in table 7:

## 12. Conclusions

In this section, results of the analysis and evaluations of this study are summarized as following:

(1) Response modification factor is affected by many factors like stiffness, fundamental period of vibration, number of bays, frame height, geometry of the structure, etc.

(2) The given values of R-factor at ECP-201 are conservative values; as the accurate values of R-factor is higher than the given values.

(3) Every frame has its unique value of R-factor. Accordingly; many parameters should be mentioned at code to simulate the actual value of R-factor for each frame.

(4) R-factor values increase when number of stories decrease.

(5) R-factor values decrease when fundamental period of vibration increases.

(6) R-factor values decrease when number of bays decrease.

(7) Changing lateral load pattern has an obvious effect on response modification factor.

(8) During pushover analysis, triangular lateral load pattern gives lower values of R-factor; consequently the more conservative values for R-factor.

(9) Finite element software such as SAP 2000 can perform pushover analysis precisely which can be used to calculate R-factor.

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