RESPONSE MODIFICATION FACTOR AND SEISMIC BEHAVIOR OF BASE-ISOLATED RC STRUCTURES

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Abstract - In this paper, one of the significant seismic design parameter as response modification factor in reinforced concrete (RC) buildings with base isolation system was evaluated. The seismic isolation system is a capable approach to absorbing seismic energy at the base and transfer to the substructure with lower response modification factor as compared to non-isolated structures. A response spectrum method and static nonlinear pushover analysis in according to Uniform Building Code (UBC-97), have been performed on building models involve 5, 8, 12, and 15-stories with fixed and isolated bases consist of identical moment resisting configurations. The isolation system is composed of lead rubber bearing (LRB) was designed with help UBC-97 parameters. The force-deformation behavior of isolators was modeled as Bi-linear hysteretic behavior which can be effectively used to create the isolation systems. The obtained analytical results highlight the response modification factor of considered base isolation system with higher values than recommended in the codes. The response modification factor is used in modern seismic codes to scale down the elastic response of structures.

Keywords - Response modification factor; Base isolation system; Pushover analysis; Lead rubber bearing; Bi-linear hysteretic.

I. INTRODUCTION

Base isolation system has been widely used to reduce the seismic force and protect against earthquake damage, and it is moving toward a very efficient tool in the seismic design of the structure. Increasing flexibility of structure is well achieved by the insertion of these additional elements between upper structure and foundation as they absorb the larger part of the seismic energy. For this purpose, several structures have been built with this system all around the world.

Implementation of base isolation system generates large fundamental period that causes the displacement during the earthquake. The displacement is proportional to the period, and the acceleration is inversely proportional to the period. An infinite and minimum value of period, depend on the flexible and rigid concept of structures. In the flexible structures with increasing the displacement and period, response acceleration reduces, but in rigid structures with zero displacements, acceleration increase [1], [2]. The earthquake causes inertia forces proportional to the product of the building mass and the earthquake ground accelerations. As the ground accelerations increase, the strength and the capacity of the building must be increased to avoid structural damage. The codes attempt to conduct the structures toward the flexibility using response modification factor (R). Ductility is one of the important factors to assess response modification factor. Ductility is a concept of allowing the structural elements to deform beyond their elastic limit in a controlled manner. Beyond this limit, the structural elements soften and the displacements increase with only a small increase in force. However, structures do not remain elastic during the earthquake and moreover, they are expected to undergo large nonlinear deformation [3], [4]. In design concept, codes allow reducing the inertia forces by response modification factor that accounts for ductility in the structure. The design procedure for the base-isolated structures using the dynamic method is based on the recommendations provided by the uniform building code (UBC-97)[5]. The numerical coefficient, R, which represents global ductility in the structure is different for isolated and non-isolated structures. Response modification factor in ATC-19 [6] and ATC-34 [7] has been split into three parts: over-strength factor, redundancy factor, and ductility factor. The behavior of response modification factor of non-isolated structures with lateral load resisting system of concentric and eccentric braced frames as CBF and EBF was investigated by various researchers. Finding values for factors such as ductility, over-strength, force reduction due to ductility and response modification factor by Mussa Mahmoudi and Mahdi Zaree [8], showed that response modification factor for buckling restrained braced frames (BRBFs) is higher than the concentric braced frames (CBFs) one, and also Asgarian and Shokrgozar [9], investigated the response modification factor of split X, and chevron V and inverted V in steel frames. They found that response modification factor with values of 8.35 and 12 can be suggested for ultimate limit state and allowable stress design methods. Kiggins and Uang [10], evaluated that Buckling- Restrained Braces

(BRB) with steel moment frames not only reduces residual story drifts rather leads to the large value of response modification factor. The present study focuses on the evaluation of over-strength, redundancy, and ductility factor of base isolated structures with nonlinear static pushover analysis that has been suggested in ATC-40 [13] with help of ETABS [11] software.

