The Failure Mechanism of Reinforced Concrete Couple Shear Walls and Coupling Beams- A Ductility Approach

Sk Amjad Hossain[1]* and Ashutosh Bagchi[2]

[1]Department of Civil Engineering and Construction Management, The University of Texas, Tyler Houston Engineering Center, USA
[2]Department of Building, Civil and Environmental Engineering, Concordia University, Montreal, Canada

Abstract

The coupled shear wall system is one of the effective potential options in midrise and high rise reinforced concrete buildings design. In seismic design consideration it is very important to ensure that the flexural displacement capacity which is known as ductility should be greater than flexural displacement demand. Non-linear dynamic analyses are performed on 12 storey slender coupled shear wall buildings in 14 different models as per Canadian Code CSA-A23.3 14, except the coupling beams are modelled with conventional reinforcements. Inelastic rotational and curvature demand of 06 coupling beams on each building at different level which are modelled with conventional reinforcements are investigated and the results are compared with US building codes, ACI-318-19, ATC 40 and FEMA-273-356. Investigation shows that coupling beams modelled with conventional reinforcements the inelastic rotational and curvature demand are within the capacities. It has been observed also the overall flexural displacement capacity of the seismic force resisting system of 12 story coupled shear wall buildings in 14 different models in ductility approach. Curvature ductility demands are calculated from those models which are the function of displacement ductility demand and also the plastic hinge length. In ductility approach calculations it was observed that the maximum flexural compression zone length limitation without confinement reinforcement as per (CSA A23.3 1994) is not working with slender coupled shear wall (tension or compression wall), while considering the plastic hinge length as per CSA- A23.3 04 or CSA-A23.3 14; Separate formula has been proposed for maximum compression zone length limitation for coupled shear wall design in ductility approach.

Keywords: Coupled shear wall • Reinforced concrete • Failure mechanism • Ductility • Flexural displacement • Building code seismic • Force resisting systems.

Introduction

The coupled shear wall system is one of the effective potential Seismic Forces Resisting System (SFRS) in that have been using in midrise and high rise reinforced concrete buildings design. In seismic force resisting system it has to be ensure that the flexural displacement capacity which is known as ductility should be greater than the flexural displacement demand. The Canadian design standards, CSA A23.3 04 & 14, when confined reinforcements are required or not both are developed as per displacement-based approach and are expressed in terms of inelastic rotations. The inelastic displacement demands are calculated in ductility approach from displacement ductility demand and yield displacements, which is basically estimated from yield curvature, on the other hand inelastic displacement demand is an input in displacement approach. In displacement-based approach the low degree of coupled shear walls is modelled similar to the separate cantilever walls, on the other hand very high degree of coupled shear walls are modelled similar to the solid walls with openings. And, it is not properly justified to model coupled shear walls as solid cantilever shear walls with openings, since mass and stiffness distribution are not same for cantilever and coupled shear walls. Whereas, in terms of ductility

perspective, one of the important phenomena that the curvature distribution in coupled shear walls at the points the system yields is completely different than cantilever shear walls. And that phenomenon is truly considered if rotational capacity and rotational demand are calculated in ductility approach in terms of reinforced concrete coupled shear walls design.

Objectives

The objectives of this study to investigate parameters that affect the load deflection behaviour of couple shear walls in ductility approach, using non-linear dynamic analysis. The analysis was performed using computer software SeismoStruct 2000. It was modelled a 12 storey building in Seismostruct 2000. Investigations were conducted on 14 different case studies with variation of different parameters like aspect ratio of the walls, depth span ratio of coupling beams, reinforcements of walls and coupling beams etc. And it will be investigating the influencing parameters that effect on the true load deflection behaviour calculating the displacement ductility demand and curvature ductility demand of the coupled shear walls and also effect of conventional reinforcement pattern on coupling beams.

Literature review: Ductility and plastic hinge length

In order to make proper seismic design of RC concrete wall, one of the important parts is to ensure flexural displacement capacity should be greater than flexural displacement demand. The inelastic portion of flexural displacement demand has been generated from a concentration of inelastic curvatures near the base of the wall. Usually, inelastic curvatures are assumed to be uniform over a height that is called plastic hinge length. Bohl A and Adebar P [1] performed a study to investigate the profile of inelastic curvatures that should be used to estimate the flexural displacement capacity of high-rise RC concrete walls. A handful numbers of study were performed earlier by different researchers summarized as follows. Not too many analysis and test results are available also in recent days in this field.

