Investigation on the Behavior of Conventional Reinforced Coupling Beams

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Abstract—Coupled shear walls consist of two shear walls connected intermittently by beams along the 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. The base of the shear walls may be designed for elastic or ductile inelastic behavior. The amount of energy dissipation depends on the yield moment capacity and plastic rotation capacity of the coupling beams. In this paper, an analytical model of coupling beam was developed to calculate the rotations and moment capacities of coupling beam with conventional reinforcement.

Keywords—Design studies, computational model(s), case study/studies, modeling, coupling beam.

I. INTRODUCTION

WHEN two shear walls are joined by beams at fixed intervals along its height, it is called a coupled shear wall; these beams are the primary factors which control the behavior of coupled shear walls. The coupling beams are designed for ductile inelastic behavior for the purpose of energy dissipation while the base of the shear walls can be designed for either elastic behavior or ductile inelastic behavior.

Coupling beams are quite short and deep and generally have span/depth ratios of two or even lower, since the widths of door and window openings usually range from 1.0 to 1.5 m. Being similar to deep beams, coupling beams with span/depth ratios lower than 2.0 have a predisposition to fail in shear rather than in flexure. During a major earthquake, if the coupling beams were very strong, the wall units might fail due to the large axial forces and bending moments induced in them without prior yielding of the coupling beams.

As the walls are taking vertical loads and are the major lateral loads resisting elements, any damage to the walls could endanger the safety of the building and render the repair after earthquake very difficult. On the other hand, if the coupling beams were not too strong, they would yield and dissipate the excessive vibration energy before the wall units yield thereby reducing the axial forces induced in the walls and protecting the walls from being damaged. Hence, the coupling beams should be designed to yield before the walls yield, but then the coupling beams would be subjected to a certain ductility demand. In any case, the earthquake resistance of a coupled shear wall structure is highly dependent on the nonlinear behavior, especially the strength and ductility of the coupling beams. The amount of earthquake energy dissipation is governed by the yield moment capacity and plastic rotation capacity of the coupling beams [9]-[23].

However [1], [6]-[9] show the inconsistent modeling parameters and inconsistent evaluative parameters of coupling beams described as follow:

1. As per [7], [8], the rotational capacities of beam depends on size of wall (t_w, L_w) which is illogical.

2. When shear span to depth ratio $\phi \leq 2$ or aspect ratio $L_b/d_b \leq 4$, the behavior of all types of RCC coupling beams is controlled by shear [21]. For this reason, as aspect ratio $\left(\dfrac{L_b}{d_b}\right)$ of diagonally reinforced beam is less than 1.5, it means that the behavior of diagonally reinforced beam is controlled by shear. Whereas, [1] and [7] show that diagonally reinforced coupling beam behavior is controlled by flexure which is not acceptable.

3. Conventional longitudinal reinforcement with non-conforming transverse reinforcement as per [1], [7], [8] is not accepted for new construction.

4. If the behavior of coupling beam is controlled by flexure [aspect ratio $\left(\dfrac{L_b}{d_b}\right)$ is greater than 4], the length of the coupling beam is quite larger. It has been observed [18] that weakly coupled shear walls can be obtained for larger span of the coupling beam and the design results of this type of coupled shear walls were inconsistent with regard to the ductility and energy dissipation during earthquake motion. Hence, it can be said that rotational capacity of coupling beams controlled by flexure as per [1], [7], [8] cannot be accepted.

5. As per [1], [7], [8], regarding the conditions of $\dfrac{Shear}{t_w L_w \sqrt{f_c'}}$ or $\dfrac{Shear}{b_w d \sqrt{f_c'}} \leq 3$ or ≥ 6 are confusing.

6. Similarly, for aspect ratio of $\dfrac{L_b}{d_b} = 1.5$ Galano and Vignoli [9] shows different results regarding the ultimate rotation of various RCC coupling beams in comparison with the results made by Englekirk [6].

Hence, in this paper an analytical model of coupling beam was developed to calculate the rotations and moment capacities of coupling beam with conventional reinforcement.

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TABLE I
INVESTIGATIVE MODEL OF COUPLING BEAM IN ATENA2D (2006) AS PER [3]-[6], [9]

Coupling Beam			
Type	**L_b (m)**	**Reinforced Steel**	
		Longitudinal	**Transverse**
Conventional reinforced coupling beam with longitudinal and transverse conforming reinforcement	0.6	8-10 ϕ	2-legged 16 ϕ @200 c/c
		8-20 ϕ	2-legged 25 ϕ @200 c/c
	0.9	8-10 ϕ	2-legged 16 ϕ @200 c/c
		8-20 ϕ	2-legged 25 ϕ @200 c/c
	1.2	8-10 ϕ	2-legged 16 ϕ @200 c/c
		8-20 ϕ	2-legged 25 ϕ @200 c/c

II. ANALYTICAL MODELING IN ATENA2D

Since the above discussions show the contradictory behavior of coupling beam, more study is required to investigate into the limitations on behavior of coupling beams. ATENA2D (2006) [2] was considered to carry out this study.

