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Response Modification Factor of Coupled Steel Shear Walls

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ABSTRACT: The present research is concerned with the determination of ductility, over-strength and response modification factors of coupled steel shear wall frames. Three structural models with various numbers of stories, bay width and coupling beam height were analyzed using static pushover and incremental nonlinear dynamic analyses. The ductility, over-strength and response modification factors for the three models are determined. Tentative values of 11.1, 11.6 and 10.6 are suggested for the response modification factor of coupled steel shear wall frames with deep and medium depth coupling beams, and uncoupled steel shear wall frames, respectively in the allowable stress design method.

Keywords: Coupled Steel Shear Walls, Ductility Reduction Factor, Incremental Nonlinear Dynamic Analysis, Over Strength Reduction Factor, Response Modification Factor, Static Pushover Analysis.

INTRODUCTION

During recent decades, ductile steel shear walls have been used as lateral load resistant systems in design and retrofit of civil engineering structures. Proper behaviour of this system regarding stiffness, strength, ductility, energy absorption and stability of hysteresis loops strengthens the idea of its application in seismic design of the structures.

A coupled shear wall system consisting of steel fill plates bounded with the column-beam system resembles a cantilever plate girder where the plate, columns and beams of the system act as the web, flanges and stiffeners of the girder, respectively. However, the strength and stiffness of the beams and columns of the steel shear wall frame have more effects on the system behaviour compared with the flanges and stiffeners of the plate girder. The flexural members coupling the shear walls increase stiffness of the system. Under lateral loads, the plate buckles and resists by forming a diagonal tension field. Even for high steel shear walls and large shear loads, high post buckling resistance of the steel plates facilitates the application of thin plates in the coupled shear wall system. The reduction of structural weight, increase of lateral stiffness, reduction of dimensions of beams and columns and easy and fast implementation are some of advantages of

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this system compared with the concrete shear wall system. Also, in contrast with braced frames, this system has the privilege of decentralization of seismic energy from beam-to-column connection joints and the increase of ductility and feasibility of replacing damaged plates after earthquakes. In this regard, Kulak (1985); Elgaaly et al. (1993); Lubell et al. (2000); Rezai (1999); Thorburn et al. (1983); Timler et al. (1998) and Tromposch et al. (1987) assessed the behavior of steel shear walls under static and dynamic loads.

Wagner (1931) and Driver et al. (1998) proposed a modified strip model for analyzing steel shear walls, consisting of thin plates without stiffener, by substituting some inclined truss members along the diagonal tension field (Figure 1). The angle of inclined truss members is determined using the principle of minimum energy consumption. Provided that the steel plate is pretty thin, and that the beams and columns are pretty stiff, α becomes 45º.

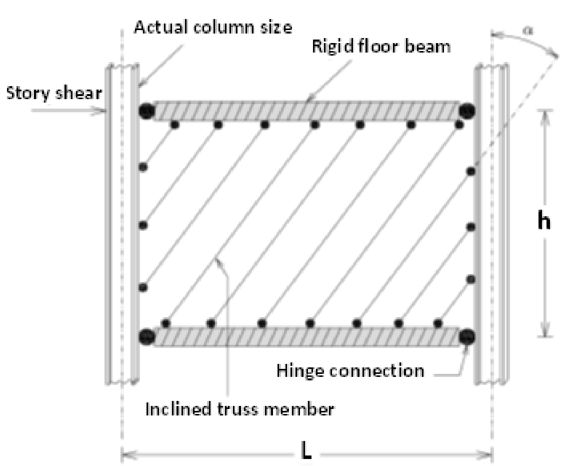

Fig. 1. Modified Strip Model (Driver et al., 1998).

Although the strip model is suitable for estimation of the ultimate load of steel shear walls with thin plate, it cannot be used for steel shear walls consisting of thick plates, those that lack stiffener or those that have opening. To overcome such deficiencies Roberts and Sabouri-Ghomi (1992) and Sabouri-Ghomi and Gholhaki (2005) presented a general model to analyze and design different types of steel shear walls regardless of whether they have thin plates, stiffener and opening. In this model, first, the behaviour of the steel plate and surrounding frame are evaluated separately. Next, the behaviour of the complete system is evaluated using the super position principle considering interaction of the plate and frame. Astaneh-Asl (2001) developed design methods of the coupled steel shear wall considering ductility and over strength of the system and ensuring that the ductile failure modes precede brittle ones. In this design method, inelasticity triggers in non-gravity loads carrying members, if necessary, spreads into gravity load carrying ones towards the end of seismic event in a controlled manner preventing progressive collapse.

