Steel and Composite Structures, *Vol. 20, No. 2 (2016) 337-355*
DOI: http://dx.doi.org/10.12989/scs.2016.20.2.337

Performance based evaluation of RC coupled shear wall system with steel coupling beam

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(*Received January 18, 2015, Revised July 29, 2015, Accepted September 13, 2015*)

Abstract. Steel coupling beam in reinforced concrete (RC) coupled shear wall system is a proper substitute for deep concrete coupling beam. Previous studies have shown that RC coupled walls with steel or concrete coupling beam designed with strength-based design approach, may not guarantee a ductile behavior of a coupled shear wall system. Therefore, seismic performance evaluation of RC coupled shear wall with steel or concrete coupling beam designed based on a strength-based design approach is essential. In this paper first, buildings with 7, 14 and 21 stories containing RC coupled shear wall system with concrete and steel coupling beams were designed with strength-based design approach, then performance level of these buildings were evaluated under two spectrum; Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE). The performance level of LS and CP of all buildings were satisfied under DBE and MCE respectively. In spite of the steel coupling beam, concrete coupling beam in RC coupled shear wall acts like a fuse under strong ground motion.

Keywords: reinforced concrete coupled shear wall; concrete and steel coupling beam; performance based evaluation; nonlinear analysis

1. Introduction

RC moment resisting frames accompanied with RC shear walls are popular in the high-rise structures. Shear walls are structures which provide resistance against lateral loads and their position with architectural and installation requirements lead to repeated openings from floor to floor throughout the height of the system and result is isolated walls connected by coupling beams. Coupling beams provide a transfer of vertical forces between adjacent walls, which creates a coupling action resisting a portion of the total overturning moment induced by the base shear (El-Tawil *et al*. 2002). This coupling action has two useful effects; it reduces the moments that must be resisted by the individual walls and therefore results in a more efficient structural system at elastic state. Thus it provides a mean by which energy is dissipated over the height of the wall system as the coupling beams undergo inelastic deformations (Aristizabal-Ochoa 1987). Coupling beam must behave in a ductile manner, yield before the wall piers and exhibit significant energy dissipation characteristics. Therefore, coupling beams should be designed to avoid over coupling,

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http://www.techno-press.org/?journal=scs&subpage=6 ISSN: 1229-9367 (Print), 1598-6233 (Online)

which causes the system to act as a single wall. Also, light coupling should be avoided as it causes the system to behave like two isolated walls (Aristizabal-Ochoa 1982, 1987, Aktan and Bertero 1981, 1984, 1987, Lybas and Sozen 1977, Shiu *et al*. 1981, 1984). Several researchers (Paulay and Binney 1974, Robert and Paulay 1975) have investigated for improving the energy absorption capacity and ductility of reinforced concrete coupling beams. For span-to-depth ratios less than 2, because of shear behavior and high energy absorption, a method using specially detailed diagonal reinforcement, developed by Paulay and Binney (1974) and Robert and Paulay (1975), but this detail may be very difficult to construct. In order to possess stable hysteretic response of RC coupling beams under seismic loading, a high level of detailing, including confinement of beam concrete and adequate containment of steel reinforcement in the connected walls, must be provided (Shahrooz *et al*. 1993). This leads to deep beams with heavy reinforcement, requiring extra formwork and labor in construction. For this reason, different techniques were proposed instead of conventional coupling beams (Shahrooz *et al*. 1993, Harries *et al*. 1992a, b, El-Tawil and Kuenzli 2002, Harries *et al*. 2000, Harries 2001, Park and Yun 2005, El-Tawil *et al*. 2010, Yahya and Qiang 2008, Nie *et al*. 2014, Khalifa 2014). Some researchers have turned to steel coupling beams, with their ends embedded in the two adjacent walls, instead of RC coupling beams (Shahrooz *et al*. 1993, Harries *et al*. 1992a, b, El-Tawil and Kuenzli 2002, Harries 2001, Park and Yun 2005, El-Tawil *et al*. 2010, Fortney *et al*. 2007). Steel coupling beams possess the necessary combination of ductility, strength and stiffness, needed for providing best overall structural performance and the suitable hysteretic response and also provide a permanent alternative to reinforced concrete coupling beams that can be replaced after a severe earthquake. Furthermore, the advantages of steel coupling beams become apparent in cases either where height restrictions do not allow the use of deep reinforced concrete coupling beams or where the required stiffness and capacities cannot be economically obtained by concrete coupling beams. Coupling beams may be detailed to dissipate more portion of the input energy by flexure or shear, depending on the coupling beam length. Also it is more advantageous to design them as shear yielding members or shear critical criteria, since such members have more desirable energy dissipation; such a choice is not possible for reinforced concrete coupling beams. El-Tawil *et al*. (2010) developed design recommendations for steel coupling beams in RC shear wall.

In order to ensure the desired plastic mechanism, i.e., that the steel or concrete coupling beams yield prior to the wall piers, the walls must be stronger than the beams that frame into them. This is similar to the preferred weak beam-strong column behavior of ductile frames. Previous studies have shown that RC coupled walls with steel or concrete coupling beam designed with strength-based design approach, may not guarantee a ductile behavior of a coupled shear wall system (Harries and McNiece 2006, Cheng *et al*. 2015). A viable alternative to strength based design approach is a performance based design approach to allow controlled non-linearity in specified structural members as long as certain structural and element performance criteria are satisfied. Analysis in performance based design is nonlinear. The nonlinear static (Pushover) analysis is taken into account a promising tool for seismic performance evaluation of existing and new structures (Krawinkler and Seneviratna 1998, Inel and Ozmen 2006, ATC-40 1996, FEMA-273 1997, FEMA-356 2000). Pushover analysis gives an estimation of seismic capacity of the structural system and its components based on the material characteristics and detailing of member dimensions. This information can be used to check the specified performance criteria (Raju *et al*. 2012). Modelling the inelastic behavior of the structural members for different levels of performance (Immediate Occupancy (IO), Life Safety (LS), Collapse prevention (CP)) which is an important step towards performance evaluation of building. In most building code applications, for instance, the desired performance of a structure is that it will satisfy LS requirements at the design-level earthquake (10% probability of exceedance in 50 years (DBE)) and collapse prevention (CP) requirements at the maximum Considered Earthquake (2% in 50 years (MCE)).

In this paper, nonlinear behavior of short building to tall building with RC coupled shear wall with concrete or steel coupling beam as their lateral bearing system were evaluated at different levels of performance.

2. Coupling beams

Coupling beams can be subjected to high loading and rotational demands, under lateral loads (i.e., earthquake or wind). Conventionally RC coupling beams with longitudinal flexural and transverse shear reinforcement may be inadequate due to brittle failures in the form of diagonal or sliding cracking (Su *et al*. 2009). A number of coupling beam designs, such as diagonally reinforced concrete coupling beams (Paulay and Binney 1974, Robert and Paulay 1975, Subedi *et al*. 1999, Kwan and Zhao 2002, Harries and McNeice 2006), steel coupling beams (Park and Yun 2005, Harries *et al*. 1993, Hosseini *et al*. 2011) based on strength approach were proposed. The degree of coupling is a function of the strength and relative stiffness of the beam and wall. Coupling individual flexural walls causes the lateral load resisting behavior changes to one, where overturning moments are resisted partly by an axial compression-tension couple, across the wall rather than by the individual flexural action of the walls. So coupling beams act like a fuse, they will tolerate until serve earthquake. But, in strong ground motion, they are not expected to behave rigid; even coupling beams shall be flexible to dissipate energy (Harries 1999, Saatcioglu *et al*. 1987).

