Civil Engineering Infrastructures Journal, 49(1): 33 – 43, June 2016
Print ISSN: 2322-2093; Online ISSN: 2423-6691
DOI: 10.7508/ceij.2016.01.003

The Effect of Spandrel Beam's Specification on Response Modification Factor of Concrete Coupled Shear Walls

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Received: 14 Dec. 2014; Revised: 06 Sep. 2015; Accepted: 12 Sep. 2015

ABSTRACT: Response modification factor (R factor) is one of the seismic design parameters to be considered in evaluating the performance of buildings during strong motions. This paper has tried to evaluate the response modification factor of concrete coupled shear wall structures with various length/depth ratios of spandrel beams. The effect of diagonal reinforcement of spandrel beam was also evaluated on the R factor. The R factor directly depends on overstrength factor and ductility reduction factor. For this purpose, three conventional structures with 5, 10 and 15 story buildings (having various spandrel beam's length/depth ratio with and without diagonal reinforcement) were selected and the nonlinear static analyses were conducted to evaluate their overstrength and ductility reduction factors. Also for a 5-story structure, nonlinear dynamic analysis (time history) was carried out in order to compare the results with nonlinear static analysis. It was concluded that the R factors using nonlinear time history analysis and nonlinear static analysis are almost the same. The results also indicate that by increasing the height of the structure, the overstrength reduction factor decreases; while the ductility reduction factor increases. Also, the response modification factor decreases with increasing length/depth ratio of spandrel beams. The coupled shear walls with diagonal reinforcement in spandrel beams have a greater R factor.

Keywords: Concrete Coupled Shear Wall, Ductility Reduction Factor, Response Modification Factor, Overstrength Factor, Spandrel Beam.

INTRODUCTION

Shear walls are generally used in multi-story buildings because of good performance under lateral loads due to earthquakes. Coupled shear walls, which are special cases of shear wall systems, are an effective earthquake-resisting structure with high stiffness and acceptable ductility due to their short span tie beams (Doran, 2003). This leads to optimal use of two adjacent shear walls. In most cases, regular openings for windows or doors in walls are inevitable. Localization of openings is such that the structural behavior of shear walls to bearing loads is desirable. It is required that the general behavior and flexural behavior of walls do not face difficulty with a significant

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decrease in cross sectional area; because the walls become brittle and can collapse before it reaches the maximum flexural strength due to shear failure. In shear walls with openings, if the wall has one or more openings at its bottom, each of the wall components located in the side openings called the shear wall pier and a part of the wall between the upper and lower openings or between the piers called spandrel beam (Figure 1).

Earthquake energy absorption by the coupled shear walls places a large demand on the spandrel beams. It is therefore concluded that in the design of shear walls, one needs to place and predict the plastic section (plastic hinge) in the flexural behavior so that no failure happens in the spandrel beam and in the walls.

Fig. 1. Spandrel beams in coupled shear walls

The benefits of coupled shear walls in terms of ductility include:

- Excellent control of lateral displacement.
- Strongly coupled systems, enabling the use of thinner walls.
- Deformation limits during ductile response are not affected by high dynamic modes.
- An appropriate reinforcement shows greater hysteretic damping than conventional buildings.

Tasnimi and Kiarash (2009) studied the effects of openings at various levels of shear wall in dual structure systems. They concluded that due to some considerations such as architectural requirements, the necessity of openings may cause less effective cross section resistance to the shear and it is possible that the limited lateral displacement of the code requirements cannot be satisfied. Therefore, openings can have a considerable effect on seismic behavior. In their study, they compared incremental nonlinear analysis, capacity curve, performance level and the target point of 12 selected structures based on UBC (1997) and Iranian seismic code (Standard no. 2800) (2005) demands. For this purpose, 12 frames with 8 to 20-stories with 3 spans having coupled shear walls were studied. The results indicated that the structures having more than 10 stories with openings of more than 10% of the wall area cannot provide the life safety performance level criteria of FEMA-356 regulations (2000). However, for structures with shallow spandrel beams, acceptance criteria of their performance levels are deferred and even with 20% openings, shear wall can provide life safety performance.