II. LEAD RUBBER BEARING ISOLATOR (LRB)

Force-displacement relationship of lead rubber bearing is based on Bi-linear model, in practice, all isolation bearings are modeled by a bilinear model that can be defined by initial stiffness (K_1), post-yield stiffness (K_2), and characteristic strength (Q). Components of lead rubber bearing is shown in Fig. 1 and a typical idealized force-displacement relationship of the lead-rubber bearing is shown in Fig.2. The elastic stiffness K_1 is either estimated from available hysteresis loops from the elastomeric bearing test or as a ratio (n= K_1/K_2) depends on the material used and considered n=0.10 for rubber isolator. Post-yield stiffness of the isolation systems K_2 is accurately estimated from the shear modulus of the rubber and the bearing design. The characteristic strength Q is estimated from the hysteresis loop for the elastomeric bearings, and for lead-plug bearing, Q is given by the yield stress in the lead and the area of the lead, as the following formula.

$$Q = A_{pb}.\sigma_y \quad (1)$$

where A_{pb} is the area and σ_y is the yield stress of lead. Which σ_y was assumed 10 (MPa). The effective stiffness, defined as the secant slope of the peak-to-peak values in a hysteresis loop [12], [13]. The seismic isolators in the system are defined as Nllink components.

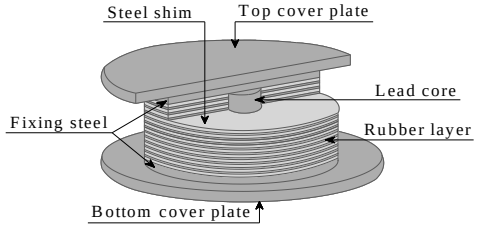


Fig. 1. Components of lead rubber bearing

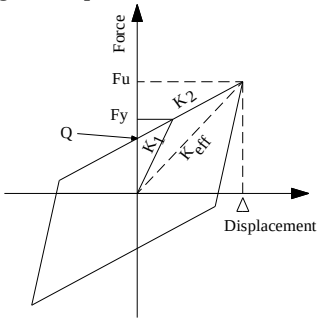


Fig. 2. Parameter of basic hysteresis loop

III. MODEL CONSIDERED FOR ANALYSIS

Typical floor plan of fixed-base and base-isolated 5, 8, 12 and 15-stories reinforced concrete moment resisting frame, are analyzed using response spectrum and equivalent static analysis, consist of shear walls (dual system), which are used with 3 bays of 5.00m in both X and Y-directions and the typical story height is 3.10m from the first level to the roof. As the subject structures in this context are illustrated in Fig. 3. The buildings are assumed to be located in seismic zone 3 and assigned a seismic zone factor Z=0.3 according to the UBC-97 code [5], and soil profile type was assumed to be S_D Based on the seismic zone factor (Z=0.3) and stiff soil profile (S_D), the seismic coefficient $C_V=C_{VD}=0.54$ and $C_A=C_{AD}=0.36$ are obtained from UBC 97. Further inputs include the unit weight of the concrete is 25 KN/m^3, the elastic modulus of concrete is 2.40×10^6 KN/m^2, the compressive strength of concrete is 25 N/mm^2, the yield strength of steel is 400 N/mm^2 and elastic modulus of steel is 2×10^5 N/mm^2. The loading of the building was assumed to be dead load and live load as 6.50 and 3.50 KN/m2, respectively. Percentage of imposed load to be considered in seismic weight calculation 40. Response reduction factor for the fixed-base (R) and base-isolated structure (R_1) consist of special moment resisting frame have taken as 8.5 and 2.0 respectively, as reflected in UBC-97.

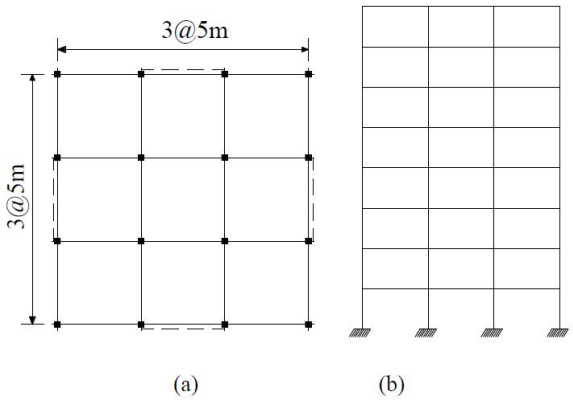


Fig. 3 Configuration of structures. (a) Plan, and (b) Elevation of the prototype buildings

IV. DESIGN OF ISOLATION SYSTEM

The lateral displacement and base shear of superstructures, stability of isolation devices, and possibility of uplift in isolators require a preliminary design, which may be based on a previous project or data from a manufacturer. After this preliminary design process is completed, examples of the final design of the isolators will be ordered and subjected to the code-mandated prototype test program. Depending on the results of the prototype tests, the preliminary design may or may not need to be modified. In order to minimize the number of interaction in the design, it is

essential to have accurate data and good design procedures in the preliminary design phase [12]. According to the UBC-97, the two basic displacements to be calculated are D_D and D_M, based on Design Basis Earthquake (DBE), and Maximum Capable Earthquake (MCE) levels for stiff soil at the center of rigidity of the isolation system are estimated using the equations (2) and (3).