As mentioned above, total resistant moment of coupled shear wall system depends on coupling ratio. Coupling ratio (CR) is defined as follows, Eq. (1)

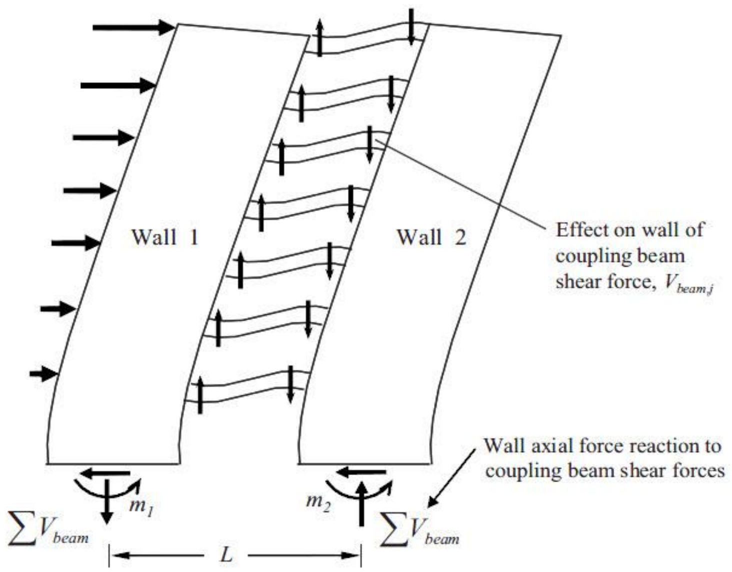


Fig. 1 Definition of the CR (El-Tawil *et al*. 2010)

$$CR = \frac{L\sum V_{beam}}{L\sum V_{beam} + \sum m_i} \tag{1}$$

Where ΣV_{beam} is accumulation of coupling beam shears acting on each wall pier, L is lever arm between the centroids of the wall piers, m_i is individual wall pier moment reaction (see Fig. 1). CR range was recommended between 20% and 55% as an efficient structural design (El-Tawil *et al*. 2010, Lequesne 2011, Cheng *et al*. 2015).

As mentioned above, since shear forces for seismic design of RC coupling beams are carried by diagonal reinforcement, details of RC coupling beams may be very difficult to construct. Consequently, steel coupling beam is suggested instead. Steel coupling beams have similar behavior and provide the same structural role as link beams in eccentrically braced frames (EBF).

3. Steel coupling beam

As noted earlier, it is more advantageous to design the steel coupling beams as shear-yielding members since a shear-critical steel coupling beam exhibits a more desirable mode of energy dissipation than a flexure-critical steel coupling beam. Therefore, in this research, the coupling beams are designed to yield in shear according to the method proposed by Harries *et al*. (1993), in conjunction with the AISC Seismic Provisions (American Institute for Steel Construction 2010), for shear links in an eccentrically braced frame. The steel coupling beam should be embedded in the wall to control cracking therefore its capacity can be developed. Number of methods may be used to calculate the necessary embedment length (Marcakis and Mitchell 1980, Mattock and Gaafar 1982). The equations proposed by Marcakis and Mitchell generally result in slightly longer embedment lengths.

3.1 Basis of design provision

Links are “fuse” elements of frame, the link rotation angle (γ_p) is the inelastic angle between the link and the beam outside of the link, when the total story drift is equal to the design story drift, Δ. The link rotation angle shall not exceed the following value: for links of length 1.6 M_p/V_p or less: 0.08 rad and for links of length 2.6 M_p/V_p or greater: 0.02 rad. where M_p is nominal plastic flextural strength, V_p is nominal shear strength of an active link. Linear interpolation between the above values is used as links of length between 1.6 M_p/V_p and 2.6 M_p/V_p.

As can be seen in Fig. 2 and according to the method proposed by Harries *et al*. (2000), (γ_p) can be obtained.

Links can be I-shaped cross sections (rolled wide-flange sections or built-up sections), or built-up box sections. HSS (i.e., hollow sections) cannot be used as links. Shear yielding will occur when $V_p = 0.6F_y \times A_w$ and $M < M_p = Z_b \times F_y$ or $e \leq 1.6M_p / V_p$. where F_y, A_w and Z_b are the I-shaped cross section characteristics; yielding strength, cross sectional area of the web and plastic section modulus, respectively. Shear yielding of steel links provide best overall structural performance for strength, stiffness and ductility. Coupled shear walls are expected to withstand significant inelastic deformations in the links when subjected to design earthquake. But links shall be flexible to dissipate energy at strong ground motions. Design of steel coupling beams based on strength approach are according to the following Eqs. (2)-(6)

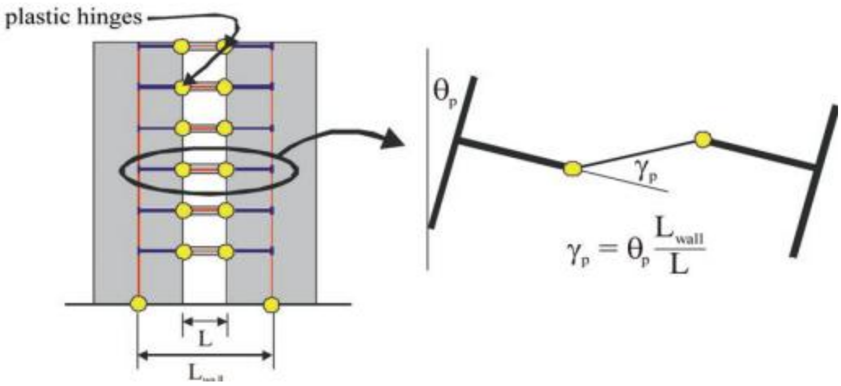


Fig. 2 Determination of coupling beam angle of rotation (Harries *et al*. 2000)

$$M(LRFD) = M_D + 1.2(M_L + M_E) \tag{2}$$

$$V(LRFD) = V_D + 1.2(V_L + V_E) \tag{3}$$

$$V_n = \min(V_p, \frac{2M_p}{e}) \tag{4}$$

$$\theta_p = \frac{0.7R\Delta_w}{h} \tag{5}$$

$$\gamma_p = \frac{L_{wall}.\theta_p}{L} \tag{6}$$

These three equations: $e \leq 1.6M_p/V_p$, V(LRFD) $\leq 0.9V_n$, $\gamma_p \leq 0.08$ were checked for design of equivalent steel coupling beams. Where M_D, M_L, M_E are flexural moments due to dead, live and earthquake load respectively, also V_D, V_L, V_E, are shear forces due to dead, live and earthquake load respectively in coupling beam. R is response modification factor (Standard No. 2800 2007), Δ_w is maximum relative lateral displacement of the story, h is story height, L_{wall} and L as it was shown in Fig. 2.

