Structural Engineering and Mechanics, Vol. 42, No. 5 (2012) 645-675
DOI: http://dx.doi.org/10.12989/sem.2012.42.5.645

A study on the behaviour of coupled shear walls

Dipendu Bhunia*[1], Vipul Prakash[2a] and Ashok D. Pandey[3b]

[1]*Civil Engineering Department, BITS Pilani, Pilani-333031, Rajasthan, India*
[2]*Department of Civil Engineering, IIT Roorkee, India*
[3]*Department of Earthquake Engineering, IIT Roorkee, India*

(Received November 14, 2009, Revised December 6, 2011, Accepted April 26, 2012)

Abstract. An effective design technique for symmetrical coupled shear walls is presented. Proposed formulation including assumptions and steps with mathematical formulation has been elaborated to make the design technique. An example has been considered to validate the technique with the DRAIN-3DX (1993) and SAP V 10.0.5 (2000) nonlinear programs. Parametric study has also been considered to find out the limitations along with remedial action of this technique. On the other hand, nonlinear static analysis is considered to determine the response reduction factor of coupled shear walls. Finally, it has been concluded in this paper that the proposed design technique can be considered to design the coupled shear walls under seismic motion.

Keywords: coupled shear walls; coupling beam; design technique; nonlinear behavior; capacity curve

1. Introduction

There are various building codes (ACI 318 1995, NZS 3101 (Part1) 1995, ACI 318 1999, AISC 2000 and ACI 318 2005) which provide guidelines to design a coupled shear walls considering its linear and nonlinear behavior. Similarly, Schulz (1961), Beck (1962), Rosman (1964), Burns (1965), Mcleod (1966), Coull *et al.* (1967), Choudhury *et al.* (1967), Schwaighofer (1967), Jain *et al.* (1967), Coull *et al.* (1968), Kratky *et al.* (1971), Swift *et al.* (1971), Kumar (1978), Smith *et al.* (1991), Shahrooz *et al.* (1993), Kuang *et al.* (1999), Kuang *et al.* (2000), Chan *et al.* (2000), Kuang *et al.* (2001), Aksogan *et al.* (2003) and Kim *et al.* (2003) described the different methods (Continuous connection method, Differential equation method, Matrix analysis, Finite element method, Wide column frame analogy etc.) to analyze the coupled shear walls based on its linear behavior. It has been observed that continuous connection method is based on Chitty (1947)'s concept.

All the above mentioned authors stated same assumptions in their papers. These are:

1. Coupled shear walls exhibit flexural behavior.
2. Coupling beams carry axial forces, shear forces and moments.

*Corresponding author, Assistant Professor, E-mail: dipendubhunia@gmail.com
[a]Associate Professor
[b]Assistant Professor

3. The axial deformation of the coupling beam is neglected.
4. The effect of gravity loads on the coupling beams has not been taken.
5. The horizontal displacement at each point of wall 1 is equal to the horizontal displacement at each corresponding point of wall 2 due to the presence of coupling beam.
6. The slopes and curvatures of the two walls are same at any level. and

In those papers, both symmetrical and unsymmetrical coupled shear walls were taken for analyses and the base of the shear walls were found as fixed base and flexible base (assuming spring) conditions. On the other hand, Hindalgo *et al.* (2002) described the analytical model to predict the inelastic seismic behavior of unsymmetrical shear walls with fixed base. Paulay (1970) also explained the elasto-plastic analysis of fixed base unsymmetrical coupled shear walls based on the continuous connection method and Winokur *et al.* (1968)'s design concept. He has analyzed the walls (ductility of 6) considering both uncracked and cracked sections with the assumption of formation of all plastic hinges in the coupling beams and at the base of the shear walls at the collapse mechanism. Nadjai *et al.* (1998) has also shown the ductility of 6 for flexible based (assuming spring) unsymmetrical coupled shear walls. Chan *et al.* (1989) has prepared the elastic design charts of stiffened unsymmetrical coupled shear walls with fixed base. Similarly, Hutchison *et al.* (1984), Wallace *et al.* (1992), Wallace (1994) have given methodologies to design the symmetrical coupled shear walls for fixed base condition during earthquake. Paulay *et al.* (1976), Lu *et al.* (2005) described the modeling of symmetrical ductile coupled shear walls with fixed bases. They have verified these models experimentally. Paulay *et al.* (1976) showed the ductility of coupled shear walls with diagonally reinforced coupling beam was greater than the conventional reinforced coupling beam; whereas, Lu *et al.* (2005) calculated the ductility about 4 for coupled shear walls with deep beam of $L_b/d_b < 1.5$. For completely ductile coupled shear walls, ductility exhibits 4; whereas, for partially coupled shear walls the ductility found as 3.5 (Chaallal *et al.* 1996 and Mitchell *et al.* 2003). Saka (1992), Harries *et al.* (1993), Munshi *et al.* (2000), Paulay (2002) have explained seismic design methodology for symmetrical coupled shear walls with fixed base conditions. Among those Paulay (2002) has shown the ductility of 5 at the collapse mechanism and Harries *et al.* (1993) has considered the steel coupling beam in coupled shear walls to obtain more ductility. In addition, some text books on RCC structures (Paulay *et al.* 1992, Englekirk 2003) provide guidelines for designing a coupled shear walls considering its linear and nonlinear behavior. In those books, seismic design approaches of symmetrical coupled shear walls with fixed bases have been described. Significance for considering different types of coupling beams has been explained and reduced sectional properties have been taken care for designing the shear walls. Both the books have taken the ductility factor of coupled shear walls as 5.5. However, Indian codes do not discuss anything about ductility factor of coupled shear walls. It has given only the response reduction factor as 5 for special moment resisting frames (IS 1893 (Part 1) 2002). Hence, in this paper an attempt has been made to develop a design technique to obtain an adequate ductility of coupled shear walls considering its ideal seismic behavior (stable hysteresis with high earthquake energy dissipation). It has also been seen from the above surveys that all the coupled shear walls were varying in between 6 to 40 stories; whereas, for the design purposes coupled shear walls were considered as symmetrical coupled shear walls. Therefore for preparing this design technique, symmetrical coupled shear walls have been considered. Design/capacity curve of coupled shear walls has been obtained at the collapse mechanism of the structure based on this technique. The design technique is applied to both fixed base and pinned base coupled shear walls. To start with, procedure is useful in selecting the preliminary dimensions of symmetrical coupled shear walls and subsequently arrives at a final design state. Further, this technique is particularly useful for designer, consultant and practicing engineer who have no access to sophisticated software packages. A case study has been performed implementing the technique with the help of Microsoft Excel Spreadsheet and the results have been validated. Parametric study has also been considered to find out the limitations along with remedial action of this technique. On the other hand, nonlinear static analysis is considered to determine the response reduction factor of coupled shear walls.

2. Proposed formulation

In Fig. 1, the coupled shear walls are subjected to a triangular variation of loading with amplitude F_1 at the roof level. The value of F_1 is obtained corresponding to the CP level of structure. Subsequently the base shear and roof displacement can be determined. The procedure involving Fig. 1, the assumptions, steps and mathematical calculation with initial value of F_1 as unity has been illustrated as follows.

2.1 Assumptions

The following assumptions are adopted for the design technique to obtain the ideal seismic behavior of coupled shear walls.

1. The analytical model of coupled shear walls is taken as two-dimensional entity.
2. Coupled shear walls exhibit flexural behavior.
3. Coupling beams carry axial forces, shear forces and moments.
4. The axial deformation of the coupling beam is neglected.
5. The effect of gravity loads on the coupling beams is neglected.
6. The horizontal displacement at each point of wall 1 is equal to the horizontal displacement at

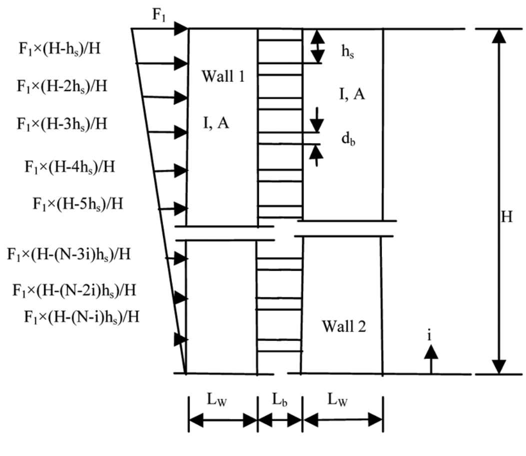

Fig. 1(a) Coupled shear walls

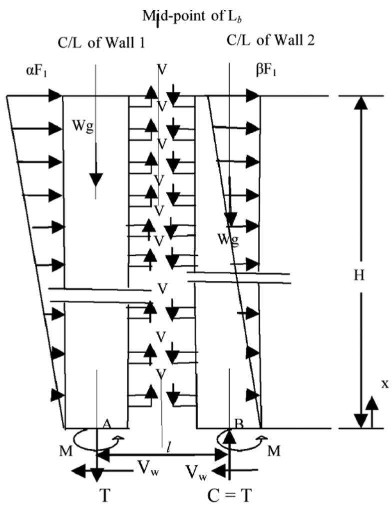

Fig. 1(b) Free body diagram of coupled shear walls each corresponding point of wall 2 due to the presence of coupling beam.

7. The slopes and curvatures of the two walls are same at any level.
8. The point of contra flexure occurs at mid point of clear span of the beam.
9. The seismic design philosophy requires formation of plastic hinges at the ends of the coupling beams. All coupling beams are typically designed identically with identical plastic moment capacities. Being lightly loaded under gravity loads they will carry equal shear forces before a collapse mechanism is formed. All coupling beams are, therefore, assumed to carry equal shear forces.
10. The nonlinear static analysis has been considered (ATC 40 1996) to carry out the study.

Hence, the distribution of the lateral loading is assumed as a triangular variation, which conforms to the first mode shape pattern.

2.2 Steps

The following iterative steps are developed in this study for the design of coupled shear walls.

1) Select type of coupling beam and determine its shear capacity.
2) Determine the fractions of total lateral loading applied on wall 1 and wall 2.
3) Determine shear forces developed in coupling beams for different base conditions.
4) Determine wall rotations in each storey.
5) Check for occurrence of plastic hinges at the base of the walls when base is fixed. For walls pinned at the base this check is not required.
6) Calculate coupling beam rotation in each storey.
7) Check if coupling beam rotation lies at collapse prevention level.
8) Calculate base shear and roof displacement.
9) Modify the value of F_1 for next iteration starting from Step (2) if Step (7) is not satisfied.