The application of coupled steel shear wall systems in the seismic resistant design of structures has seen a considerable increase. On the other hand, common structural analysis methods proposed in seismic codes for typical low-rise and regular high-rise structures are equivalent static and spectral analysis procedures. These issues expose the need for providing more theoretical materials of coupled steel shear wall systems. One of these materials is the response modification factor that relates linear analysis and nonlinear behaviour of structures in the aforementioned analysis methods. This is the main focus of the present research. In this study, three categories of coupled steel shear wall systems designed based on various provisions are evaluated using static nonlinear and dynamic nonlinear analyses. The ductility reduction factor, over-strength reduction factor and response reduction factor are finally proposed for typical layouts of this system.

RESPONSE MODIFICATION FACTOR

In the equivalent linear static method, as the common method proposed in most codes (UBC97, 1997; NBCC, 2005, and BHRC, 2005) for seismic analysis of structures, the lateral seismic loads are reduced by response modification factor to be indirectly taken into account for nonlinear behaviour of structures. Mazzoni and Piluso (1996) evaluated several theoretical approaches such as the low cycle fatigue theory, energy method and maximum plastic deformation technique to evaluate the response modification factor of structural systems. As seen in Figure 2, the response modification factor is the product of three factors including the ductility reduction factor, Rμ, over-strength reduction factor, Rs, and redundancy factor, Y. In this figure, the response modification factor is represented as follows:

$$R = \frac{V_e}{V_d} \tag{1}$$

where V_e is the maximum base shear assuming elastic response for the structure and V_d is the design base shear of the real structure.

For the load and resistance factor design method (AISC, 1999) and the allowable stress design method (MHUD, 2006), the value attributed to V_d is V_s as the base shear corresponding to the first plastic hinge formation in the structure, and V_w as the base shear corresponding to the first allowable stress exceedance in the structure, respectively. Thus, Eq. (1) can be written as:

$$R_u = \frac{V_e}{V_s} \tag{2}$$

$$R_w = \frac{V_e}{V_w} \tag{3}$$

where R_u and R_w are the response modification factors in the load and resistance factor design method and allowable stress design method, respectively. The following relation can relate these two variable one to another:

$$Y = \frac{R_w}{R_u} = \frac{V_s}{V_w} \tag{4}$$

where Y is the redundancy factor determined based on the attitude of design codes for design stress (i.e. yield stress and allowable stress). The redundancy factor usually varies between 1.4 to 1.7 and UBC97 (1997) adopts the value of 1.4 for this factor.

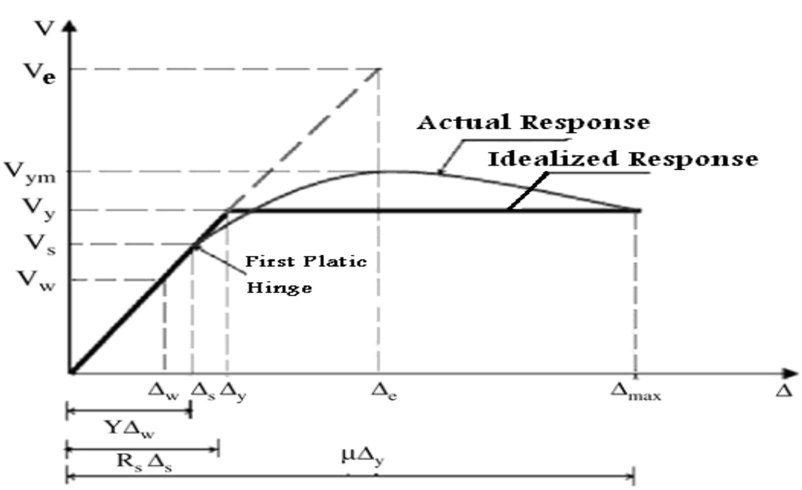

Fig. 2. Definition of nonlinear parameters (Uang, 1991).

$$R_w = 1.4R_u \tag{5}$$

Considering the ductility and over strength, the response modification factor is defined as:

$$R_u = \frac{V_e}{V_s} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = R_\mu \times R_s \tag{6}$$

where V_y as yield base shear is the maximum base shear in the idealized elastic-perfectly-plastic structure, $R_\mu = \frac{V_e}{V_y}$ is the reduction factor resulting from the ductility and $R_s = \frac{V_y}{V_s}$ is the reduction factor resulting from the over strength.

DAMAGE STANDARDS AND STRUCTURAL MODELS

The performance of plastic hinges is qualitatively evaluated using FEMA 273 (1997). The evaluation is undertaken based on the ratio of hinge deformation to corresponding yield deformations, and the damage state of the whole structure is specified by the status of performance point as shown in Figure 3. In this figure, Q is the action (i.e. moment or shear) at hinge, QCE is the corresponding yield limit and Δ and θ are the displacement and rotation of the hinge, respectively. By having the performance point in hand, the deformation of structural elements and, hence, strengthening requirements can be determined. In the present study, nonlinear hinge characteristics are defined using FEMA 273 (1997). In addition to that, response modification factors of structures consisting of the special moment resisting frame combined with coupled steel shear walls are evaluated via nonlinear static (pushover), linear dynamic and nonlinear dynamic analyses using the SAP 2000 software (CSI, 1997).