Shahbakhti and Heshmati (2007) studied the effect of reinforcement on ductility and response modification factor of concrete shear walls with openings. The results indicated that increase in reinforcement rate increases the ductility of walls. Response modification factor of shear walls with minimum reinforcement in walls of 8 and 12-story buildings were about the same as 4 and 5-story buildings. It was stated that the amount of longitudinal reinforcement used in the walls, (especially in high rise structures), has a significant effect on their strength and ductility. They demonstrated that in walls with lower percentage reinforcement, when the base shear increased then tensile failure in the foundation level and lower story occurred; but in the walls with higher percentage reinforcement, the piers resist well till the fracture of the coupled beams occurs.

Hosseini et al. (2011) presented a comparison between the nonlinear behavior of steel and concrete spandrel beams in coupled shear wall systems using Finite Element analysis. In this study, due to the role spandrel beam plays in seismic behavior of coupled shear walls, a prefabricated concrete beams as link beam of the coupled shear wall system previously tested under cyclic load were analyzed. Then they replaced that with a steel spandrel beam and reanalyzed to specify any discrepancy between cyclic behavior of the concrete spandrel beam and steel spandrel beam. Steel spandrel beam, with and without stiffener were used in order to observe the seismic behavior improvements. The results indicated that the steel spandrel beams are better in absorbing energy up to 3 times more than concrete coupling ones. The use of stiffener in steel spandrel beam has little effect on their hysteretic behavior and ability for energy absorption with only 10% improvement.

Bazargani and Adebar (2015) calibrated a nonlinear Finite Element model with experimental results and confirmed that large shear strains occur in flexural tension regions of concrete walls due to vertical tension strains in the presence of diagonal cracks and in the absence of demand on horizontal shear reinforcement.

Baradaran et al. (2014) studied the performance of shear wall building with gravity-induced lateral demands.

Abdollahzadeh and Malekzadeh (2013) determined the ductility, over-strength and response modification factors of coupled steel shear wall frames using static pushover and incremental nonlinear dynamic analyses.

Ranjbar et al. (2013) analyzed, a reinforced concrete elevated water tank (similar to shear wall) of 900 cubic meters capacity, exposed to three pairs of earthquake records in time history using mechanical and Finite Element modeling techniques.

In this regard, Meftah and Mohri (2013), McGinnis et al. (2013), Hadidi et al. (2003), Abdollahzadeh and Malekzadeh (2013), Eljadei (2012), Bhunia et al. (2013) and Khatami et al. (2012) assessed the behavior of shear coupled walls under static and dynamic loads.

Response modification factor (R factor) is one of the seismic design parameters to be considered in the nonlinear performance of building structures during strong motions. Mahmoudi (2003) evaluated the relationship between overstrength and members ductility of RC moment resisting frames by static nonlinear analysis. Also Mahmoudi and Zaree (2010, 2011) evaluated the response modification factors of concentrically braced steel frames.

Response modification factor in seismic design plays an important role and basic design philosophy is based on it, but regulations are not sufficiently accurate in determining the minimum values, which in some cases, may cause uncertainty in seismic design. In other words, we cannot ensure that using these factors will result in an appropriate design. In appropriated design, the structures meet seismic requirements in strong earthquakes such as ductility and good resistance. Today, many researches on this case have been carried out. Generally, the response modification factor in earthquake codes in different countries is set based on engineering judgment and understanding the behavior of the members from these structures which experience different earthquakes and still, no regulations mentioned a specific method to determine response modification factor. Some parameters under different conditions have different effects on this factor.

The aim of this study was to determine a reasonable value for response modification factor of buildings with coupled shear wall system of three structures specifically with 5, 10 and 15- stories; as an example of short, medium and tall buildings. Each response modification factor was determined using a nonlinear static analysis to be compared with the value in codes.