2.3 Mathematical calculation

The steps which are described in the above have been illustrated here as follows:

Step 1

The type of coupling beam can be determined according to Table 1 and shear capacity can be calculated as follows.

i. Reinforced concrete coupling beam

Shear capacity of coupling beam with conventional reinforcement $V_{sp} = 2A_s f_y(d-d')/L_b$ whereas, shear capacity of coupling beam with diagonal reinforcement $V_{sp} = 2A_s f_y(d-d')/L_b + 4A''_s f_y(d_b/2-d')/L_b$ and shear capacity of coupling beam with truss reinforcement $V_{sp} = 2A''_s f_y(d-d')/L_{d1} + 2(A_s-A''_s)f_y(d-d')/L_{d2}$. where, $L_{d1} = \sqrt{(L_b/2)^2+(d-d')^2}$ and $L_d = L_{d2} = \sqrt{(L_b)^2+(d-d')^2}$. All three shear capacities must be less than equal to $(0.08f'_c b_b d_b/\lambda_0)$. where, λ_0 is member over strength factor of 1.25. Moment capacity for all three types of reinforcements can be written as: $M_p = V_{sp} \times L_b/2$. where, d is the effective depth of the coupling beam, d' is the distance from extreme compression fiber to centroid of compression reinforcement, A_s is the total area of reinforcement and A''_s is the area of truss diagonal reinforcement.

ii. Shear dominant steel coupling beam

For I-section type of steel coupling beam, shear capacity for shear dominant steel coupling beam is denoted as $V_{sp} = 0.6 f_y t_{w'} (D - 2t_f)$ and moment capacity is $M_p = Z_p f_y$; which, f_y is yield stress of steel, $t_{w'}$ is web thickness, D is the overall depth of the section, t_f is flange thickness and Z_p is plastic section modulus.

iii. Flexure dominant steel coupling beam

The transferable shear force (V_{nf}) for flexure dominant steel coupling beam is the lesser of $2M_p/e$ and V_{sp}; where, M_p is the moment capacity which is $Z_p f_y$.

Step 2

In Fig. 1(b), free body diagram of coupled shear walls has been shown; α and β are fractions of total lateral loading incident on wall 1 and wall 2, such that

$$\alpha + \beta = 1.0 \tag{1}$$

For symmetrical coupled shear walls, moments of inertias of two walls are equal for equal depths and thicknesses at any level. Further, curvatures of two walls are equal at any level. Hence based on the Assumption (7), Eq. (1) can be written as

$$\alpha = \beta = 0.5 \tag{2}$$

Step 3

In this step, it is explained how to calculate the shear force developed in the coupling beams for different types of boundary conditions. CSA (1994), Chaallal *et al.* (1996) defined the degree of coupling which is written as

Table 1 Modified parameters governing the coupling beam characteristics controlled by shear

Type of coupling beam	Shear Span to Depth Ratio	$\frac{L_b}{d_b}$	Type of detailing	Plastic Rotation Capacity (Radians): $\frac{Shear}{b_w d \sqrt{f_c'}}$	Plastic Rotation Capacity (Radians): CP
Reinforced concrete coupling beam	$\alpha \leq 2$	No limit	Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	0.015
				≥6	0.010
		< 1.5	Diagonal Reinforcement (strength is an overriding consideration and thickness of wall should be greater than 406.4 mm)	-	< 0.03
		1.5 to 4.0	Truss Reinforcement (additional experimentation is required)	-	0.03-0.08
Steel coupling beam	$e \leq \frac{1.6M_p}{V_{sp}}$		Shear dominant	-	$\frac{0.15}{L_b}$

Table 2 Values of constant k and exponents a, b and c

N	k	a	b	c
6	2.976	0.706	0.615	0.698
10	2.342	0.512	0.462	0.509
15	1.697	0.352	0.345	0.279
20	1.463	0.265	0.281	0.190
30	1.293	0.193	0.223	0.106
40	1.190	0.145	0.155	0.059

$$DC = \frac{T \times l}{M_{ot}} \tag{3}$$

where, $l = L_w + L_b$; T is the axial force due to lateral loading; M_{ot} is total overturning moment at the base of the wall produced due to lateral loading. For fixed base condition DC varies from 0 to 1 and Eq. (3) can also be written as

$$DC = k\frac{(d_b)^a}{(L_w)^b \times (L_b)^c} \tag{3a}$$

The above Eq. (3a) is proposed by Chaallal *et al.* (1996); where, k is constant and a, b and c are exponents which are given in Table 2.

So based upon the above criteria and considering Eqs. (3) and (3a), shear force developed in the coupling beam could be determined as follows:

Fixed base condition:

For fixed base condition following equation can be written as

$$C = T = \sum_{i=1}^{N} V_i = \frac{M_{ot}}{l} \times k\frac{(d_b)^a}{(L_w)^b \times (L_b)^c} \tag{4}$$

where, N is the total number of stories and M_{ot} is total overturning moment at the base due to the lateral loading.

Therefore, based on the Assumption (9) shear force in coupling beam at each storey is

$$V = \frac{\sum_{i=1}^{N} V_i}{N} \tag{5}$$

Pinned base condition:

In this study, pinned base condition has been introduced as one of the possible boundary conditions for coupled shear walls. It is expected that stable hysteresis with high earthquake energy dissipation can be obtained for considering this kind of base condition; although no previous papers described this kind of boundary condition.

DC is 1 for pinned base condition from the Eq. (3). Hence, the equation can be written as

$$C = T = \sum_{i=1}^{N} V_i = \frac{M_{ot}}{l} \tag{6}$$

Therefore, based on the Assumption (9) shear force in coupling beam at each storey is

$$V = \frac{\sum_{i=1}^{N} V_i}{N} \tag{7}$$

Step 4

After getting α, β and V at each storey for the particular value of F_1, bending moment values in each storey could be determined for each wall. Subsequently, curvature diagram for each wall is generated by using moment area method as adopted in the Microsoft excel spreadsheet which is required to determine the wall rotation in each storey. The following equations are considered to calculate the wall rotation.

Overturning moment at a distance 'x' from base with respect to each wall can be written as

$$M_{ox}(x) = \sum_{j=0}^{N-i} \left\{ 0.5 \times \frac{F_1}{H}(H - jh_s)(H - x - jh_s) \right\} \tag{7a}$$

where, i is storey number and it is considered from the base as 0, 1, 2, 3, …, N. Resisting moment in wall due to shear force in the coupling beam at a distance 'x' from base can be written as

$$M_{wr}(x) = \left(\frac{L_w}{2} + \frac{L_b}{2}\right) \sum_{j=i}^{N} V_j \tag{7b}$$

whereas, net moment in the wall at a distance 'x' from base generated due to overturning moment and moment due to shear force in the coupling beam, can be written as

$$M_{net}(x) = M_{ot}(x) - M_{wr}(x) \tag{7c}$$

Wall rotation at ith storey for fixed base can be written as

$$\theta_{wi} = \frac{\int_0^{ih_s} M_{net}(x)dx}{EI_w} \tag{7d}$$

where

$$I = \frac{t_w \times L_w^3}{12} \tag{7e}$$

For plastic hinge rotation at the fixed base of wall or rotation at the pinned base of wall, Eq. (7d) could be written as

$$\theta_{wi} = \frac{\int_0^{ih_s} M_{net}(x)dx}{EI_w} + \theta_{w0} \tag{7f}$$

Step 5

i. Tensile forces at the base of wall 1 (T) as well as compressive forces at the base of wall 2 (C) are calculated due to lateral loading.
ii. Compressive loads at the bases of wall 1 and wall 2 are calculated due to gravity loading.
iii. Net axial forces at the bases of wall 1 and wall 2 are calculated, i.e., Net axial force = Tensile or

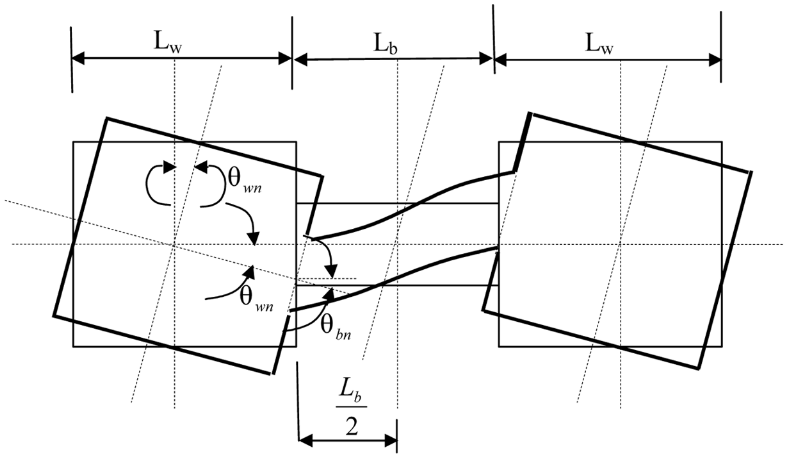

Fig. 2 Deformed shape of a *i*th storey symmetrical coupled shear walls

Compressive force due to lateral loading (*T* or *C*) ± Compressive load due to gravity loading.

iv. Then, according to these net axial forces for the particular values of f_{ck}, b, d and p, the yield moment values at the bases of wall 1 and wall 2 can be determined from P-M interaction curve (IS 456 1978, Jain 1999). where f_{ck}, b, d and p are yield strength of concrete, breadth of a section, depth of that section and percentage of minimum reinforcement in that particular section,; and P is the axial force and M is the moment; here net axial force is considered as P in the P-M interaction curve.

v. Therefore, if calculated bending moment value at any base of the two walls is greater than yield moment value, plastic hinge at that base would be formed, otherwise no plastic hinge would be formed.

Step 6

The rotation of coupling beam in each storey is determined as follows:

Rotation of coupling beam at *i*th storey for symmetrical walls (Englekirk 2003) according to Fig. 2 is given by

$$\theta_{bi} = \theta_{wi}\left(1 + \frac{L_w}{L_b}\right) \tag{8}$$

where, θ_{wi} is rotation of wall at *i*th storey and can be calculated according to Eq. (7d), L_w = depth of wall, L_b = length of coupling beam.

For plastic hinge rotation at the fixed base of wall or real hinge rotation at the pinned base of wall, Eq. (8) could be written as

$$\theta_{bi} = L_{wb}\{\theta_{wi}\} \tag{9}$$

where, θ_{wi} can be calculated as per Eq. (7f) for fixed base of wall or for pinned base of wall and

$$L_{wb} = \left(1 + \frac{L_w}{L_b}\right) \tag{10}$$

Step 7

The rotational limit for collapse prevention level of different types of RCC coupling beams and steel beams are given in Table 1. Check whether the rotations of beams attain their rotational limit of CP level at the collapse mechanism of the structure simultaneously.

Step 8

The roof displacements can be calculated as per the following equations

$$(\Delta_{roof}) = h_s \times \left(\sum_{i=0}^{N} \theta_{wi} \right) \tag{11}$$

The Base shear can be calculated as follows

$$V_B = \frac{F_1 \times (N+1)}{2} \tag{12}$$

Step 9

The F_1 will be modified as follows if the condition of Step 7 is not satisfied:

To obtain the collapse mechanism of the structure, it is required to increase F_1 with equal increment until coupling beams attain their rotation limit of CP level simultaneously.

