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Research Article

Seismic Analysis of Monolithic Coupling Beams of Symmetrical Coupled Shear Wall System

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Abstract

It is a well-known fact that the Coupled shear walls are common lateral load-resisting structural components in high-rise Reinforced Concrete (RC) buildings, in which the coupling beams spanning across the openings are normally the most critical elements. Because of architectural considerations and the need to accommodate service voids, these beams are usually of small dimensions and small stiffness as compared with the wall piers in turn. Therefore, the coupling beams often experience very high induced bending and shear stresses under lateral loads. The seismic response of high rise buildings with different height, 10, 20, and 40 stories building are investigated for the three real earthquake excitations, namely; El-Centro, 1940, Northridge, 1994, and Loma prieta, 1989 to analyze and evaluate the effect of the geometry parameters (Span/depth) ratios (1, 2, 4 and 6), and the aspect ratio of the shear wall height to building width effects on the monolithic reinforcement concrete coupling beams of symmetrical coupled shear wall system.

Key words: *Seismic response; coupling beams; coupled shear wall; high rise building.*

1. Introduction

Concrete structural walls can be designed as efficient primary lateral load resisting systems particularly suited for ductile response with very good energy dissipation characteristics when regular patterns of openings (e.g., windows, doors, and/or mechanical penetrations) are arranged in a rational pattern (Park, R. and Paulay,T. 1975; Paulay, T.; and Binney, J.R, 1974; Liao, W. C., et al., 2006, Naish, D., et al., 2009) Examples are shown in Fig. 1; where wall piers are interconnected or coupled to each other by beams at the floor and roof levels.

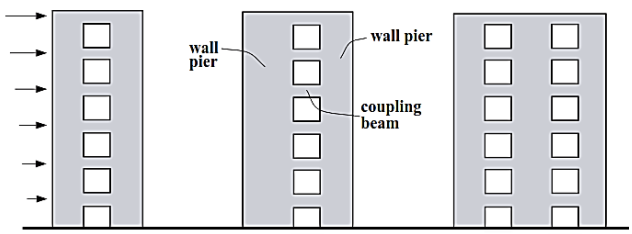

Fig. 1 Coupling wall structures.

Coupling-beams can be thought of as large-scale shear connectors providing composite action between distinct sub-cores. First yield will often occur in coupling-beams and spread vertically. The span/depth ratio of these beams is determined by non-structural considerations and usually well into the 'deep beam' range prone to brittle behavior. The desired failure mechanism for a coupled wall structural system involves the formation of plastic hinges in most or all of the coupling beams and also at the base of each wall pier. In this manner, the dissipation of seismic input energy is distributed over the height of the structure (rather than being concentrated in a few stories), similar to the strong column-weak girder design philosophy for ductile moment resisting frames. The behavior and mechanisms of lateral resistance of a single (i.e., uncoupled) wall and two coupled wall systems are compared in Fig. 2. The gravity loads acting on the walls are ignored for this example and it is assumed that a lateral force in the plane of the walls is applied at the top. The base moment resistance, $M_{w,unc}$ of the uncoupled wall Fig. 2(a) is developed in the traditional form by flexural stresses, while axial forces as well as moments are resisted in the coupled shear wall systems Figs. 2(b) and 2(c). When a coupled shear wall system is pushed from left to right under lateral loads, tensile axial forces (N_{twb}) develop in the left wall pier and compressive axial forces (N_{cwb}) develop in the right wall pier due to the coupling effect.

2. Judging the efficiency of a coupled shear wall system

The magnitude of these wall axial forces is equal to the sum of the shear forces of all the coupling beams at the upper floor and roof levels; and thus, depends on the stiffness and strength of those beams.

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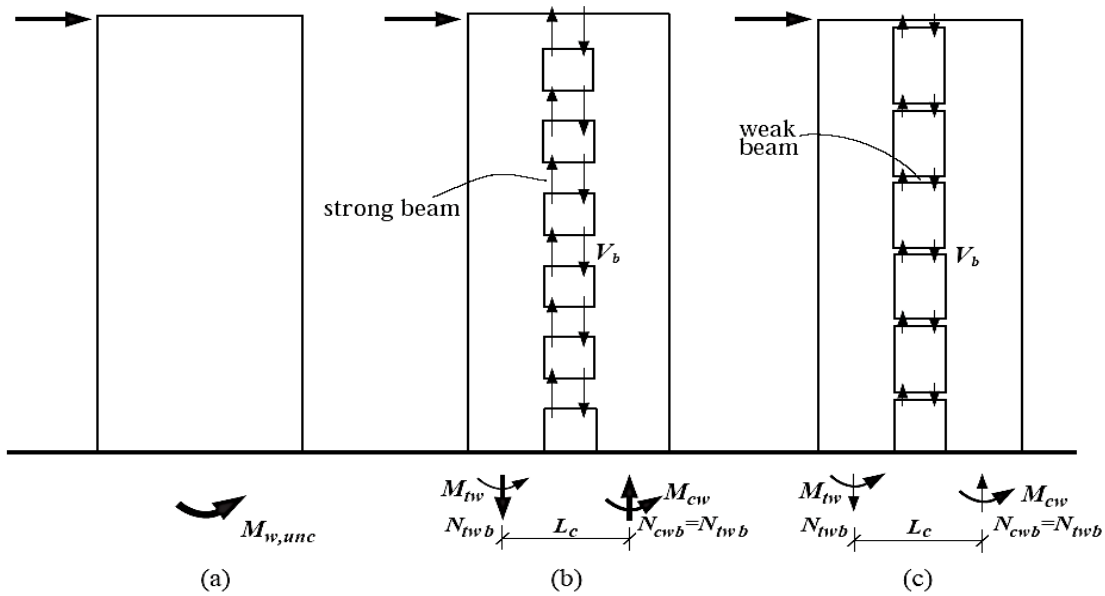

Fig. 2 Wall systems- (a) Uncoupled wall; (b) Coupled wall with strong beams; and (c) Coupled wall with weak beams.

As a result of the axial forces that develop in the walls, the lateral stiffness and strength of a coupled wall system is significantly larger than the combined stiffness and strength of the individual constituent walls (i.e., wall piers) with no coupling. The total base moment, *Mw* of the coupled wall structures in Figs. 2(b) and 2(c) can be written as:

$$M_w=M_{tw}+M_{cw}+N_{cwb}L_c \quad (1)$$

Where, M_{tw} and M_{cw} are the base moments in the tension and compression side walls, respectively, $N_{cwb} = N_{twb}$, and L_c is the distance between the centroids of the tension and compression side walls. Then, the contribution of the wall axial forces from coupling to the total lateral resistance of the system can be expressed by the Coupling Degree, *CD* as:

$$CD=\frac{N_{cwb}L_c}{M_W}=\frac{N_{CWb}L_C}{M_{tw}+M_{cw}+N_{cwb}\,L_c} \quad (2)$$

The coupling degree, which can be controlled by changing the strength and stiffness of the beams relative to the wall piers as shown in Figs. 2(b) and 2(c), is an important parameter for the seismic behavior and design of coupled wall structures.