Structural Models

To accomplish the study, in the first phase, three concrete structures of 5, 10 and 15 stories were designed according to Iranian National Building Code (part 9): concrete structures design (2009). The models used in this research are three residential buildings with similar plan as shown in Figure 2. Story height at ground story is 3.3 m and for others is 3 m. Iranian National Building Code (part 6): loading (2009) and Iranian code of practice for seismic resistance design of buildings, Standard no. 2800 (2005) were used for gravity and seismic loading of structures.

Shear walls are located in long direction of the structure (Figure 2) and the earthquake load is considered in the long direction only. Thus the earthquake load is applied in one direction but the structure is 3-dimensionaly analyzed.

It was assumed that the material specifications of concrete are as follows: f_c = 210 Kg/cm^2 and $E = 2.1\times10^6$ Kg/cm^2. The members' geometrical specifications of the shear walls are shown in Table 1.

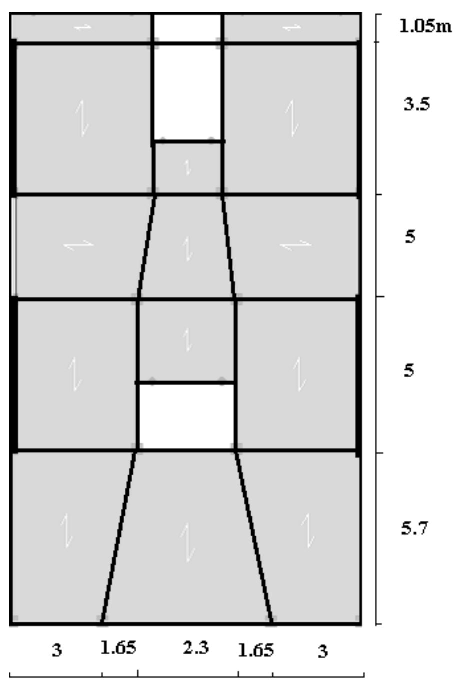

Fig. 2. Plan of 5, 10 and 15 story models (all dimensions in meters)

Table 1. Geometrical specifications of the shear walls for 5-story building

Number of Story	Thickness (cm)	Length (cm)	Height (m)	Vertical Reinforcement (%)	Horizontal Reinforcement (%)
1	40	460	3.3	0.0035	0.0042
2	40	460	3	0.0035	0.0042
3	40	460	3	0.0035	0.0042
4	30	460	3	0.003	0.0035
5	30	460	3	0.003	0.0035

The load-deflection model used for nonlinear analysis is presented in Figure 3. In this figure, IO represents immediate occupancy; LS represents life safety and CP represents collapse prevention.

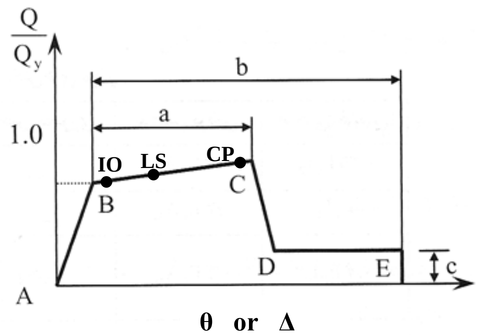

Fig. 3. Load-deflection model of members

METHODOLOGY

To evaluate response modification factors, nonlinear static (pushover) analysis is performed by subjecting a structure to monotonically increasing lateral forces in an invariant height-wise distribution. The models selected in the previous section were first analyzed under pushover analysis, by Perform-3D software (2006). When the first plastic joint reached life safety performance level (building performance that includes significant damage to both structural and non-structural elements during design earthquakes in one of the structural members), the analysis was stopped. The roof displacements versus base shear curves were designed for each of the models. The roof displacement curves versus base shear (pushover curve), was smoothed to a bilinear diagram to interpret the results. The parameters (Δ_d, Δ_y and Δ_{max}) needed for determination of R factors was estimated using the curve shown in Figure 4. Δ_{max} is related to roof displacement achieving first structural element to a level of life safety performance, Δ_y is roof displacement as the initial yield observed in the structure and value of Δ_d is the roof displacement in design base shear level.