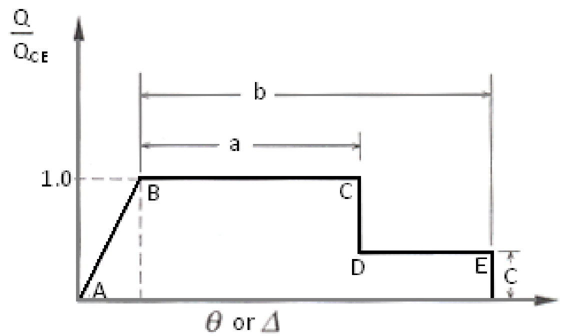

Fig. 3. An example of force -deformation curve of nonlinear hinges according to FEMA 273 (1997).

To assess the parameters affecting the response modification factor, three categories of structural models are investigated including (A) coupled shear wall systems with deep coupling beam, (B) coupled shear wall systems designed based on Astaneh-Asl (2001) recommendations, AISC Seismic Provisions AISC (1997) and FEMA-368 Provisions (FEMA 368, 2000) and (C) uncoupled shear wall systems. In each group three different number of stories (i.e. 10, 12 and 15 stories) and three different values of coupling beam bay width (i.e. 3, 4 and 5 meters) are selected. In total, 27 structural models are to be investigated. The code of each model consists of the group code, the number of bays, the number of stories and the width of coupled bay sequentially; for example, A_3_12_4 represent the frame from group A with 3 bays, 12 stories and coupled bay width of 4 m. For instance, the layout of the 10-story frame is shown in Figure 4. In all models, the typical story height and width of bays normal to the frame plane are 3 meters and 4 meters, respectively.

Dead load and live load of floors are 550 kg/m^2 and 200 kg/m^2, respectively, and the partitions have the weight of 130 kg/m^2. The site has high seismic risk (i.e. Zone 3) and subsoil of type III using the standard described in BHRC (2005), (average shear wave velocity from 175 m/s to 375 m/s in 30 meter depth top layer of soil). Equivalent static lateral loads are used in the seismic design procedure (BHRC, 2005) and a preliminary value of 11.2 is used for the response modification factor based on Astaneh-Asl (2001). The modules of elasticity, yield stress and ultimate stress of the structural steel are 2.04×106 kg/cm^2, 2400 kg/cm^2 and 3700 kg/cm^2, respectively. Other assumptions are as follow:

- The steel plates are replaced with a series of truss members (struts) along the tension field.
- All frame members are connected together by rigid connection except in group C that the coupling beams are connected to columns by the simple connection. In steel structures, these connections have considerable moment capacity and behave in a "semi-rigid" manner rather than acting as a pin connection as the current practice assumes.
- For the dynamic analysis, story masses are located in story levels considering the rigidity of floor diaphragms.
- Idealized elastic-plastic behaviour with maximum ductility of 4 and strain hardening of 2% is considered for the members.
- The P-Δ effect is considered to include geometric nonlinearities.

