Evaluating Seismic Design Factors for Reinforced Concrete Frames Braced with Viscoelastic Damper Systems

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ABSTRACT

This study attempts to highlight the effects of viscoelastic damper bracings on the design seismic factors used for the design of reinforced concrete (RC) moment frames. The aim is to evaluate and provide the seismic response modification factor of RC ordinary moment frames (OMFs) which are braced with viscoelastic damping systems. Ductility and overstrength factors are also evaluated. These design factors have been also investigated for similar, but unbraced frames. The effect of number of stories has been considered in this study through using four-, eight-, twelve- and sixteen-story models. The frame members were designed by using linear response history analysis procedure. The analysis has been performed using nine different earthquake records to include the variability in seismic parameters of ground motions. Moreover, a nonlinear pushover analysis was employed with assigning nonlinear material behavior and plastic hinges to beams and columns, according to FEMA 356 code. It was found that the response modification, ductility and overstrength factors increased with increasing the number of stories. However, adding viscoelastic dampers further increased these seismic factors significantly.

KEYWORDS: Moment frames, Viscoelastic bracing, Response modification factor, Ductility, Overstrength.

INTRODUCTION

Structures can be designed and constructed with much less strength than their elastic strength demand in order to reduce high cost related to elastic strength demand (Moghaddam et al., 2005). This reduction has additional structural requirements of ductility, energy dissipation and self-centering capacity (Kim and Choi, 2005). Such requirements can be evaluated and determined using explicit inelastic dynamic analysis (Barakat et al., 1997; Shatnawi et al., 2008; Shatnawi, 2001).

Received on 18/9/2017.
Accepted for Publication on 15/10/2017.

Considering the complexity and time-consuming nature of explicit inelastic dynamic analysis, one may use simplified procedures to design problems with practical and relatively quick solutions (Bojórquez et al., 2014). Such simplifications exist in terms of seismic structural properties and force reduction factors (Izadinia et al., 2012). Figure 1 conveys this concept by showing the inelastic response of a structure to seismic excitation. If the structure is designed to remain elastic during excitation, it will be subject to an induced elastic force and elastic displacement equal to elastic base shear, V_e, and elastic displacement, Δ_e. However, inelastic dynamic analysis shows that structures can be designed with strength less than the elastic strength demand, resulting in maximum inelastic displacement, Δ_M.

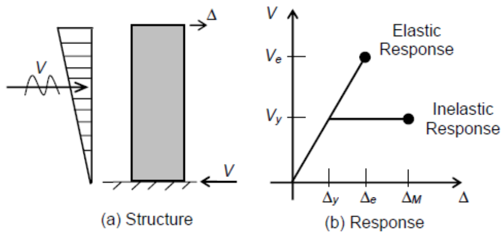


Figure (1): Elastic *versus* inelastic behavior of structures

Attempts by many researches have been performed to predict the values assigned to response modification factors for different structural systems with different analysis methods (Barakat et al., 1997; Zafar, 2009; Mahmoudi and Zaree, 2010; Izadinia et al., 2012; Abdi et al., 2015, among others).

Zafar (2009) conducted a parametric study using a single-bay three-story building to find the effects of geometry and material characteristics on the response modification factor value and concluded that decreasing the bay length and story height leads to an increase in the structure response modification factor. Also, he found that the response modification factor increased with the change in stiffness of the structure due to changes in material or geometric properties. Moreover, Izadinia et al. (2012) studied the demand seismic parameters, such as ductility reduction factor, response modification factor and overstrength factor, using different pushover analysis methods, including conventional and adaptive pushover analyses and found that a maximum relative difference of 16% in the response modification factor exists between the conventional and adaptive pushover analysis methods. Furthermore, Esmaeili et al. (2013) studied the influence of increasing the building height on the seismic behavior of dual structural systems in the form of steel moment-resisting frames accompanied with reinforced concrete shear walls. Their analytical results showed that ductility and response force reduction factors increased with increasing the structural height, while the overstrength factor was clearly decreasing.

Abdi et al. (2015) investigated response modification, ductility and overstrength factors using a nonlinear statistical analysis for different steel structures equipped with viscous damping systems and found that the response modification factors for steel buildings increased when adding the damping systems.

Viscoelastic bracing system has been adopted for various high-rise buildings in order to reduce lateral force effects as wind loads. Such passive dissipation system was found effective, reliable and with a significant reduction of lateral deformations (Habibi et al., 2013; Hejazi et al., 2014; Sebetahd and Zandi, 2012; Mazza, 2016, among others). For example, Sabetahd and Zandi (2012) studied the effect of viscoelastic dampers on the reduction of seismic effects of 9-, 14- and 17-story structures. Their study showed that maximum base shear for the three structures decreased (25%) when adding viscoelastic dampers having (25%) damping ratio, whereas roof displacement reduced by (65%). They concluded that viscoelastic dampers have a significant effect on reducing seismic structural displacement response to earthquake vibrations. As an application, Lavan and Amir (2014) proposed an approach for performance-based optimal sizing seismic retrofitting of 3D irregular frame structures using viscous dampers. Such approach optimizes the whole size group of viscous dampers. Moreover, Mazza (2016) proposed a displacement-based design method using hysteretic damped braces (HYDBs) and rubber pads for the seismic retrofitting of irregular-plan reinforced concrete framed buildings and checked the procedure effectiveness and reliability using nonlinear static analysis.