According to Eq. (1) the response modification factor of the structures will be obtained by multiplying R_S and R_μ (Whittaker et al. 1999):

$$R = R_s \times R_\mu \times R_r \tag{1}$$

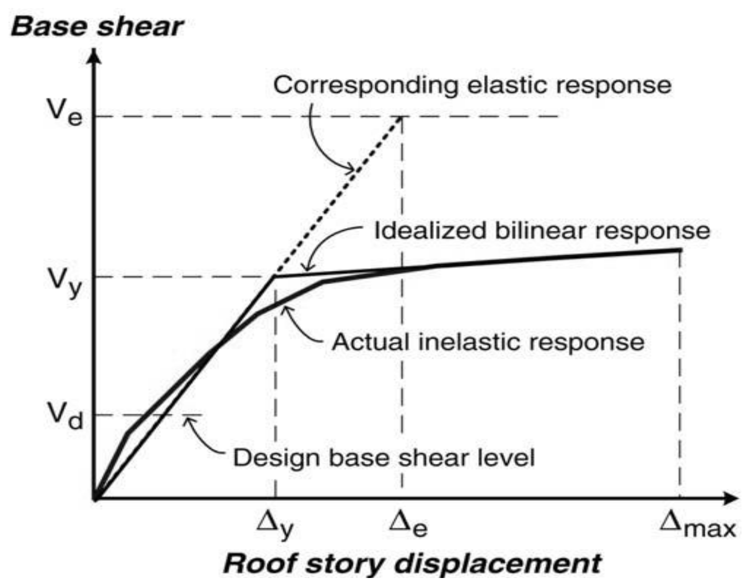

Fig. 4. Lateral load-roof displacement relationship of a structure (pushover curve)

where R_S: is the overstrength factor, $R\mu$: is the ductility reduction factor and R_r: is the redundancy factor to quantify improved reliability of seismic framing systems constructed with multiple lines of strength. Values of R_S are determined in accordance with Eq. (2) and the values of R_μ is determined according to the Newmark-Hall (1970) relationship (Eqs. (3-5)). In this regard, μ (ductility factor) is determined according to Eq. (6). In this study, it is assumed that the redundancy factor is equal to 1.0 (Borzi and Elnashai, 2000).

$$R_s = \frac{\Delta_y}{\Delta_d} = \frac{V_y}{V_d} \tag{2}$$

$$R_\mu = 1 \qquad T < 0.03 \text{ sec} \tag{3}$$

$$R_\mu = \sqrt{2\mu - 1} \qquad 0.03 < T < 0.12 \text{ sec} \tag{4}$$

$$R_\mu = \mu \qquad T > 0.12 \text{ sec} \tag{5}$$

$$\mu = \frac{\Delta_{\max}}{\Delta_y} \tag{6}$$

In the Eq. (2) V_d: is the design base shear level and V_y: is the base shear corresponding to overall yielding of the structure.

Verifying the Results

In order to verify the results, the results of nonlinear time history dynamic analyses were compared to nonlinear static analysis. For this reason, the 5 story building model was subjected to Northridge, Cape Mendocino and San Fernando acceleration time history records. The specifications of these acceleration time history records are shown in Table 2.

To achieve this aim, at first the structure subjected to the above records is analyzed by nonlinear time history till the first hinge limit of life safety (LS) in the structure is formed. Then structure subjected to the above records is analyzed by linear time history dynamic analysis. Having base shear resulting from the above mentioned analysis the response modification factor of a structure can be determined as follows:

$$R = R_{sd} \times R_\mu \tag{7}$$

where R_{sd}: is the overstrength factor due to nonlinear time history analysis which is the same as pushover analysis method (Eq. 8).

$$R_{sd} = \frac{V_y}{V_d} \tag{8}$$

where V_d: is base design shear that the structure designed is based on and V_y base shear related to first member reaches life safety performance level.