4. Design and modeling

4.1 Overview of prototype structures

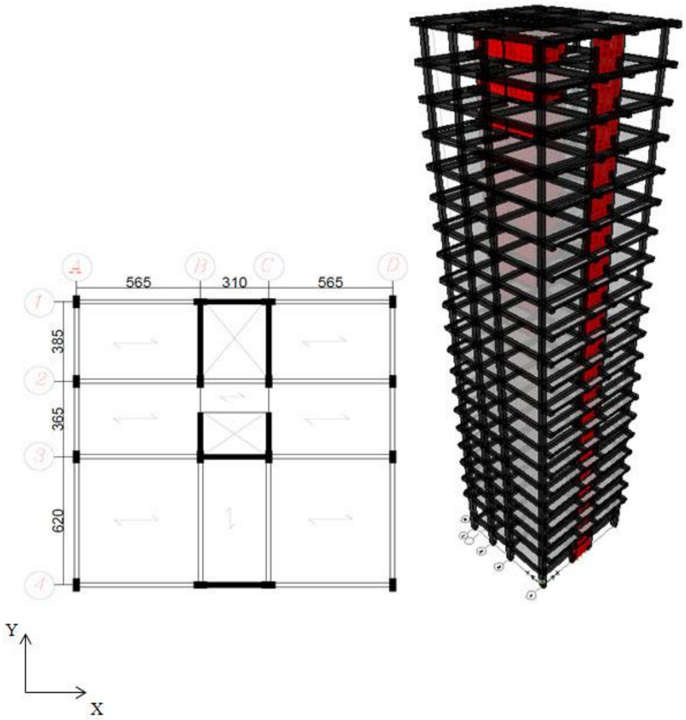
In this study, six structural models were used to specify the trend of this research that are defined as follows: 7, 14, 21-story building in the form of concrete moment resisting frame accompanied with RC coupled shear wall, first with concrete, then with steel coupling beams (see Table 1). The height of the first story is 2.9 m, second story 4 m and the rest 3.2 m. According to the plan dimensions and height of 21-storey, core wall system was used to satisfy the provision of design under lateral load. The core of the structure consists of two U-shaped reinforced concrete walls (Fig. 3(a)). The gap between the walls allows to access the elevator lobby and other architectural and installation requirements at each floor, is spanned by 1.2 m coupling beams. In addition to core wall system, a single shear wall was used in X-direction. Also 7 and 14-story models were used to investigate the height effects on behavior of coupled shear wall. The steel material used in the sections of the structural members is of ST37 type with yielding strength of 2400 kg/cm^2 and ultimate strength of 3700 kg/cm^2. The compressive strength of concrete material f'_c, used in the shear walls is 240 kg/cm^2, and yielding strength of steel bar 4000 kg/cm^2. In order to calculate earthquake load, the spectrum dynamic method was used based on reference Standard No. 2800 (Standard No. 2800 2007). American Concrete Institute Requirements (ACI 318-05 2005) were used to design intermediate RC shear wall and frame respectively. Also Eqs. 2-6 were

Table 1 Dual systems under investigation

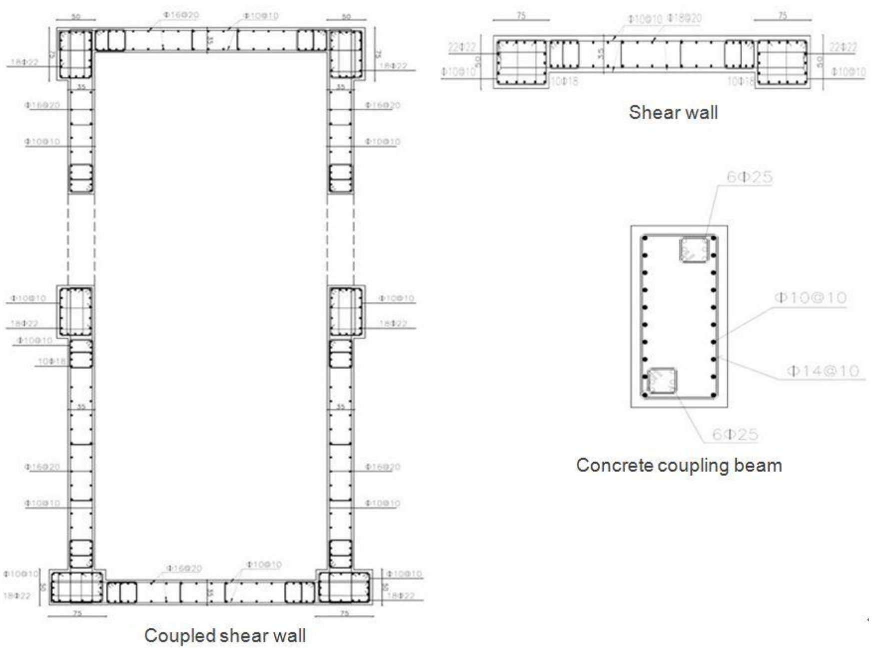
Number	Model	Symbol
1	7 story with concrete coupling beam	7st-conc
2	14 story with concrete coupling beam	14st-conc
3	21 story with concrete coupling beam	21st-conc
4	7 story with steel coupling beam	7st-steel
5	14 story with steel coupling beam	14st-steel
6	21 story with steel coupling beam	21st-steel

Table 2 Details of RC coupled shear wall and concrete coupling beam of buildings

Thickness of shear wall		Details of reinforcement bar			Detailes of reinforcement bar	
Story	Thickness (cm)	Longitudinal bar in web	Horizontal bar in web	Longitudinal bar in boundary zone	Story	Diagonal reinforcement bar of each side
21 story						
1-2	40	Φ18@20	Φ10@10	18Φ25	1-2	6Φ25
3-4	35	Φ16@20	Φ10@10	18Φ22	3-4	6Φ25
5-6	35	Φ16@20	Φ10@10	18Φ18	5-6	4Φ25
7	30	Φ14@20	Φ10@10	16Φ18	7	4Φ25
8-12	30	Φ12@20	Φ10@10	16Φ16	8-12	4Φ20
13-16	25	Φ12@20	Φ10@10	16Φ16	13-16	4Φ18
17	20	Φ10@20	Φ10@10	12Φ16	17	4Φ18
18-21	20	Φ10@20	Φ10@10	12Φ16	18-21	4Φ16
14 story						
1-5	30	Φ14@20	Φ10@10	16Φ18	1-5	4Φ25
6-9	25	Φ12@20	Φ10@10	16Φ16	6-11	4Φ20
10-14	20	Φ10@20	Φ10@10	12Φ16	12-14	4Φ18
7story						
1-2	25	Φ12@20	Φ10@10	16Φ16	1-2	4Φ25
3-7	20	Φ10@20	Φ10@10	12Φ16	3-5	4Φ20
					6-7	4Φ18



(a)



(b)

Fig. 3 (a) The structural plan & elevation of the models; (b) details of RC coupled shear wall and concrete coupling beam of the fourth story in 21-story building employed to design of steel coupling beam. For example, details of RC coupled shear wall and concrete coupling beam of the fourth story in 21-story building are given in Fig. 3(b). Also details of RC coupled shear wall and concrete coupling beam for all the buildings are summarized in Table 2. After examining various sections for steel coupling beam, finally IPE400 was chosen for all the stories. Based on AISC 2010, links with length less than $1.6\frac{M_P}{V_P}$ must be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - \frac{d}{5})$ for a link rotation angle (γ) of 0.08 rad or $(52t_w - \frac{d}{5})$ for link rotation angles of 0.02 rad or less. Linear interpolation can be used for values between 0.08 and 0.02 rad. Therefore, intermediate web stiffeners is used in steel coupling beam to prevent lateral buckling. Also as mentioned above (Section 2), the coupling ratio (CR) of all structures were obtained and range from 25% to 40%.