Too little coupling (i.e., too small a coupling degree) yields a system with behavior similar to uncoupled walls and the benefits due to coupling are minimal. Too much coupling (i.e., too large a coupling degree) will add excessive stiffness to the system, causing the coupled walls to perform as a single pierced wall with little or no energy dissipation provided by the beams, and will result in large axial forces in the foundation. The desirable "range" for the amount of coupling lies in between these two extremes and should be selected properly in seismic design as investigated by (El-Tawil et al., 2002).

Although *CD* reflects a true judgment to the behavior of the coupled shear wall, it is a post analysis result which requires full analysis of the system. In other wards it is worth to have a pre-analysis coefficient depends only on the geometric characteristics of the coupled system of walls. This coefficient can simply be used to eliminate number of trials judge the system or going through to many sophisticated analysis procedure and calculations. In this paper the authors suggest two factors depend on the geometric configuration of the coupled shear wall system (constitute walls and coupling beams) and defined as follows:

$$\textit{Stiffness Coefficient } (K_f) = \frac{\sum K_b}{\sum K_w} \quad (3)$$

$$\textit{Inertia Coefficient } (I_f) = \frac{\sum I_w}{I_{gross}} \quad (4)$$

Where:

K_b: Stiffness of each individual coupling beam acts as a beam fixed at both ends = $\frac{12EI_b}{{L_b}^3}$

K_w: Stiffness of each individual wall acts as a cantilever subjected to concentrated load at the top of the building = $\frac{3EI_w}{{L_w}^3}$

I_w: Second moment of inertia of each individual wall

I_{gross}: Second moment of inertia of each gross wall considering the overall dimensions of the system (i.e. considering the system as a single solid wall without any openings)

Both K_f and I_f are dimension less factors that reflects the geometric configuration of the system. K_f accounts for the geometric parameters of the system in the vertical direction such as beams depth, building height and number of coupling beams. On the other side I_f accounts for the

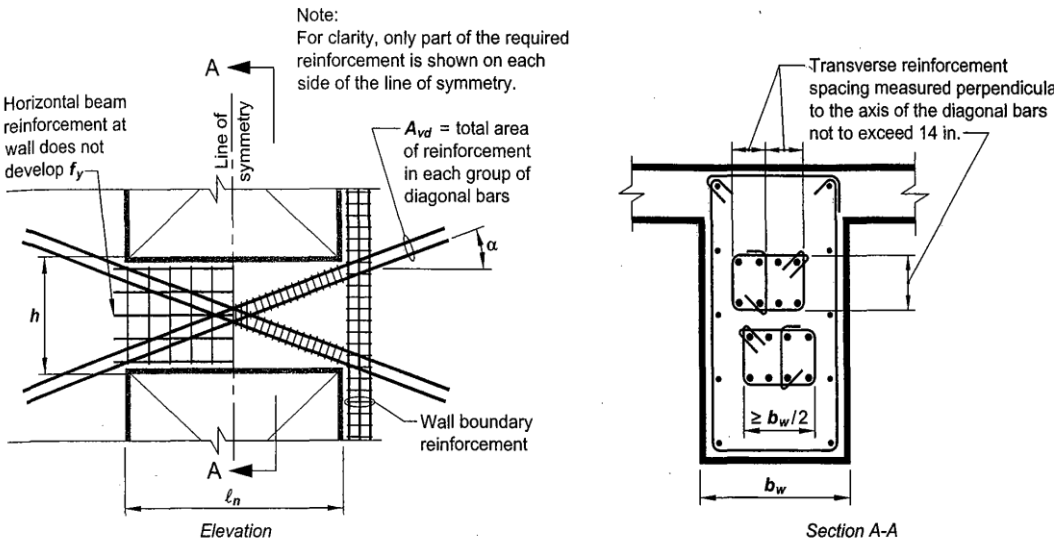

Fig. 3 Diagonally oriented reinforcement for deep coupling beams

spacing between walls and the width of the building which is the geometric parameters in the horizontal direction. Combining both together to produce Coupling Index (*CI*) as a dimension less pre-analysis coefficient depends on the geometric configuration of the coupled shear wall system and defined as follows:

$$CI = K_f \, x \, I_f \tag{5}$$

An analytical discussion for the preceding factors and coefficients is presented in section 5 of this paper. More over some correlations between these factors and the induced forces in the beams and some tips for design are discussed also.

3. Behavior of cast-in-place concrete coupling beams

Indeed coupling beam behaves as any laterally loaded element where load transfer mechanism governs the behavior and the failure mode of the element. In general reinforced concrete bending members (RC beams) are classified according their shear-span/depth ratio (a/h) into four categories as it was mentioned in (ACI 318M-05,2005; Saatcioglu, M., et al., 1987), 1) deep ($a/h \leq 1$); 2) short ($1 < a/h \leq 2.5$); 3) slender ($2.5 < a/h \leq 6$); and 4) very slender ($6 < a/h$), where (a/h) is the shear span to depth ratio. Very slender beams fail in flexure, while slender beams without any stirrups experience diagonal tension failure. Short beams without any stirrups fail in either shear-tension (i.e. principal tensile stress exceeds allowable) or shear-compression (i.e. principal compression stress exceeds allowable) before their flexural capacity is reached. On the other hand, load transfer of deep beams without web reinforcement is carried out by tied-arch action. The most common mode of failure in deep beams is anchorage failure at the end of the tension tie combined with dowel splitting. For coupling beam, direct loads have no significant effect in the same time beam internal forces are induced mainly due to coupling action. According it is reasonable to consider that the shear span is the total length of the beam (i.e. $a = L_b$) and this can be only considered for coupling beams which governed mainly by coupling action.

On the other side, (ACI 318R-08, 2008) identified a sharp threshold ($L_b/h = 4$) to distinguish between shear and bending failure modes. According to ACI-318 requirements; coupling beams with aspect ratio, (L_b/h) ≥ 4, have to satisfy the requirements of flexural members of Special Moment Frames (SMF). While coupling beams with aspect ratio, (L_b/h) < 4, shall be permitted to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the mid-span. Each diagonal element consists of a cage of at least four longitudinal bars confined with transverse reinforcement as shown in Fig. 3.

From all the above it could be concluded that there are four main stations (L_b/h) = 1, 2.5, 4 and 6. These stations control the behavior of the coupling beam and significantly affect the overall efficiency of the system. In this paper these stations are adopted to be the main parameter of the research as illustrated in details in the next section.