Table 2a Dimensions and material properties of coupled shear walls

Depth of the wall (L_w)	4 m	Width of coupling beam (b_b)	300 mm
Length of coupling beam (L_b)	1.8 m	Storey height (h_s)	3.0 m
Depth of coupling beam (d_b)	600 mm	Modulus of concrete (E_c)	27.0 GPa
		Modulus of steel (E_s)	200.0 GPa
Number of stories (N)	20	Steel yield strength (f_y)	415 MPa
Wall thickness (t_w)	300 mm		

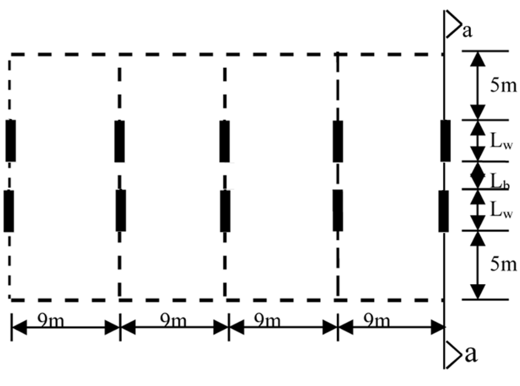

Fig. 3(a) Plan view of building

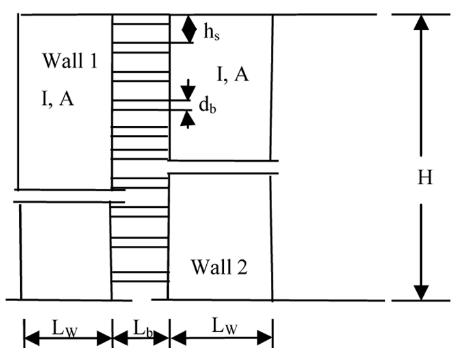

Fig. 3(b) Coupled shear walls at section 'a-a'

3. Validation of the proposed design technique

The following numerical example has been considered to validate the propose design technique. In this study plan and elevation with dimensions and material properties of the coupled shear walls have been adopted as given in Chaallal *et al.* (1996).

3.1 Numerical example

The results obtained from the proposed method are compared with the results obtained in SAP V 10.0.5 (2000) and DRAIN-3DX (1993) software packages. These walls are subjected to triangular variation of lateral loading. Table 2a mentions the different parameters with dimensions and material properties which have been considered in the study.

Fig. 3(a) and Fig. 3(b) show the plan and sectional elevation of the coupled shear wall building.

3.2 Loading consideration

Dead loads (DL) of 6.7 kN/m^2 and live loads (LL) of 2.4 kN/m^2 have been taken as given in Chaallal *et al.* (1996). Total gravity loading on coupled shear walls at section ‘a-a’ has been calculated as the sum of dead load plus 25% LL using IS 1893 (part 1) (2002) for floor; however, in case of roof only dead load is considered.

3.3 Modeling of coupled shear walls in proposed design technique

The modeling of coupled shear walls involving Fig. 1, assumption and steps with mathematical calculation in design technique is already described.

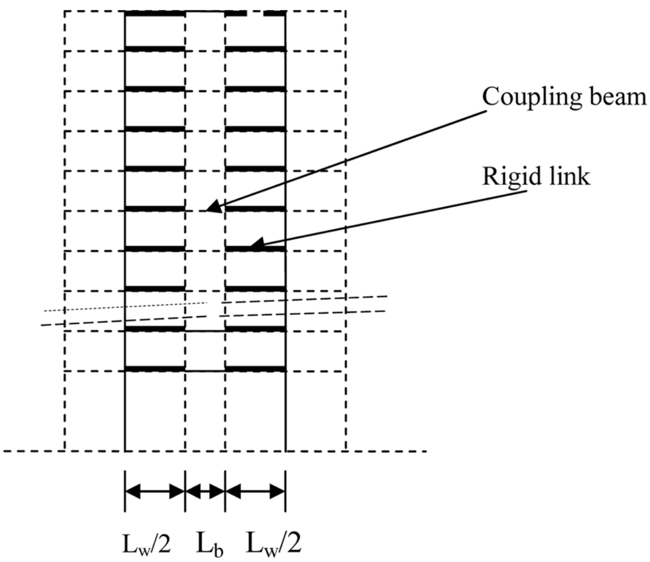

Fig. 4 Modeling in SAP V 10.0.5 (2000) and DRAIN-3DX (1993)

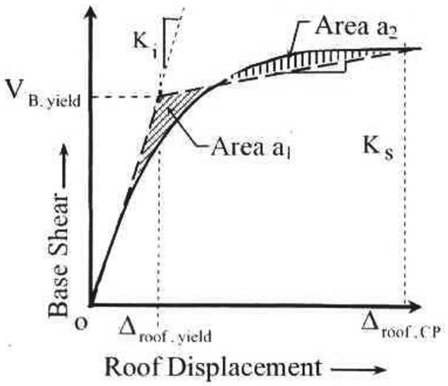

Fig. 5 Bilinear representation for capacity curve

3.4 Modeling of coupled shear walls in SAP and DRAIN-3DX

Wide column frame analogy (Mcleod 1966) has been considered for modeling in SAP V 10.0.5 (2000) and DRAIN-3DX (1993) according to Fig. 4. In this analogy, shear walls are represented as two line elements (centre line of shear wall) and beams are represented as line elements (centre line of beam) by joining with each other with rigid link. Beam column elastic element (Type-17) and inelastic element (Type-15) are used for modeling.

3.5 Calculation of ductility

The obtained design/capacity curve from the proposed design technique, SAP V 10.0.5 (2000) and DRAIN-3DX (1993) is bilinearized. The bilinear representation is prepared in the following manner based on the concepts given in ATC 40 (1996).

It can be seen from Fig. 5 that bilinear representation can be due to the basis of initial tangent stiffness and equal energies (Area a_1 = Area a_2).

Subsequently, ductility of the coupled shear walls has been calculated as

$$\mu_s = \frac{\Delta_{roof,CP}}{\Delta_{roof,yield}} \tag{13}$$

4. Results and discussions

Coupled shear walls at section ‘a-a’ as shown in Fig. 3 are considered for conducting the study.

RCC coupling beam with Conventional longitudinal reinforcement with conforming transverse reinforcement: RCC coupling beam with Conventional longitudinal reinforcement with conforming transverse reinforcement in each storey has been selected as per Table 1 for the study. The results of this study for fixed base as well as pinned base conditions have been shown in the following manner.

Discussions of numerical results:

Figs. 6(a) and 6(b) shows that the results obtained from proposed design technique for both fixed

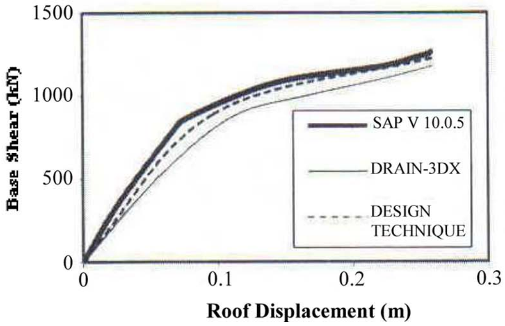

Fig. 6(a) Capacity curve for fixed base condition

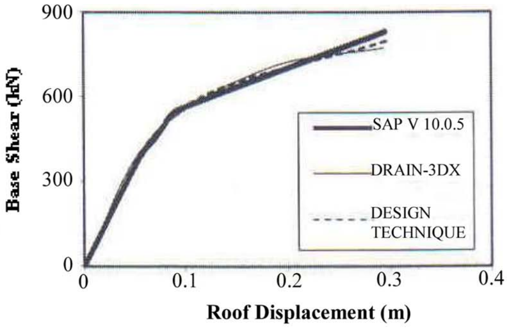

Fig. 6(b) Capacity curve for pinned base condition

Table 3 Ductility of coupled shear walls considering different approaches

Method	Ductility	
	Fixed base	Pinned base
Proposed Design Technique	7	7.5
DRAIN-3DX (1993)	6.75	7.45
SAP V 10.0.5 (2000)	6.92	7.47

and pinned base conditions are almost similar to the results obtained from DRAIN-3DX (1993) and SAP V 10.0.5 (2000). Ductility of coupled shear walls (Table 3) is obtained for all three methods. It is noticed that ductility for pinned base condition is greater than fixed base conditions for all three methods. However, it is necessary to find the limitations of the proposed design technique. Therefore, parametric study is described to detect the limitations of the proposed design technique in the following manner.

4. Parametric study

It has been seen from the CSA (1994) and Chaallal *et al.* (1996)'s papers that the behavior of the ductile coupled shear walls depends on degree of coupling. whereas, degree of coupling depends upon depth and length of the coupling beam as well as depth and height of the coupled shear walls (Park *et al.* 1975, Paulay *et al.* 1992).

Therefore, study has been restricted on length of the coupling beam and number of stories as basic variables and other parameters are taken as constant. These parameters have been considered in proposed method to make out effect on the behavior of coupled shear walls. Further, modifications to achieve desirable behavior according to the proposed method have been included in this paper.

4.1 Model for parametric study

Fig. 3 and DL of value 6.7 kN/m^2 & LL of value 2.4 kN/m^2 have been considered to carry out the parametric study.

Table 4 Dimensions and material properties of coupled shear walls for parametric study

Depth of the wall (L_w)	4 m	Width of coupling beam (b_b)	300 mm
Length of beam (L_b)	1 m, 1.5 m and 2 m	Storey height (h_s)	3.6 m
Depth of beam (d_b)	800 mm	Modulus of concrete (E_c)	22.4 GPa
Number of stories (N)	10, 15 and 20	Yield strength of steel (f_y)	415 MPa
Wall thickness (t_w)	300 mm		

4.2 Parameters

Table 4 mentions the different parameters with dimensions and material properties which have been considered to carry out the parametric study.

4.3 Expected behavior

Overturning moment is generated when coupled shear walls are subjected to lateral loading. This moment is resisted by moment in each wall and axial force induced in the wall. Overturning moment at a distance 'x' from base with respect to each wall can be calculated as per the Eq. (7a).

From the Eq. (7a) it is evident that overturning moment increases with increase in F_1 and H.

Resisting moment in wall due to shear force in the coupling beam at a distance 'x' from base can be calculated by the Eq. (7b).

whereas, net moment in the wall at a distance 'x' from base generated due to overturning moment and moment due to shear force in the coupling beam, can be calculated by the Eq. (7c).

According to the Eq. (7c), if net moment at that the base of the wall is greater than its yield moment capacity, plastic hinge occurs at the base for fixed base condition. But for pinned base condition, net moment at the base is zero.

Wall rotation at ith storey can be calculated by the Eq. (7d). For plastic hinge rotation at the fixed base of wall or for rotation at the pinned base of wall, wall rotation at ith storey can be calculated by the Eq. (7f).

From Eqs. (7d) and (7e), it is clear that wall rotation increases with decrease in L_w. However, beam rotation decreases with decrease in wall rotation and increase in L_b, (Eqs. (8), (9) and (10)). Thus triangular variation of lateral loading with amplitude of F_1 at the roof level increases until coupling beams attain their rotational limit of CP level. The storey displacement increases towards the roof as storey displacement is the summation of the wall rotation from base to roof (Eqs. (11) and (12)). For symmetrical coupled shear walls, displacement, wall rotation and bending moment of two walls are equal due to their equal flexural rigidity.

4.4 Analysis using proposed design technique

The above mentioned building has been studied by the design technique. The results for different parameters have been described in this section.

4.5 Observed behavior

To study the influence of length of the coupling beam (L_b) on the behavior of coupled shear walls, length of the coupling beam is considered as 1 m, 1.5 m and 2 m for both fixed and pinned base conditions. RC coupling beam with Conventional longitudinal reinforcement with conforming transverse reinforcement has been selected. Shear capacity in the coupling beam is calculated as in Step 1. The rotational limit of coupling beam has been selected as per Step 7. The study has been done for coupled shear walls with number of stories 20, 15 and 10 for both fixed and pinned bas conditions.