This design approach is based on strength, therefore, for more accurate assessment of RC coupled shear wall with steel coupling beam under different level of earthquake, must be used from nonlinear analysis and performance based evaluation approach. In most building code applications, for instance, the desired performance of a structure will satisfy life safety (LS) requirements at the design basis earthquake (10% probability of exceedance in 50 years) and collapse prevention (CP) requirements at the maximum considered earthquake (2% in 50 years). Structural performance of all buildings will be verified at both LS and CP levels.

Nonlinear static analysis was used for all structural models in PERFORM3D software. Structures were simulated in 3-Dimention. The moment-rotation characteristics of the plastic hinges for RC column and beam are obtained from section analysis using appropriate nonlinear constitutive laws. In this research, FEMA beam and column plastic hinge properties (FEMA356 2000) were assigned for nonlinear behavior of beams and columns in PERFORM3D software (Fig. 4). PERFORM3D model of structures is shown in Fig. 5.

Nonlinear characteristics of RC shear wall and coupling beam will be described in the next sections. For control point of the displacement of structure in all analysis, the center of mass at the roof level is selected. Since, for plotting the capacity curves of the structures, the relative lateral displacement (i.e., drift) of roof is used as a reference relative lateral displacement. Two approaches have been used to regulate the relative lateral displacement of structure. The first criterion for finishing the analysis is when the deformation capacity of each element is reached and the second one is the limitation of reference drift and inter-story drift for the structure, which is 2% of building height, based on Table C1-3 of FEMA-356 (Federal Emergency Management Agency-356 2000). Therefore, the analysis will be stopped, when these drifts exceed from the mentioned limit.

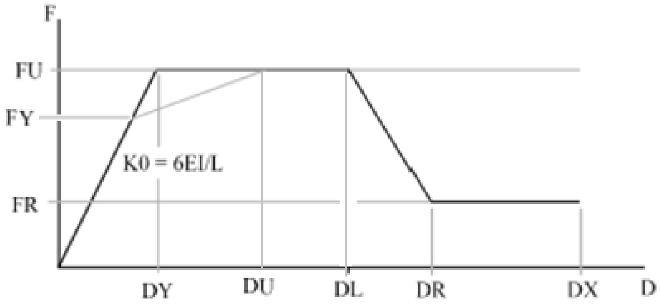


Fig. 4 Modeling of the nonlinear behavior of RC beams and columns in PERFORM3D

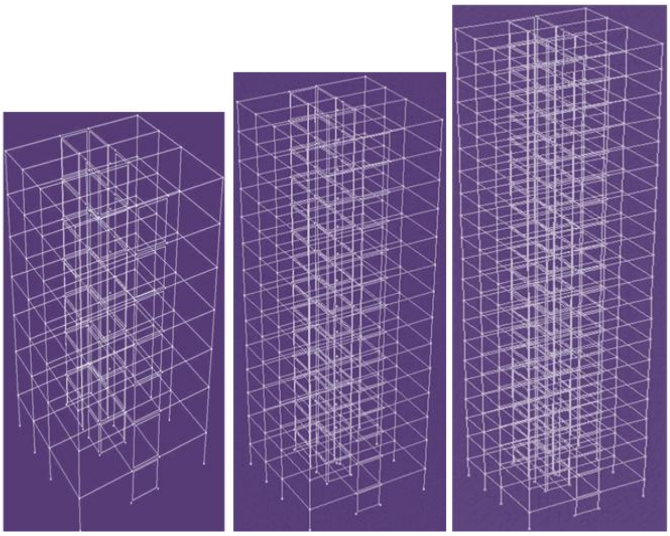


Fig. 5 The structural models in PERFORM3D

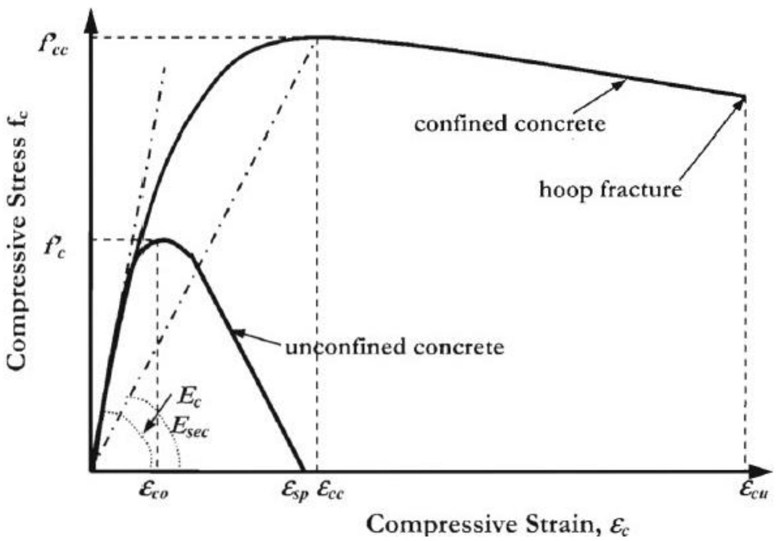


Fig. 6 Stress-strain relationship for concrete in compression (Priestley *et al*. 2007)

4.2 Nonlinear modeling of RC shear wall

To make the RC coupled shear wall sections, defining the linear and nonlinear characteristics of concrete and steel materials are necessary. The fiber cross-section elements consisting of steel and concrete fibers were used to model RC shear wall. ACI 318-05 (2005) requires confinement in boundary zones, when structural walls do not have the ability to deform their maximum displacement without exceeding ultimate concrete compressive strains. Adding confinement allows the concrete to exhibit higher compressive strains without a significant degradation in strength, as illustrated in Fig. 6. In PERFORM3D software, the stress-strain curve of confinement

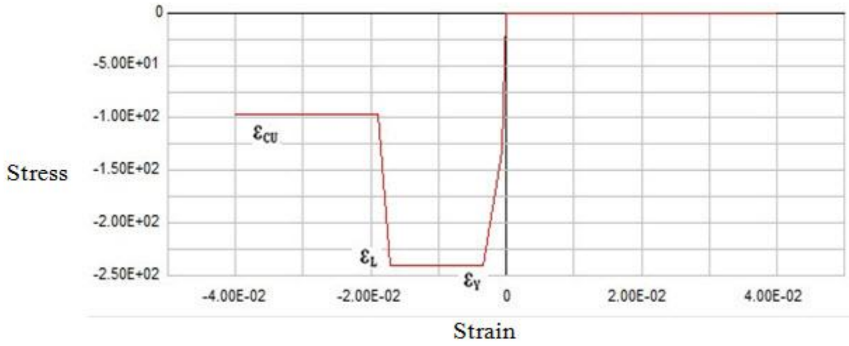


Fig. 7 Nonlinear properties of concrete material in PERFORM3D

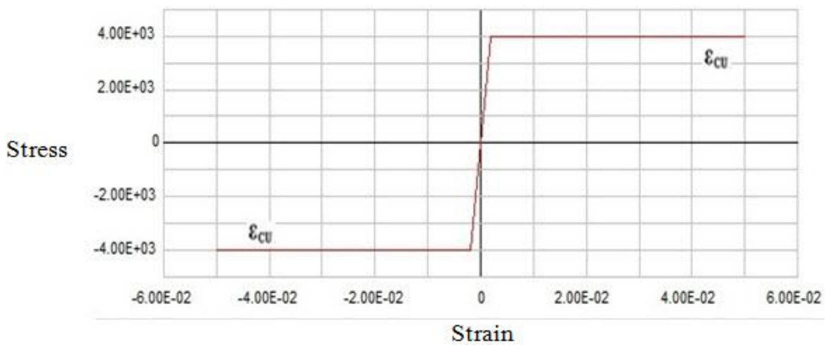


Fig. 8 Modeling of the steel behavior in PERFORM3D

concrete is selected in the form of trilinear with strength loss and its tension strength is ignored. Fig. 7 shows that, the strain of ultimate strength of concrete, ε_L, is taken 0.0171, the strain of crushing limit of concrete ε_{cu}, 0.04 and the strain of yielding strength of concrete ε_Y, 0.0034. Also E_c (modulus of elasticity), is 200000 kg/cm^2. The stress-strain relationship of steel bar needs to be bilinear (elastic-perfectly plastic), without strength loss. The modulus of elasticity, E_s, is taken 2100000 kg/cm^2 and the ultimate strain ε_{su}, 0.05 according to the Fig. 8. Also, yielding strength, F_y, is 4000 kg/cm^2.