4. Case study

The results of induced forces in the shear walls as well as shear force distributions in coupling beams are compared by a Finite element 3D nonlinear –modeling regular building provided with symmetrical coupled shear walls. These walls are subjected to three real earthquakes excitations, (El-Centro, 1940, Northridge, 1994, and Loma prieta, 1989). Fig. 4 presents the response spectrum and

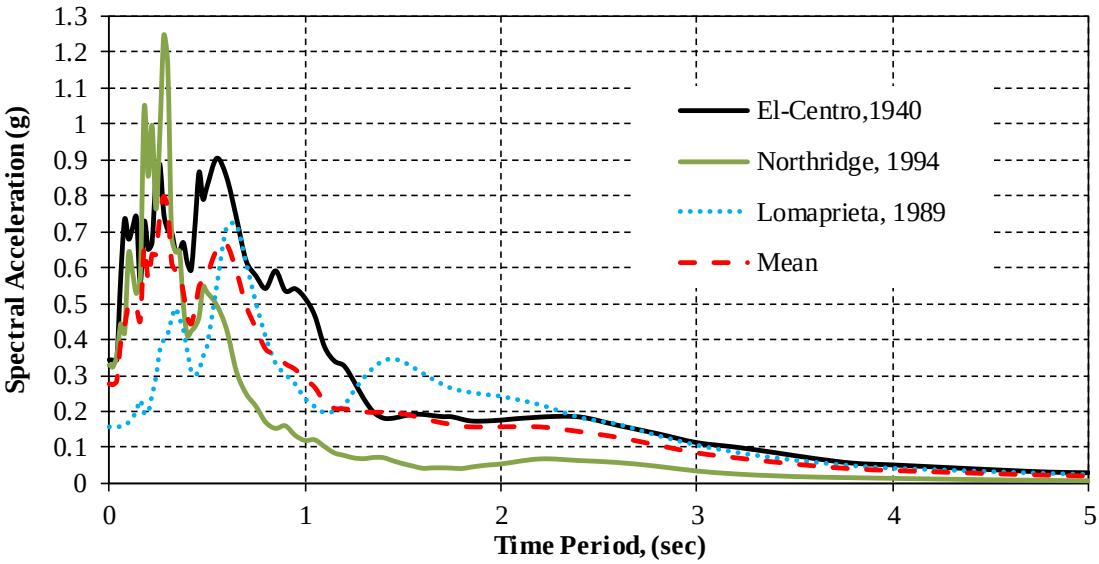

Fig. 4 The response spectrum curves and mean for three real earthquakes.

mean response spectrum for these earthquakes, respectively.

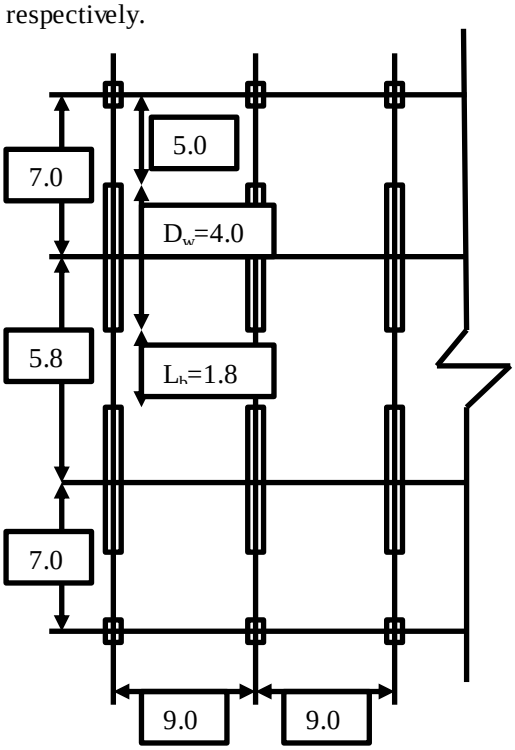

Fig. 5 Plan view of coupled shear wall building system

The dimensions are length of wall L_w= 4.0 m, Length of beam L_b=1.8 m. Therefore the total length of the coupling system B = 9.8m. The depth of beam h will be varied based on (L_b/h) as shown in Table 1, total wall height H= n (floor number) x 3.0 m (floor height); Three wall heights were adopted based on number of floors [n=10, 20, and 40], and wall thickness t_w= 200, 400-200, and 800-400-200mm; respectively.

In order to generalize the study, building height is reflect in terms of aspect ratio of the coupling system (H/B). In other words (H/B) varied (3.06, 6.12 and 12.24) based on n (10, 20 and 40) respectively.

Note that the breadth of coupling beam is equal to the corresponding wall thickness in all cases. Also it is assumed that young`s modulus of the used concrete $E_c$= 27 GPa and young`s modulus of steel E_s=*200 GPa*. The case study geometry description has been given in Figs.5 and 6.

The main geometry parameters corresponding to the various coupling beam clear span to depth ratio (L_b/h) are shown in Table 1 and Figs. 7 and 8.

In the current case study the opening width to the total length of the coupling system ratio (W_{open}/B) = 18.4%. Therefore the corresponding I_f (as defined in Eqn 4) for this system is 13.6%. This value is a characteristic value for the system used to determine the *CI* factor as defined in Eqn 6.

$$CI = 0.184 \times K_f \quad (6)$$

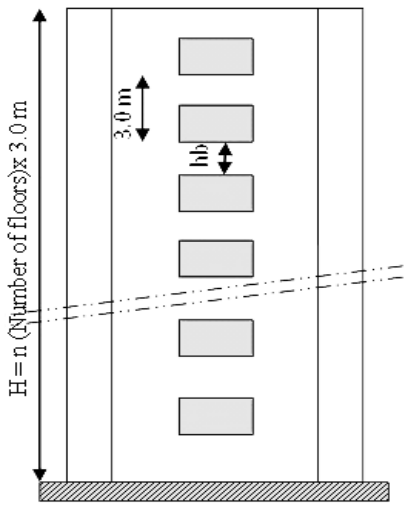

Fig. 6 Section elevation of coupled shear walls

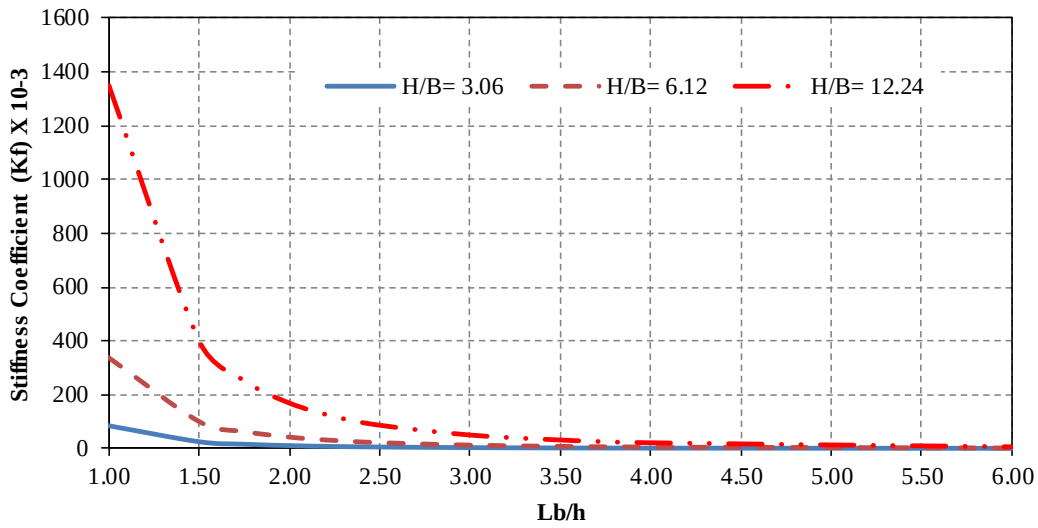

Fig. 7 Stiffness coefficient (Kf) versus coupling beam span to depth (Lb/h) ratio.