Four parts are created in the PART module

i) Shear wall 1
ii) Shear wall 2
iii) Reinforcement
iv) Coupling beam

A. Shear Wall

Dimensions of the shear wall were 300mm thick with 4m length and 3m height. Minimum reinforcement in the shear wall was taken as 0.25% of its gross area @ 450 c/c.

B. Reinforcement Layouts

There were six RCC coupling beams with conventional reinforcement layout considered in the analytical program using ATENA2D [2]. For the layout, the cross section of the coupling beam was considered as 600mm (depth, d_b) × 300mm (width, b_b) and the beam span-depth ratio ($\frac{L_b}{d_b}$) was considered as 1, 1.5 and 2. The chosen length and depth of coupling beam are 0.6 x 0.6, 0.9 x 0.6, and 1.2 x 0.6.

C. Material Properties in ATENA 2D

Material no. 1 [Concrete (M20 grade)]: shear wall 1 and shear wall 2
Name: concrete_Plane Stress Elastic Isotropic
Type: CCPlaneStressElastIsotropic
Elastic modulus E = 2.240E+04 [MPa]
Poisson's ratio sm = 0.200 [-]
Specific material weight Rho = 2.300E-02 [MN/m^3]
Coefficient of thermal expansion Alpha = 1.200E-05 [1/K]
Material no. 2
Name: SBeta Material: coupling beam
Type: CCSBETAMaterial
Elastic modulus E = 2.240E+04 [MPa]
Poisson's ratio sm = 0.200 [-]
Tensile strength F_t = 3.130E+00 [MPa]
Compressive strength F_c = -2.000E+01 [MPa]
Type of tension softening: Exponential
Specific fracture energy G_f = 4.421E-05 [MN/m]

Crack model: Fixed
Compressive strain at compressive strength in the uniaxial compressive test Eps_C = -2.000E-03 [-]
Reduction of compressive strength due to cracks CompRed = 0.800 [-]
Type of compression softening: Crush Band
Critical compressive displacement Wd = -5.0000E-04 [m]
Shear Retention Factor Variable
Tension-compression interaction: Linear
Specific material weight Rho = 2.300E-02 [MN/m^3]
Coefficient of thermal expansion Alpha = 1.200E-05 [1/K]
Material no. 3 [Steel (Fe 415 grade)]
Name: Reinforcement
Type: CCReinforcement
Type: BiLinear
Elastic modulus E = 2.100E+05 [MPa]
Sigma Y = 415.000 [MPa]
Specific material weight RHO = 7.850E-02 [MN/m^3]
Coefficient of thermal expansion ALPHA = 1.200E-05 [1/K]

$$V = \frac{F \times L_w}{L_w + L_b} \tag{1}$$
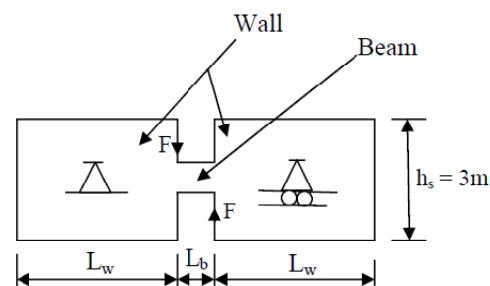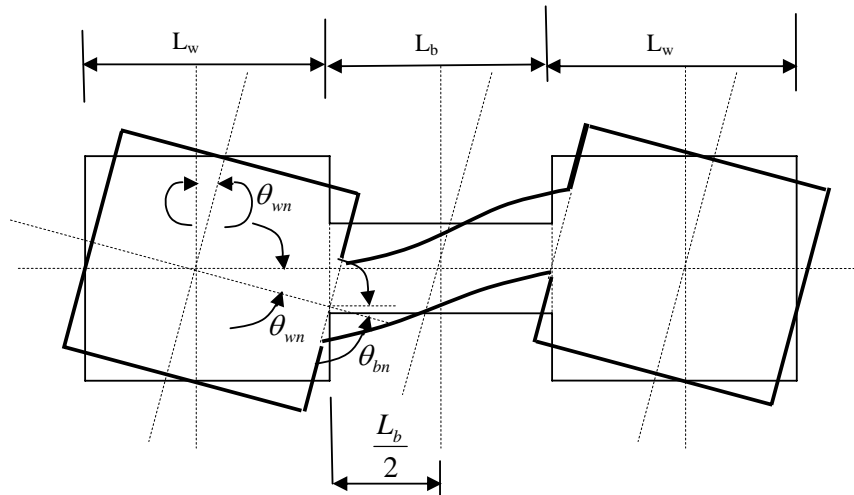
where, V is shear force in the beam

Fig. 1 Sketch of the investigative model

Fig. 2 Deformed shape of a ith storey symmetrical coupled shear walls

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

Rotation of coupling beam at i *th* storey for symmetrical walls [6] as per Fig. 2 is given by

$$\theta_{bi} = \theta_{wi}\left(1 + \frac{L_w}{L_b}\right) \qquad (2)$$

where, θ_{wi} is rotation of wall at i *th* storey and L_w = depth of

wall, L_b = length of coupling beam.