Response modification, ductility and overstrength factors for reinforced concrete frames which are specifically braced with viscoelastic damping systems are not available in design building codes such as IBC and EC. This study attempts to contribute to providing such factors to be utilized in the analysis and design of these types of reinforced concrete moment frame with such damping systems when subjected to seismic forces.

Ductility Factor

Ductility is defined as the capacity of the structure to sustain large inelastic deformations without collapse and any major reduction in strength or stiffness degradation. Therefore, the response extents of earthquake-induced vibrations are dependent on the energy dissipation level of the structures, which is a function of their capability to absorb and dissipate energy by ductile deformations (Elnashai and Di Sarno, 2008). Thus, most structures are designed and constructed to behave inelastically under severe earthquakes for economic reasons.

According to the above main philosophy in seismic design, which is known as the concept of strong column-weak beam, displacement ductility as the most common indicator of seismic design is defined as the ratio between ultimate displacement and yield displacement and is given as follows:

$$\mu_s = \frac{\Delta_u}{\Delta_y} \tag{1}$$

where Δ_u and Δ_y are displacement at ultimate and yield points, respectively.

Force Reduction Factor, *R*

Force reduction factor, *R*, is the relation between the elastic strength demand and the actual yield level of the structure. Figure 2 illustrates the force and displacement at first significant yield (*i.e.*, F_y and Δ_y, respectively). Figure 2 shows that the force reduction factor, *R*, may be expressed in terms of forces as:

$$R = \frac{F_e}{F_y} \tag{2}$$

where F_e and F_y are the maximum and yield forces due to linear elastic response of the structure during the entire time of excitation caused by the earthquake.

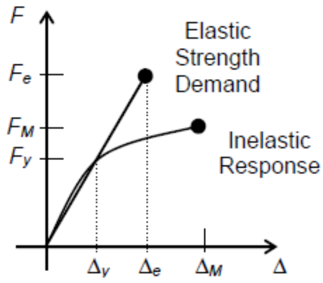


Figure (2): Definition of response modification factor

The factor *R* may also be expressed in terms of displacements as follows:

$$R = \frac{\Delta_e}{\Delta_y} \tag{3}$$

where Δ_e and Δ_y are the maximum and yield displacements due to linear elastic response of the structure during the entire time of excitation caused by the earthquake.

However, the inelastic response of actual structures is not well defined by straight line segments as in the ideal elastic case. Thus, the first significant yield is not necessarily the formation of the first plastic hinge in the structure. Figure 3 provides various definitions of the first significant yield.

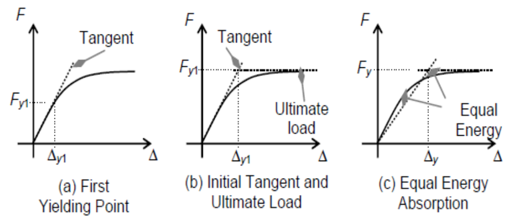


Figure (3): Definitions of first significant yield

Relationship between Force Reduction Factor and Ductility

Force reduction factor and global ductility demand can be established through statistical or probabilistic means because of randomness and uncertainty of earthquakes. Experience and inelastic dynamic analysis show that ductility demand is a function of the amount of force reduction. Generally, ductility increases as force reduction increases as shown in Figure 4, which indicates that a reduction of the actual strength of the structure from elastic strength demand, F_e, to level F_{y1} will result in the maximum inelastic displacement of Δ_{M1}. Each level of reduction needs ductility demand associated with that level as shown in Figure 4. The envelope also shows that larger reduction in strength requires larger ductility demand from the structure.

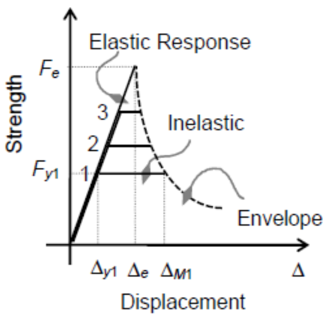


Figure (4): Structural response

Newmark and Hall (1973, 1982) have suggested two approximate relationships between R and μ with reference to structural period. They have found that for long-period structures, the maximum elastic displacement and maximum inelastic displacement remain within the same range. Alternatively, for short-period structures, they found that the maximum elastic energy and the maximum inelastic energy absorbed by the structure remain within the same range. Accordingly, they have proposed Equal Displacement Criterion (EDC) for long-period structures and Equal Energy Criterion (EEC) for short-period structures. Also, they concluded that in structures with a very short period of less than 0.2 second, a force reduction factor of 1 (*i.e.*, no force reduction) should be used, since ductility does not help in reducing the response of the structure.