4.3 Nonlinear modeling of coupling beams

To define nonlinear characteristics of concrete coupling beam, model of shear hinge-displacement type in PERFORM3D was used (Fig. 9). To assign nonlinear characteristics of concrete coupling beams according to the Table (6-18) of FEMA-356 (2000), plastic hinge rotation of diagonal reinforcement (θ) is estimated 0.05. Therefore $\Delta = L\theta$, where L is coupling beam length. Shear force takes into account the two components: $V_s + V_c$, where V_s is the contribution of diagonal reinforcement and V_c is the contribution of concrete and calculated based on ACI 318-05 (2005) as follows

$$V_s = 2A_s F_y \sin\alpha \tag{7}$$

$$V_c = 0.53\sqrt{f_c'}b_w d \tag{8}$$

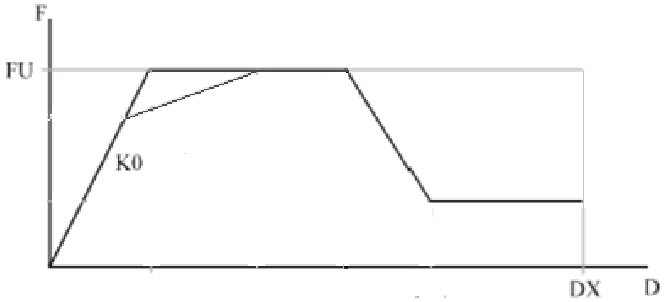


Fig. 9 Modeling of the nonlinear behavior of coupling beam in PERFORM3D

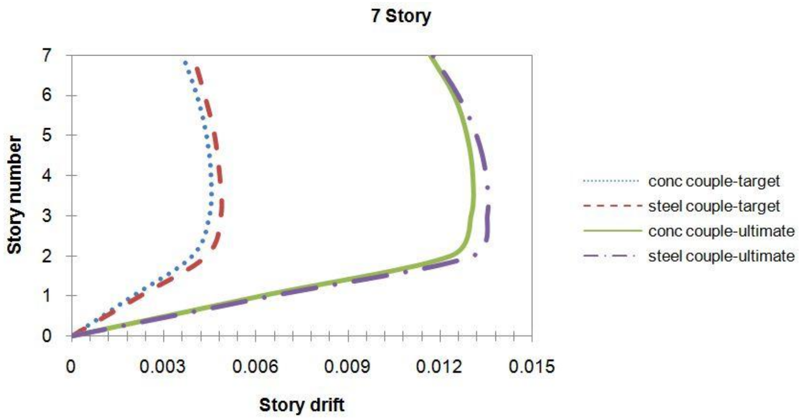


Fig. 10 Story drift ratio at two levels for 7-story building

where A_s, F_y and α are cross-section area, yielding stress and angle of diagonal rebar with respect to the horizontal line in concrete coupling beam and b_w and d are width and effective depth of concrete coupling beam section.

As previously mentioned, steel coupling beams provide the same structural role as link beams in eccentrically braced frames. Also, to define nonlinear characteristics of steel coupling beam, model of shear hinge-displacement type in PERFORM3D was used (Fig. 9). According to the Table (5-6) of FEMA-356, since intermediate web stiffeners were sufficiently used in steel coupling beams to prevent buckling, plastic hinge rotation (θ) and shear force were taken 0.17 and $0.6F_yA_w$ respectively.

5. Discussion and results of nonlinear analysis

The story drift ratio plot for RC coupled shear wall systems consist of steel and concrete coupling beams at target and ultimate levels are shown in Figs. 10-12. The target level of structures is performance point under DBE. These illustrate that, although steel coupling beams were designed based on the criterion of sufficient strength and shear yielding members; but both of them (i.e., steel and concrete coupling beams system) approximately have the same drift distribution over the height. The values of the fundamental periods in the *X* and *Y* directions and

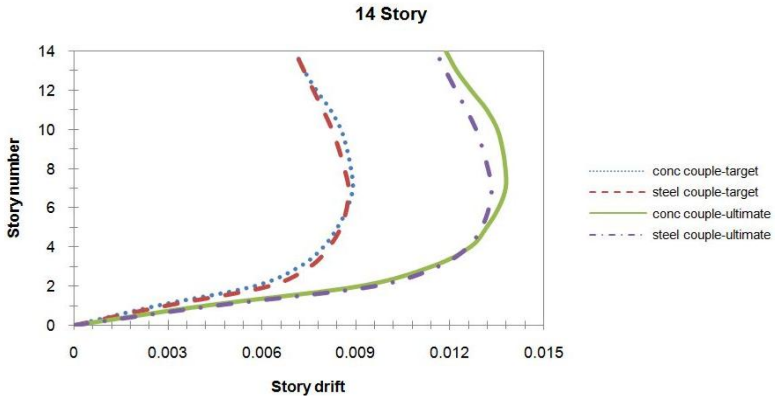


Fig. 11 Story drift ratio at two levels for 14-story building

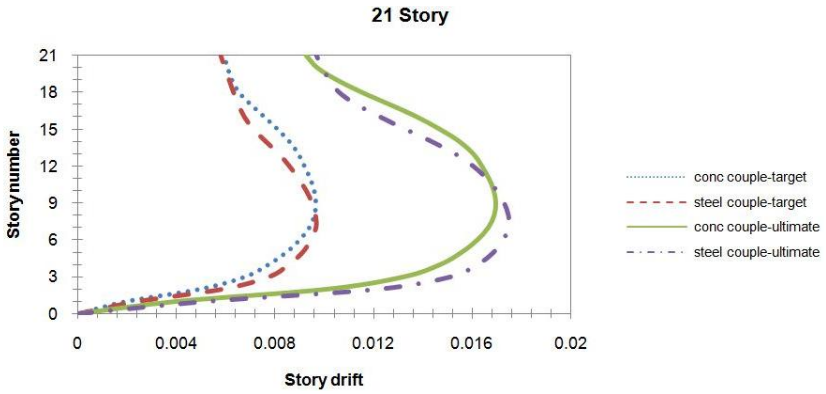


Fig. 12 Story drift ratio at two levels for 21-story building

Table 3 Fundamental periods of structures

Model	T_x (sec)	T_y (sec)	T_{tortion} (sec)
7st-conc	0.653	0.413	0.482
14st-conc	1.612	1.125	0.939
21st-conc	2.738	2.037	1.380
7st-steel	0.758	0.522	0.550
14st-steel	1.712	1.234	1.008
21st-steel	2.844	2.146	1.448

also for torsional mode of vibration are given in Table 3. As can be seen in result of tables, fundamental periods of RC coupled shear wall with concrete and steel coupling beam are almost the same.