For Number of Stories $N = 20$

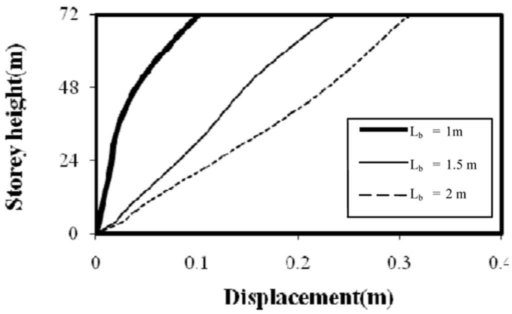

Fig. 7(a) Storey displacement for fixed base condition at CP level

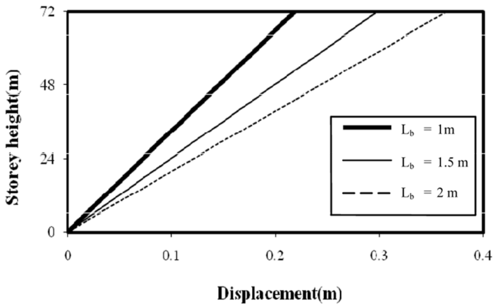

Fig. 7(b) Storey displacement for pinned base condition at CP level

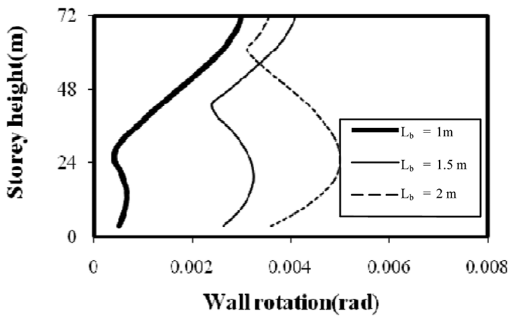

Fig. 8(a) Wall rotation for fixed base condition at CP level

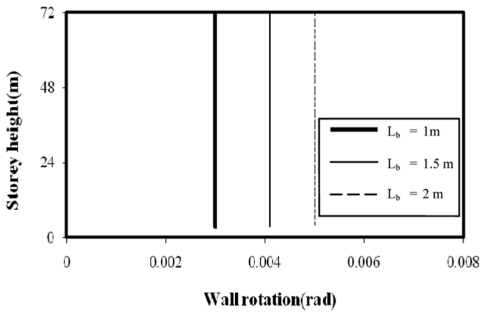

Fig. 8(b) Wall rotation for pinned base condition at CP level

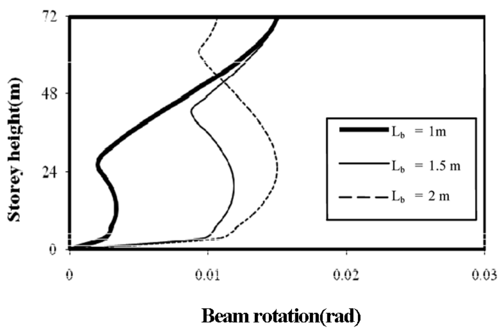

Fig. 9(a) Beam rotation for fixed base condition at CP level

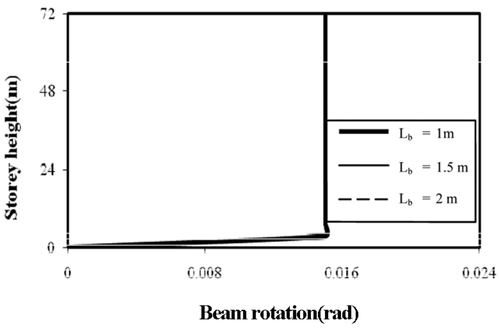

Fig. 9(b) Beam rotation for pinned base condition at CP level

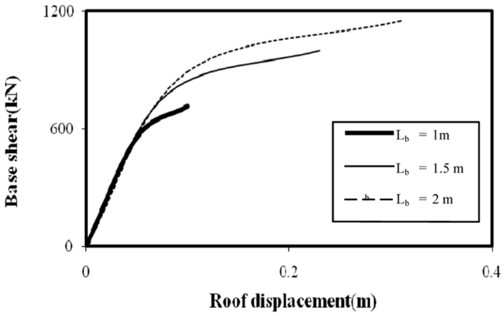

Fig. 10(a) Capacity curve for fixed base condition

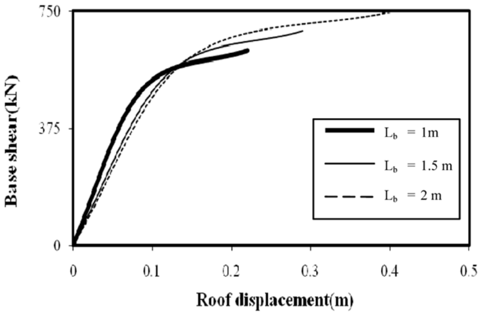

Fig. 10(b) Capacity curve for pinned base condition

4.6 Discussion of results for N = 20

The deflection for the case of pinned base condition is much higher than the case of fixed base (Fig. 7), whereas, the base shear for the case of pinned base condition is lower than the case of fixed base (Fig. 10). It means that the results obtained from the proposed method are satisfactory. Since wall rotation is proportional to the length of the beam (Fig. 8) and deflection is the summation of the wall rotation, deflection is proportional to the length of the beam (Fig. 7). It has been also observed that all beams reach to their rotational limit of CP level for pinned base condition; whereas, most of the beams reach to their rotational limit of CP level for fixed base condition (Fig. 9). Hence, coupled shear walls is behaving as a rigid body motion for pinned base condition; which is expected. However, explanations of the results as per Fig. 9 are given in the following manner for fixed base condition:

i) The rotation of the cantilever wall is maximum at the free end of the wall. This rotation decreases towards the base of the wall and is zero at the base for fixity.
ii) Fixed base coupled shear walls with short span coupling beam behaves as a cantilever wall (L_b = 1 m of Fig. 8). It means that this kind of coupled shear walls behaves like a single shear wall; which is expected as per behavior concerned.
whereas, fixed base coupled shear walls with long span coupling beam does not behave as a cantilever wall (L_b = 1.5 m and L_b = 2 m of Fig. 8). It shows unexpected behavior.
iii) Beam rotation is proportional to the wall rotation.

Therefore, it can be said from the above observations that coupled shear walls with short span coupling beam (L_b = 1 m) will be more acceptable in comparison with the long span coupling beam

Table 5 Ductility of coupled shear walls for N = 20

Base condition	Length of the coupling beam (L_b)	Values
Fixed	1 m	3.33
	1.5 m	4.8
	2 m	6.3
Pinned	1 m	5.11
	1.5 m	6.35
	2 m	7.1

(L_b = 1.5 m and L_b = 2 m) although the behavior of all three coupling beam is governed by shear according to Table 1.

With the help of Eq. (13), Fig. 5 and Fig. 10, ductility for pinned base condition and fixed base condition has been calculated according to the following table.

It has been seen from the above table that ductility is found more for pinned base condition in comparison with the fixed base condition and ductility is increased with increase in length of the coupling beam.

For Number of Stories N = 15

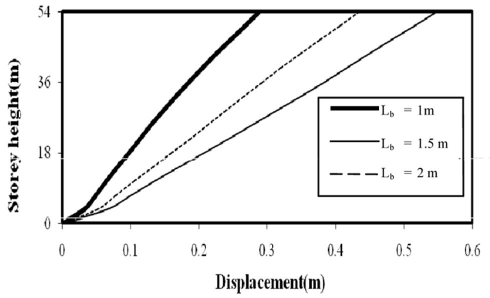

Fig. 11(a) Storey displacement for fixed base condition at CP level

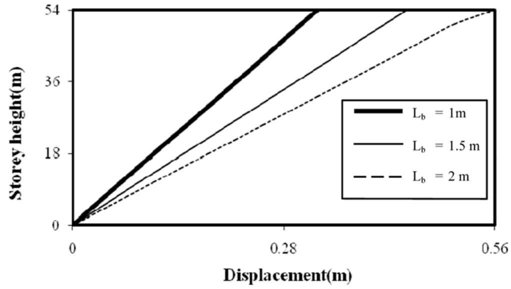

Fig. 11(b) Storey displacement for pinned base condition at CP level

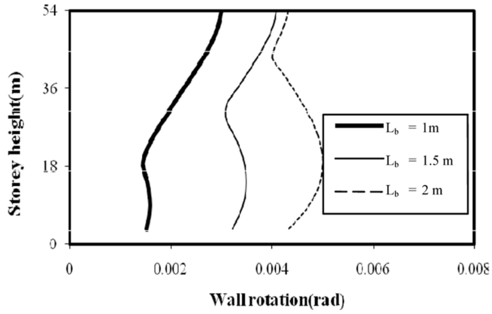

Fig. 12(a) Wall rotation for fixed base condition at CP level

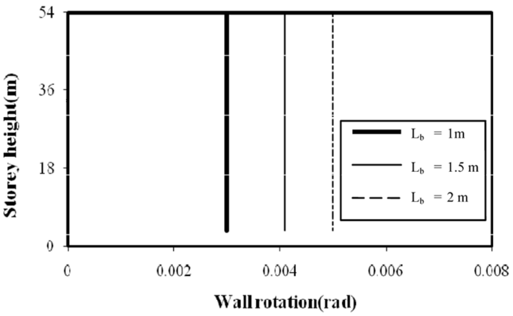

Fig. 12(b) Wall rotation for pinned base condition at CP level

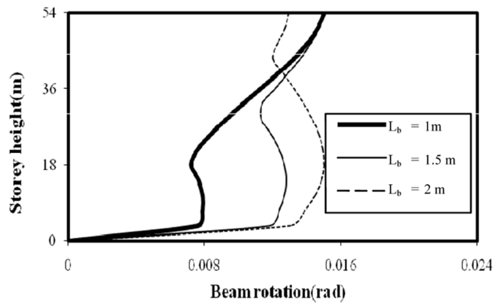

Fig. 13(a) Beam rotation for fixed base condition at CP level

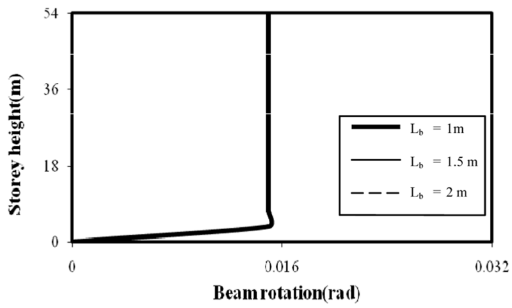

Fig. 13(b) Beam rotation for pinned base condition at CP level

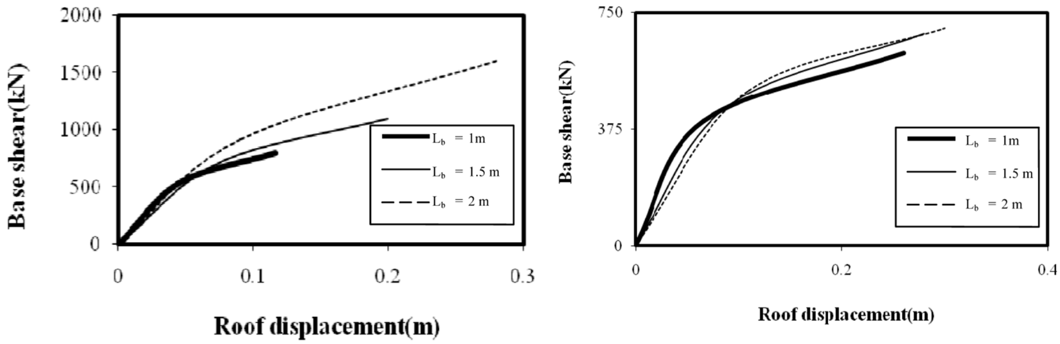

Fig. 14(a) Capacity curve for fixed base condition

Fig. 14(b) Capacity curve for pinned base condition

Table 6 Ductility of coupled shear walls for $N = 15$

Base condition	Length of the coupling beam (L_b)	Values
Fixed	1 m	2.93
	1.5 m	4.0
	2 m	5.9
Pinned	1 m	4.5
	1.5 m	5.85
	2 m	6.87

4.7 Discussion of results for N = 15

With the help of Eq. (13), Fig. 5 and Fig. 14, ductility for pinned base condition and fixed base condition has been calculated as per the following table.

