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A PROCEDURE FOR THE EVALUATION OF COUPLING BEAM CHARACTERISTICS OF COUPLED SHEAR WALLS

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ABSTRACT

The behavior of coupled shear walls is governed by coupling beams. This paper presents a simple technique for the purpose of design to determine an appropriate level of yield moment capacity for the coupling beams. This technique is checked against nonlinear static pushover analysis performed using DRAIN-3DX for the usual case of symmetric coupled shear walls with different types of coupling beams. The assumption of pinned base in the shear walls with steel coupling beams yields results which agree closely with those of DRAIN-3DX. For the case of fixed base shear walls, the design technique is expected to be conservative.

Keywords: coupled shear walls, coupling beam, rotation, capacity

1. INTRODUCTION

Coupled shear walls consist of two shear walls interconnected by beams along their height. The behavior of coupled shear walls is mainly governed by the coupling beams. The coupling beams are designed for ductile inelastic behavior in order to dissipate energy to provide damping during an earthquake. The base of the shear walls may be designed as pinned or may be designed for ductile inelastic behavior. The amount of energy dissipation depends on the yield moment capacity and plastic rotation of the coupling beams. If the yield moment capacity is too high, then the coupling beams will undergo only limited rotations and dissipate little energy. On the other hand, if the yield moment capacity is too low, then the coupling beams may undergo rotations much larger than their plastic rotation capacities. Therefore, the coupling beams should be provided with an optimum level of yield moment capacities depending on the plastic rotation capacity available. The plastic rotation capacity in coupling beams depends upon the type of coupling beam – steel beam with shear-dominant coupling beam, steel beam with flexure-dominant coupling beam, R.C.C. beam with conventional flexural and shear reinforcement, R.C.C. beam with diagonal reinforcement, and R.C.C. beam with rhombic reinforcement. Coupling beam characteristics

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in a coupled shear walls are controlled by various factors, i.e. type of material, size, type of detailing, yield moment capacity and plastic rotation capacity. Table 1 summarizes the above mentioned factors which characterize the coupling beams as determined experimentally and analytically as per different sources [1-8] and [14].

Table 1. Factors governing the coupling beam characteristics

Type of material		Size	Type of detailing	Shear capacity	Moment Capacity (M_p)	Plastic Rotation Capacity (Radians)			
						$\frac{Shear}{t_w l_w \sqrt{f_{c'}}}$	IO	LS	CP
Reinforced concrete coupling beam	$\alpha \le 2$	No limit	a	$V_{sp} = \frac{0.08 f_c' b h_b}{\lambda_0}$	$\frac{V_{sp} \times L_b}{2}$	≤ 3	0.006	0.015	0.020
						≥ 6	0.005	0.012	0.016
			b	$V_{sp} = \frac{0.08 f_c' b h_b}{\lambda_0}$	$\frac{V_{sp} \times L_b}{2}$	≤ 3	0.006	0.008	0.010
						≥ 6	0.004	0.006	0.007
		<1.5	Diagonal Reinforcement	$V_{sp} = \frac{0.08 f_c' b h_b}{\lambda_0}$	$\frac{V_{sp} \times L_b}{2}$	-	0.006	0.018	0.03
		1.5 to 4.0	Rhombic Reinforcement	$V_{sp} = \frac{0.08 f_c' b h_b}{\lambda_0}$	$\frac{V_{sp} \times L_b}{2}$	-	NA	NA	NA
Steel coupling beam	$e \le \frac{1.6 M_p}{V_{sp}}$		Shear dominant	$V_{sp} = 0.6 F_y t_{w'} (d - 2t_f)$	$Z_x F_y$	-	$\frac{0.015}{L_b}$	$\frac{0.12}{L_b}$	$\frac{0.15}{L_b}$
Steel coupling beam	$e \ge \frac{2.6 M_p}{V_{sp}}$		Flexure dominant	$V_{sp} = \frac{2M_p}{e}$	$Z_x F_y$	$\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_y}}$ and $\frac{h}{t_{w'}} \le \frac{418}{\sqrt{F_y}}$	$1\theta_y$	$6\theta_y$	$8\theta_y$
						$\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_y}}$ and $\frac{h}{t_{w'}} \ge \frac{640}{\sqrt{F_y}}$	$0.25\theta_y$	$2\theta_y$	$3\theta_y$