$$D_D = \frac{(g/4\pi^2)C_{VD}T_D}{B_D} \qquad (2)$$

$$D_M = \frac{(g/4\pi^2)C_{VM}T_M}{B_M} \qquad (3)$$

where g is the gravitational acceleration, C_{VD} and C_{VM} are seismic coefficients, T_D is the period at the design displacement (DBE), and T_M is the period at the maximum displacement (MCE), and BD and BM are damping coefficient corresponding to the DBE and MCE level responses, respectively. For preliminary design purposes, effective damping β_{eff}=15% and 20% critical are assumed. Damping is the characteristic of a structural system that opposes the motion and tends to return the system to rest when it is disturbed [14]. The first step in the design procedure is to select a trial effective isolated period (T_D), in a time separation of 3 is considered [16]. Then, the stiffness of the isolation system is estimated using the total seismic weight, W, by the following equations.

$$K_{eff} = \frac{W}{g}\left(\frac{2\pi}{T_D}\right)^2 \qquad (4)$$

And other important mechanical characteristic of the isolation bearing are the following formulas. And horizontal stiffness is defined as:

$$K_H = \frac{G.A}{t_r} \qquad (5)$$

And vertical stiffness is estimated from the following equation:

$$K_V = \frac{E_c.A}{t_r} \qquad (6)$$

Where E_c is given by $E_c=6GS^2$. Maximum shear strain is taken:

$$\gamma = \frac{D}{t_r} \qquad (7)$$

where G is the shear modulus that assumed 0.40 (MPa), A is the full cross-section area, t_r is the total thickness of the rubber, and S is the shape factor that assumed 10. The maximum horizontal displacement (D) is related to the maximum shear strain γ=150%. Two different types of lead rubber bearing (LRB) has preferred, which will be denoted LRB-A to keep beneath the 4 inner columns, and LRB-B to keep beneath the 12 outer columns. In this context with this information, a design of elastomeric bearings with a lead plug based on the requirements of the UBC-97 code [12], [15] is illustrated in Table 1 and Table 2.

No. stories	5-stories		8-stories	
Type of LRBs	LRB-A	LRB-B	LRB-A	LRB-B
β_{eff} (%)	15	15	15	15
$B_D=B_M$	1.35	1.35	1.35	1.35
T_D (s)	2.5	2.5	2.5	2.5
T_M (s)	2.7	2.7	2.77	2.77
D_D (m)	0.25	0.25	0.25	0.25
D_M (m)	0.572	0.572	0.587	0.587
Ø[a](mm)	85	50	110	60
K_{eff} (kN/m)	981.75	279.25	1413.7	436
Q (kN)	56.72	19.62	94.98	28.26
K_H (kN/m)	981.75	279.25	1413.7	436
K_V (kN/m)	5.89 ×10^5	1.67 ×10^5	8.48 ×10^5	2.62 ×10^5

a: Ø is the lead core diameter

Table 1. Summary of Mechanical Properties of Lead Rubber Bearing (LRB) for 5 and 8- stories.

No. stories	12- stories		15- stories	
Type of LRBs	LRB-A	LRB-B	LRB-A	LRB-B
β_{eff} (%)	20	20	20	20
$B_D=B_M$	1.5	1.5	1.5	1.5
T_D (s)	2.8	2.8	3.0	3.0
T_M (s)	2.53	2.53	3.41	3.41
D_D (m)	0.25	0.25	0.268	0.268
D_M (m)	0.482	0.482	0.65	0.65
Ø[a](mm)	125	70	150	80
K_{eff} (kN)	1744	528	1744	528
Q (kN)	122.66	38.46	176.63	50.24
K_H (kN/m)	1744	528	1744	528
K_V (kN/m)	1.05 ×10^6	3.16 ×10^6	1.05 ×10^6	3.16 ×10^6

Table 2. Summary of Mechanical Properties of Lead Rubber Bearing (LRB) for 12 and 15- stories.