The capacity curves (Base shear vs. roof displacement capacities) of six structures are obtained by nonlinear analysis as shown in Figs. 13(a)-(f). Performance level of all the structures were evaluated based on two spectrums, first with Design Basis Earthquake (DBE) and then Maximum

Considered Earthquake (MCE) as can be seen in Fig. 14. Performance point of structures was obtained from Displacement Coefficient Method (DCM) of FEMA356 (2000). Due to ability of PERFORM3D, Displacement Coefficient Method (DCM) was employed because of its accuracy and simplicity. Therefore, as shown in Fig. 13, performance point of six structures based on DBE and MCE levels can be obtained. According to the performance point of structures, amount of deformation demand (D) of all members of them were obtained at two levels of earthquake and ultimate point. As described, ultimate point is minimum of drift 2% and drift corresponding to ultimate load. On the other hand, deformation capacity (C) of all members in three performance levels (IO, LS and CP) was defined based on FEMA 356 (2000). Then the ratio of deformation demand to capacity (D/C) of all members of structures at different performance levels and different earthquake levels were determined. The ratio of average D/C of beams, columns, RC shear walls and concrete or steel coupling beams for each of the six structures are listed in Tables 4-9. The D/C ratio of members of structures shows the amount of damage at each of the performance levels. In other words, if average D/C ratio of members at each of the performance level is less than 1, average amount of deformation of members will be less than the limit of that performance level.

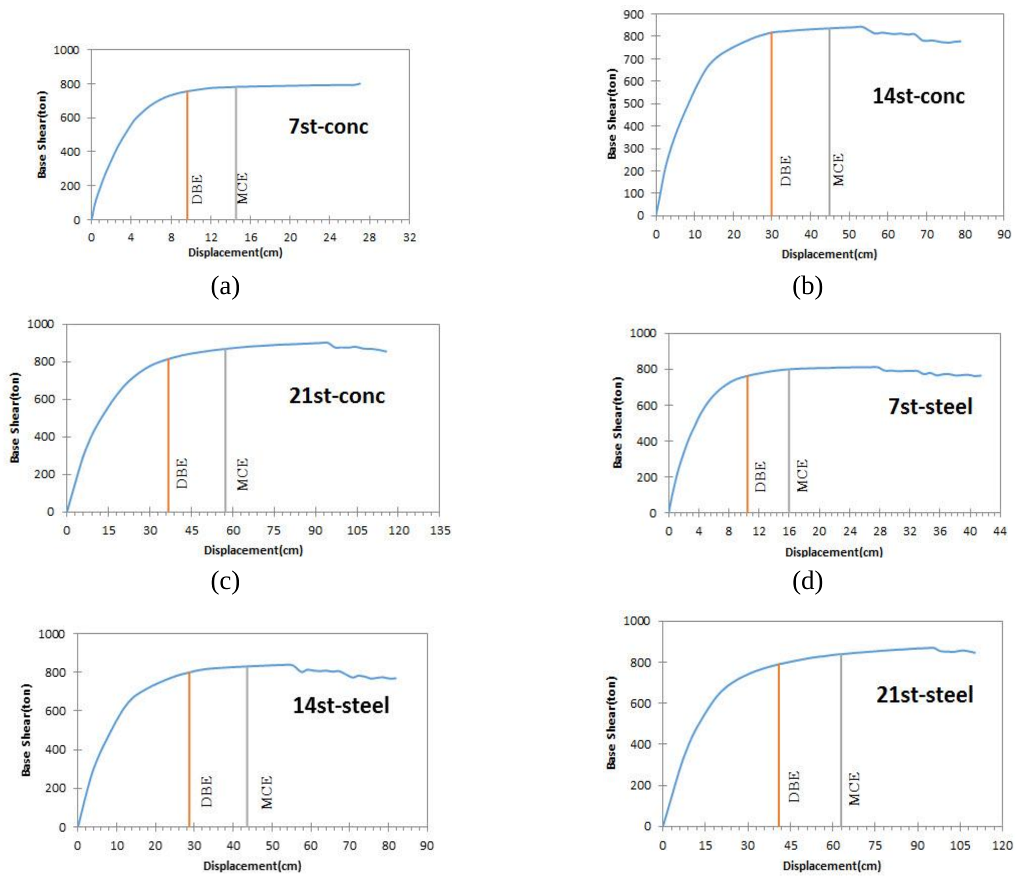


Fig. 13 Capacity curves of evaluated structures

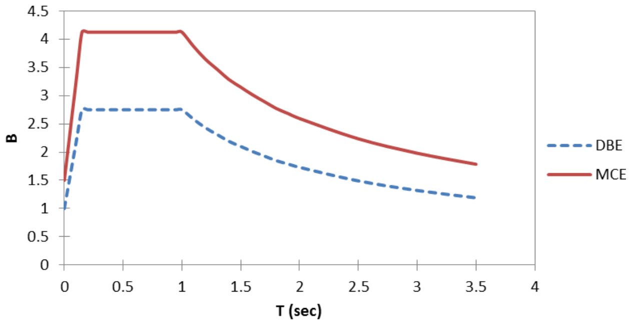


Fig. 14 Spectra for design basis earthquake and maximum considered earthquake

As can be seen in Tables 4-9, by changing the spectrum from DBE to MCE, the ratio of average D/C of members increases. Average D/C ratio of members for all structures consist of concrete or steel coupling beam under DBE is less than 1 at LS performance level. Therefore, design of these structure based on strength approach under DBE satisfies performance of them at LS level. Also average D/C ratio of members for all structures under MCE is less than 1 at CP performance level, except 14-story building with concrete coupling beam that D/C is slightly greater than 1, so requirement of Code is satisfied at this performance level approximately. The results in Tables 4-6 show that in structures with concrete coupling beam, coupling beams have ratio of average D/C higher than shear wall under MCE at CP level; in other words, the amount of damage of concrete coupling beams is higher than RC shear walls, hence concrete coupling beams dissipate more energy than RC shear wall. Also concrete coupling beams dissipate more energy than RC columns and RC beams of these structures. This result is true even at ultimate point. Therefore, concrete coupling beam acts like a fuse in RC coupled shear wall system. Average D/C ratio of columns is

Table 4 $\frac{D}{C}$ Ratio for 7-story building with concrete coupling beam

Model	Element	Performance level	$\frac{D}{C}$ Ratio at performance point		$\frac{D}{C}$ Ratio at ultimate point
			DBE	MCE	
7st-conc	Beam	IO	0.748	1.083	1.830
		LS	0.374	0.546	0.915
		CP	0.299	0.433	0.732
	Column	IO	0.434	0.727	1.499
		LS	0.144	0.242	0.499
		CP	0.108	0.181	0.374
	Coupling beam	IO	2.059	3.69	8.338
		LS	0.686	1.233	2.779
		CP	0.411	0.739	1.668
	Shear wall	IO	0.719	1.177	2.323
		LS	0.359	0.588	1.161
		CP	0.239	0.392	0.774