To calculate ductility reduction factor, in the dynamic time history analysis method, the following equation is used:

$$R_\mu = \frac{V_e}{V_y} \tag{9}$$

where V_e: is the base shear which is obtained from the dynamic analysis of linear time history analysis. V_y: is the base shear related to the first formation of LS performance in members in nonlinear time history analysis.

Table 2. Characteristics of the acceleration time history records

Earthquake Total Time	Maximum Displacement	Maximum Velocity	Maximum Acceleration	Distance to Fault	Earthquake Magnitude	Date of Event	Name
sec	PGD (cm)	PGV (cm/s)	PGA (g)	km	M		
29.97	2.45	15.7	0.324	24.9	6.6	09/02/1971	San Fernando
39.94	9.55	51.9	0.568	22.6	6.7	17/01/1994	Northridge
35.94	1.161	6.89	0.229	33.8	7.1	25/04/1992	Cape Mendocino

RESULTS

After modeling and analyzing the structures, the roof displacement versus the base shear diagrams for each structure (5, 10 and 15 story) are shown in Figures 5-7, respectively. In these Figures, the horizontal axis shows the roof displacement of the structures and the vertical one shows the base shear.

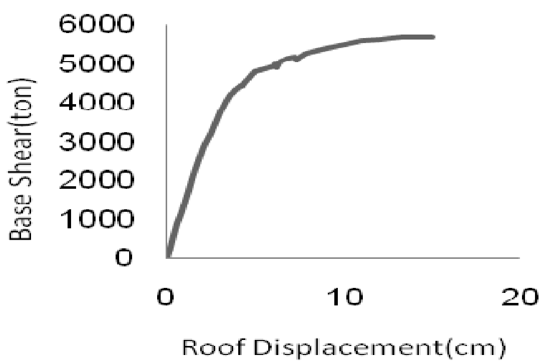

Fig. 5. Base shear - roof displacement curve for five story structure

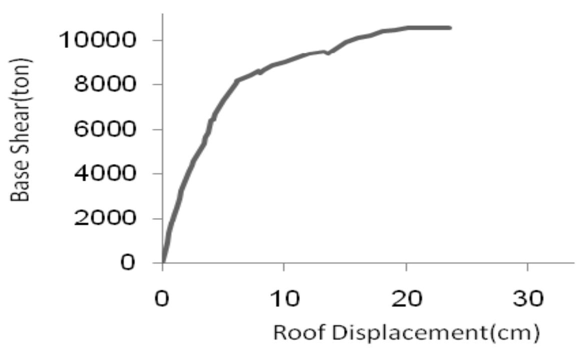

Fig. 6. Base shear - roof displacement curve for ten story structure

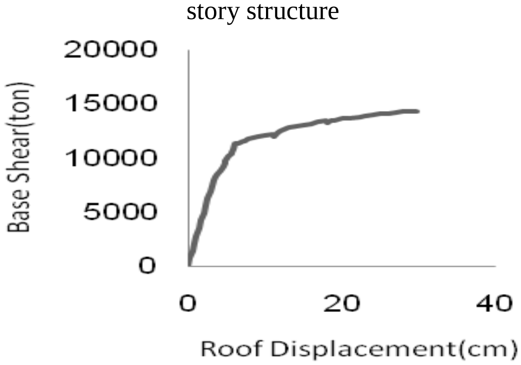

Fig. 7. Base shear - roof displacement curve for fifteen story structure

Finally, by making these diagrams into bilinear diagrams, the values of roof displacement, overstrength factor, ductility reduction factor and overall structural response modification factor for 5, 10 and 15 story structures are obtained using pushover analysis as shown in Table 3.

Using nonlinear time history dynamic analysis, the overstrength reduction factor, ductility reduction factor and final response modification factor for 5 story structure are calculated and shown in Table 4. As shown in Table 4, all response modification factors that belong to the records are approximately the same but the lowest response belongs to the second record.