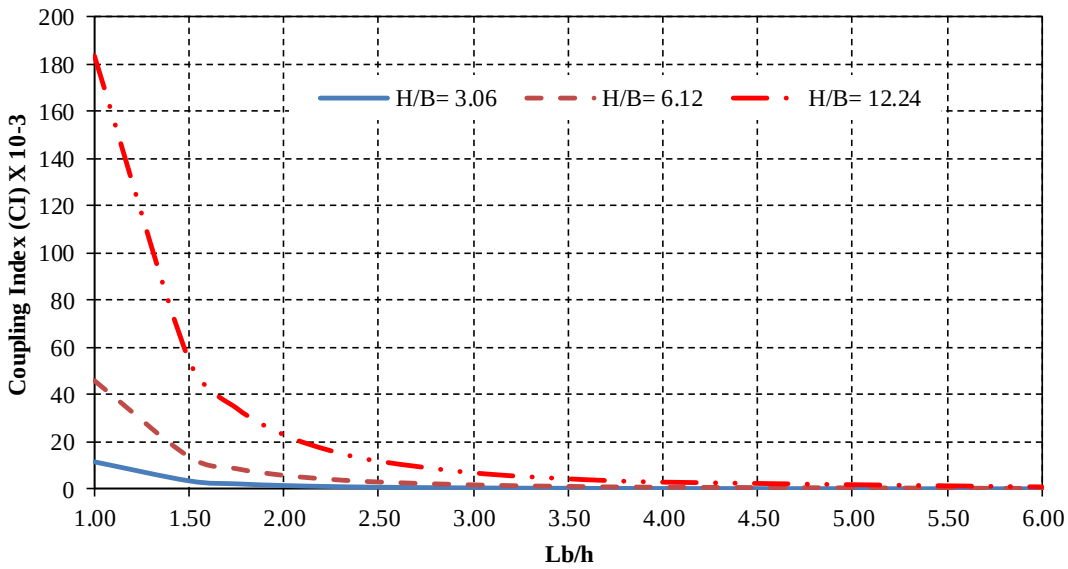

Fig. 8 Coupling index (CI) versus coupling beam span to depth (Lb/h) ratio.

Table 1: Geometric parameters and factors that were used in the parametric study.

Coupling beam aspect ratio (L_b/h)	Coupling beam depth (h) (mm)	Stiffness coefficient (Kf) x10^{-3}			Coupling Index (CI) x 10^{-3}		
		H/B= 3.06	H/B= 6.12	H/B= 12.24	H/B= 3.06	H/B= 6.12	H/B= 12.24
1.0	1800	84.38	337.50	1350.00	11.475	45.899	183.597
2.5	720	5.40	21.60	86.40	0.734	2.938	11.750
4.0	450	1.32	5.27	21.09	0.179	0.717	2.869
6.0	300	0.39	1.56	6.25	0.053	0.212	0.850

5. Modeling of monotonic cast-in-suite concrete coupled wall systems

As shown in Fig. 9, previous researches (Lequesne, R. D., et al., 2010; Parra-Montesinos, G. J., et al., 2010; Fintel, M. and Ghosh, S., 1982; Wallace, J., 2007, Weldon, B. and Kurama, Y., 2006.), has often adopted an "equivalent frame" analogy for the nonlinear inelastic hysteretic modeling and analysis of coupled wall structural systems. The properties of the wall piers and the beams are concentrated at the cetroid of each member. In the analytical model, rigid end zones (or kinematic constraints) are typically necessary to model where the coupling beams frame into the walls. Rigid end zones may also be needed in the wall piers depending on the stiffness of the coupling beams.

It is emphasized that the validity of the frame analogy used to model the behavior of coupled wall structures can vary considerably depending on the stiffnesses assumed for the members and the lengths of the rigid end zones. Furthermore, nonlinear shear deformations in deep coupling beams and the axial "elongation" and "shortening" of the tension and compression side wall piers due to axial-flexural interaction cannot be represented using frame analogy,(Park, H. and Eom, T., 2007). Base on the above mentioned analog model, the paper presents a 3-D finite element model (FEM) for the whole

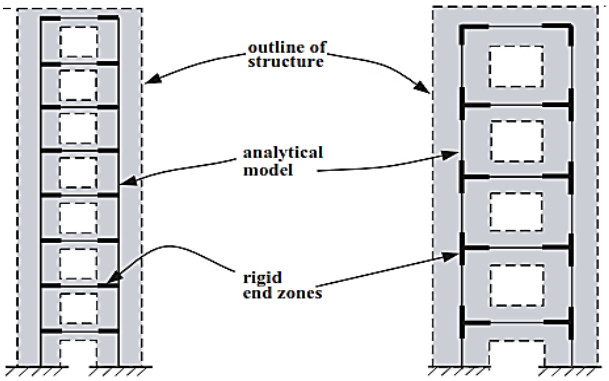

Fig. 9 Equivalent frame analytical models, as presented by M. Fintel, and S. Ghosh (1982).

building is adopted to simulate the behavior of the predefined case study to take into consideration the elongation and shortening of the tension and compression side wall piers which failed in the simulation for frame analogy .

In the FEM walls and slabs are modeled using four-nodded shell element, while columns and beams are modeled as two nodded frame elements. Coupling beam was modeled as a shell element to ensure joint connectivity and to account for shear deformations in the coupling beam. Walls and coupling beams are defined as piers and spandrels respectively in order to simplify extracting the induced straining actions as calculated at the centroid of each member of these members. Figs. 10, 11, and 12 present a perspective view of 3-D finite element model for 10, 20, 40 stories buildings, respectively.

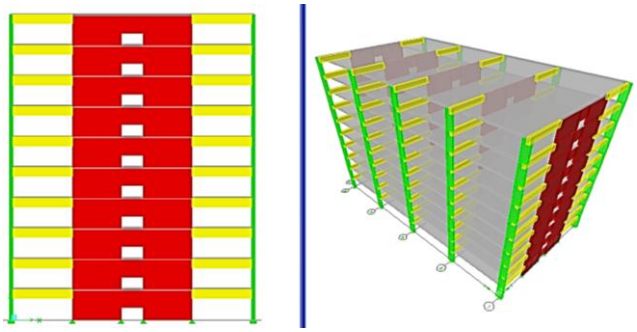

Fig. 10 Perspective view for 3-D finite element model for 10 stories building.

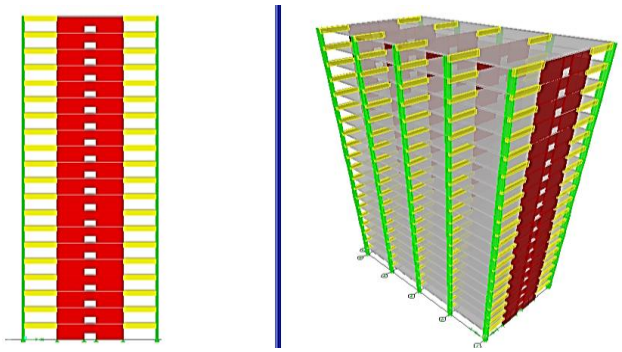

Fig. 11 Perspective view for 3-D finite element model for 20 stories building.