Table 5 $\frac{D}{C}$ Ratio for 14-story building with concrete coupling beam

Model	Element	Performance level	$\frac{D}{C}$ Ratio at performance point DBE	$\frac{D}{C}$ Ratio at performance point MCE	$\frac{D}{C}$ Ratio at ultimate point
14st-conc	Beam	IO	1.269	2.114	4.079
		LS	0.6346	1.057	2.039
		CP	0.507	0.845	1.632
	Column	IO	0.6708	1.364	1.564
		LS	0.223	0.454	0.521
		CP	0.167	0.341	0.391
	Coupling beam	IO	3.006	5.683	8.350
		LS	1.002	1.894	2.783
		CP	0.601	1.137	1.67
	Shear wall	IO	0.779	1.288	1.829
		LS	0.389	0.644	0.914
		CP	0.259	0.429	0.609

Table 6 $\frac{D}{C}$ Ratio for 21-story building with concrete coupling beam

Model	Element	Performance level	$\frac{D}{C}$ Ratio at performance point DBE	$\frac{D}{C}$ Ratio at performance point MCE	$\frac{D}{C}$ Ratio at ultimate point
21st-conc	Beam	IO	1.089	1.519	3.620
		LS	0.544	0.759	1.810
		CP	0.435	0.607	1.448
	Column	IO	0.333	0.634	1.495
		LS	0.111	0.211	0.498
		CP	0.083	0.158	0.373
	Coupling beam	IO	1.737	3.459	7.523
		LS	0.579	1.153	2.508
		CP	0.347	0.691	1.505
	Shear wall	IO	0.494	0.809	2.246
		LS	0.247	0.404	1.123
		CP	0.1647	0.269	0.748

much less than RC shear wall for all structures with concrete coupling beam until ultimate point, therefore gravity resistant system will be damaged slightly at severe earthquake. As shown in Tables 7-9, in structures with steel coupling beam, coupling beams have ratio of average D/C less than shear walls under MCE at CP level; in other words, amount of damage of RC shear walls is higher than steel coupling beams, hence RC shear walls dissipate more energy than steel coupling beams. This conclusion holds even at ultimate point. The energy dissipation of RC beams in these

Table 7 $\frac{D}{C}$ Ratio for 7-story building with steel coupling beam

Model	Element	Performance level	$\frac{D}{C}$ Ratio at performance point DBE	$\frac{D}{C}$ Ratio at performance point MCE	$\frac{D}{C}$ Ratio at ultimate point
7st-steel	Beam	IO	0.762	1.280	4.064
		LS	0.381	0.640	2.032
		CP	0.304	0.512	1.626
	Column	IO	0.451	0.833	1.922
		LS	0.150	0.277	0.640
		CP	0.112	0.208	0.480
	Coupling beam	IO	2.316	4.522	15.200
		LS	0.105	0.205	0.690
		CP	0.082	0.161	0.542
	Shear wall	IO	0.745	1.418	3.717
		LS	0.372	0.709	1.859
		CP	0.248	0.472	1.239

Table 8 $\frac{D}{C}$ Ratio for 14-story building with steel coupling beam

Model	Element	Performance level	$\frac{D}{C}$ Ratio at performance point DBE	$\frac{D}{C}$ Ratio at performance point MCE	$\frac{D}{C}$ Ratio at ultimate point
14st-steel	Beam	IO	1.324	2.096	4.171
		LS	0.662	1.048	2.085
		CP	0.529	0.838	1.668
	Column	IO	0.744	1.381	1.574
		LS	0.248	0.460	0.524
		CP	0.186	0.345	0.393
	Coupling beam	IO	4.295	7.336	11.740
		LS	0.195	0.333	0.533
		CP	0.153	0.262	0.419
	Shear wall	IO	0.926	1.438	2.221
		LS	0.463	0.719	1.111
		CP	0.308	0.479	0.740

structures is higher than steel coupling beams, RC shear walls and columns. Therefore, steel coupling beam does not act like a fuse in RC coupled shear wall system. This may be caused from the fact that steel coupling beams were designed based on shear yielding criteria with conventional steel (f_y = 2400 kg/cm^2). Average D/C ratio of columns is much less than RC shear wall for all structures with steel coupling beam until ultimate point, therefore similar to previous structures, gravity resistant system will be damaged slightly at severe earthquake. As shown in Tables 4-9,

Table 9 $\frac{D}{C}$ Ratio for 21-story building with steel coupling beam

Model	Element	Performance level	$\frac{D}{C}$ Ratio at performance point DBE	$\frac{D}{C}$ Ratio at performance point MCE	$\frac{D}{C}$ Ratio at ultimate point
21st-steel	Beam	IO	1.071	1.622	3.371
		LS	0.535	0.811	1.685
		CP	0.428	0.649	1.348
	Column	IO	0.375	0.743	1.491
		LS	0.125	0.253	0.497
		CP	0.093	0.185	0.372
	Coupling beam	IO	3.375	6.861	13.170
		LS	0.153	0.311	0.598
		CP	0.120	0.245	0.470
	Shear wall	IO	0.661	1.119	2.308
		LS	0.330	0.559	1.154
		CP	0.220	0.373	0.769

average D/C ratio of steel coupling beam is higher than concrete coupling beam for all structures under MCE at IO level, therefore demand ductility of steel coupling beam is higher than concrete coupling beam.

6. Conclusions

In this paper, seismic performance evaluation of short buildings to tall buildings consist of RC coupled shear walls with concrete or steel coupling beam designed based on strength approach were studied. For nonlinear analysis of structures, nonlinear stress-strain curves for confined concrete in shear wall and nonlinear plastic hinge for RC beam, RC column and concrete and steel coupling beam were used. Performance level of buildings was evaluated under DBE and MCE. Some of the key results obtained from this evaluation are as follows:

- The structural systems consist of RC coupled shear walls with concrete and steel coupling beams showed approximately the same fundamental period and drift distribution over the height.
- Seismic performance of the structural systems consist of RC coupled shear walls with steel and concrete coupling beams were satisfied at level of LS and CP under DBE and MCE respectively.
- D/C average ratio of coupling beams in building with concrete coupling beam was higher than average ratio of D/C of RC shear wall under MCE and ultimate point; Therefore, concrete coupling beam acted like a fuse.
- D/C average ratio of steel coupling beams was less than D/C average ratio of RC shear wall under MCE and ultimate point; therefore the amount of damage of RC shear walls is higher than steel coupling beams under MCE and ultimate point.

- Average D/C ratio of columns is much less than RC coupled shear wall with concrete and steel coupling beam at ultimate point; therefore gravity resistant system will be damaged slightly at severe earthquake.

References

ACI Committee 318 (2005), Building Code Requirements for Structural Concrete, American Concrete Institute, Farmington Hills, MI, USA.

AISC (2010), Specification for structural steel buildings, American Institute of Steel Construction, Chicago, IL, USA.

Aktan, A.E. and Bertero, V.V. (1981), "The seismic resistant design of R/C coupled structural walls", Report No. UCB/EERC-81/07; Earthquake Engineering Research Center, University of California, Berkeley, CA, USA.

Aktan, A.E. and Bertero, V.V. (1984), "Seismic response of R/C frame-wall structures", *Struct. Div.*, **110**(8), 1803-1821.

Aktan, A.E. and Bertero, V.V. (1987), "Evaluation of seismic response of RC building loaded to failure", *Struct. Div.*, **113**(5), 1092-1108.

Aristizabal-Ochoa, J.D. (1982), "Dynamic response of coupled wall systems", *Struct. Div.*, **108**, 1846-1857.