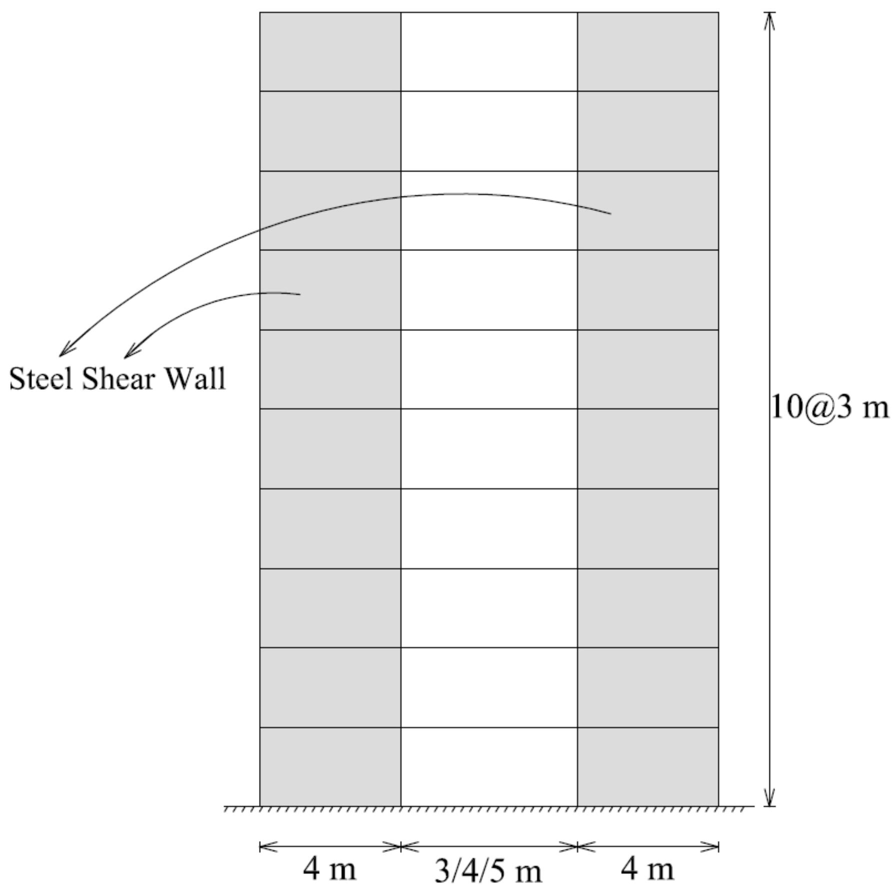

Fig. 4. Layout of the 10- story frame.

NUMERICAL RESULTS

To determine Vs, it is supposed that the linear ultimate limits of the structure in the nonlinear static analysis and nonlinear dynamic analyses are the same Asgarian and Shokrgozar (2009). After modelling the frames, the base shear force is calculated using the equivalent static loading model specified in the Iranian Standard No. 2800 (BHRC, 2005). Using the model, loading is distributed in height of the structures and the structures are then subjected to non-linear static (pushover) analysis.

To determine Vy, three severe Iranian strong ground motion (Table 1) are initially selected. To scale these records with respect to the design spectrum, their PGAs are altered iteratively in a way that the calculated time history makes the structure reach one of following failure criteria:

- Based on Iranian Standard No. 2800 (BHRC, 2005), the maximum inter-story drift is limited to 0.025 H and 0.020 H for the frames with the fundamental period less than 0.7 sec and more than 0.7 sec, respectively, where H is the story height.
- If story mechanism or overall mechanism forms in the frame lead to loss of its stability before reaching the inter-story drift limit, the nonlinear dynamic analysis is terminated, and the last scaled earthquake base shear is selected as the ultimate limit state, Vy.

The models are then analyzed using the incremental nonlinear dynamic analysis under the scaled records and the maximum nonlinear base shear, Vy, is determined under each record. Finally, the maximum linear base shear, Ve, is determined using the linear dynamic analysis of the structure under the same scaled records. The numerical values obtained for Ve and Vy under various earthquake records are shown in Table 2. Averaged values of Ve and Vy under these records are calculated for each model, and the over strength reduction factor, Rs, and ductility reduction factor, Rμ, are determined using Eq. (6).

As the frames are designed based on the preliminary response modification factor, once the tentative values of the factor are obtained, the models are then amended. The modified models are analyzed again to determine new values of response modification factors. This procedure is continued two or three times to determine the final seismic response modification factors. Results are shown in Table 2. The response modification factor of the models in the allowable stress design method, Rw, is determined using a value of 1.4, as recommended in UBC97 (1997) for Y. These results are presented in Table 3. As results indicate, the mean response modification factors are 11.1, 11.6 and 10.6 for structural models of groups A, B and C, respectively.

AstanehAsl (2001) has recommended the value of 11.2 for the response modification factor of dual systems with special steel moment frames and steel plate shear walls, and this value is concordant with the findings of the present research. The value of 10.6 for the response modification factor of the uncoupled steel shear wall systems is almost concordant with the value of 10 presented in the National Building Code of Canada (NBCC, 2005) as the response modification factor of these systems.

Figure 5 shows the ductility reduction factor of studied frames. Generally, the increase of stories or the structure height results in a decrease in the ductility reduction factor. This decrease is more significant in phasing from 12-story frames to 15-story frames than that in phasing from 10-story frames to 12-story frames. This can be attributed to effects of higher modes that exert more lateral loads on higher stories of high rise structures.