In EDC, the maximum elastic displacement and the maximum inelastic displacement remain in the same range during excitation. This leads to the following relationship, since $\Delta_e = \Delta_M$:

$$R = \mu \tag{4}$$

In EEC, the maximum elastic and inelastic absorbed energies remain the same during excitation, leading to the following relationship:

$$R = \sqrt{2\mu - 1} \tag{5}$$

However, Equation (4) is only valid for long-period structures, whereas Equation (5) is only valid for short-period structures with the period term being only a relative quantity associated with the dominant period of the earthquake.

Nassar and Krawinkler (1991) have derived a relationship between R and μ based on statistical evaluation of different hysteresis models to actual earthquakes as follows:

$$R = [C\,(\mu - 1) + 1]^{\,1/C} \tag{6}$$

where C is defined as:

$$C = \frac{T^a}{1 + T^a} + \frac{b}{T} \tag{7}$$

where a and b are constants and T is the period of the structure.

Nassar and Krawinkler (1991) used nonlinear regression for a bilinear model with 10% post-yielding stiffness to produce values of a = 0.8 and b = 0.29. Figure 5 shows a plot of C parameter *versus* the period T using these values of a and b. The relationship derived by Nassar and Krawinkler showed the same trend as EDC and EEC when C equals 1 and 2, respectively.

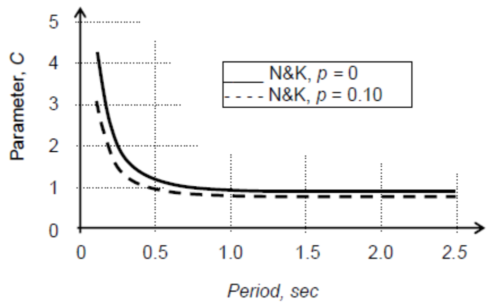


Figure (5): Relationship between parameter *C* and structural period

In conclusion, the assignment of *R* and μ in seismic codes is based on engineering judgment in part, experiments and experience from previous earthquakes, as well as from practice as a result of randomness of structural response to earthquakes. Seismic design codes always emphasize that the requirements are minimal. Generally, EDC and EEC are still the most popular criteria in practice. However, EEC may be seen as an upper bound, since it needs more ductility demand than EDC.

Overstrength Factor

Overstrength factor, Ω, is defined as the ratio between the required strength and the actual strength of a component, material or structural system. Overstrength factor may be expressed as given in Equation (8) below; where V_{max} and V_y are the actual maximum and design yield base shear of the system as illustrated in Figure 6.

$$\Omega = \frac{V_{max}}{V_y} \quad (8)$$

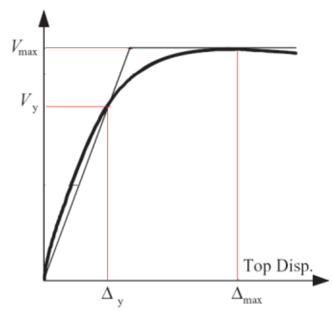


Figure (6): Definition of overstrength factor

CASE STUDY AND ANALYSIS PROCEDURE

Case Study and Building Description

Reinforced concrete OMF force-resisting system will be used in this study in order to evaluate force reduction factor, *R*, ductility factor, μ, and overstrength factor, Ω. The reinforced concrete buildings for the analysis are four-, eight-, twelve- and sixteen-story buildings. Such numbers of stories have been used in literature to evaluate response modification factor of steel structure with viscous damper device (Abdi et al., 2015) as well as for analytical investigation of *R*-factor for RC frames with steel chevron bracing (Akbari and Maheri, 2013).

All the model buildings consist of RC OMFs in both orthogonal directions and have the same floor plan geometry as illustrated in Figure 7. All selected models have three bays with 5.0 m spacing in both horizontal and transverse directions. Similar plan geometry has been adopted in Mahmoudi and Zaree (2010) and Mahmoudi and Abdi (2012) to evaluate response modification factors for steel structures with different bracing systems. As well, Akbari and Maheri (2013) used similar plans for analytical study on *R*-factor for RC frames with steel chevron bracing.