α=Shear span to depth ratio, a = Conventional longitudinal reinforcement with conforming transverse reinforcement, b= Conventional longitudinal reinforcement with non-conforming transverse reinforcement, IO = Immediate occupancy level, LS = Life safety level, CP = Collapse prevention level, Size = Ratio between clear span and depth of coupling beam, $\theta_y = \frac{Z_x F_y L_b}{6 E_s I_b}$ =Yield rotation, E_s = Modulus of elasticity of steel and I_b = Moment of inertia of beam

The capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. Nonlinear pushover analysis is required to obtain the capacity curve beyond elastic limit [12]. There are various programs like DRAIN-3DX [13] which can be used to perform nonlinear analysis to determine the capacity curve.

2. PROPOSED FORMULATION

2.1 Assumptions

1. Coupled shear walls exhibit flexural behavior.
2. Point of contra flexure occurs at mid point of clear span of the beam.
3. Axial deformations of the beams can be neglected.
4. Coupling beam carries axial force, shear force and bending moment.
5. The lateral loading has a triangular variation.
6. The horizontal displacement in each point of wall 1 is equal to the horizontal displacement in each point of wall 2 due to the presence of coupling beam.
7. All coupling beams have identical moment capacities. They are plastified or carry equal amount of shear forces simultaneously before collapse mechanism is formed, i.e. all beams reach the rotational level at collapse prevention simultaneously as well as all coupling beams reach the rotational level at yield point at the same instant.
8. The curvatures of the two walls are same at any level.

In Figure 1(a), the coupled shear walls are subjected to a triangular variation of point loadings in each storey with amplitude of F_1 at the roof level. The value of F_1 could be determined so that all coupling beams reach their rotational limit for collapse prevention level as well as yield level simultaneously, then subsequently the base shear, roof displacement and shear force developed in coupling beams could be determined. The procedure including steps as well as mathematical calculation has been illustrated as follows with initial value of F_1 as 1:

2.2 Steps

1) Select type of coupling beam and determine its shear capacity.
2) Determine the fractions of total lateral loading subjected on wall 1 and wall 2 respectively.
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. For walls hinged at the base this check is not required.
6) Calculate coupling beam rotation in each storey.
7) Check if coupling beam rotation lies at yield level or collapse prevention level.
8) Modify the value of F_1 for next iteration starting from step (2) if step (7) is not satisfied as per the assumption (7). Otherwise go to step (9).
9) Calculate base shear and roof displacement.

2.3 Mathematical Calculation

Step 1:

For various types of reinforced (conventional, diagonal or rhombic) coupling beams, limiting value of shear capacity is given by table 1,

$$V_{sp} = \frac{0.08 f_{c'} b h_b}{\lambda_0} \quad (1)$$

Where, breadth of coupling beam is b; depth of coupling beam is h_b; $f_{c'}$ is specified compressive strength of concrete and young's modulus of concrete E_c depends on $f_{c'}$ and λ_0 is a factor of value 1.25.

For Steel shear dominant type of coupling beam, limiting value of shear capacity is given by table 1,

$$V_{sp} = 0.6F_y t_{w'}(d - 2t_f) \tag{2}$$

For Steel flexural dominant type of coupling beam, limiting value of shear capacity is given by table 1,

$$V_{sp} = \frac{2M_p}{e} \tag{3}$$

Where, $M_p = Z_x F_y$, Z_x is plastic section modulus, F_y is yield stress of steel and young's modulus of steel E_s depends on F_y, $t_{w'}$ is web thickness, d is the depth of the section and t_f is flange thickness.