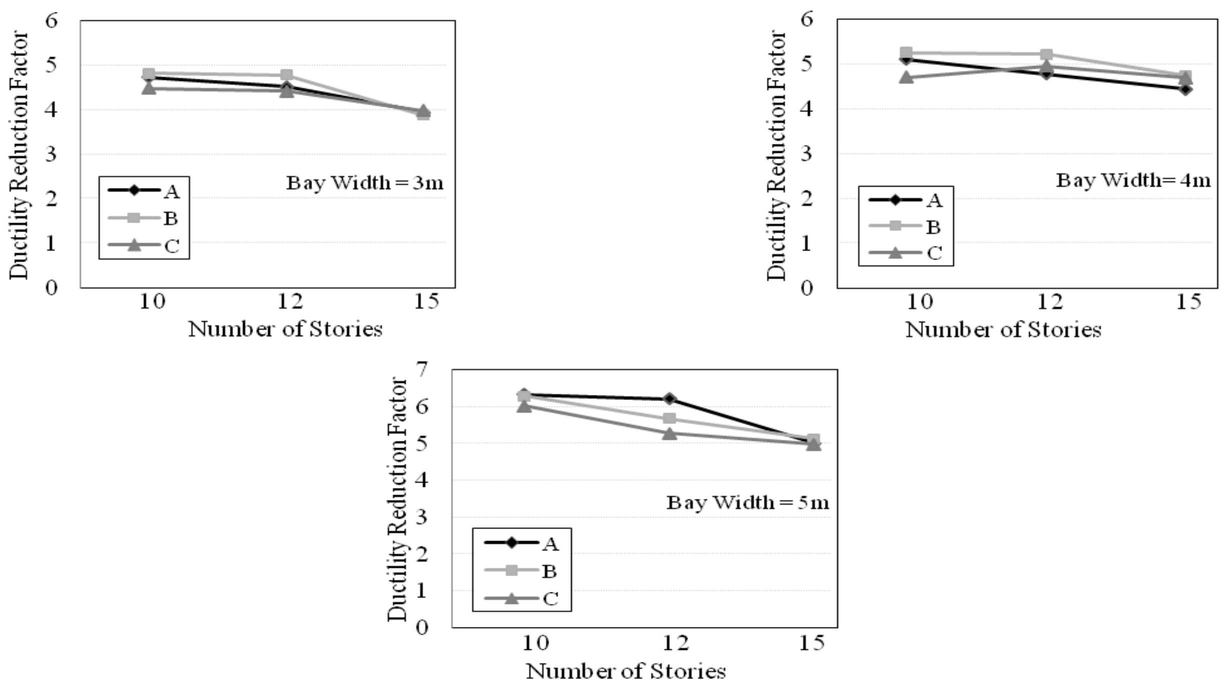

Fig. 5. Variation of ductility reduction factor for different number of stories.

Figure 6 represents the variation of the ductility reduction factor with respect to the coupled bay width. As seen, an increase in the coupled bay width has led to the ductility reduction factor rise in all models. It can be generally concluded that the width of coupled bay has significant effect on the ductility of coupled steel shear walls. Generally, in 10- and 12-story frames, coupled shear walls have more drops in the ductility reduction factor than uncoupled ones. In 15-story frames coupling the shear walls cannot increase the factor significantly except for the frames with the coupled bay width of 4 m. Hence, it can be said that the coupling is more effective in low and midrise frames than in high rise ones. With respect to normal-depth coupling beams (i.e. Category B), using deep coupling beams (i.e. Category A) not only cannot increase the ductility reduction factor in some cases but also leads to the reduction of this factor in most cases. Hence, this technique is not an appropriate choice for the ductility increase of coupled steel shear walls.

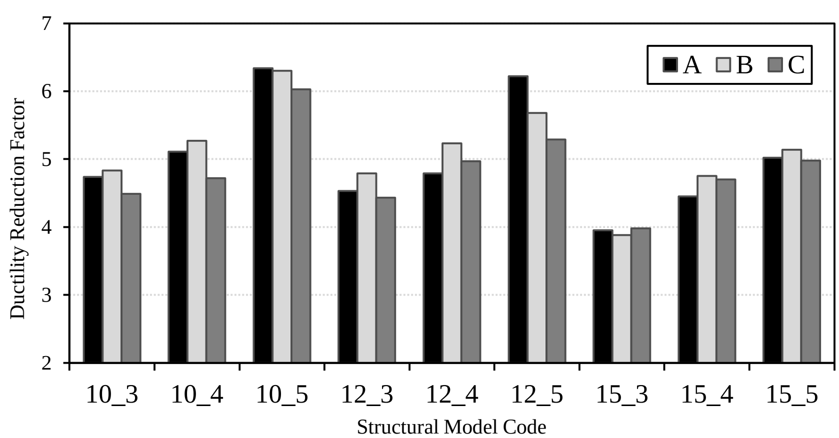

Fig. 6. Ductility reduction factors of different models.

Figure 7 shows the variation of the over strength reduction factor against the number of stories. For the coupled bay width of 3 m, the over strength reduction factor decreases with the increase of the number of stories or height of the structure. For frames with the coupled bay width of 4 m, the over strength reduction factor remains almost unchanged by the increase of the number of stories. In the case of frames with the coupled bay width of 5 m, the increase of the number of stories has resulted in that the over strength reduction factor has gone up. Given the complexity seen in variations of the over-strength reduction factor, the selection of an appropriate value for the coupled bay width is an important issue in the design of coupled steel shear wall systems.