The model ground story has a height of 3.8 m, while all the other stories have a height of 3.0 m each. The design live load was taken as 2.0 kN/m^2 on roof floor and 4.0 kN/m^2 on all other floors. The design dead load was calculated for a slab thickness of 200 mm. The concrete slab has been modeled and defined as a rigid diaphragm. All columns and beams have rectangular cross-sections of different sizes. Element sizes have been determined based on linear response history analysis and design of different numbers of stories. All concrete elements are designed according to ACI318-14 provisions. The unit weight of reinforced concrete was taken as 24.0 kN/m^3. All reinforced concrete elements have a concrete compressive strength of 30 MPa. The definitions of concrete and steel stress-strain diagrams are shown in Figure 8 as utilized in the software ETABS 2013. A concrete damping ratio of 5% has been selected. ETABS defined this damping as stiffness weighted, which is also known as composite modal damping. For illustration, Figure 9 shows elevations of both lateral force-resisting systems of the OMF building with four stories.

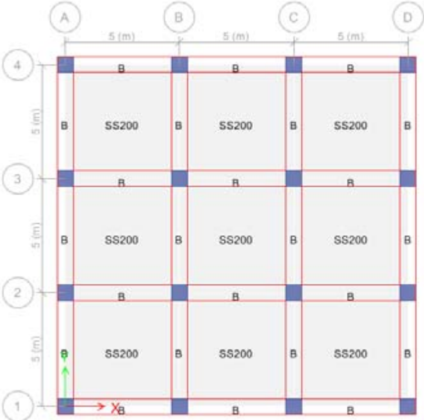
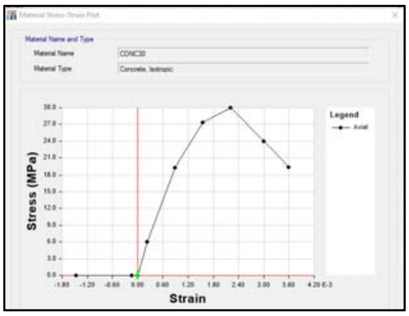
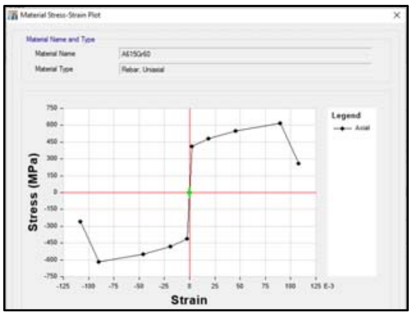


Figure (7): Plan view of the OMF buildings



(a) Concrete stress-strain curve definition in ETABS



(b) Steel stress-strain curve definition in ETABS

Figure (8): Concrete and steel stress-strain diagrams

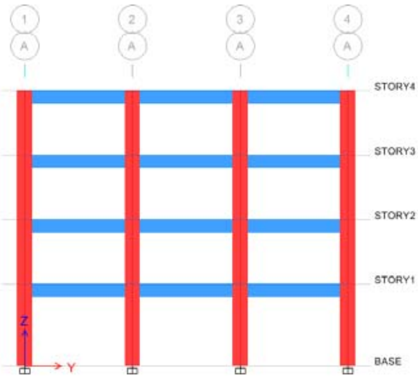


Figure (9): Elevation view of the 4-story OMF building

In this study, R, μ and Ω factors of RC OMF with and without viscoelastic dampers were evaluated as well for four-, eight-, twelve- and sixteen-story buildings. Kelly (2001) provides design guidelines for in-structure damping and energy dissipation and stated that the most effective damping coefficients for viscoelastic dampers are in the range of 5,000 *kN-sec/m* to 10,000 *kN-sec/m*. Viscoelastic dampers provide a velocity-dependent damping force and also have a corresponding elastic effective stiffness, K_{eff}, with a numerical value equal to 2 times the damping coefficient, C, in units of *kN/m*. The most common type of viscoelastic dampers consists of two layers of polymer bonded between a central driving steel plate and two outer steel plates as shown in Figure 10. The dampers are typically installed as pads with maximum dimensions of 200 mm × 200 mm with two pads per unit. Each unit provides an area of 80,000 mm^2 (Kelly, 2001).

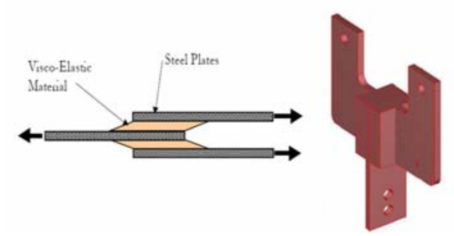


Figure (10): Viscoelastic damping element

In this study, viscoelastic dampers with damping coefficient of 5,000 *kN-sec/m* and effective stiffness of 10,000 *kN/m* (2 × damping coefficient) will be used in the outer frames in each direction at the central bays of each story as shown in Figures 11 and 12. Such damping coefficient and stiffness values are based on practical guidelines for such system found in literature (Kelly, 2001). Viscoelastic dampers were defined using link element property with the mentioned damping coefficient and effective stiffness properties. The determination of strength, deformability and reinforcement steel of each element of the buildings for the corresponding quality and quantity of the built-in material is carried out using ETABS 2013 computer software.