Step 2:

In Figure 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 respectively, such that,

$$\alpha + \beta = 1.0 \tag{4}$$

Based on the assumption (8), following equation can be written as

$$\frac{M_1(x)}{E_c I_1} = \frac{M_2(x)}{E_c I_2} \tag{5}$$

or,

$$\frac{M_1(x)}{I_1} = \frac{M_2(x)}{I_2} \tag{6}$$

If n' = storey number counted from the roof of the system (ranging from 1 to n), n is total number of stories, $L_{01} = \frac{L_b}{2} + \frac{L_{w_1}}{2}$, and $L_{02} = \frac{L_b}{2} + \frac{L_{w_2}}{2}$ then the moment about the center line of wall 1 at a distance 'x' from roof in fig. 1(b) for x>0, is

$$M_1(x) = \sum_{i=1}^{n'} \left\{ \alpha \frac{F_1}{H}(H - (i-1)h_s)(x - (i-1)h_s) \right\} - L_{01}\left(\sum_{i=1}^{n'} V_i\right) \tag{7}$$

Similarly, the moment about the center line of wall 2, is

$$M_2(x) = \sum_{i=1}^{n'} \left\{ \beta \frac{F_1}{H}(H-(i-1)h_s)(x-(i-1)h_s) \right\} - L_{02}\left(\sum_{i=1}^{n'} V_i\right) \tag{8}$$

Substituting (7) & (8) into (6) leads to the following equation

$$\frac{1}{I_1}\sum_{i=1}^{n'} \left\{ \alpha \frac{F_1}{H}(H-(i-1)h_s)(x-(i-1)h_s) \right\} - \frac{1}{I_1} L_{01}\left(\sum_{i=1}^{n'} V_i\right) = \frac{1}{I_2}\sum_{i=1}^{n'} \left\{ \beta \frac{F_1}{H}(H-(i-1)h_s)(x-(i-1)h_s) \right\} - \frac{1}{I_2} L_{02}\left(\sum_{i=1}^{n'} V_i\right) \tag{9}$$

For simplifying the above and considering equation (4) in conjunction,

$$\alpha = \frac{I_1}{I_1 + I_2} + \frac{\sum_{i=1}^{n'} V_i}{F_1(I_1+I_2)\left[\frac{1}{H}\sum_{i=1}^{n'}(H-(i-1)h_s)(x-(i-1)h_s)\right]}(L_{01}I_2 - L_{02}I_1) \tag{10}$$

Step 3:
The definition of degree of coupling could be written as [9],

$$DC = \frac{T \times l}{M_{ot}} \tag{11}$$

Where, T is the axial force at the base of the wall and M_{ot} is total overturning moment. For fixed base condition the degree of coupling varies from 0 to 1 whereas for the case of pinned base condition the degree of coupling is 1.

So based upon the above criteria and considering equation (11), shear force developed in the coupling beam could be determined as follows. Here $V_0 = 0$ since there is no coupling beam beyond the roof as per figure 1(b).

Fixed base condition:

$$\sum_{n'=1}^{n} V_{n'} = \frac{M_{ot}(H) * [1. - \frac{L_b}{(L_{w_1} + L_{w_2})}]}{l} \tag{12}$$

Where, M_{ot} is total overturning moment at the base due to the lateral loadings.

Therefore, shear force in $n'th$ coupling beam at a distance ‘x’ from roof is,

$$V_{n'}(x) = \frac{M_{ot}(x) * [1. - \frac{L_b}{(L_{w_1} + L_{w_2})}]}{l} - \sum_{i=1}^{n'} V_{i-1} \tag{13}$$

Pinned base condition:

$$\sum_{n'=1}^{n} V_{n'} = \frac{M_{ot}(H)}{l} \tag{14}$$

Therefore, shear force in n' coupling beam at a distance 'x' from roof is,

$$V_{n'}(x) = \frac{M_{ot}(x)}{l} - \sum_{i=1}^{n'} V_{i-1} \tag{15}$$

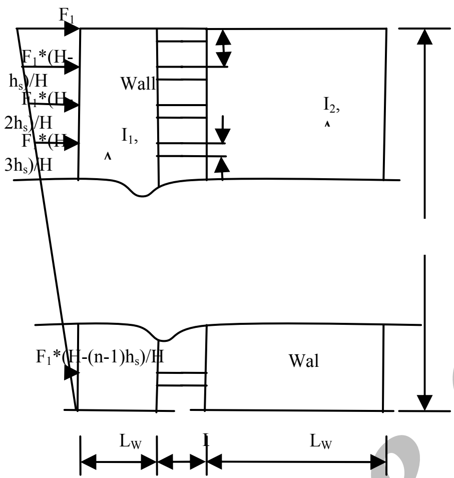

Figure 1(a). Coupled shear walls

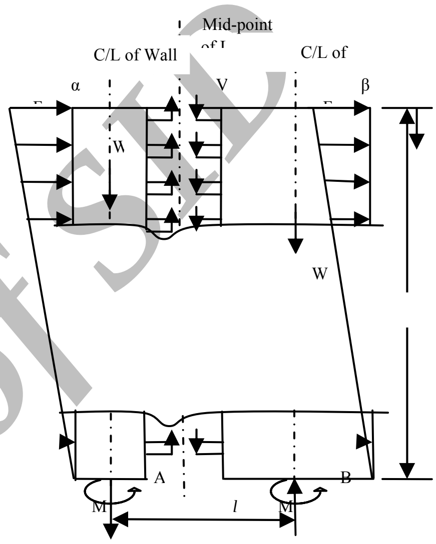

Figure 1(b). Free body diagram of coupled shear walls

Step 4:
After getting α, β and $V_{n'}$ at each storey for the particular value of F_1, bending moment values in each storey could be determined for each wall. After that, curvature diagram for each wall is generated from which wall rotation in each storey for the walls could be determined.

Step 5:
i. Tensile forces in wall 1 as well as compressive forces in wall 2 are calculated due to lateral loadings in each level.
ii. Compressive loads in wall 1 and wall 2 are calculated in each storey due to gravity loadings.
iii. Net axial forces in wall 1 and wall 2 in each storey are calculated.
iv. Then, according to these net axial forces for the particular values of f_{ck}, b, d and p, the limiting moment values in each storey in wall 1 and wall 2 could be determined from P-M interaction curve [10-11]. 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 respectively. All these limiting values are basically for linear

behavior of that particular section.

v. So if calculated bending moment value at the base is greater than limiting moment value, then plastic hinge in that particular storey would be formed otherwise no plastic hinge would be formed.

vi. If base moment of wall is zero then real hinge would be formed at the base of wall.

Step 6:

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

Rotation of coupling beam at any level x for symmetrical walls [1],

$$\theta_{bx} = \theta_{wx}\left(1 + \frac{L_w}{L_b}\right) \quad (16)$$

Where, θ_{wx} is rotation of wall at any level x, $L_{w1} = L_{w2} = L_w$ = depth of wall, L_b = length of coupling beam.

Equation (16) can be written as follows,

$$(\theta_{bx})_1 = (\theta_{wx}) + \frac{(\theta_{wx})\frac{L_{w_1}}{2} + (\theta_{wx})\frac{L_{w_2}}{2}}{L_b} \quad (17)$$

or

$$(\theta_{bx})_2 = (\theta_{bx})_1 \quad (18)$$

Equations (17) and (18) are for unsymmetrical walls. For post-yield rotation at the base of the walls (θ_{wp}), Equations (16) and (17) could be written as,

$$\theta_{bx} = L_{wb}(\theta_{wx} + \theta_{wp}) \quad (19)$$

$$(\theta_{bx})_1 = (\theta_{bx})_2 = L_1\{(\theta_{wx}) + (\theta_{wp})\} \quad (20)$$

For real hinge rotation at the base of wall (θ_{w0}), Equations (16) and (17) could be written as,

$$\theta_{bx} = L_{wb}(\theta_{wx} + \theta_{w0}) \quad (21)$$

$$(\theta_{bx})_1 = (\theta_{bx})_2 = L_1\{(\theta_{wx}) + (\theta_{w0})\} \quad (22)$$

Where, $L_{wb} = \left(1 + \frac{L_w}{L_b}\right)$ and $L_1 = \left(1 + \frac{L_{w_1} + L_{w_2}}{2L_b}\right)$

Step 7:

The rotational limit for collapse prevention level & immediate occupancy level (assuming yield level) of different types of RC coupling beams and steel beams are given in table 1. Here assuming rotational limit for rhombic reinforced type of coupling beam is equal as rotational limit for diagonal reinforced coupling beam. Check whether the rotations of all beams lies at yield level or collapse prevention level, otherwise go to step 8 where magnitude of F_1 is being modified for different types of bases conditions of walls.