In Figure 8, we can trace the variation of the over strength reduction factor with respect to the coupled bay with. In all models, the over strength reduction factor decreases as the coupled bay width increases. The coupled bay width has more effect on the over strength reduction factor in low-rise frames than that it does in mid and high-rise frames. It can be said that the increase of stories hinders the effect of the coupled bay width on the over strength reduction factor. Similar to the case of the ductility reduction factor, deep coupling beams (i.e. category A) cannot enhance the performance of the models regarding the over strength reduction factor. This fact re-implies the inappropriateness of the deep coupling beams application for improving the performance of coupled steel shear walls.

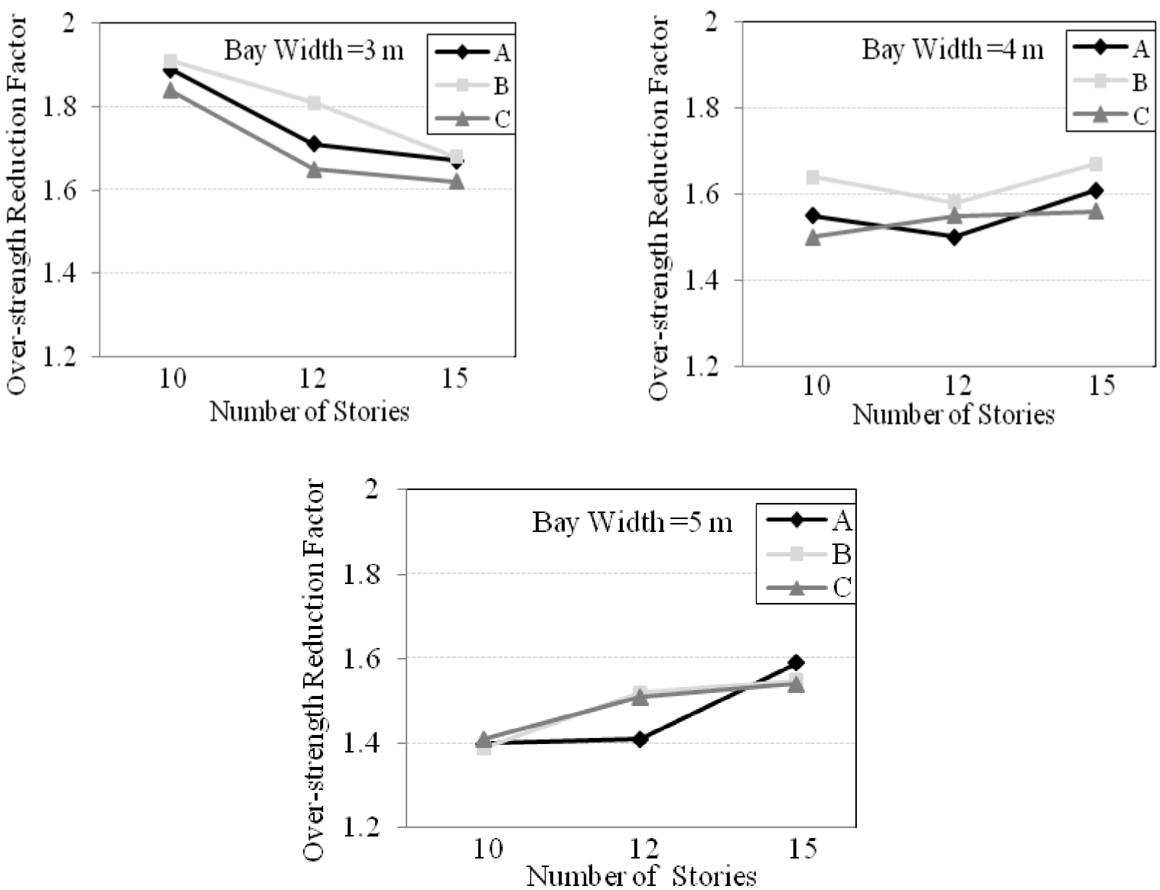

Fig. 7. Variation of over strength reduction factor for different number of stories.