Especially for coupled shear wall, Adebar P, et al. [2] commented that, "To the

author's knowledge, there are no recommendations for plastic hinge length of coupled walls." Chan WWL [3] proposed that the plastic hinge length is equal to the length over which reinforcement is yielding in any kind of wall. Sawyer HA [4] proposed a seminal equation for plastic hinge length in beams, where he assumed M_y/M_{max} =0.85 and determined l_p* = 0.15 z from Chan equation. He recognized that inelastic curvatures vary linearly from the maximum value to zero over this length and that the same inelastic rotation results from the maximum inelastic curvature being uniform over half of this length. He assumed that plasticity would spread over a length of d/4. Mattock AH [5] and Corley WG [6] performed tests on a large number of beams up to 750 mm deep and observed that plastic hinge length increases with effective depth d and shear span ratio (z/d) and decreases with the quantity of flexural tension reinforcements. Mattock AH [5] proposed modified plastic hinge length value in Sawyer HA equation.

Paulay T and Uzumeri SM [7] modified Sawyer's equation by assuming that d = 0.8l_w and z = h_w to apply for concrete walls. Pauley T and Priestley MJN [8] recommended α =0.25 and β =0.044 in modified Sawyer's equation for a lower bound estimate of plastic hinge length, l_p of concrete walls. Bohl A and Adebar P [1] proposed a simplified equation for maximum curvature at the base of the walls,

$$\phi_{max,2} = \phi_{y,1} + (\phi_{max,1} - \phi_{y,2})(\frac{l_{p,1}}{l_{p,2}});$$

Where ϕ_{max} = maximum curvature at the base of the wall, ϕ_y = yield curvature and the subscripts 1 and 2 refer to the longer and shorter wall respectively.

In couple shear wall buildings, a system of couple shear wall is connected together by coupling beams. And, the systems of high degree of coupling are subjected to lateral loading, the shear force in the coupling beams induce high degree of axial force in the walls. One wall will be subjected to tension and other will be subjected to compression. Geometry of the coupling beam, span and the axial forces (tension and compression) are expected to have some effect on plastic hinge prediction.

The configuration of couple shear is consisting of by interconnecting beams connected along the height of the two coupled walls. The behavior of couple shear wall is mostly depend on the behavior of coupling beams and the failure of coupling beam is one of the failure modes of couple shear walls. The coupling beams should be designed as ductile inelastic manner for properly energy dissipations. The energy dissipations of the coupling beams depend on the yield moment capacity and the plastic rotation capacity of the coupling beams. If the yield moment capacity of the coupling beams is very high then it would undergo limited rotation and dissipate limited energy, on the other hand, if the yield moment capacity is not high, the couple beams may undergo much more than the plastic rotation capacity. Hence, the yield moment capacity should be optimized in order to properly design the coupling beams of the couple shear wall.

And, the yield moment capacity of the coupling beams also depends on the plastic rotation capacity.

Conventional reinforcement's patterns are defined as the formation of longitudinal flexural reinforcements and transverse shear reinforcements. Longitudinal reinforcements consist of top and bottom reinforcement's parallel to the longitudinal axis of the coupling beams and transverse reinforcements consists of formation of closely spaced ties along the length of the coupling beams.

Diagonal reinforcements in coupling beams consists of placement of reinforcements diagonally. It requires minimum four bars per diagonal for formation of the diagonal reinforcements. Though, it was not modelled the coupling beams under investigations with the diagonal reinforcements in this study. The diagonal reinforcements may be buckled if it is subjected to compressive loading in coupling beams. Under the reverse loading the reinforcement already buckled in one direction with compressive loading may be stressed in tension again. But this action could lead to low-cycle fatigue failure. To avoid this kind of circumstances, it is necessary to ensure that the reinforcements should not buckle. In the different building codes there is a minimum spacing required between two adjacent diagonal bars to prevent buckling due to the compressive loads.

Design provisions of coupling beams with different building codes:

US Code ACI-318 – 19/ ATC 40/ FEMA 273:

Conventional reinforcements (Tables 1-4)

Table 1. Rotation capacities for coupling beams controlled by flexure as per FEMA 273 and FEMA 356.

Type of Coupling Beam	Conditions Shear/twLw √fc'	Plastic Rotation Capacity (Radians)		
		IO	LS	CP
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.006	0.015	0.025
	≤ 6	0.005	0.010	0.015
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤ 3	0.006	0.012	0.020
	≤ 6	0.005	0.008	0.010

Table 2. Rotation capacities for coupling beams controlled by shear as per FEMA 273 and FEMA 356.