Comparing the results in Tables 3 and Table 4 for 5 story building, shows that the R factor determined using pushover analysis, being 9.43 is approximately the same as factor obtained from time history analysis, being 9.67 (resulted from the second record) with low different. This issue arises in both methods the V_d factors are the same but the V_y and V_e factors are different. The V_y and V_e factors in time history analysis depend on earthquake record contents such as frequency of time-acceleration records. Also the R_μ factors in pushover analysis are calculated using approximate formula but in time history analysis are determined directly using Eq. (9).

The Effect of Structures' Height on R Factor

The changing trend of ductility reduction factor (R_μ), overstrength (R_s) and the overall response modification factor of the coupled shear wall structures with varying heights are shown in Figure 8. As shown, increasing the height of the structure decreases the response modification factor and overstrength reduction factor (R_s) and increase ductility reduction factor (R_μ).

Table 3. Response modification factors from pushover analysis of the structures

Number of Stories	Δ_d (cm)	Δ_y (cm)	Δ_{max} (cm)	R_s	R_μ	R
5 story	1.4	4	15	2.86	3.3	9.43
10 story	2.46	5.8	23.5	2.35	3.85	9.08
15 story	3.6	7.2	29.8	2	4.13	8.26

Table 4. Response modification factors for 5 story structure from time history analysis

Earthquake	V_d (ton)	V_y (ton)	V_e (ton)	R_s	R_μ	R
Northridge	1731	4900	17105	2.83	3.49	9.87
Cape Mendocino	1731	4765	16820	2.75	3.52	9.67
San Fernando	1731	4960	17350	2.86	3.49	9.97

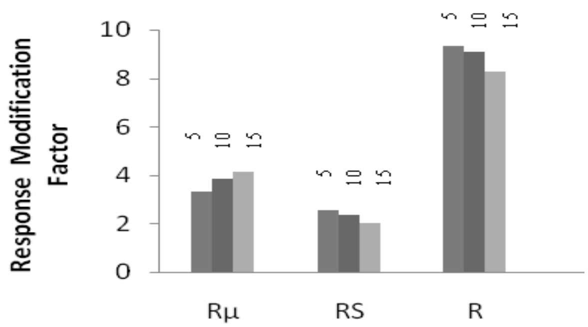

Fig. 8. Response modification factors due to increasing height of structures

Abdollahzadeh and Malekzadeh (2013) evaluated the response modification factor of the coupled steel shear walls with deep spandrel beam. They proposed a quantity of 11 for response modification factor of coupled steel shear walls. Comparing the results of R factor of concrete shear wall (about 9) with steel one show that the coupled steel shear walls has more ductility than coupled concrete shear walls.

Mahmoudi (2003) evaluated the overstrength factor for R/C moment resisting frames and find out that the over strength of factor decreases when the height of the structures increase. Mahmoudi and Zare (2010) studied the variation in R factors for concentrically braced steel frames and they showed that with increasing the height of the structure, the R factor decreases. The same results were extracted in this research.

Comparing the response modification factor of buildings with coupled shear wall and response modification factor set out in standard 2800, it can be concluded that, the guidelines set for these structures should provide new values for the response modification factor.

The Effect of Length to Spandrel Beam's Depth Ratio and Diagonal Bracing Ratio on R Factor

To evaluate the effect of length on spandrel beam depth ratio, and also in order to show the presence or absence of diagonal reinforcement (Figure 9) on the response modification factor, the following spandrel beams in a 5 story structure are implemented:

- Spandrel beam with length to depth ratio of approximately 3 (2.67), using diagonal reinforcement that its response modification factor is equal to 9.43.
- Spandrel beam with length to depth ratio of approximately 3 (2.67), without diagonal reinforcement.
- Spandrel beam with length to depth ratio of approximately 2, using diagonal reinforcement.

- Spandrel beam with length to depth ratio of approximately 2, without diagonal reinforcement.
- Structure with non- deep beams.

With nonlinear static analysis, the response modification factor for each mode was determined as illustrated in Tables 5 and 6.