Step 8:

The modified F_1 will be as follows:

$$(F_1)_{\text{yieldlevel}} = \frac{\sum_{j=1}^{n} V_j}{\left(\sum_{i=1}^{n} V_i\right)_{F_1=1}} \tag{23}$$

Where, the above equation represents modified value of F_1 for yield level and none or few or more beams carry equal amount of shear capacity in beam at the yield level.

$$(F_1)_{\text{collapseprevention}} = \frac{n \times V}{\left(\sum_{i=1}^{n} V_i\right)_{F_1=1}} \tag{24}$$

Where, the above equation represents modified value of F_1 for collapse prevention level; V is shear capacity in coupling beam.

Step 9:

The roof displacements can be calculated as per following equations:

$$\left(\Delta_{roof}\right) = h_s \times \left[\theta_{wx1} + \theta_{wx2} + ... + \theta_{wxn}\right] \tag{25}$$

$$\left(\Delta_{roof}\right)_{wall1} = h_s \times \left[(\theta_{wx1})_1 + (\theta_{wx2})_1 + ... + (\theta_{wxn})_1\right] \tag{26}$$

$$\left(\Delta_{\text{roof}}\right)_{\text{wall2}} = \left(\Delta_{\text{roof}}\right)_{\text{wall1}} \tag{27}$$

The equation (25) is for symmetrical coupled shear walls; (26) and (27) are for unsymmetrical coupled shear walls.

The Base shear can be calculated as follows:

$$V_B = F_1 + F_1(H - h_s)/H + F_1(H - 2h_s)/H + \ldots\ldots + F_1(H - (n-1)h_s)/H \qquad (28)$$

The methodology discussed above is referred as "Design Technique" hereafter.

3. CASE STUDY

The results of capacity curve as well as shear force distributions in coupling beams at collapse prevention level (cp level) and at yield level are compared by Design Technique and DRAIN-3DX for symmetrical coupled shear walls. These walls are subjected to triangular variation of lateral loadings. The dimensions are depth of wall D_w = 4.0 m, length of beam L_b = 1.8 m, depth of beam H_b = 600 mm, total wall height h_w = 60 m (n=20), and wall thickness t_w = 300 mm = b_b breadth of coupling beam. Note that E_c = 27 GPa; dead load, D = 6.7 kN/m^2 and live load, L = 2.4 kN/m^2 [9], $f_{c'}$ = 29.16 MPa; assuming young's modulus of steel E_s = 200 GPa. The figures have given as Figure 2(a) and Figure 2(b).

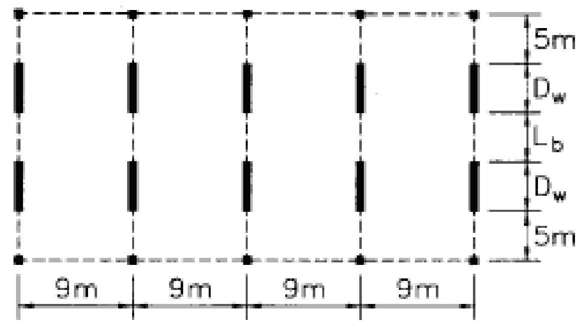

Figure 2(a). Plan view of building

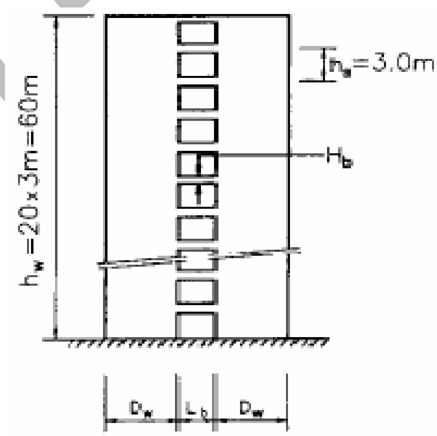

Figure 2(b). Coupled shear walls

3.1 Modeling in DRAIN-3DX

Wide column frame analogy has been used for modeling in DRAIN-3DX as per following Figure. In this analogy, shear wall elements are represented as two line elements (centre line of shear wall) and beams are represented as line elements (centre line of beam) and connected with each other with rigid link. Beam column elastic element (Type-17) and inelastic element (Type-15) are used for modeling. F_y = 415 MPa is used for the case of reinforced concrete section and F_y = 250 MPa is used for the case of steel section.