It has been observed from Figs. 11 to 14 and Table 6 that the results obtained for $N = 15$ are similar with the results of $N = 20$ for fixed base condition and pinned base condition.

For Number of Stories $N = 10$

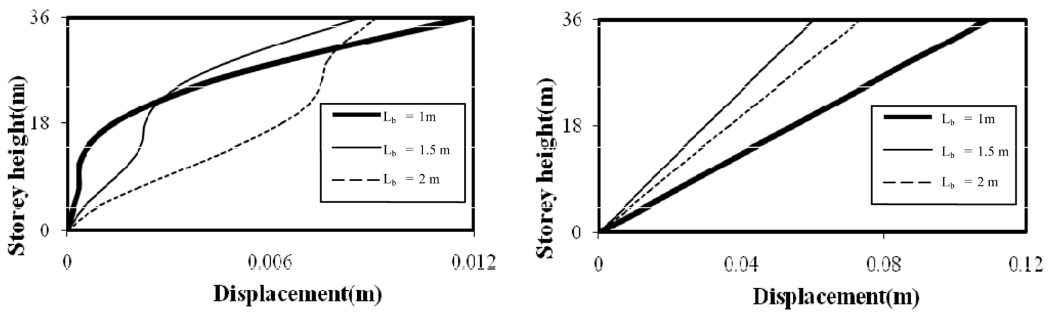

Fig. 15(a) Storey displacement for fixed base condition at CP level

Fig. 15(b) Storey displacement for pinned base condition at CP level

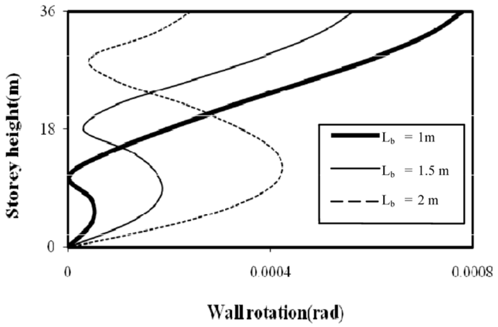

Fig. 16(a) Wall rotation for fixed base condition at CP level

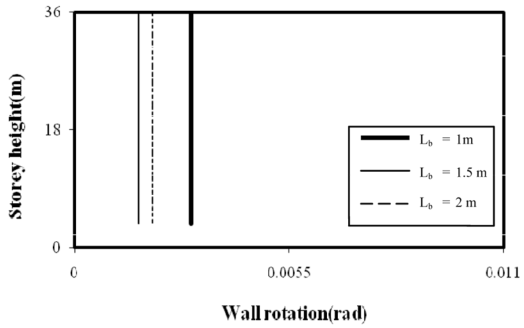

Fig. 16(b) Wall rotation for pinned base condition at CP level

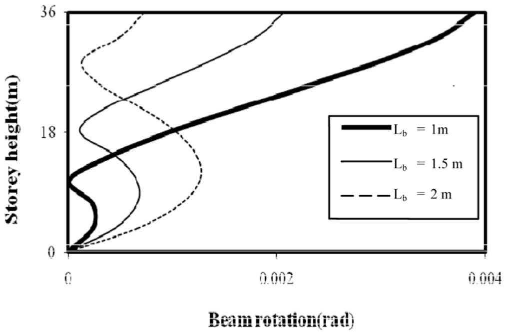

Fig. 17(a) Beam rotation for fixed base condition at CP level

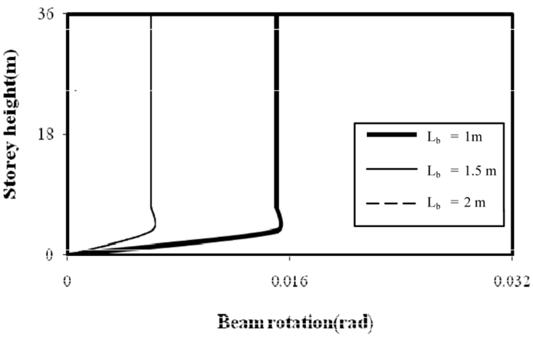

Fig. 17(b) Beam rotation for pinned base condition at CP level

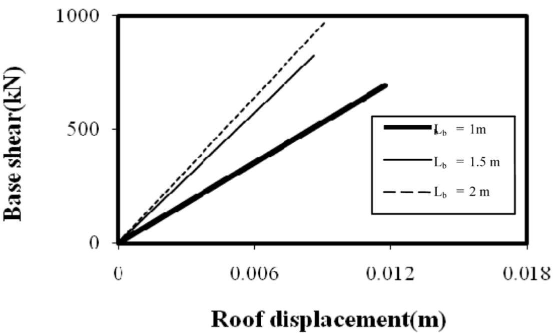

Fig. 18(a) Capacity curve for fixed base condition

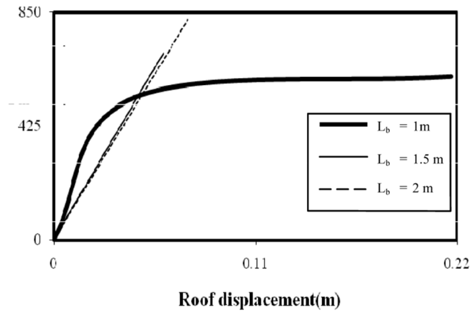

Fig. 18(b) Capacity curve for pinned base condition

4.8 Discussion of results for N = 10

Figs. 15 to 18 show that capacity curve reaches CP level for the case of L_b = 1 m with pinned base condition only. However, capacity curve does not reach the CP level for the other cases. It means that ideal seismic behavior of coupled shear walls has only been achieved for L_b = 1 m with pinned base condition. Now, remedial action has been considered in the following manner to obtain the ideal seismic behavior.

Remedial Action for $N = 10$

The remedy for the cases of L_b = 1 m, 1.5 m and 2 m with fixed base condition and L_b = 1.5 m and 2 m with pinned base condition to achieve CP level is mentioned below.

To obtain the CP level, it is required to increase the wall rotation. Since wall rotation (Eqs. (7d) and (7e)) is inversely proportional to the L_w^3, it is required to decrease the L_w.

It has been observed that the desirable behavior of coupled shear walls has been achieved (Figs. 19 to 22).

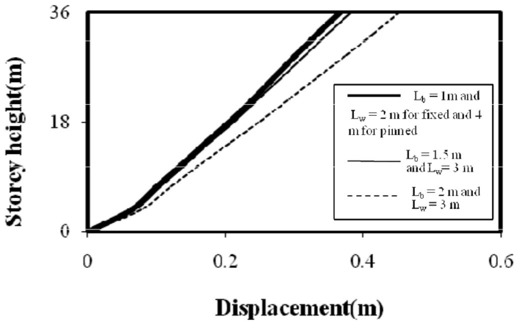

Fig. 19(a) Storey displacement for fixed base condition at CP level

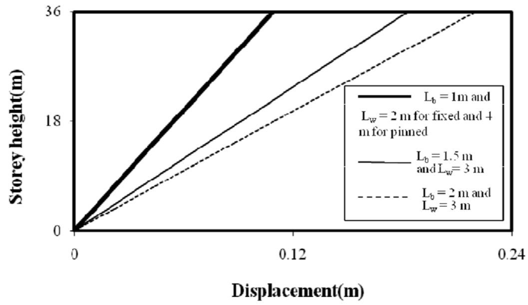

Fig. 19(b) Storey displacement for pinned base condition at CP level

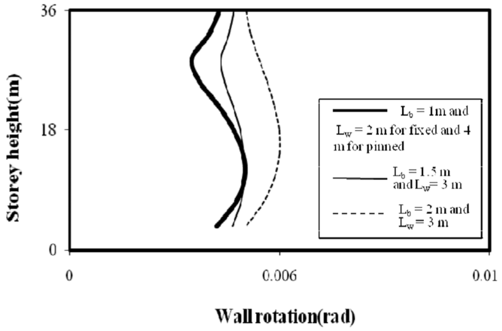

Fig. 20(a) Wall rotation for fixed base condition at CP level

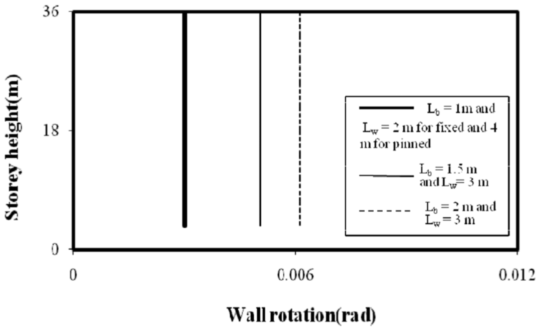

Fig. 20(b) Wall rotation for pinned base condition at CP level

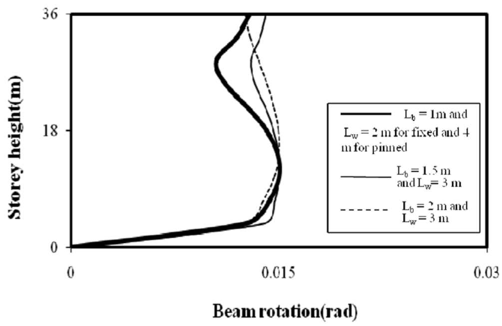

Fig. 21(a) Beam rotation for fixed base condition at CP level

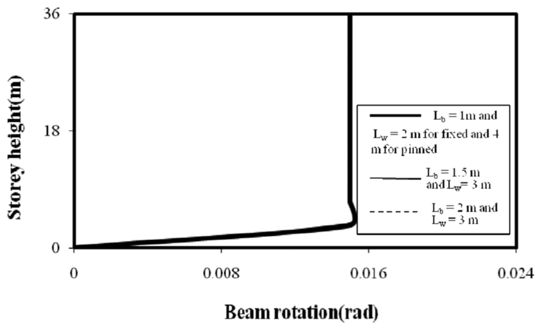

Fig. 21(b) Beam rotation for pinned base condition at CP level

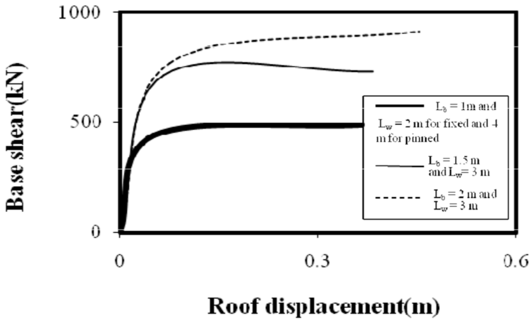

Fig. 22(a) Capacity curve for fixed base condition

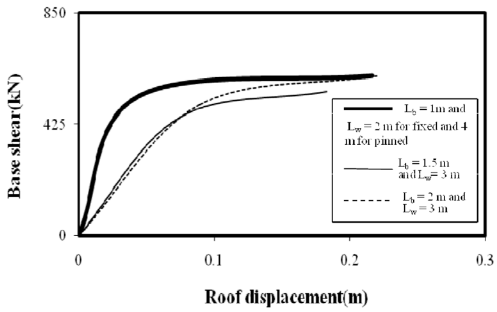

Fig. 22(b) Capacity curve for pinned base condition

4.9 Discussion of the above results

Figs. 20(a) and 20(b) show expected behavior for pinned base condition and unexpected behavior for fixed base condition according to the explanations given in discussion of results for N = 20 although ideal seismic behavior has been achieved.