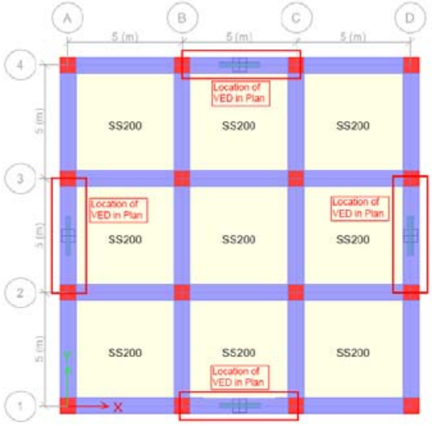


Figure (11): Viscoelastic dampers location in plan of reinforced concrete moment frame building

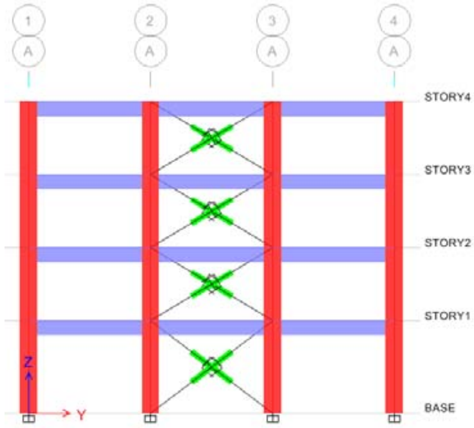


Figure (12): Viscoelastic dampers location in elevation of reinforced concrete moment frame building

Procedure of Analysis

Using the software ETABS 2013, three-dimensional finite element analyses were carried out for the various structures subjected to a selected set of earthquake records. The software is applied to perform nonlinear static pushover analysis as well as linear dynamic time history analysis. As per ASCE7-10, the frames have effective seismic weights including the design dead load and one-fourth of the design live load (*i.e.*, $D + 0.25L$). Each building of concern is analyzed using linear time history analysis, applying earthquake records with 5% damping ratio and designed according to ACI318-14 load combinations including the effect of earthquake forces, E. The sizes of the elements and steel reinforcement obtained from linear response history analysis and design load combinations are then used in the nonlinear static pushover analysis. The plastic hinges have been assigned at the beginning and end of beam and column elements with properties conforming with FEMA 356. The design concrete members' sizes and steel reinforcement are carefully selected for each building following the requirements of ACI 318-14 Code.

Selection of Ground Motions

The time history analysis requires ground motion records to reflect site characteristics and seismic hazard. ASCE7-10 specified that a minimum of three earthquake records shall be used in such type of analysis. Table 1 shows the earthquakes' accelerogram characteristics used in this study. These seismic records which have different characteristics have been used in ETABS 2013 to perform the modal time history analysis so that it includes the variability in seismic parameters of different ground motions.

Table 1. Description of earthquake records used in time history analysis

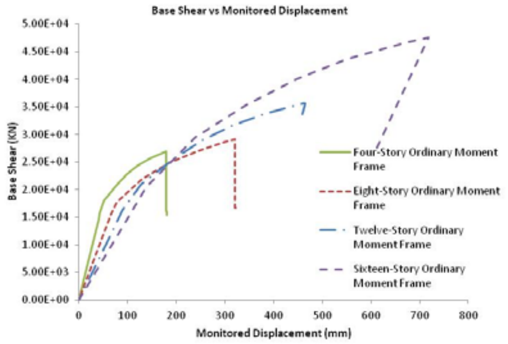
Earthquake	Year	M	Epicentral Distance (*km*)	Component	Max. Acc. A (g)	Soil Condition
Parkfield, California	1966	5.6	5	N85W	0.434	Rock
San Fernando, California	1971	6.4	4	S74W	1.075	Rock
Nahanni N.W.T., Canada	1985	6.9	7.5	LONG	1.101	Rock
Imperial Valley, California	1940	6.6	8	S00E	0.348	Stiff Soil
San Fernando, California	1971	6.4	41	N37E	0.199	Stiff Soil
Monte Negro, Yugoslavia	1979	7	17	N00E	0.171	Rock
San Fernando, California	1971	6.4	41	S38W	0.119	Rock
Near E. Coast of Honshu, Japan	1968	7.9	290	N00E	0.226	Stiff Soil
Near E. Coast of Honshu, Japan	1973	7.4	112	N00E	0.205	Stiff Soil