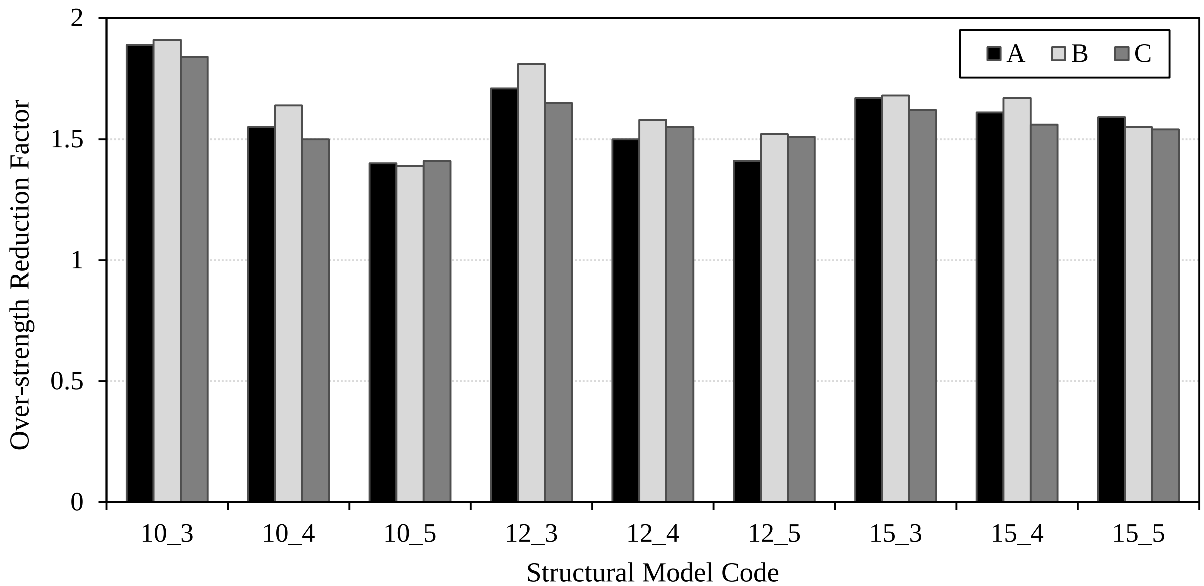

Fig. 8. Over-strength reduction factors of different models.

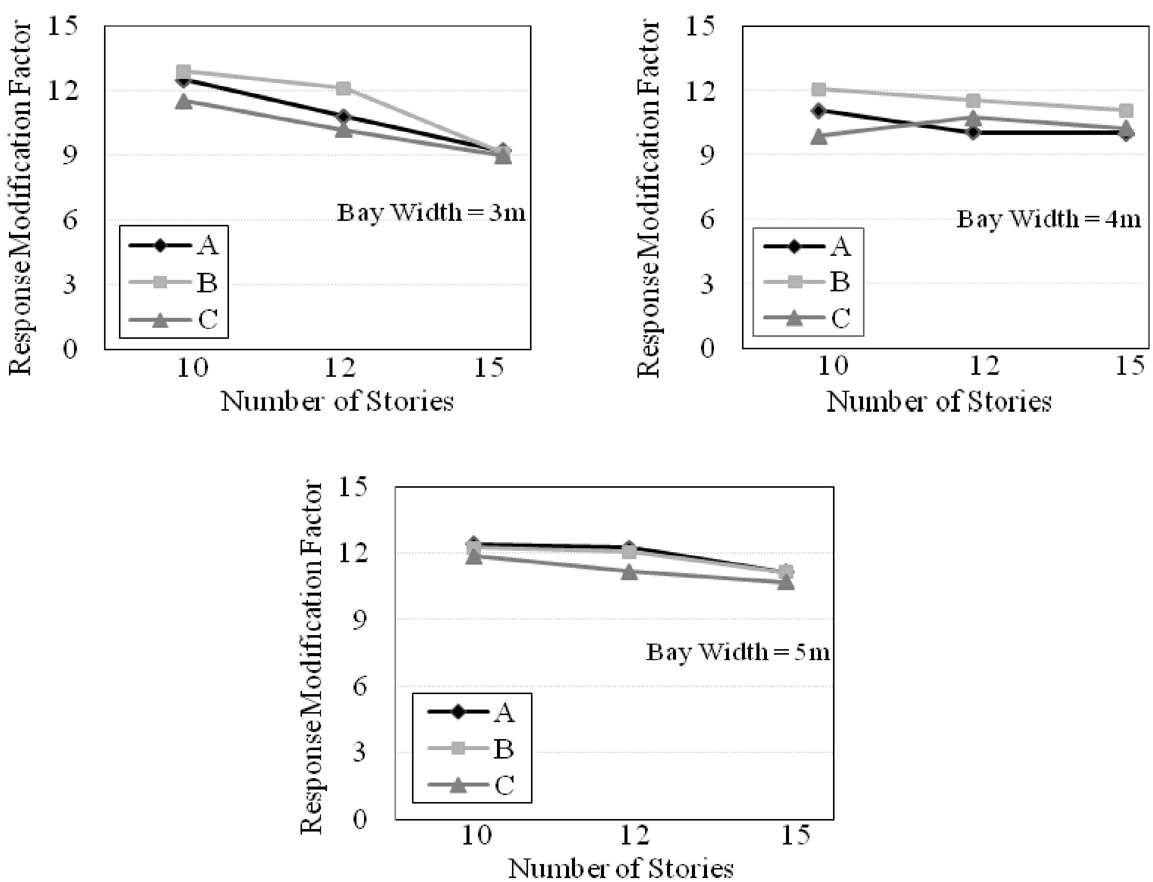

Fig. 9. Variation of response modification factor for different number of stories.