The cracked behavior of shear walls and other objects are modeled in ETABS (Nonlinear Ver.9.7.1, 2010), by using property modifiers to adjust their stiffness. ACI-318, recommendations for (EI) modifiers are correlated with either I_{22} or I_{33} for beams and columns, both M_{11} and M_{22} for slabs and either f_{11} or f_{22} for walls. Default settings align shear walls such that their 1-axis is horizontal and their 2-axis is vertical. As a result, the flexural modifier (EI) is applied to f_{22} for wall piers and f_{11} for spandrels (i.e. coupling beams). No recommendation is made for shear.

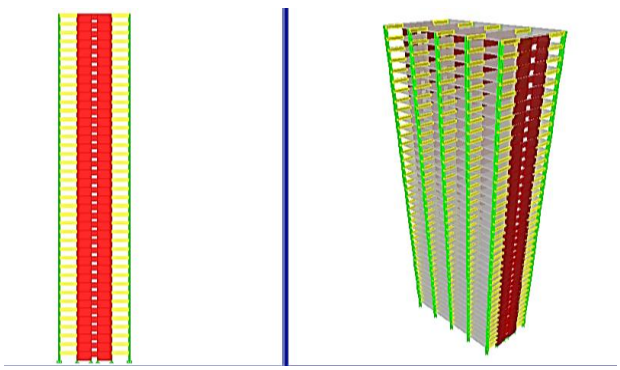

Fig. 12 Perspective view for 3-D finite element model for 40 stories building.

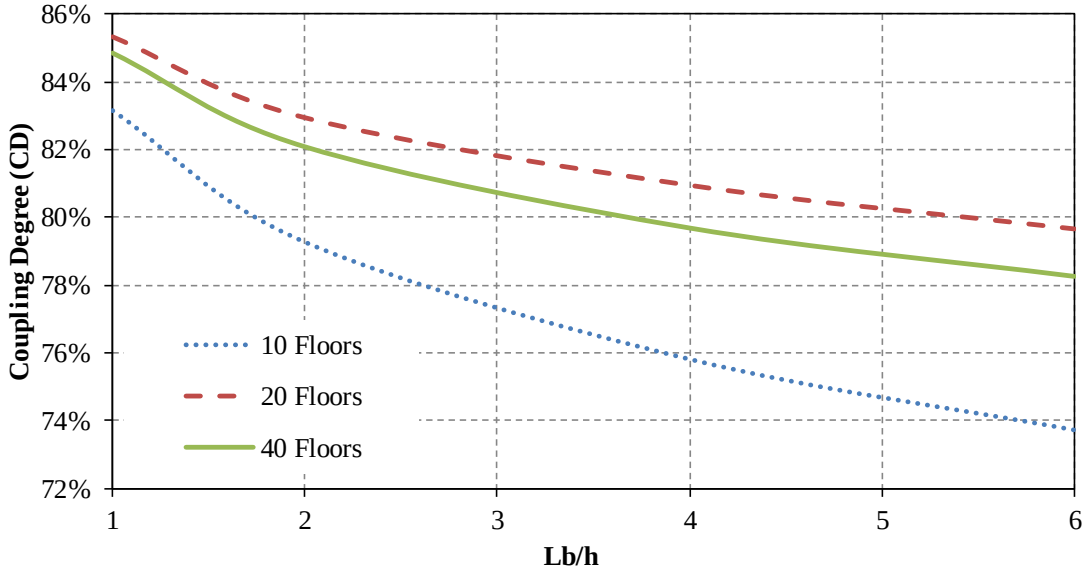

Fig. 13 Coupling degree (CD) versus beam (span/depth) ratio under El-Centro, 1940 earthquake

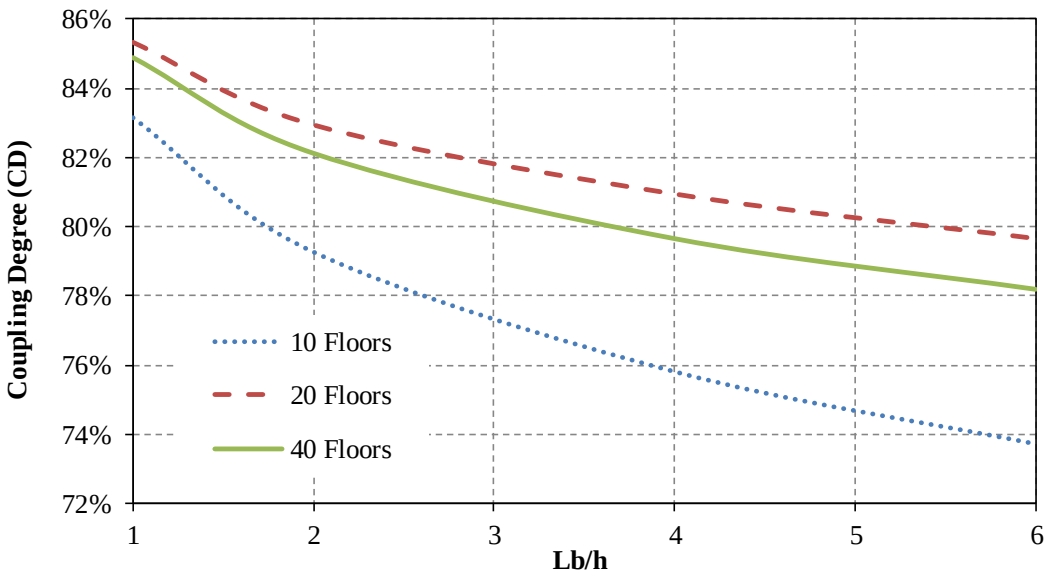

Fig. 14 Coupling degree (CD) versus beam (span/depth) ratio under Northridge, 1994 earthquake.

6. Discussion of analysis outputs of finite element 3-d model

In order to assess the behavior of the coupled shear wall and the influence of the size of the coupling beam on the system, the following parameters are selected to be studied and discussed in this section:

- Coupling Degree (CD)
- Induced shear force in the coupling beam
- Building drift
- Induced Normal force in the individual shear wall
- Induced Bending moment in the individual shear wall

6.1 Coupling degree

Figs. 13, 14, and 15 present the coupling degree (CD) percentage versus to beam (span/depth), (L_b/h) ratio for different buildings stories numbers 10, 20, and 40, respectively. The seismic analysis for these cases was