V. RESPONSE MODIFICATION FACTOR

Elastic analysis of structures under earthquake force can generate base shear force and stress which are so noticeably bigger than real structural response. The structures can absorb a lot of earthquake energy and resists when it enters the inelastic range of deformation. The response modification factor or response reduction factor (R) reflects the capacity of the structure to dissipate energy through inelastic behavior. Whereas, it is a reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra [16], [17]. The response modification factor was calculated as the product of three factors: Ductility Reduction factor (R_μ), Over-strength Reduction factor (R_s), and Redundancy factor, (R_R) [6]. As shown in Fig. 4, the base shear versus roof displacement relationship of a structure which can be developed by a nonlinear static pushover analysis. On the structure level, an elastic–perfectly-plastic idealization of the real system is employed, since no clear yield point is present. Hereby actual force-displacement response curve from pushover analysis is idealized by a bilinear

elastic-perfectly plastic response curve [18]. The parameters of response modification factor can be defined as following.

$$R = R_{\mu} \times R_S \times R_R \qquad (8)$$

Regarding ductility reduction factor significant research works have been reported for the case of R_{μ} such as Nassar and Krawinkler [19], Riddell, and Hidalgo and Cruz. Researchers proposed the different formulation in order to determine the ductility reduction factor. Subsequently, in the present research, the formulation proposed by Miranda and Bertero was used. Miranda presented below equations using 124 ground motions recorded on a wide range of soil conditions and assumed five percent of critical damping. Their equation for the ductility factor is given as following.

$$R_{\mu} = \frac{\mu - 1}{\varnothing} + 1 \geq 1 \qquad (9)$$

where Ø is a function of ductility ratio $\mu=\Delta_{max}/\Delta_y$ (Eq.16), fundamental period of structure (T), and soil conditions at the site, and is given by:

For rock sites

$$\varnothing = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} exp[-\frac{3}{2}(\ln T - \frac{3}{5})^2] \qquad (10)$$

For alluvium

$$\varnothing = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} exp[-2(\ln T - \frac{1}{5})^2] \qquad (11)$$

For soft soil sites

$$\varnothing = 1 + \frac{T_g}{3T} - \frac{3T_g}{4T} exp[-3(\ln \frac{T}{T_g} - \frac{1}{4})^2] \qquad (12)$$

where T_g is the predominant period of the ground motion, defined as the period at which the maximum relative velocity of a 5% damped linear elastic system is maximum throughout the whole period range. According to the soil profile type, as stated in UBC-97, alluvium was considered to treat the S_D as selected in this study.

Structural over-strength has an important role in collapse prevention of the buildings, over-strength helps the structure to stand safely not only against saver tremors but reduces the elastic strength demand, as well as, this object is performed using the reduction factor [28]. The over-strength factor is defined as the ratio of the maximum base shear coefficient in actual behavior V_y to first significant yield strength or first plastic hinge in structure V_s [20].

$$R_s = \frac{V_y}{V_s} \qquad (10)$$

The redundancy R_R can not, however, be larger than one [6]. Typically redundancy factor was set equal to 1.0. Moreover, the over-strength and redundancy are considered as one component, as has been adopted by many investigators including some of the ATC researchers. This is because the over-strength parameter implicitly accounts for redundancy through redistribution of actions, which leads to higher over-strength. The force reduction factor can be therefore defined as the product of the ductility reduction factor (R_{μ}) and the over-strength factor (R_S) [21].

$$R = R_{\mu} \times R_S \qquad (11)$$

Therefore, the response modification factor accounts show the difference in the level of stresses considered in the design of the structure. It is generally expressed in the following form. As mentioned previous sentences, response modification factor for ultimate strength design method is established in Eq. (15).

$$R = \frac{V_e}{V_s} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = R\mu \times R_S \qquad (12)$$

where V_e, simply represent the maximum lateral force, which would develop in a structure, responding entirely linear elastic behavior under the specified ground motion.

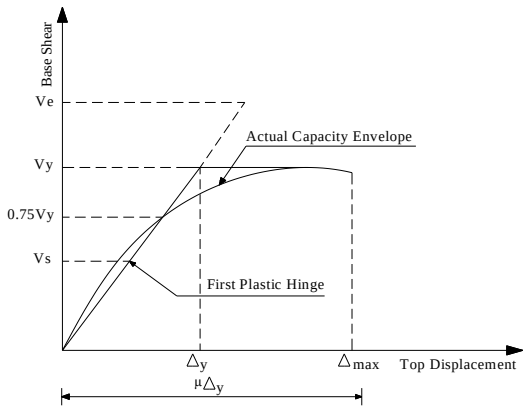


Fig. 4 Illustration of seismic performance factors

As previously shown in Fig. 4, the most significant parameter is mainly defined as the ability of a material, component, connection or structure to undergo inelastic deformation as defined ductility of the structure. The general analytical definition of ductility is given below.

$$\mu = \frac{\Delta_{max}}{\Delta_y} \qquad (13)$$

Δ_{max} was defined as the ultimate deformation, corresponding to the peak strength when the post-peak load is fallen 15%. And the yield displacement (Δ_y) is defined as the displacement of the intersection point of the following two lines: the strength line that passes through the origin and 75% V_{max} of the curve and is horizontal to the x-axis [22].