Figure 9 shows the variation of the response modification factor in the allowable stress design method, Rw, with respect to the number of stories. Generally, the increase of the number of stories has decreased the response modification factor in all models except in the models of Category C with the coupled bay width of 4 m. In this special case, the response modification of 10-story frame is lower than those of 12- and 15-story frames. By tracing this issue, it can be seen that similar conditions hold for the ductility reduction factor and over strength reduction factor of these frames. Based on these results, it can be said that the increase of the coupled bay width generally hinders the effect of frame height on the response modification factor.

Figure 10 represents the variation of the response modification factor with respect to the coupled bay width. As seen, the coupled bay width has low effect on this factor; no systematic relation can be recognized between the response modification factor and the coupled bay width. However, in all cases, frames with uncoupled shear walls generally have lower response modification factor than those with coupled shear walls. Also, as mentioned in the case of ductility reduction factor and over-strength reduction factor, using deep coupling beams is not an appropriate technique to improve the performance of coupled steel shear walls.

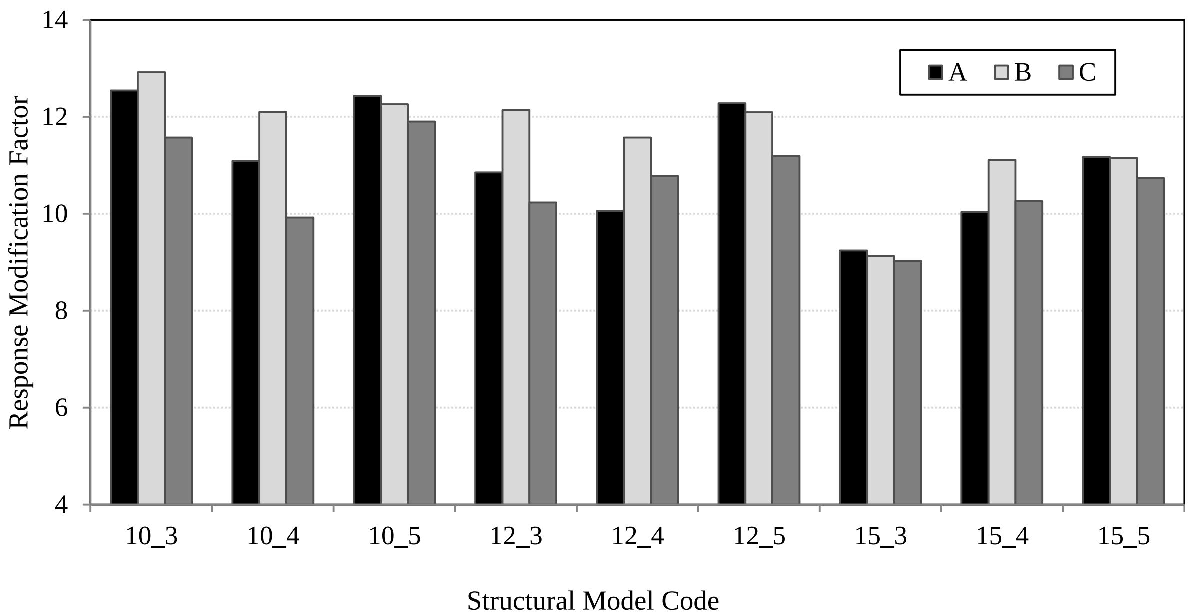

Fig. 10. Response modification factors of different models.

CONCLUSIONS

The ductility reduction factor, over-strength reduction factor and response modification factor of coupled steel shear wall frames and uncoupled ones with various numbers of stories, coupled bay width and height of coupling beam are evaluated using static pushover, linear dynamic and incremental nonlinear dynamic analyses. The results of the present study can be summarized as follows:

- The mean response modification factor of the coupled steel shear walls with deep coupling beam (i.e. Category A) in the allowable stress design method is almost 11.1. This value is almost 11.6 for the mean response modification factor of coupled steel shear walls with normal depth coupling beams (i.e. Category B). The corresponding value of 10.6 is obtained for the uncoupled shear walls (i.e. Category C).

- In each category, for a given bay width of the coupling beam, the response modification factor and ductility reduction factor decrease as the number of stories increases.
- In each category, for a given number of stories, the increase of the coupled bay width increases the ductility reduction factor, but it decreases the over strength reduction factor. Consequently, the response modification factor shows little and unsystematic changes.
- In each category, when the ratio of the frame height to the distance between the two shear walls increases, the response modification factor and ductility reduction factor generally decrease so that 15-story models with the bay width of 3 meters have the minimum values of response modification factor and ductility reduction factor.
- The application of deep coupling beams is not an appropriate technique for the increase of the ductility reduction factor, over strength reduction factor and response reduction factor in none of categories and considered geometries for the frames.

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