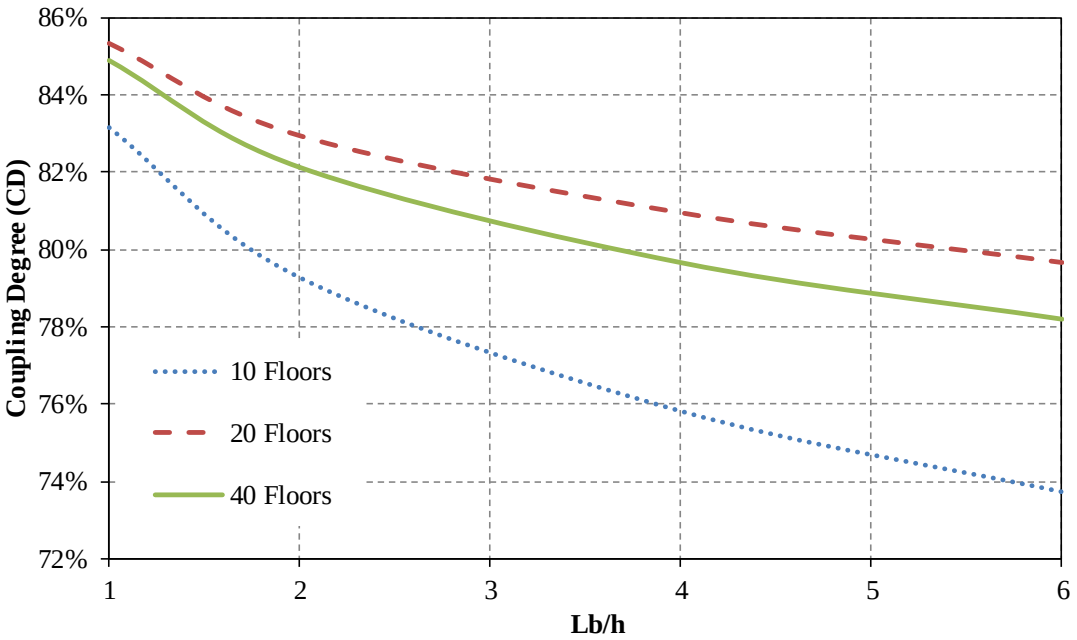

Fig. 15 Coupling degree (CD) versus beam (span/depth) ratio under Loma prieta, 1989 earthquake

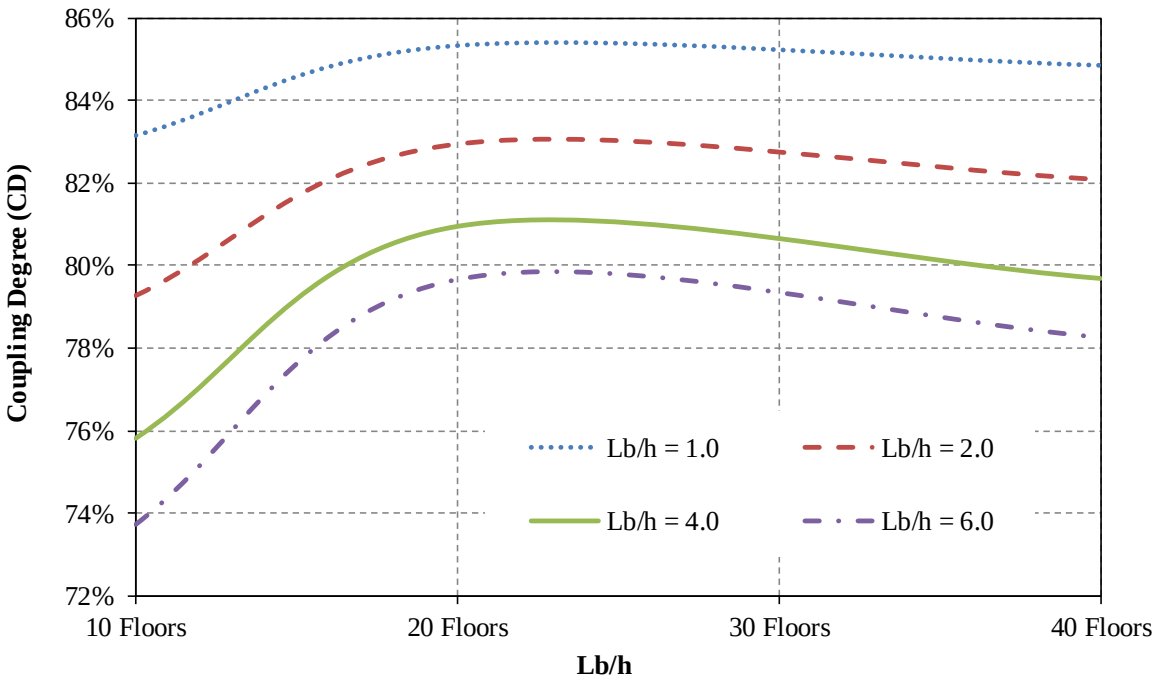

Fig. 16 Coupling Degree (CD) versus number of building stories.

done under different real earthquakes El-Centro 1940, Northridge, 1994, and Loma prieta, 1989.

Fig. 16 presents the same relation between the CD versus to different building stories number of different coupling beam (span/depth) ratio in trial to expect the optimum value of beam (span/depth) with high coupling degree percentage.

All the above mentioned three figures have the same trend, where the Coupling degree is inversely proportional to the beam stiffness. The efficiency of the coupled shear wall systems increases by the increase of the slenderness ratio (H/B) of the building system until a certain value (in this study until slenderness of 6.12. i.e. 20 stories building, as it was cleared in Fig. 16) after this value the system