Aristizabal-Ochoa, J.D. (1987), "Seismic behavior of slender coupled wall systems", *Struct. Div.*, **113**(10), 2221-2234.

ATC-40 (1996), Seismic evaluation and retrofit of concrete buildings, Applied Technology Council, **1**(2), CA, USA.

Cheng, M.Y., Fikri, R. and Chen, C.C. (2015), "Experimental study of reinforced concrete and hybrid coupled shear wall systems", *Eng. Struct.*, **82**, 214-225.

El-Tawil, S. and Kuenzli, C. (2002), "Pushover of hybrid coupled walls II: Analysis and behavior", *Struct. Div.*, **128**(10), 1282-1289.

El-Tawil, S., Kuenzli, C. and Hassan, M. (2002), "Pushover of hybrid coupled walls I: design and modeling", *Struct. Eng.*, **128**(10), 1272-1281.

El-Tawil, S., Harries, K., Fortney, P., Shahrooz, B. and Kurama, Y. (2010), "Seismic design of hybrid coupled wall systems: State of the art", *Struct. Eng.*, **136**(7), 755-769.

FEMA-273 (1997), NEHRP Guidelines for the seismic rehabilitation of buildings, Federal Emergency Management Agency, Building Seismic Safety Council, Washington, USA.

FEMA-356 (2000), Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Building Seismic Safety Council, Washington, USA.

Fortney, P.J., Shahrooz, B.M., Gian, A. and Rassati, G.A. (2007), "Seismic performance evaluation of coupled core walls with concrete and steel coupling beams", *Steel Compos. Struct.*, *Int. J.*, **7**(4), 279-301.

Harries, K.A. (1999), "Ductility and deformability of coupling beams in reinforced concrete coupled walls", *Proceedings of the 8th Canadian Conference on Earthquake Engineering*, Vancouver, Canada, June, pp. 475-481.

Harries, K.A. (2001), "Ductility and deformability of coupling beams in reinforced concrete shear walls", *Earthq. Spectra*, **17**(3), 457-478.

Harries, K.A. and McNeice, D.S. (2006), "Performance-based design of high rise coupled wall systems", *Struct. Des. Tall Spec. Build.*, **15**(3), 289-306.

Harries, K.A., Mitchell, D., Cook, W.D. and Redwood, R.G. (1992a), "Concrete walls coupled by ductile steel link beams", *Proceedings of the Earthquake Engineering 10th Word Conference Balkema*, Rotterdam, The Netherlands, pp. 3205-3210.

Harries, K.A., Mitchell, D., Cook, W.D. and Redwood, R.G. (1992b), "Seismic response of steel beams coupling reinforced concrete walls", *Struct. Div.*, **119**(12), 3611-3629.

Harries, K.A., Mitchell, D., Cook, W.D. and Redwood, R.G. (1993), "Seismic response of steel beams

coupling concrete walls", *Struct. Eng.*, **119**(12), 3611-3629.
Harries, K.A., Gong, B. and Shahrooz, B.M. (2000), "Behavior and design of reinforced concrete, steel and steel–concrete coupling beams", *Earthq. Spectra*, **16**(4), 775-799.
Hosseini, M., Sadeghi, H. and Habiby, S.A. (2011), "Comparing the nonlinear behaviors of steel and concrete link beams in coupled shear walls system by finite element analysis", *Procedia Eng.*, **14**, 2864-2871.
Inel, M. and Ozmen, H.B. (2006), "Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings", *Eng. Struct.*, **28**(11), 1494-1502.
Khalifa, E.S. (2014), "Analytical model for steel fiber concrete composite short-coupling beam", *Composites: Part B*, **56**, 318-329.
Krawinkler, H. and Seneviratna, G.D.P.K. (1998), "Pros and cons of a push-over analysis of seismic performance evaluation", *Eng. Struct.*, **20**(4-6), 452-464.
Kwan, A.K.H. and Zhao, Z.Z. (2002), "Cyclic behavior of deep reinforced concrete coupling beams", *Proceedings of the Institution of Civil Engineers-Structures and Buildings*, **152**, 283-293.
Lequesne, R.D. (2011), "Behavior and design of high-performance fiber-reinforced concrete coupling beams and coupled-wall systems", Ph.D. Thesis; Department of Civil and Environmental Engineering, The University of Michigan-Ann Arbor, MI, USA.
Lybas, J.M. and Sozen, M.A. (1977), "Effect of beam strength and stiffness on dynamic behavior of reinforced concrete walls", Structural research series No. 444, University of Illinois, Urbana-Champaign, IL, USA.
Marcakis, K. and Mitchell, D. (1980), "Precast concrete connections with embedded steel members", *Prestressed Concrete Institute*, **25**(4), 88-116.
Mattock, A.H. and Gaafar, G.H. (1982), "Strength of embedded steel section as brackets", *ACI*, **79**(2), 83-93.
Nie, J.G., Hua, H.S. and Eatherton, M.R. (2014), "Concrete filled steel plate composite coupling beams: Experimental study", *Construct. Steel Res.*, **94**, 49-63.
Park, W.S. and Yun, H.D. (2005), "Seismic behavior of steel coupling beams linking reinforced concrete shear walls", *Eng. Struct.*, **27**(7), 1024-1039.
Paulay, T. and Binney, J.R. (1974), "Diagonally reinforced concrete beams for shear walls", *ACI Special Publication SP 42 - Shear in Reinforced Concrete*, 579-598.
Raju, K.R., Cinitha, A. and Iyer, N.R. (2012), "Seismic performance evaluation of existing RC buildings designed as per past codes of practice", *Indian Academy of Sciences*, **37**(2), 281-297.
Robert, P. and Paulay, T. (1975), *Reinforced Concrete Structures*, Wiley and Sons, New York, NY, USA.
Saatcioglu, M., Derecho, A.T. and Corley, W.G. (1987), "Parametric study of earthquake-resistant coupled walls", *Struct. Div.*, **113**(1), 141-157.
Shahrooz, B.M., Remmetter, M.A. and Qin, F. (1993), "Seismic design and performance of composite coupled walls", *Struct. Div.*, **119**(11), 3291-3309.
Shiu, N.K., Barney, G.B., Fiorato, A.E. and Corley, W.G. (1981), "Earthquake resistant walls-coupled wall test", Report to NSF submitted by Portland Cement Association, Research and Development, Skokie, IL, USA.
Shiu, N.K., Takayangi, T. and Corley, W.G. (1984), "Seismic behavior of coupled wall systems", *Struct. Div.*, **110**(5), 1051-1066.
Standard No. 2800 (2007), *Iranian Code of Practice for Seismic Resistant Design of Buildings*, (3th Edition), Building and Housing Research Center, Tehran, Iran.
Subedi, N.K., Marsono, A.K. and Aguda, G. (1999), "Analysis of reinforced concrete coupled shear wall structures", *Struct. Des. Tall Spec. Build.*, **8**(2), 117-143.
Su, R.K.L., Lam, W.Y. and Pam, H.J. (2009), "Experimental study of plate reinforced composite deep coupling beams", *Struct. Des. Tall Spec. Build.*, **18**(3), 235-257.
Yahya, C.K. and Qiang, S. (2008), "Seismic design and response evaluation of unbounded post-tensioned hybrid coupled wall structures", *Earthq. Eng. Struct. Dyn.*, **37**(14), 1677-1702.

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