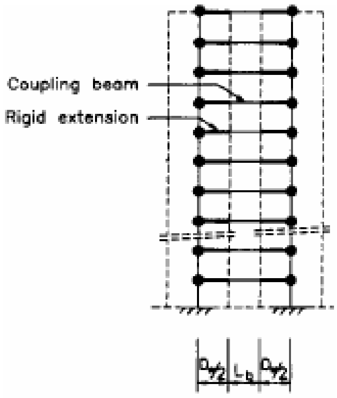

Figure 3. Modeling in drain-3dx

4. RESULTS AND DISCUSSIONS

a) Conventional type of reinforced coupling beam: Assuming longitudinal reinforcement with conforming transverse reinforcement.

It has been seen from the Figure 4(a) that for the case of RC beam with fixed base condition, the results of the initial part of the capacity curve are nearly matched but there are small differences of the end part of the capacity curve obtained both from Design technique and DRAIN-3DX respectively. The results of the shear force distributions of RC coupling beams are not matched obtained both from Design technique and DRAIN-3DX respectively as per the Figures 5(a).

It has been also seen from the Figures 4(b) that for the case of RC beam with pinned base condition, the result of the capacity curve is in lower side in the case of Design technique against the result obtained from DRAIN-3DX. The results of the shear force distributions of coupling beams are nearly similar patterns obtained both from DRAIN-3DX and Design technique respectively as per the Figures 5(b).

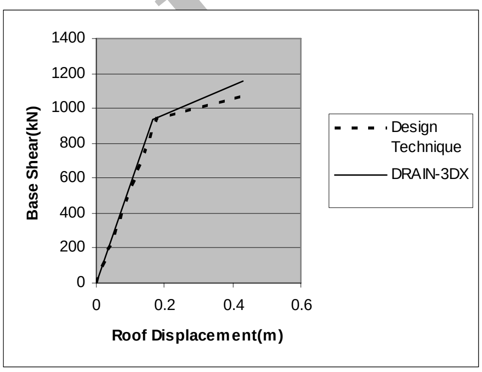

Figure 4(a). Capacity curve for fixed base condition

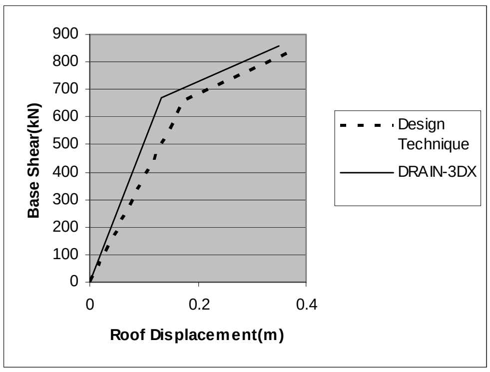

Figure 4(b). Capacity curve for pinned base condition

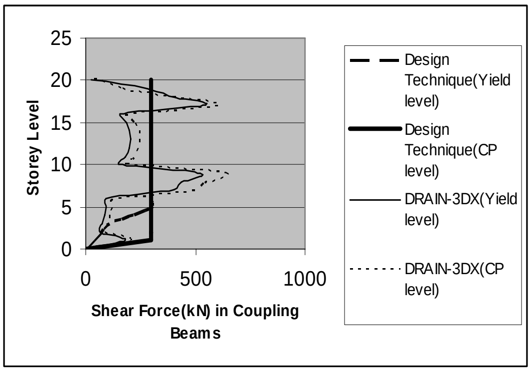

Figure 5(a). Shear force distribution in coupling beams for fixed base condition

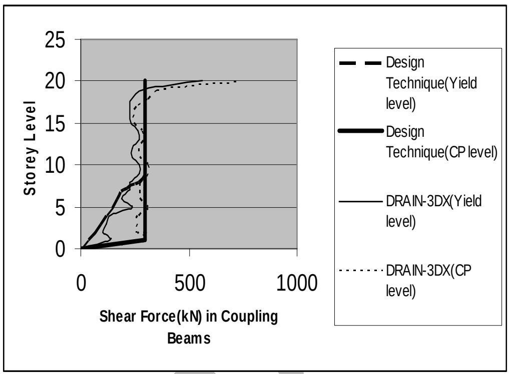

Figure 5(b). Shear force distribution in coupling beams for pinned base condition

b) Diagonal/Rhombic type of coupling beam: Assuming rotational level for rhombic type is same as rotational level for diagonal type.