D. Reinforcements

Reinforcement property was modeled according to Table I for each type of beam and results were analyzed by using Solution parameters with Newton-Raphson Method.

III. RESULTS AND DISCUSSIONS

Following tables show the results including discussions for the conventional reinforced coupling beams with conforming transverse reinforcement which were modeled in ATENA-2D [2] as per Fig. 1.

TABLE II
COMPARING THE MODELING PARAMETERS AND NUMERICAL ACCEPTANCE CRITERIA WITH FEMA 273 (1997) AND FEMA 356 (2000) [7]-[8]

Longitudinal reinforcement and transverse reinforcement	$Shear$ $\frac{}{t_w L_w \sqrt{f_c'}}$	Rotational limit at collapse prevention level (CP) in radians								Crack width in coupling beam at CP level in meters by ATENA2D		
		Member controlled by flexure		Member controlled by shear		ATENA2D						
		FEMA273	FEMA356	FEMA273	FEMA356	Reinforcement	$L_b = 0.6m$	$L_b = 0.9m$	$L_b = 1.2m$	$L_b = 0.6m$	$L_b = 0.9m$	$L_b = 1.2m$
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.025	0.025	0.015	0.020	8-10 ϕ and 2-legged 16 ϕ @200 c/c	0.023	0.028	0.032	0.005	0.015	0.019
	≥ 6	0.015	0.02	0.010	0.016	8-20 ϕ and 2-legged 25 ϕ @200 c/c	0.019	0.020	0.022	0.017	0.018	0.021

TABLE III
COMPARING THE MODELING PARAMETERS AND NUMERICAL ACCEPTANCE CRITERIA WITH ATC 40 (1996) [1]

Longitudinal reinforcement and transverse reinforcement	$\dfrac{Shear}{b_w d \sqrt{f_c'}}$	Member controlled by flexure ATC 40	Member controlled by shear ATC 40	Rotational limit at collapse prevention level (CP) in radians ATENA2D Reinforcement	$L_b = 0.6m$	$L_b = 0.9m$	$L_b = 1.2m$	Crack width in coupling beam at CP level in meters by ATENA2D $L_b = 0.6m$	$L_b = 0.9m$	$L_b = 1.2m$
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.025	0.018	8-10 ϕ and 2-legged 16 ϕ @200 c/c	0.023	0.028	0.032	0.005	0.015	0.019
	≥ 6	0.015	0.012	8-20 ϕ and 2-legged 25 ϕ @200 c/c	0.019	0.020	0.022	0.017	0.018	0.021

TABLE IV
COMPARING THE VALUES OF SHEAR PARAMETERS OF FEMA 273 (1997), FEMA 356 (2000), ATC 40 AND ATENA 2D [1], [2], [7], [8]

L_b/d_b	FEMA 273, FEMA 356 $\dfrac{Shear}{t_w L_w \sqrt{f_c'}}$	ATENA 2D	ATC 40 $\dfrac{Shear}{b_w d \sqrt{f_c'}}$	ATENA 2D	ATENA 2D $\dfrac{Shear}{t_b L_b \sqrt{f_c'}}$
1	≤ 3	0.08	≤ 3	0.56	0.56
	≥ 6	0.18	≥ 6	1.19	1.19
1.5	≤ 3	0.05	≤ 3	0.32	0.22
	≥ 6	0.12	≥ 6	0.70	0.47
2	≤ 3	0.04	≤ 3	0.28	0.14
	≥ 6	0.10	≥ 6	0.58	0.29

It was observed from the Tables II and III that the rotational limit at collapse prevention level (CP) were in the similar trends in FEMA 273, FEMA 356, ATC 40 and the results of ATENA 2D. It was also seen that the crack widths were more but rotations were less for the conditions of more shear, which is quite well understood. In addition, Tables II-IV were showing incremental rotational capacities with lesser shear and moment capacities with increases of L_b or L_b/d_b ratio. As per the literatures [18], greater L_b/d_b ratio gives weakly coupled shear walls which are not accepted. Table IV was also showing that there was no case where $\dfrac{Shear}{t_w L_w \sqrt{f_c'}}$ or $\dfrac{Shear}{b_w d \sqrt{f_c'}}$

achieves ≤ 3 or ≥ 6. However, based on the conditions of $\dfrac{Shear}{b_w d \sqrt{f_c'}}$ or $\dfrac{Shear}{t_b L_b \sqrt{f_c'}}$ greater ratios were obtained specially in the case of $\dfrac{Shear}{b_w d \sqrt{f_c'}}$.

IV. CONCLUSIONS

➢ Rotational capacities of beam depend on size of the beam, i.e. L_b, b_w and d.
➢ The behavior of the coupling beam should be governed by shear.
➢ The aspect ratio (L_b/d_b) of the coupling beam should be less than 1.5.
➢ The condition in the table given in FEMA 273, FEMA 356 and ATC 40 should not never be either ≤ 3 or ≥ 6.
➢ Finally, rotational limit at collapse prevention level (CP) for conventional reinforced coupling beam should be in the range of 0.01-0.02 radian.

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