VI. NONLINEAR STRUCTURAL ANALYSIS RESPONSE SPECTRUM METHOD

Pushover and the response spectrum dynamic analysis is employed to evaluate over-strength, global yield limit state, ductility, and response modification factors

of base isolated structures. Pushover analysis is an advanced tool to carry out static nonlinear analysis of framed structures. Pushover analysis is used to evaluate nonlinear behavior and gives the sequence and mechanism of plastic hinge formation. The pushover curve that is a relationship between the shear forces versus roof displacement gives the actual ultimate capacity of the structure in the nonlinear zone [23]. The capacity of the structure is represented by a force-displacement curve, obtained by nonlinear static pushover analysis. Pushover analysis can be performed as force-controlled or as displacement-controlled. In the displacement-controlled procedure, the structure is pushed to reach the specified target displacement, however value of applied load is not known in advance. However, static pushover analysis is more appropriate for low-rise and short period frame structures. For well-designed buildings but with structural irregularities, the results of the procedure also show good correlation with the dynamic analysis [24].

VII. EVALUATION AND RESULT

The load-displacement curve for the monotonic lateral pushover, shown in Fig. 4, indicate the capacity curve for the eight models classes in the case that components of the response modification factor are considered. The pushover curves are idealized as stated in the previous section. Figs. 5-8 state that ultimate shear force in isolated structures are varied from 82.98, 157.66, 242.05 to 302.55 (ton), in 5, 8, 12 and 15-stories, respectively. And similarly, in fixed base structures, the ultimate shear force has the values of 1415.61, 879.85, 825.60, and 660.02 (ton) as the same stated earlier in the story, respectively. From these statements, can be observed that ultimate bearing capacity of isolated structures increase more rapidly with 264.6% whereas in fixed base structures ultimate bearing capacity is reduced by 53.37%, when the number of stories increase. Subsequently, as shown in Fig. 9, backbone curve of fixed base evolves downward to coincide along the isolated model curve with increasing height of the structures.

Components of response modification factor have been estimated, which is tabulated in table 3. According to Table 4, the isolated structures with ductility (μ) values of 3.80, 3.75, 3.38 and 3.22 are more flexible in compared to fixed base structures in ductility values of 1.20, 1.25, 1.52 and 1.55. Subsequently, from the variation of over-strength (R_S) values, can be seen that in isolated structures the over-strength (R_S), increase when the number of stories increase, against the fixed base structures the over-strength (R_S) decrease as the height of structures increase. As compared the response reduction factor in the ultimate limit state (R), with reflected value of R=2 in UBC-97, based on isolation system, it is evident that R- value has been considered from the code, has approximately 80% reduction, indicate that is more conservative.

No. stories	R_μ	R_S	μ	R
5	1.16	3.74	1.20	4.33
8	1.31	1.32	1.25	4.35
12	1.67	3.11	1.52	5.11
15	1.60	2.41	1.55	3.86

Table 3. Comparison Response Modification Factor for fixed-base

No. stories	R_μ	R_S	μ	R
5	3.83	2.60	2.80	9.95
8	3.75	3.53	3.75	13.23
12	3.43	3.01	3.38	10.32
15	3.2	3.17	3.22	10.14

Table 4. Comparison Response Modification Factor for isolated-base.