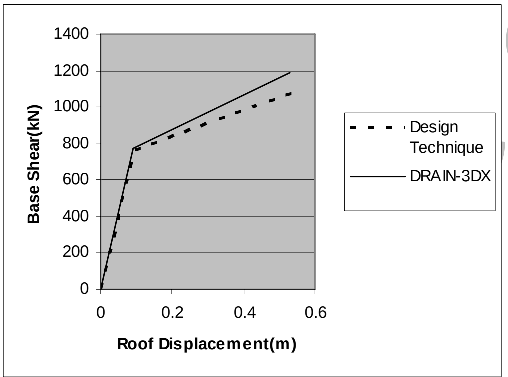

Figure 6(a). Capacity curve for fixed base condition

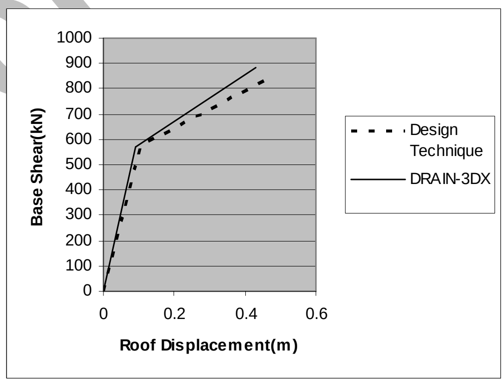

Figure 6(b). Capacity curve for pinned base condition

It has been seen from the above figures that for the case of RC beam with fixed base condition, the results of the initial part of the capacity curve are nearly matched but there are small differences of the end part of the capacity curve obtained both from Design technique and DRAIN-3DX respectively.

Whereas for the case of RC beam with pinned base condition, the result of the capacity curve is in lower side in the case of Design technique against the result obtained from DRAIN-3DX.

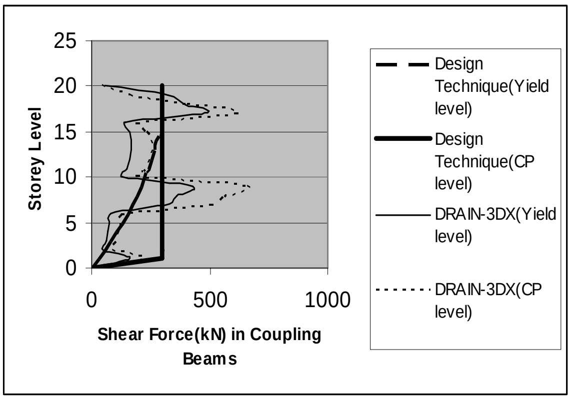

Figure 7(a). Shear force distribution in coupling beams for fixed base condition

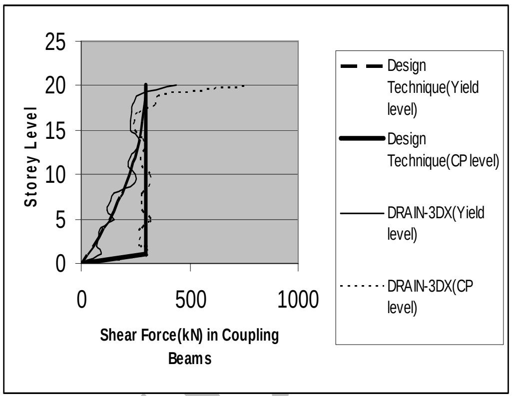

Figure 7(b). Shear force distribution in coupling beams for pinned base condition

The results of the shear force distributions of RC coupling beams are not matched which are obtained both from Design technique and DRAIN-3DX respectively as per the Figures 7(a).