RESULTS AND DISCUSSION

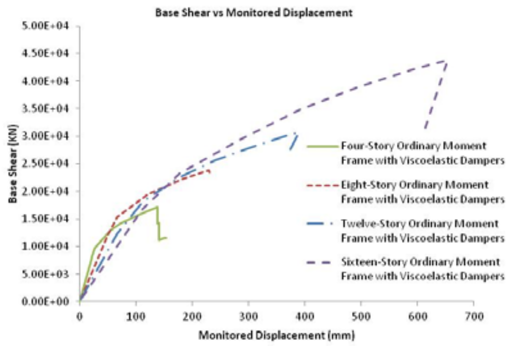
Static Pushover Curve

Static pushover curve demonstrates the relation between the base shear force applied to the building (Y-axis) and monitored top roof displacement (X-axis) in the direction of applied force up to structure failure. Such procedure is very helpful extracting the yield and ultimate displacement and force of the building and evaluating local and global ductility. Takeda model (Takeda et al., 1970) has been used to model the elastoplastic behavior of reinforced concrete members. This model is very similar to the kinematic model, but uses a degrading hysteretic loop based on the Takeda model (Berkeley, C. S. I., 2012). This simple model, is more suitable for reinforced concrete than for steel structures (Berkeley, C. S. I., 2012).

Nonlinear static pushover analysis was performed using ETABS 2013 software to obtain static pushover curves of all the types of buildings of concern in this study. Figure 13a shows static pushover curves of RC OMF buildings without viscoelastic dampers, while Figure 13b shows static pushover curves of such buildings with viscoelastic dampers. It is very important to indicate that the first significant yield displacement point is related to the formation of the first plastic hinge in the structure. It can be noted from Figure 13 that the maximum base shear for OMF without viscoelastic dampers is higher than the one for OMF with viscoelastic dampers, since viscoelastic dampers reduce the amount of reinforcement required for all the concrete elements, which leads to decrease the maximum shear force resisted by the structure.



(a) OMF without viscoelastic dampers



(b) OMF with viscoelastic dampers

Figure (13): Static pushover curves of RC OMF building for different numbers of stories

Overall Ductility Factor

Overall ductility factor, μ, is defined as the ratio between ultimate displacement and yield displacement of the top roof of the building (*i.e.*, $\mu = \Delta_u/\Delta_y$). Overall ductility factor is also known as deflection amplification factor as defined in ASCE7-10.

Yield displacement and ultimate displacement were obtained from static pushover curves by using static pushover analysis. Then, these displacements were used to calculate overall ductility. It is obvious that the first significant yield displacement point is related to the formation of first plastic hinge in the structure. Ductility factors of RC OMF building without viscoelastic dampers are summarized in Table 2 for different numbers of stories. It is noted from the results that the overall ductility factor is increased as the number of stories increases, as shown in Figure 14. Such result can be related to the increase of axial compressive force on columns by increasing the number of stories which has an effect on increasing overall ductility factor. In IBC 2012/ASCE 7-10, a value of 2.5 is assigned for deflection amplification factor for RC OMFs. This assigned value is lower than the least value of overall ductility (4.48 for 4-story buildings) that has been obtained by pushover analysis by 44%.

Ductility factors of RC OMF buildings with viscoelastic dampers are summarized in Table 3 for different numbers of stories. It is noted from the results that the overall ductility factor is increased as the number of stories increases, as also shown in Figure 14. It is observed that adding viscoelastic dampers to RC OMFs increased the overall ductility factors by11%, 6%, 14% and 11% for 4-, 8-, 12- and 16-sotry building, respectively. This can be related to the increase of building stiffness and damping by adding the viscoelastic dampers which have an effect on increasing the overall ductility factor.

Table 2. Overall ductility factor of RC OMF buildings without dampers

No. of Stories	Yield Displacement Δ_y (mm)	Ultimate Displacement Δ_u (mm)	μ
4	40	179.1	4.48
8	65.8	318.6	4.84
12	93	462	4.97
16	138.5	718	5.18

Table 3. Overall ductility factor of RC OMF buildings with viscoelastic dampers

No. of Stories	Yield Displacement Δ_y (mm)	Ultimate Displacement Δ_u (mm)	μ
4	27	136	5.04
8	46	238	5.17
12	67	388	5.79
16	112	654	5.84

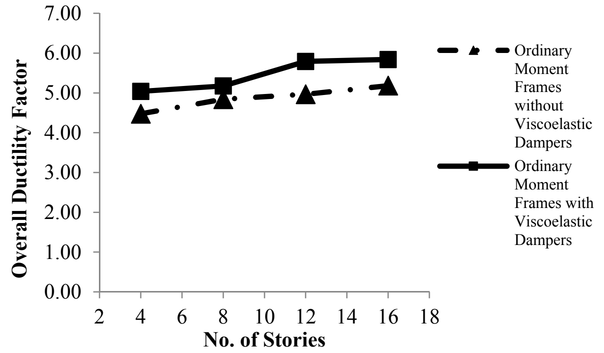


Figure (14): Overall ductility factor of RC OMF buildings with and without viscoelastic dampers