The following salient features have been illustrated based on the above studies:

Coupled shear walls with pinned base condition shows rigid body motion, which is expected as per behavior concerned. Pinned base condition also shows better nonlinear behavior (adequate ductility with high earthquake energy dissipation) in compare to the fixed base condition. On the other hand, coupled shear walls with fixed base condition shows surprising behavior with regard to the wall rotation for long span coupling beam and reduced depth of wall, ; although desirable behavior (adequate ductility with high earthquake energy dissipation) has been achieved. For this reason, to obtain consistency between the behavior with respect to the wall rotation and the behavior based on the high earthquake energy dissipation, coupled shear walls ($N \geq 15$ with equal storey height h_s = 3.6 m) should be designed with an optimum ratio of L_b/L_w = 0.25 for L_b/d_b = 1.25 and $I_b/I_w = 8 \times 10^{-03}$. It also shows that the behavior of coupling beam should be governed by shear.

5. Nonlinear static analysis

In this paper, nonlinear static analysis is also carried out to determine the response reduction factors of coupled shear walls at different earthquake levels.

5.1 Calculation of performance point

It is required to compute the performance points at different earthquake levels to find response reduction factors of coupled shear walls as per ATC 40 (1996). Thus, it is necessary to discuss the following parameters for calculation of performance point one by one in brief. Selection of coupling beam

There are different types of RC coupling beams and steel coupling beams whose geometry, rotational capacities and moment capacities are different. The behavior of coupled shear walls is governed by these coupling beams. Therefore, it is necessary to select the proper type of coupling beam based on its geometry, rotational capacity, moment capacity and the demand for the particular zone. Table 1 describes the different types of coupling beams and the parameters which govern the coupling beam characteristics.

Capacity curve

Two key elements of a performance based design procedure are capacity and demand. Capacity is a representation of the structure's ability to resist the seismic demand; whereas, demand is a representation of the earthquake ground motion. The structure must have the adequate capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of design. ATC 40 (1996) provides the procedure of construction of capacity spectrum and demand spectrum for NSP in PBSD. Matsagar and Jangid (2008) and Prasanth *et al.* (2008) have considered this procedure for their work.

Demand curve

The demand curve is the curve plotted as spectral acceleration versus time period. The demand curve is considered as 5% damped elastic response spectra for the particular zone as per IS 1893 (Part 1) (2002).

Modal analysis

The modal analysis is carried out to calculate the modal frequencies and corresponding mode shapes which are taken up for calculation of modal participation factor and modal mass coefficient for the first natural mode. SAP V 10.0.5 (2000) is considered for modal calculations. Wide column frame analogy has been considered for modeling of coupled shear walls in SAP V 10.0.5 (2000) as shown in Fig. 4. In this analogy, shear walls are represented as two line elements (centre line of shear wall) and beams are represented as line elements (centre line of beam) by joining each other with rigid link.

Identification of performance point

There are different methods which identify the performance point for the particular zone given in the ATC 40 (1996). This paper considers procedure 'A' for calculation of performance point.

Roof drift and ductility

The roof drift is the displacement at roof level when the structure is subjected to external forces. In this paper, roof drift (Pore 2007) is taken as the roof displacement at the performance point (pp). Calculation of ductility has already been discussed based on the concept of bilinearization (ATC 40 1996) and conventional definition of ductility, i.e., $\mu_\Delta = \Delta_{roof,CP}/\Delta_{roof,yield}$. However, this definition cannot represent the reduction in energy dissipation capacity on account of bilinear form of capacity curve. This formula is meant only for elastic-perfectly-plastic (EPP) behavior under monotonic loading and unloading. It is also implied in the definition that ultimate displacement is at least few times of yield displacement which is meant that energy dissipation is required to be significantly greater than energy recovery. The following two equations (Pore 2007) are then considered to find out the ductility, i.e.

$$\mu_{\delta e1} = \frac{\mu_\delta}{1+\alpha_1(\mu_\delta - 1)} \quad \text{for monotonic loading} \tag{14}$$

where, $\alpha_1 = k_1/k$; k_1 is post yield stiffness and k is unloading stiffness.

$$\mu_{\delta e2} = \frac{\alpha_e \mu_\delta}{\alpha_e(\mu_\delta + 1) - \mu_\delta} \quad \text{for cyclic loading} \tag{15}$$

where, $\alpha_e = 1 + \alpha_1(\mu_\delta - 1)$

These two equations are proposed on the basis of energy dissipation and energy recovery involved in the problem for two cases of loading – monotonic and cyclic. These equations adopted for general case of bilinear behavior with hardening is able to provide results consistent with the conventional EPP idealization as well as accommodates the elastic behavior where no energy dissipation is involved. These equations are also expressed in terms of the area under hysteretic loop. This helps to incorporate the stiffness and/or strength degradation effects. Relationship among μ_δ and the two proposed energy based forms is

$$\mu_\delta \ge \mu_{\delta e1} \ge \mu_{\delta e2}$$

5.2 Calculation of response reduction factor

Response reduction factor depends on three parameters –
(i) Over strength factor
(ii) Ductility factor
(iii) Redundancy factor

However, There has no proper definition available in IS codes which is required to classify those three factors individually. The response reduction factor serves as calibration factor (Prakash 2004, Prakash *et al.* 2006, Pore 2007) to incorporate the observed seismic performance of a particular structure type under earthquake shaking. This factor is the only link between linear or elastic behavior and non linear behavior (Pore 2007). It provides an important contribution to the present PBSD methodologies (Bertero 1986, Miranda 1997). IS 1893 (Part 1) (2002) says that response reduction factor depends on overstrength factor and ductility factor. Therefore, response reduction factor, R using IS 1893 (part 1) (2002) can be as

$$\mathrm{R}_{\mathrm{IS1893}} = R_s \times R_\mu \tag{16}$$

where, R_s is overstrength factor which can be calculated as (Pore 2007)

$$R_s = \frac{\text{Base shear at limiting response or the maximum or ultimate level base shear, whichever is LOWER}}{\text{Base shear at first yield or (non-factored) design base shear, whichever is LOWER}} \tag{17}$$

Based on the studies of 13 formulations for R_μ the ductility factor, Pore (2007) proposed formulation as follows

$$R_\mu = \frac{\text{Base shear corresponding to elastic response, i.e., for } \mu = 1}{\text{Base shear corresponding to limiting performance level, i.e., for } \mu \ge 1} \tag{18}$$

If redundancy factor R_R is to be included in the above Eq. (16), the modified equation can be written as

$$R_{modified} = R_s \times R_\mu \times R_R \tag{19}$$

where, R_R is redundancy factor which can be calculated as (Pore 2007)

$$\left.\begin{aligned} R_R &= \frac{\text{Base shear at maximum or ultimate level}}{\text{Base shear at limiting response}} \geq 1.0 \text{ for } \delta_l \leq \delta_u \\ R_R &= 1.0 \quad \text{for } \delta_l > \delta_u \end{aligned}\right\} \tag{20}$$

where, δ_u is the roof displacement at ultimate strength capacity and δ_l is the displacement at limiting response.

ATC 40 (1996) describes the following reduction factors, i.e.

$$R_{\mu\xi} = \frac{1.65}{2.31 - 0.41 \times 1n(100\kappa\xi_{eq} + 5)} \text{ for constant acceleration region} \tag{21}$$

and

$$R_{\mu\xi} = \frac{2.12}{3.21 - 0.68 \times 1n(100\kappa\xi_{eq} + 5)} \text{ for constant velocity region} \tag{22}$$

where, $\xi_{eq} = \left(\frac{2}{\pi} \times \frac{\mu_{\delta e2} - 1}{\mu_{\delta e2}} \times 100\right)$ and $\kappa = \left\{1.13 - 0.51 \times \left(\frac{\mu_{\delta e2} - 1}{\mu_{\delta e2}}\right)\right\} \leq 1.0$

where $\mu_{\delta e1}$ and $\mu_{\delta e2}$ can be calculated from Eqs. (14) and (15).

Pore (2007) proposed a formula to find out the total response reduction factor

$$R = R_{S\min} \times R_{STratio} \times R_{\mu\xi} \times R_{\mu Telong} \times R_R \text{ for constant velocity region} \tag{23}$$

$$= R_{S\min} \times R_{STratio} \times R_{\mu\xi} \times R_R \text{ for constant acceleration region} \tag{24}$$

where, $R_{\mu_{Telong}} = \sqrt{\mu_{\delta e1}}$; $R_{ST_{ratio}} = \frac{(Sa/g)_{Tappro}}{(Sa/g)_{Tdyn.}}$; R_R is already defined above; $R_{S\min}$ can be found with the help of the Eq. (17).

However, Pore (2007) has shown that the above two Eqs. (22) and (23) having few shortcomings associated with formulation for κ and $R_{\mu\xi}$. Then he proposed following two modified approaches to find out the response reduction factor, i.e.

(a) First Method of Energy-Ductility Based Response Reduction

$$R_{\mu_{IDRS}} = \mu_{eq} = \mu_{\delta e1} \text{ for constant velocity region} \tag{25}$$

$$R_{\mu_{IDRS}} = \sqrt{2 \times \mu_{eq} - 1} = \sqrt{2 \times \mu_{eq2} - 1} \text{ for constant acceleration region} \tag{26}$$

Table 7 Dimensions and material properties of coupled shear walls for nonlinear static analysis

Depth of the wall (L_w)	4 m	Width of coupling beam (b_b)	300 mm
Length of beam (L_b)	1 m	Storey height (h_s)	3.6 m
Depth of beam (d_b)	800 mm	Modulus of concrete (E_c)	22.4 GPa
		Modulus of steel (E_s)	200.0 GPa
Number of stories (N)	20 and 15	Steel yield strength (f_y)	415 MPa
Wall thickness (t_w)	300 mm		

(b) Second Method of Energy-Ductility Based Response Reduction:

The First method is based on the same set of assumptions as that of the ATC 40 (1996) procedure up to the point of finding μ_{eq}; whereas, in this method all those assumptions are eliminated and the response reduction factor for ductility effects has been computed.

$$R_{\mu_{IDRS}} = \sqrt{2\mu_{eq}-1} \text{ for the constant acceleration region} \tag{27}$$

$$R_{\mu_{IDRS}} = \mu_{eq} \text{ for the constant velocity region of spectra.} \tag{28}$$

where,
$$\mu_{eq} = \left(\frac{\mu_{\delta e2}-1}{\mu_{\delta e2}} \times \mu_{\delta e1}\right) + 1 \tag{29}$$

5.3 Design example

Table 7, Fig. 3 and DL of value 6.7 kN/m^2 & LL of value 2.4 kN/m^2 have been considered to carry out the non linear static analysis of coupled shear walls. These walls are subjected to triangular variation of lateral loading. The base of the walls is assumed as fixed. Soil type for Roorkee is assumed as medium (Type II); maximum considered earthquake (MCE) level and design basis earthquake level (DBE) are considered for the study.