The results of the shear force distributions of coupling beams are nearly similar patterns obtained both from DRAIN-3DX and Design technique respectively as per the Figures 7(b).

c) Steel shear dominant type of coupling beam:

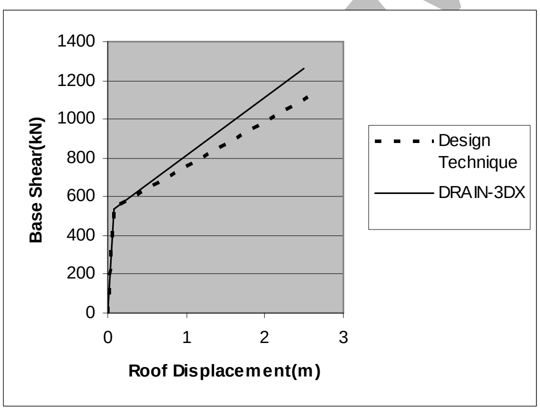

Figure 8(a). Capacity curve for fixed base condition

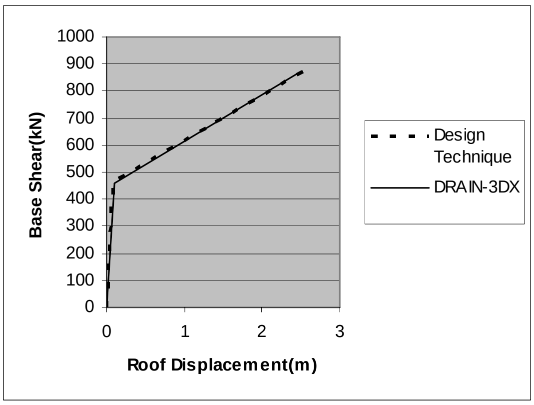

Figure 8(b). Capacity curve for pinned base condition

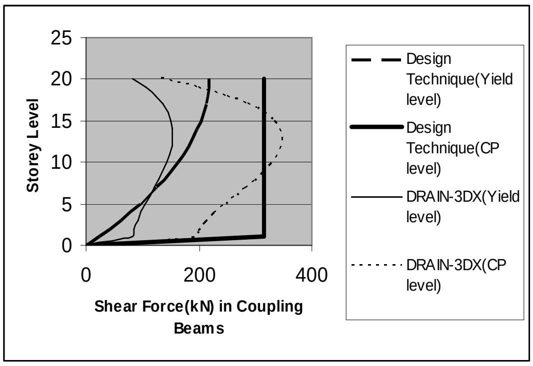

Figure 9(a). Shear force distribution in coupling beams for fixed base condition

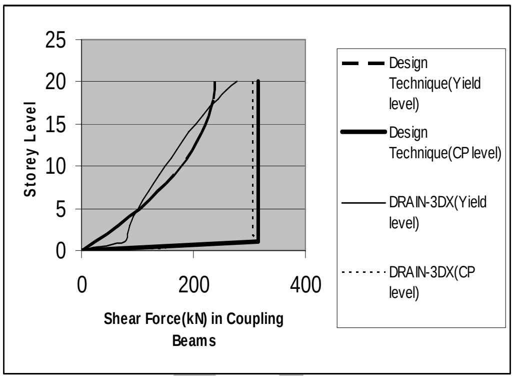

Figure 9(b). Shear force distribution in coupling beams for pinned base condition

The above figures show that for the case of steel coupling beam the results by proposed Design technique and DRAIN-3DX are nearly same.

It is observed that the use of steel coupling beam, in contrast with conventional RC, leads to increased to roof displacement while the base shear is only marginally affected. It is therefore imperative that the type of coupling beam to be adopted be judiciously selected.

5. CONCLUSIONS

1. The assumption of pinned base in the shear walls with steel coupling beams yields results which agree closely with those of DRAIN-3DX.
2. For the case of fixed base shear walls, the design technique is conservative.
3. The type of coupling beam is judiciously chosen to make the design of the coupled shear walls optimal for a particular zone.
4. The results are encouraging and the simple technique proposed may be effectively employed in design office practice.

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