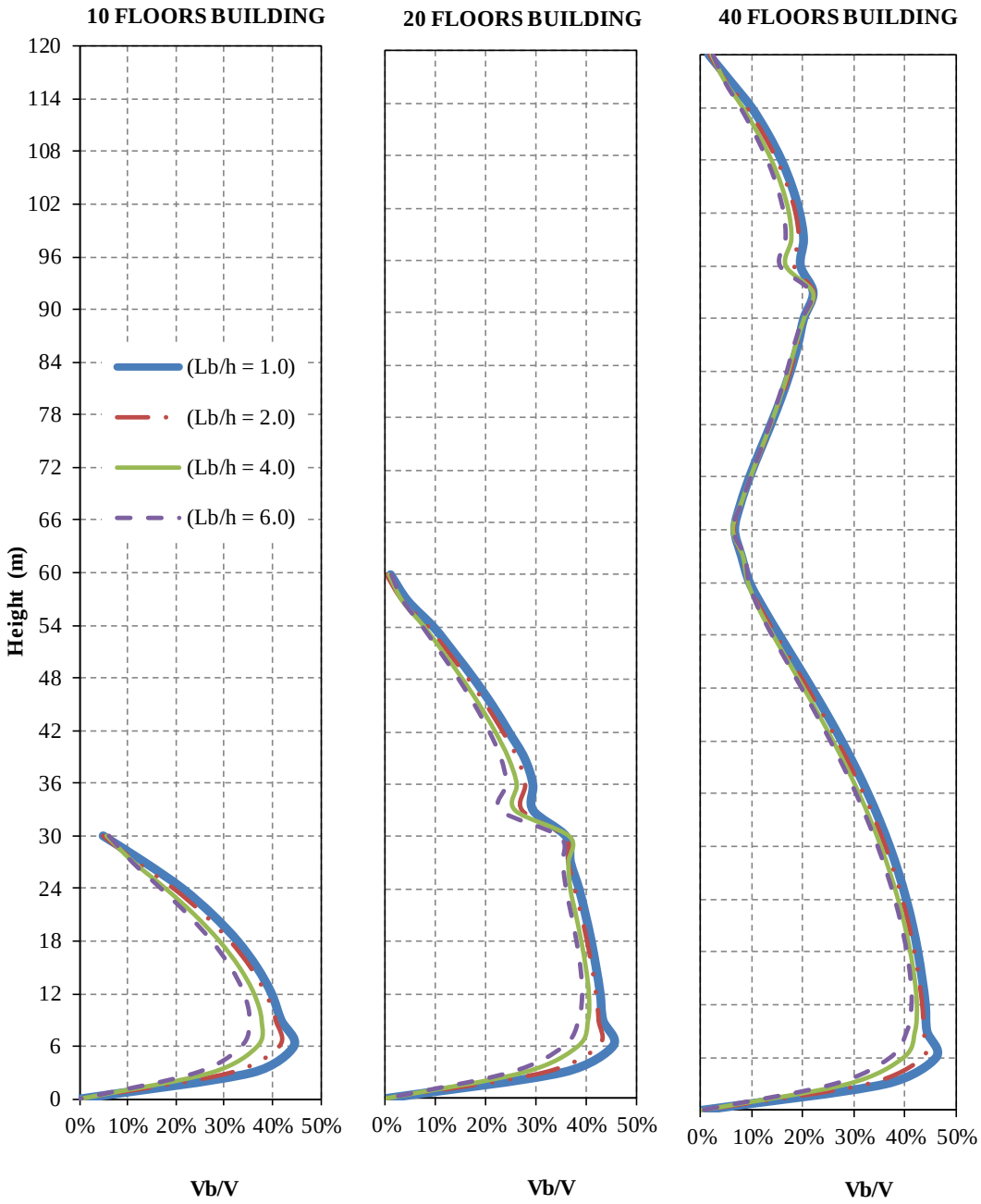

Fig. 17 Coupling shear ratio (Vb/V) versus building height.

showed much lower efficiency. To conclude there is an optimum slenderness ratio for coupling beam system depends on the dimensioning of the system and door openings size and location.Otherwise, the coupling shear wall as lateral resistance system of seismic load will be not sufficient to improve the performance of building system and may be it will be necessary to adding additional system to resist lateral load of building depend on the building slenderness ratio (H/B).

6.2 Induced shear force in the coupling beam

In this study and in order to generalize the concept, induced shear for in the coupling beam (V_b) is proportioned to the applied base shear of the building (V). As shown in Fig. 17, coupling beam exhibits the maximum shear at the second floor. For the current case study the maximum (V_b/V) is 45%. It is worth to note that the maximum ratio of (V_b/V) do not affected by the slenderness of the building system, in other words maximum (V_b/V) is constant for a particular coupling beam system whatever it's height where in the current case study the opining with to the total length of coupling system ratio (W_{open}/B) has a constant percent for all cases.

6.3 Building drift

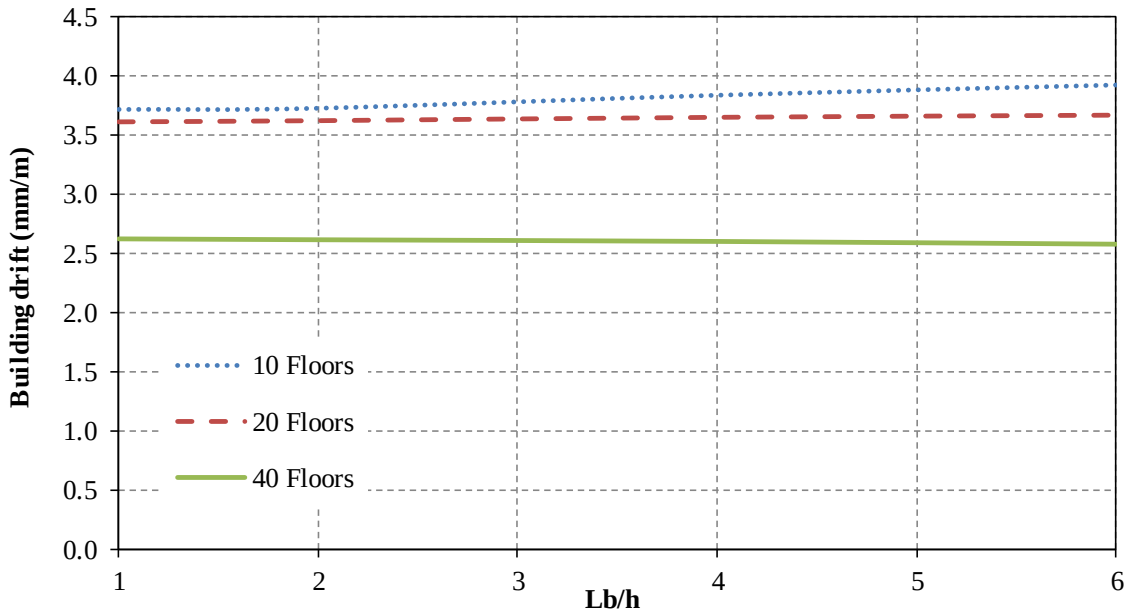

Fig. 18 Building drift versus beam span-to-depth ratio for Elcentro, 1940.

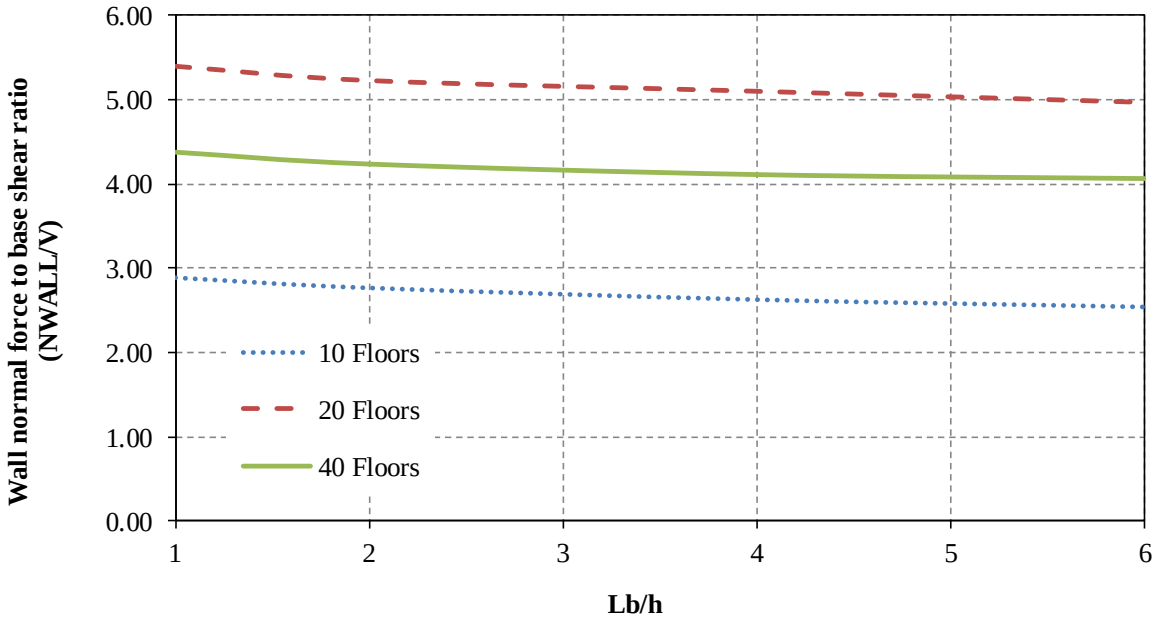

Fig. 19 Wall normal force to base shear ratio versus beam span-to-depth ratio for Elcentro, 1940.