Results and discussions

The results and discussions have been discussed in the following manner for determining the response reduction factors at DBE and MCE levels. The coupling beam is selected here as coupling beam with conventional reinforcement.

Calculation of response reduction factor at the performance point

Table 8 shows different response reduction factors for MCE and DBE level. These are calculated at different performance points.

From the above table, response reduction factor of coupled shear walls is varying between 1.22 to 2.05 for maximum considered earthquake (MCE) level; which is almost same as the provision of CSA (1994) for coupling beam with conventional reinforcement.

From the above studies it is observed that response reduction factor for coupled shear walls ($N \geq 15$ with equal storey height h_s = 3.6 m and L_b/L_w = 0.25 for L_b/d_b =1.25 and I_b/I_w = 8 $\times 10^{-03}$) with conventional reinforced coupling beam is found almost same as per the provision provided by CSA (1994) standard.

Table 8 Response reduction factors for DBE and MCE levels

Parameters		$\mu_{\delta e1}$	$\mu_{\delta e2}$	$R_{\mu\xi}$	$R_{\mu IDRS}$ [First Method of Energy-Ductility Based Response Reduction]	$R_{\mu IDRS}$ [Second Method of Energy-Ductility Based Response Reduction]	*Rd* as per CSA (1994)
n = 20	DBE	1.04	1.004	1.02	1.04	1.00	1.5 or 2 for coupled shear walls with conventional reinforced coupling beam
	MCE	2.05	1.2	1.58	2.05	1.34	
n = 15	DBE	1.01	1.00	1.002	1.01	1.00	
	MCE	1.87	1.13	1.39	1.87	1.22	

6. Conclusions

(i) The result obtained from design technique exhibits ideal seismic behavior of coupled shear walls.
(ii) For the case of pinned base, ductility was obtained more than the fixed base condition.
(iii) Coupled shear walls with pinned base condition showed rigid body motion, is expected.
(iv) Pinned base condition showed better nonlinear behavior (adequate ductility with high earthquake energy dissipation) in comparison to the fixed base condition.
(v) Coupled shear walls ($N \geq 15$ with equal storey height h_s = 3.6 m) should be designed with an optimum ratio of L_b/L_w = 0.25 for L_b/d_b =1.25 and $I_b/I_w = 8 \times 10^{-03}$ to obtain consistency between the behavior with respect to the wall rotation and the behavior based on the high earthquake energy dissipation.
(vi) Ductile response reduction factor for coupled shear walls ($n \geq 15$ with equal storey height h_s = 3.6 m and L_b/L_w = 0.25 for L_b/d_b =1.25 and $I_b/I_w = 8 \times 10^{-03}$) with conventional reinforced coupling beam was found almost same as per the provision provided by CSA (1994) standard.
(vii) Response reduction factors obtained at DBE and MCE levels were found as consistent.

Finally, it can be concluded that proposed design technique can be considered to design the coupled shear walls under seismic motion.

References

American Concrete Institute (1995), *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (318R-95)*, ACI 318-95 and ACI 318R-95, Farmington Hills, Michigan.

American Concrete Institute (1999), *Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary (318R-99)*, ACI 318-99 and ACI 318R-99, Farmington Hills, Michigan.

American Concrete Institute (2005), *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05)*, ACI 318-05 and ACI 318R-05, Farmington Hills, Michigan.

American Institute of Steel Construction, Inc. (2000), *Seismic Provisions for Structural Steel Buildings*, Supplement No. 2, Chicago, IL.

Aksogan, O., Arslan, H.M. and Choo, B.S. (2003), "Forced vibration analysis of stiffened coupled shear walls using continuous connection method", *Eng. Struct.*, **25**, 499-506.

Applied Technology Council: ATC-40 Report (1996), *Seismic Evaluation and Retrofit of Concrete Buildings*, Volume I, Redwood City, California.

Beck, H. (1962), "Contribution to the analysis coupled shear walls", *ACI J.*, **59**, 1055-1070.
Bertero, V.V. (1986), "Lessons learnt from recent earthquakes and research and implications for earthquake resistant design of building structures in united states", *Earthq. Spectra*, **2**(4), 825-858.
Bureau of Indian Standards: IS-456 (1978), *Design Aids for Reinforced Concrete*, New Delhi, India.
Bureau of Indian Standards: IS-1893 (2002), *Criteria for Earthquake Resistant Design of Structures: Part 1 General provisions and Buildings*, New Delhi, India.
Burns, R.J. (1965), "An approximate method of analysis coupled shear walls subjected to triangular loadings", *Proceedings of the 3rd world conference on Earthquake Engineering*, Auckland.
Canadian Standards Association: CAN3-A23.3-M94 (1994), "*Design of Concrete Structures for Buildings*", Rexdale, Ontario, Canada.
Chaallal, O., Gauthier, D. and Malenfant, P. (1996), "Classification methodology for coupled shear walls", *J. Struct. En.-ASCE*, **122**(12), 1453-1458.
Chan, H.C. and Kuang, J.S. (1989), "Elastic design charts of stiffened coupled structural walls", *J. Struct. Eng.-ASCE*, **115**(2), 247-267.
Chan, W.T. and Kwan, A.K.H. (2000), "Effective stiffness of coupling beams connected to walls in out-of-plane directions", *Comput. Struct.*, **75**, 385-394.
Canadian Standards Association: CAN3-A23.3-M94 (1994), "*Design of Concrete Structures for Buildings*", Rexdale, Ontario, Canada.
Choudhury, J.R. and Coull, A. (1967), "Stresses and deflections in coupled shear walls", *ACI J.*, **64**(2), 65-72.
Coull, A. and Choudhury, J.R. (1967), "Analysis of coupled shear walls", *ACI J.*, **64**(9), 587-593.
Coull, A. and Puri, R.D. (1968), "Analysis of pierced shear walls", *J. Struct. Div.*, *Proc. ASCE*, **94**(ST1), 71-82.
Englekirk, R.E. (2003), *Seismic Design of Reinforced and Precast Concrete Buildings,* John Wiley, NY.
Federal Emergency Management Agency: FEMA-273 (1997), *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, Washington, DC, U.S.A.
Federal Emergency Management Agency: FEMA-356 (2000), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, Washington, DC, U.S.A.
Federal Emergency Management Agency: FEMA-440 (2006), *Improvement of Nonlinear Static Seismic Analysis Procedures*, Washington, DC, U.S.A.
Galano, L. and Vignoli, A. (2000), "Seismic behavior of short coupling beams with different reinforcement layouts", *ACI Struct. J.*, **97**(6), 876-885.
Gajalakshmi, P. and Sengupta, A.K. (2006), "Modeling of low-rise shear walls for pushover analysis of buildings", *Ind. Concr. J.*, **80**(6), 41-46.
Gong, B., Harries, K.A. and Shahrooz, B.M. (2000), "Behaviour and design of reinforced concrete, steel, and steel-concrete coupling beams", *Earthq. Spectra*, **16**(4), 775-799.
Harries, K.A., Mitchell, D., Cook, W.D. and Redwood, R.G. (1993), "Seismic response of steel beams coupling concrete walls", *J. Struct. Eng.-ASCE*, **119**(12), 3611-3629.
Hindalgo, P.A., Jordan, R.M. and Martinez, M.P. (2002), "An analytical model to predict the inelastic seismic behavior of shear-wall, reinforced concrete structures", *Eng. Struct.*, **24**, 85-98.
Hutchison, D.L. and Geldermalsen, T.J.V. (1984), "Optimum design of reinforced concrete shear walls", *Bulle. NZ Nat. Soc. Earthq. Eng.*, **17**(3), 185-196.
Jain, A.K. (1999), *Reinforced Concrete Limit State Design*, Nem Chand & Bros, Roorkee.
Jain, J.P. and Chandra, R. (1967), "Analysis of coupled shear walls by influence coefficient method", *Ind. Concr. J.,* **41**(6), 226-231.
Kim, H.S. and Lee, D.G. (2003), "Analysis of shear wall with openings using super elements", *Eng. Struct.*, **25**, 981-991.
Kratky, R.J., Puri, S.P.S. and Pandit, G.S. (1971), "Discussion of a paper by Bryan Stafford Smith Modified beam method for analyzing symmetrical interconnected shear walls", *ACI J.*, **68**(6), 472-474.
Kuang, J.S. and Chau, C.K. (1999), "Dynamic behaviour of stiffened coupled shear walls with flexible bases", *Comput. Struct.*, **73**(1), 327-339.
Kuang, J.S. and Ng, S.C. (2000), "Coupled lateral-torsion vibration of asymmetric shear-wall structures", *Thin Wall. Struct.*, **38**, 93-104.
Kuang, J.S. and Ng, S.C. (2001), "Dynamic coupling of asymmetric shear wall structures: an analytical

solution", *Int. J. Solids Struc.*, **38**, 8723-8733.

Kumar, V. (1978), *Analysis of Coupled Shear Walls Subjected to Lateral Loads*, A seminar report, Department of Earthquake Engineering, University of Roorkee, India.

Lu, X. and Chen, Y. (2005), "Modeling of coupled shear walls and its experimental verification", *J. Struct. Eng.-ASCE*, **131**(1), 75-84.

Macleod, I.A. (1966), *Lateral Stiffness of Shear Walls with Openings*, Department of Civil Engineering, Glasgow University.

Matsagar, V.A. and Jangid, R.S. (2008), "Base isolation for seismic retrofitting of structures", *Prac. Period. Struct. Des. Constr., ASCE,* **13**, 175-185.

Menon, D. and Reddy, Y.N. (1995), "Finite element modeling of tall slender tubular towers", *J. Struct. Eng. SERC*, **24**, 243-246.

Miranda E. (1997), *Strength Reduction Factors in Performance Based Design*, EERC-CUREe Symp. in Honor of V. V. Bertero, 31/01 to 01/02/97, California.

Mitchell, D., Tremblay, R., Karacabeyli, E., Paultre, P., Saatcioglu, M. and Anderson, D.L. (2003), "Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada", *Can. J. Civil Eng.*, **30**, 308-327.

Munshi, J.A. and Ghosh, S.K. (2000), "Displacement-based seismic design for coupled wall systems", *Earthq. Spectra*, **16**(3), 621-642.

Nadjai, A. and Johnson, D. (1998), "Elastic and elasto-plastic analysis of planar coupled shear walls with flexible bases", *Int. J. Comput. Struct.*, **68**, 213-229.