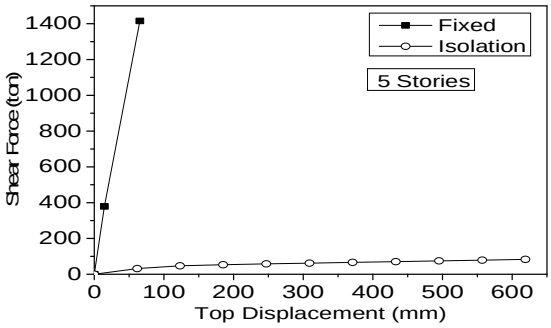


Fig. 5 Roof displacement-base shear curve (5- stories)

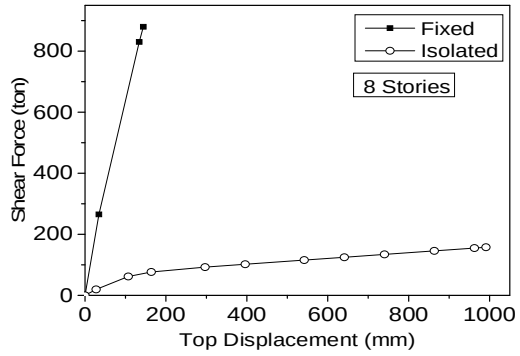


Fig. 6 Roof displacement-base shear curve (8- stories)

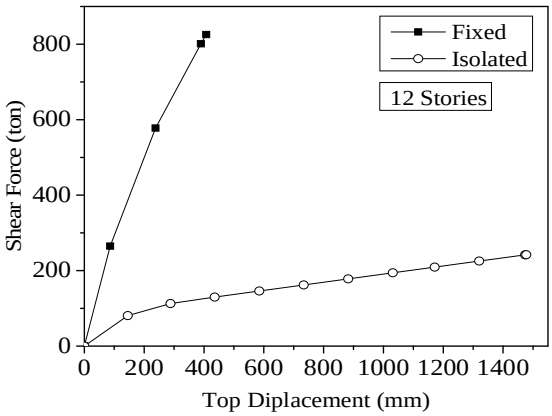


Fig. 7 Roof displacement-base shear curve (12- stories)

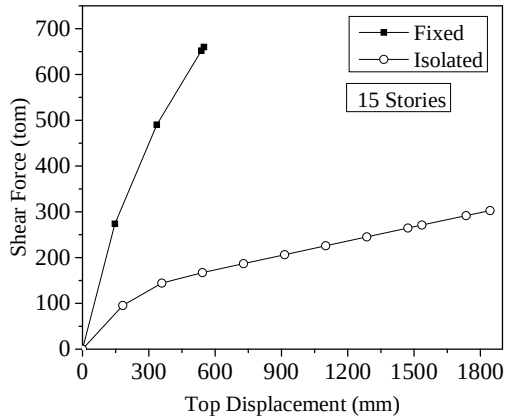


Fig. 8 Roof displacement-base shear curve (15- stories)

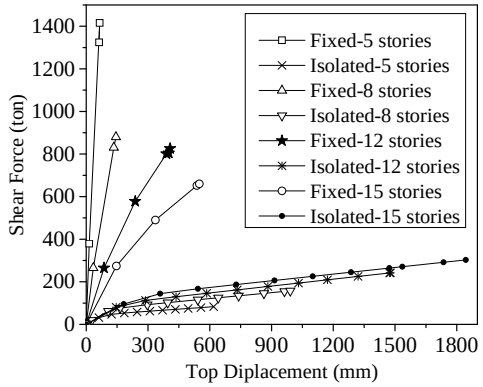


Fig. 9 Roof displacement-base shear curve for 5, 8, 12, and 15-stories

CONCLUSION

Response modification factor of isolated structures as compared with non-isolated structures was considered in this study. The structures have been analyzed by nonlinear static analysis for assessment the components of response reduction factor. This study investigated that seismic base isolation can reduce the seismic effects, and therefore base shear by lengthening the natural period of vibration. The following conclusions can be drawn from this study.

1. Based on the pushover curves, in the isolation system the ultimate bearing capacity is increased more rapidly due to the increasing the height of the structure, whereas non-isolated structures reduce the ultimate bearing capacity, therefore in multistory building, capacity curves of structures reach close to each other, this statement indicate that isolation systems in high-rise building cannot be more capable.

2. As compared the calculated response modification factor of isolation system in nonlinear analyses with the reflected values from the code, it can be seen that code reduce the value of R factor approximately by 80% in isolated structures. In fact, the response modification factor is used in modern seismic codes to scale down the elastic response of structures to keep the behavior of isolated structure in the elastic range. Based on this result response modification factor for isolation system is more conservative, for this purpose, it can be modified to 4 or 5 to keep the construction somewhat economic.

3. The over strength (R_S) factor in the isolated structures increases as the number of stories increase, unlike, in the fixed base structures over-strength factor decrease as the height of structures increase.

4. It can be observed that ductility of isolated structures is higher than the fixed base structures.

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