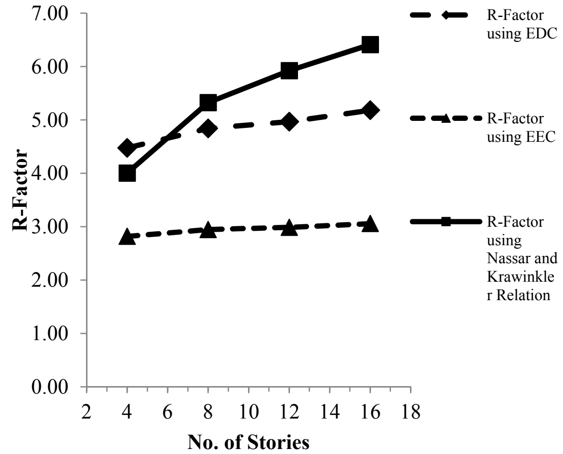


Figure (15): *R*-factor *versus* number of stories for RC OMF buildings without viscoelastic dampers

Seismic Force Reduction Factor, *R*

Seismic force reduction factor, *R*, is determined using the overall ductility factors that are related together by Equal Displacement Criterion (EDC), Equal Energy Criterion (EEC) and Nassar and Krawinkler relationship given in Equation (6).

Figure 15 illustrates *R*-factors obtained by the previously mentioned relations for RC OMF buildings without viscoelastic dampers. It is noted that *R*-factor increases as the number of stories increases. Also, it is noted that *R*-factor values obtained using EEC represent the lower bound values of *R*-factors. *R*-Factors obtained using Nassar and Krawinkler relationship are higher than *R*-Factors obtained by either EEC or EDC, except for the 4-story building, where EDC gave higher value by 11%. ASCE 7-10 provides $R = 3$ for RC OMFs, while the obtained *R*-factors by EDC and Nassar and Krawinkler relationship are higher than this value and values obtained using EEC are less than the ASCE 7-10 value, except for the 16-story building. Since *R*-factors are related and directly proportional to the overall ductility factors, this explains the reason why they are increased by increasing the number of stories when overall ductility factor is increasing.

R-factors of RC OMF buildings with viscoelastic dampers are shown in Figure 16 for different numbers of stories. It is noted that the *R*-factor is increased as the number of stories increases. It is also observed that adding viscoelastic dampers to RC OMFs increased the *R*-factor by a range of 9% to 20%.

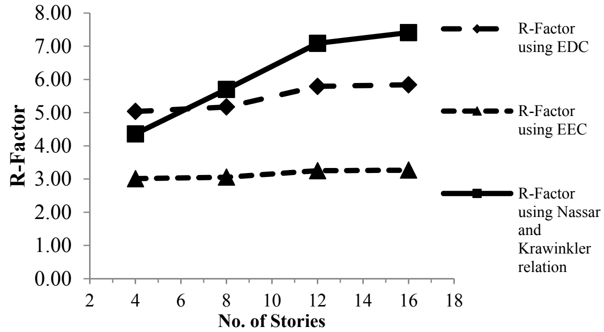


Figure (16): *R*-factor *versus* number of stories for RC OMF buildings with viscoelastic dampers

Overstrength Factor

As previously mentioned, the overstrength factor, Ω, is defined as the ratio of maximum base shear in the actual behavior, V_{max}, to base shear force at first significant yield, V_y, and is expressed as V_{max}/V_y. The maximum base shear in the actual behavior, V_{max}, and base shear force at first significant yield, V_y, are obtained by static pushover analysis using pushover curve. Then, overstrength factor is calculated as V_{max}/V_y. Overstrength factors and base shear values of RC OMF buildings without viscoelastic dampers are given in Table 4 and shown in Figure 17 for different numbers of stories. It is noted from the results obtained that the overstrength factor is increased as the number of stories increases. Such result can be related to the increase of axial compressive force on columns by increasing the number of stories which has an effect on increasing the overstrength factor. ASCE 7-10 assigns a value of 2.5 for overstrength factor for RC OMFs. The value assigned by ASCE 7-10 is higher than all the values of overstrength factors that are shown in Figure 17.

Overstrength factors of RC OMF buildings with viscoelastic dampers are also shown in Figure 17 and results are summarized in Table 5 for different numbers of stories. It is observed that adding viscoelastic dampers to RC OMFs increases the overstrength factors from 4% to 14% approximately. This can be related to the increase of building stiffness and damping by adding the viscoelastic dampers which have an effect on increasing the overall ductility factor.