New Zealand Standard: NZS 3101 (1995), *The Design of Concrete Structures*. Wellington, NZ.

Park, R. and Paulay, T. (1975), *Reinforced Concrete Structures*, John Wiley & Sons, Inc., NY.

Paulay, T. (1970), "An elasto-plastic analysis of coupled shear walls", *ACI J.*, **67**(11), 915-922.

Paulay, T. and Santhakumar, A.R. (1976), "Ductile behavior of coupled shear walls", *J. Struct. Div.-ASCE*, **102** (ST1), 93-108.

Paulay, T. and Priestley, M.J.N. (1992), *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley & Sons, Inc., NY.

Paulay, T. (2002), "The displacement capacity of reinforced concrete coupled walls", *Eng. Struct.*, **24**, 1165–1175.

Penelis, G.G. and Kappos, A.J. (1997), *Earthquake-resistant Concrete Structures*, E&FN SPON, NY.

Pore, S.M. (2007), *Performance Based Seismic Design of Low to Medium Rise RC Framed Buildings for India*, Department of Earthquake Engineering, IIT Roorkee.

Prakash, V. (2004), "Whither performance based engineering in India?", *J. Ind. Soc. Earthq. Technol.*, **41**(1), 201-222.

Prakash, V., Pore, S.M. and Jain, A.K. (2006), "Role of response reduction factor and importance factor in fifth revision of IS 1893", *Proceedings of the 13th Symposium on Earthquake Engineering*, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, India, December.

Prasanth, T., Ghosh, S. and Collins, K.R. (2008), "Estimation of hysteretic energy demand using concepts of modal pushover analysis", *Earthq. Eng. Struct. D.*, **37**(6), 975-990.

Rosman, R. (1964), "Approximate analysis of shear walls subject to lateral loads", *ACI J.*, **61**(6), 717-732.

Saka, M.P. (1992), "Optimum design of multistory structures with shear walls", *Comput. Struct.*, **44** (4), 925-936.

SAP2000: Advanced 10.0.5 (2006), *Static and Dynamic Finite Element Analysis of Structures*, Computers and Structures Inc., Berkeley, CA.

Schulz, M. (1961), "Analysis of reinforced concrete walls with openings", *Ind. Concrete J.*, **35**(11), 432-433.

Schwaighofer, J. (1967), "Door openings in shear walls", *ACI J.*, **64**(11), 730-733.

Shahrooz, B.M. and Remmetter, M.E. (1993), "Seismic design and performance of composite coupled walls", *J. Struct. Eng.-ASCE*, **119**(11), 3291-3309.

Smith, B.S. and Coull, A. (1991), *Tall Building Structures (Analysis and Design)*, John Wiley and Sons, Inc., New York.

Swift, R.D. and Heidebrecht, A.C. (1971), "Analysis of asymmetrical coupled shear walls", *J. Struct. Div.*, *Proc. ASCE*, **97**(ST5), 1407-1422.

Wallace, J.W. and Moehle, J.P. (1992), "Ductility and detailing requirements of bearing wall buildings", *J. Struct.*

Eng.-ASCE, **118**(6), 1625-1644.
Wallace, J.W. (1994), "New methodology for seismic design of RC shear walls", *J. Struct. Eng.-ASCE*, **120**(3), 863-883.
Winokur, A. and Gluck, J. (1968), "Ultimate strength analysis of coupled shear walls", *ACI J.*, **65**(12), 1029-1035.

Notations

A	Area of Symmetrical Coupled Shear Walls
A_{cw}	Area of Concrete Section of an Individual Pier, Horizontal Wall Segment, or Coupling Beam Resisting Shear in in^2 as per ACI 318 (2005)
A_g	Gross Area of Concrete Section in in^2 For a Hollow Section, A_g is the Area of The Concrete Only and does not Include the Area of the Void (S) as per ACI 318 (2005)
A_s''	Reinforcing Steel in One Diagonal as per Englekirk (2003)
A_s	Area of Nonprestressed Tension Reinforcement as per Englekirk (2003)
A_{sd}	Reinforcement along each Diagonal of Coupling Beam as per IS 13920 (1993)
A_{vd}	Total Area of Reinforcement in Each Group of Diagonal Bars in a diagonally reinforced coupling beam in in^2 as per ACI 318(2005)
b_b	Width of Coupling Beam
b_f	Flange width of I-beam as per FEMA 273 (1997) and FEMA 356 (2000)
b_w	Web Width of the Coupling Beam as per FEMA 273 (1997) and FEMA 356 (2000)
C	Compressive Axial Force at the Base of Wall 2
CP	Collapse Prevention Level
D	Overall Depth of the Steel I-Coupling beam Section
DC	Degree of Coupling
DL	Dead Loads
DBE	Design Basis Earthquake
d	Effective Depth of the Beam
d_b	Depth of the Coupling Beam
d'	Distance from Extreme Compression Fiber to Centroid of Compression Reinforcement as Per Englekirk (2003)
Δ_b	Displacement at V_b
Δ_e	Elastic displacement ($\Rightarrow V_e$)
Δ_l	Displacement at Limiting Response
Δ_{roof}	Roof Displacement
$\Delta_{roof,\,CP}$	Roof Displacement at CP Level
$\Delta_{roof,\,yield}$	Roof Displacement at Yield Level
Δ_u	Displacement at Ultimate Strength Capacity
Δ_y	Displacement at Yield Strength Capacity
Δ_{ya}	Actual displacement at V_{ya}
E_c	Modulus of Elasticity of Concrete
E_{cb}	Young's Modulus for Concrete in Beam
E_{cw}	Young's Modulus for Concrete in Wall
EPP	Elastic-Perfectly-Plastic
EQRD	Earthquake Resistant Design
E_s	Modulus of Elasticity of Steel as per FEMA 273 (1997) and FEMA 356 (2000)
E_{sb}	Young's Modulus for Steel in Beam
E_{sw}	Young's Modulus for Steel in Wall
e	Clear Span of the Coupling Beam+2×Concrete Cover of Shear Wall as per Englekirk (2003)
ε_c	Strain in Concrete
F	Force
F_1	Maximum Amplitude of Triangular Variation of Loading
FEMA	Federal Emergency Management Agency
F_u	Ultimate Force
F_y	Yield stress of Structural Steel
f_c'	Specified Compressive Strength of Concrete Cylinder
f_{ck}	Characteristic Compressive Strength of Concrete Cube
f_y	Specified Yield Strength of Reinforcement

H	Overall Height of the Coupled Shear Walls
h	Distance from inside of compression flange to inside of tension flange of I-beam as per FEMA 273 (1997) and FEMA 356 (2000)
h_s	Storey Height
I	Moment of Inertia of Symmetrical Coupled Shear Walls
I_b	Moment of Inertia of Coupling Beam
IO	Immediate Occupancy Level
i	Storey Number
k	Unloading Stiffness
k_1	Post Yield Stiffness
k_e	Elastic Stiffness
k_i	Initial Stiffness
$k_{\sec}$	Secant Stiffness
L_b	Length of the Coupling Beam
L_d	Diagonal Length of the Member
LL	Live Loads
LS	Life Safety Level
L_w	Depth of Coupled Shear Walls
l	Distance between Neutral Axis of the Two Walls
λ_0	Member Over strength factor as per Englekirk (2003)
M	Moment of Symmetrical Coupled Shear Walls
M_1	Moment at the Base of the Wall 1
M_2	Moment at the Base of the Wall 2
MCE	Maximum Considered Earthquake
MDOF	Multi-Degree of Freedom
M_n	Nominal Flexural Strength at Section in lb-in as per ACI 318 (2005)
M_p	Moment Capacity of Coupling Beam as per Englekirk (2003)
M_{ot}	Total Overturning Moment due to the Lateral Loading
MRF	Moment Resistant Frame
μ	Displacement Ductility Capacity Relied on in the Design as per NZS 3101 (1995)
μ_Δ	Ductility
$\mu_{\Delta e1}$	Energy based proposal for ductility under monotonic loading and Unloading
$\mu_{\Delta e2}$	Energy based proposal for ductility under Cyclic Loading
N	Total Number of Storeys
NA	Not Applicable
NEHRP	National Earthquake Hazard Reduction Program
NSP	Non-linear Static Procedure
P	Axial Force as per IS 456 (1978)
PBSD	Performance Based Seismic Design
p	Percentage of Minimum Reinforcement
ϕ	Shear span to depth ratio
pp	Performance Point
R	Response Reduction Factor
RCC	Reinforced Cement Concrete
R_d	Ductility Related Force Modification Factor
R_μ	Ductility Factor
R_R	Redundancy Factor
R_s	Overstrength Factor
S_a	Spectral Acceleration
S_d	Spectral Displacement
SDOF	Single-Degree of Freedom
T	Tensile Axial Force at the Base of Wall 1

T_1	Tensile Strength of One Diagonal of a Diagonal Reinforced Coupling Beam
T_d	Tensile Strength of Truss Reinforced Coupling Beam's Diagonal as per Englekirk (2003)
T'	The Residual Chord Strength as per Englekirk (2003)
t_f	Flange Thickness of Steel I-Coupling beam as per Englekirk (2003)
θ	Inclination of Diagonal Reinforcement in Coupling Beam
θ_b	Coupling Beam Rotation
θ_{Lu}	Rotational Value at Ultimate Point
$\theta_{u,\max}$	Maximum Rotational Value
θ_w	Wall Rotation
θ_y	Yield Rotation as per FEMA 273 (1997) and FEMA 356 (2000)
t_w	Wall Thickness
$t_{w'}$	Web Thickness of Steel I-Coupling beam
V	Shear Force in the Coupling Beam
V_1	The Shear or Vertical Component of One Diagonal in a Primary Truss Travelled along the Compression Diagonal as per Englekirk (2003)
V_2	The Shear in a Secondary Truss produced by the Residual Tension Reinforcement Activated the Load Transfer Mechanism as per Englekirk (2003)
V_B	Base Shear
V_b	Non-factored design base shear
V_d	Factored design base shear may be less than or greater than V_{ya}
V_e	Base shear for elastic response
V_l	Base shear at limiting response
V_n	Nominal Shear Strength in lb as per ACI 318 (2005)
V_{nf}	The transferable shear force for flexure dominant steel coupling beam as per Englekirk (2003)
V_{sp}	Shear Capacity of Coupling Beam as per Englekirk (2003)
V_{s1}	Shear Strength of Closed Stirrups as per ATC 40 (1996), FEMA 273 (1997) and FEMA 356 (2000)
V_u	Capacity corresponding to Δ_u (may be the maximum capacity)
V_{u1}	Factored Shear Force as per IS 13920 (1993)
V_{u2}	Factored Shear Force at Section in lb as per ACI 318 (2005)
V_w	Shear Force at the Base of the Shear Wall
V_{w1}	Shear Force at the Base of Wall 1
V_{w2}	Shear Force at the Base of Wall 2
V_y	Base shear at idealized yield level
V_{ya}	Actual first yield level
v_n	Total Nominal Shear Stress in MPa as per NZS 3101 (1995)
W_g	Total Gravity Loading for Symmetrical Coupled Shear Walls
w	Compressive Strut Width as per Englekirk (2003)
Z	Zone Factor
Z_p	Plastic Section Modulus of Steel Coupling Beam