Table 5 shows that when the Length/depth increases, the R factor decreases. Also it is concluded that the coupled shear walls with diagonal reinforcement have a high response modification factor. Comparing Tables 5 and Table 6, it can be seen that deep beam in couple shear wall increases the R factor.

The response modification factor variation for nonlinear static analysis and 3 records of time history analysis are shown in Figure 10.

The response modification factor variation with respect to the length to beam depth ratio and the presence or absence of diagonal reinforcement in spandrel beam is shown in Figure 11.

Fig. 9. Diagonal reinforcement bracing in spandrel beam

Table 5. Response modification factor for 5 story structure with deep beams

Length/Depth	Diagonal Reinforcement	Δ_d (cm)	Δ_y (cm)	Δ_{max} (cm)	R_s	R_μ	R
2.67	with	1.4	4	15	2.86	3.3	9.43
2.67	without	1.4	3.8	12.2	2.71	2.9	7.86
2	with	1.4	4.3	13.5	3.07	2.82	8.65
2	without	1.4	3.7	11.6	2.64	2.75	7.26

Table 6. Response modification factor for 5 story structure with non-deep beams

Δ_s (cm)	Δ_y (cm)	Δ_{max} (cm)	R_s	R_μ	R
1.4	3.5	12.8	2.5	2.95	7.37

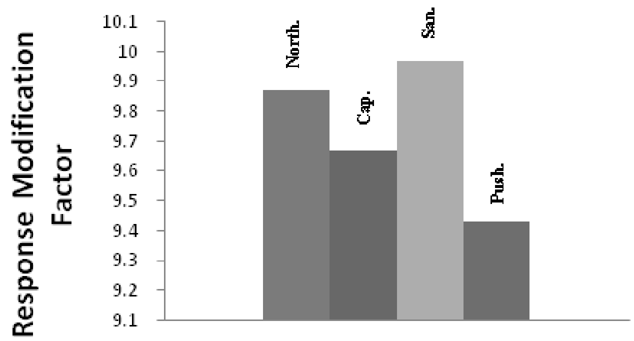

Fig. 10. Comparison of the response modification factors due to different records

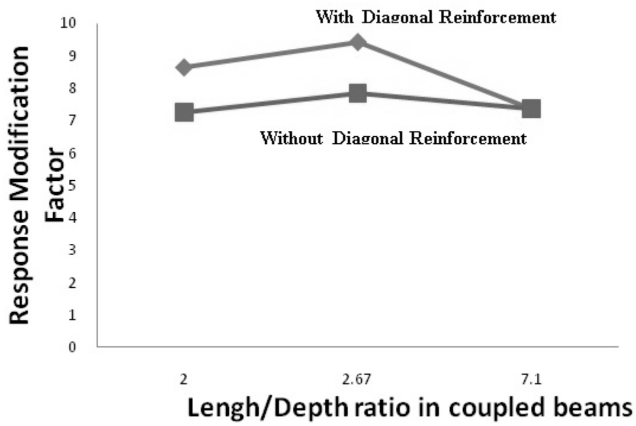

Fig. 11. Variation of the response modification factors

CONCLUSIONS

This paper has tried to evaluate the response modification factor of concrete coupled shear wall structures at various length/depth ratios of spandrel beams. The effect of diagonal reinforcement of spandrel beam was also evaluated on the R factor.

1. It was observed that by increasing the height of the structure, the ductility reduction factor (R_μ) increases and the overstrength factor (R_s) decreases. Therefore the response modification factor decreases from 9.43 for a 5 story structure to 8.26 for a 15 story structure.
2. Comparing the response modification factor of buildings with coupled shear wall and response modification factor set out in standard 2800, it can be concluded that the guidelines set for these structures should provide new value for the response modification factor.
3. Response modification factor estimated by nonlinear time history analysis method for 5 story structures is equal to 9.97, which this value is approximately equals to resulted value of the nonlinear static analysis, 9.43.
4. The response modification factor decreases with increasing length/depth ratio of spandrel beams.
5. The coupled shear walls with diagonal reinforcement in spandrel beams have greater R factor.

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