Table 4. Overstrength factor for RC OMF buildings without dampers

No. of Stories	Yield Base Shear, V_y (*kN*)	Max. Base Shear, V_{max} (*kN*)	Overstrength Factor, Ω
4	14582	26887	1.84
8	15536	29105	1.87
12	16261	35646	2.19
16	20458	47590	2.33

Table 5. Overstrength factor for RC OMF buildings with viscoelastic dampers

No. of Stories	Yield Base Shear, V_y (kN)	Maximum Base Shear, V_{max} (kN)	Overstrength Factor, Ω
4	9034	17317	1.92
8	11597	23822	2.05
12	12289	30698	2.50
16	16806	43875	2.61

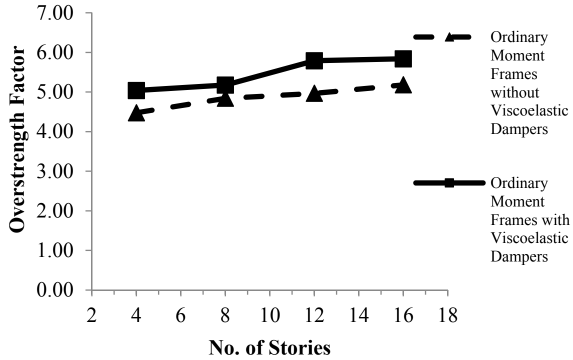


Figure (17): Overstrength factor of RC OMF buildings with and without viscoelastic dampers for different numbers of stories

Elastic Displacement

Elastic displacements, Δ_e, at top roof of 4- , 8-, 12- and 16-story buildings were obtained using linear time history analysis for each case study as shown in Figure 18. Elastic displacement results of each case of interest are shown in Figure 18. It is noted that elastic displacements for RC OMFs are increased as the number of stories increases. Adding viscoelastic dampers to OMFs decreased elastic displacement by 50%, 25%, 18% and 10% for 4-, 8-, 12- and 16-story buildings, respectively. This is due to the fact that viscoelastic dampers increase the building stiffness that will decrease the lateral elastic movements.

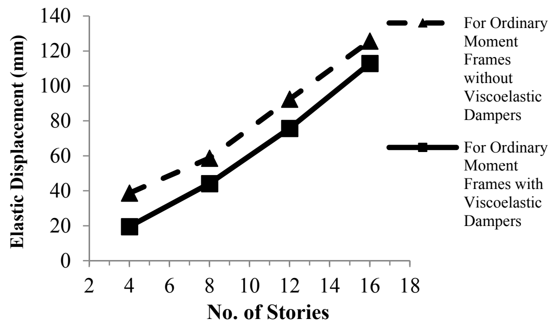


Figure (18): Comparison of elastic displacement for RC OMF buildings with and without viscoelastic dampers

Fundamental Period of the Structure

The fundamental period of the structure, T_n, is defined as the time required for the structure to complete one cycle. The natural period of structure, T_n, resulting from linear time history analysis for each case studied is shown in Figure 19. Natural period of structure for RC OMFs is increased as the number of stories increase. Adding viscoelastic dampers to OMFs has little effect on the natural period of structure as shown in Figure 19, where adding viscoelastic dampers decreases the natural period of structure, since viscoelastic dampers add more stiffness to the structure.

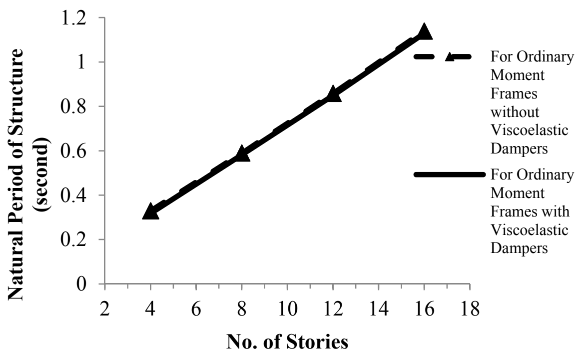


Figure (19): Comparison of natural period of structure for RC OMF buildings with and without viscoelastic dampers

CONCLUSIONS

The following conclusions can be drawn from the results of this study:

- The calculated seismic force reduction factor*s, R*, increase with the number of stories for RC OMFs with or without viscoelastic dampers. However, adding a bracing of viscoelastic dampers to RC OMFs causes a further increase in the *R*-factors.
- The overall ductility factor increases with the number of stories for RC OMFs with or without viscoelastic dampers. However, adding a bracing of viscoelastic dampers to RC OMFs causes a further increase in the overall ductility factor.
- The overstrength factor is increased as the number of stories increases for RC OMFs. However, adding a bracing of viscoelastic dampers to RC OMFs causes a further increase in the overall ductility factors. On the contrary, it has been found that adding viscoelastic dampers to RC OMFs decreases the elastic displacement.
- The response of structures to seismic effects is known to impose high ductility demand on short-period structures that is much higher than that of long-period structures. Such high demand for short-period structures may not even be achieved. This issue is overlooked in seismic codes; the codes ought to revise the concept of force reduction and to distinguish between long-period structures and short-period structures. Short-period structures may need additional provisions to be provided with enough safety measures.
- The study also indicates that viscoelastic dampers have little effect on the behavior of structures with a period of 0.2 seconds or less. Therefore, structures of short period need additional measures to elongate the period rather than increase damping, which may be achieved through using seismic isolation systems.

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