In order to generalize the concept, the term building drift is defined as the ratio between lateral displacements at top divided by the total height of the building. Fig. 18 presents the building drift ratio versus to coupling beam span to depth ratio (L_b/h) for different building height under seismic analysis. For short buildings (in this study 10 stories building), coupling beam slightly enhances building drift. On the other hand it does not enhance building drift for medium and high rise buildings (in these study 20 and 40 stories, respectively); which can be explained based on the aspect ratio between shear wall heights values to the coupled shear wall width (H/B). In the short building the (H/B) ratio was less than 4.0 but for the medium and high rise buildings increase to 6.12 and 12.44 based on the study assumption as shown in Table 1.

6.4 Induced Normal force in the individual shear wall

Induced normal force in the individual wall (N_w) is proportioned to the applied base shear of the building (V). As shown in Fig. 19, the induced normal force is much higher that applied base shear force. In the current study the induced normal force is ranging from 3 to 6 times the base shear. In the same time it is important to note that the capacity of the individual wall is much higher in case of normal force than shear force. So it is beneficial to make a

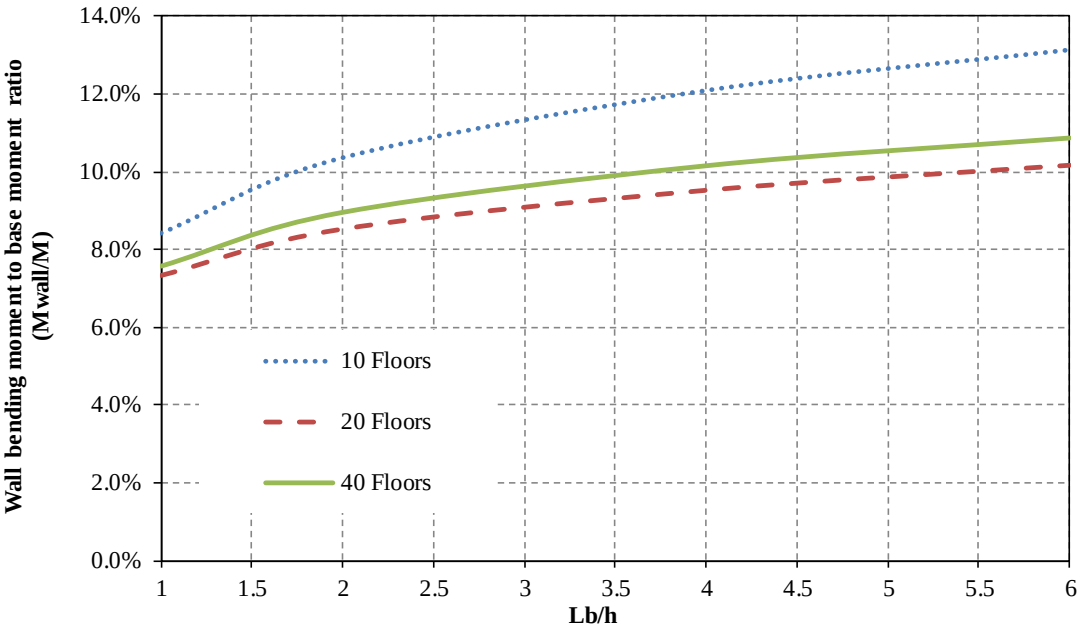

Fig. 20 Wall bending moment to base moment ratio versus beam span-to-depth ratio for Elcentro, 1940.

system that resist lateral loads by the minimum of axial resistance of the members. The induced normal force in the individual wall is directly proportional to the beam size. As the efficiency of the coupled shear wall systems increases by the increase of the slenderness ratio (H/B) of the system until a certain value, the induced normal force to base shear ratio also increased in the same manner until it reaches an optimum value at a critical slenderness ratio after that this ratio of the induced normal force to base shear started to decrease. The optimum value of normal force and the critical slenderness ratio for coupling beam system varies from system to another depending on the dimensioning of the system and door openings size and location.

6.5 Induced Bending moment in the individual shear wall

In order to generalize the concept, induced bending moment in the individual wall (M_w) is proportioned to the applied base moment of the building (M). As shown in Fig. 20, the induced bending moment is ranging from 8% to 12% the base moment.

The induced bending moment in the individual wall is inversely proportional to the beam size. As the efficiency of the coupled shear wall systems increases by the increase of the slenderness ratio (H/B) of the system until a certain value, the induced bending moment in the individual wall to base moment ratio also decreased until it reaches a minimum value at a critical slenderness ratio after that this ratio of the induced bending moment to base moment started to increase again. The minimum value of induced bending moment in the individual wall and the critical slenderness ratio for coupling beam system varies from system to another depending on the dimensioning of the system and door openings size and location.

Conclusion

The seismic response of high rise buildings with different height, 10, 20, and 40 stories building are investigated for the three real earthquake excitations, to analyze and evaluate the effect of the geometry parameters (Span/depth) ratios (1, 2, 4 and 6), and the aspect ratio of the shear wall height to coupled shear wall width (H/B) effects on the monolithic reinforcement concrete coupling beams of symmetrical coupled shear wall system. The main value points can be summarized in the following points:

1- There is an optimum slenderness ratio for coupling beam system depends on the dimensioning of the system and door openings size and location.

2- The maximum ratio of (V_b/V) ratio does not affected by the slenderness of the system. Where, the coupling beam exhibits the maximum shear force at the second floor level. The maximum (V_b/V) is 45%. It is worth to note that the maximum ratio of (V_b/V) does not affected by the slenderness of the building system.

3- For short buildings, coupling beam slightly enhances building drift. On the other hand it does not enhance building drift for medium and high rise buildings. Building drift affected by the slenderness of the wall building height to the coupled shear wall width (H/B) ratio.

4- The induced normal force in the individual wall is directly proportional to the beam size. Also, the induced normal force is ranging from 3 to 6 times the base shear. In the same time it is important to note that the capacity of the individual wall is much higher in case of normal force

than shear force. So it is beneficial to make a system that resist lateral loads by the minimum of axial resistance of the members. The optimum value of normal force and the critical slenderness ratio for coupling beam system varies from system to another depending on the dimensioning of the system and door openings size and location.

5- The induced bending moment in the individual wall is inversely proportional to the beam span to beam depth ratio (L_b/h).

6- The coupling shear wall as a lateral resistance system of seismic load will be not sufficient to improve the performance of building system and may be it will be necessary to adding additional resistance lateral load for building system depend on the building slenderness ratio (H/B).

In other wards it is worth to have a pre-analysis coefficient depends only on the geometric characteristics of the coupled system of walls. This coefficient can simply be used to eliminate number of trials judge the system or going through to many sophisticated analysis procedure and calculations.

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