Type of Coupling Beam	Conditions Shear/twLw √fc'	Plastic Rotation Capacity (Radians)		
		IO	LS	CP
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.006	0.012	0.015
	≤ 6	0.004	0.008	0.010
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤ 3	0.006	0.008	0.010
	≤ 6	0.004	0.006	0.007

Table 3. Rotation capacities for coupling beams controlled by flexure as per ATC 40.

Type of Coupling Beam	Conditions Shear/twLw √fc'	Plastic Rotation Capacity (Radians)		
		IO	LS	CP
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.006	0.015	0.025
	≤ 6	0.005	0.010	0.015
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤ 3	0.006	0.012	0.020
	≤ 6	0.005	0.008	0.010

Table 4. Rotation capacities for coupling beams controlled by shear as per ATC 40.

Type of Coupling Beam	Conditions Shear/twLw $\sqrt{fc'}$	Plastic Rotation Capacity (Radians)		
		IO	LS	CP
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.006	0.012	0.015
	≤ 6	0.004	0.008	0.010
Conventional longitudinal reinforcement with non-conforming transverse reinforcement	≤ 3	0.006	0.008	0.010
	≤ 6	0.004	0.006	0.007

Table 5. Geometry considered in 14 different coupled shear walls.

No of Cases	Depth of the Wall (m)	Length of the Beam (m)	Depth of the Coupling beam (mm)	Width of the Coupling Beam (mm)	Height of the Wall (m)
Model #01	2.5	2.0	800	650	43.2
Model #02	2.5	2.0	800	650	43.2
Model #03	2.5	2.0	800	650	43.2
Model #04	2.5	2.0	800	650	43.2
Model #05	2.5	2.0	800	650	43.2
Model #06	2.5	2.0	750	650	43.2
Model #07	2.5	2.0	700	650	43.2
Model #08	3.0	2.0	1000	650	43.2
Model #09	3.0	2.0	800	650	43.2
Model #10	2.5	2.0	800	650	43.2
Model #11	2.5	2.0	800	650	43.2
Model #12	2.5	2.0	800	650	43.2
Model #13	2.5	2.0	800	650	43.2
Model #14	2.5	2.0	1000	650	43.2

Modeling in seismostruct

Geometric non-linearities are considered in seismostruct, modeling. Large displacement/ rotations and second order (nonlinear) effect that occurs when an axial load acts upon a lateral displacement generating an additional moment called (p-delta effect) are considered in the modeling through a total co-rotational formulation, which was developed by Correia AA and Virtuoso FBE [9]. Material non-linearity also considered in Seismostruct modeling. In seismostruct, fiber approach modeling is used to represent the cross-sectional behavior, where each fiber is modeled with uniaxial stress-strain relationship. Then, the sectional stress-strain state of beam column element is calculated through the integration of the non-linear uniaxial stress strain response of the individual fiber in which the section is subdivided, i.e., the discretization of the reinforced concrete section. In seismostruct modeling, it has been considered the distributed inelasticity instead of lumped inelasticity. In this research modelling recent developed Forced Based (FB) elements are considered for distributed inelastic finite element modeling. In terms of material inelasticity, it does have one advantage over Displacement Based (DB) elements model. With a displacement-based model refined discretization (meshing) are required, on the other hand, in Forced Based (FB) model it does not require. FB formulation does not depend on sectional constitutive behavior; hence the solution is always exact.

Methodology

A twelve-story building with a simple configuration with 14 different models (3 bays by 3 bays) are analyzed in SeismoStruct [10]. The seismic design parameters are considered for Vancouver location. It has been considered real seismic ground motions those are taken from PEER (Pacific Earthquake Research Institute). The selection of ground motion considers the ratio of peak Acceleration (A) to peak Velocity (V) close to 1, which represents the seismicity of Vancouver. The height of the building is considered 45.0 m. The design live load is equal to 2.4 kN/m² for all floors except the first storey which is 4.8 kN/m². The snow load is calculated to be 2.3 kN/m². The dead load is 0.85 kPa exterior walls, 1.0 kPa for partition on floors, 0.5 kPa for ceiling and mechanical services for all floors and 0.5 kPa for roofing. The walls are considered ductile

partially couple shear walls (NBCC-2015) with R_d=3.5 and R_o=1.6 for the lateral force resisting systems, where R_o and R_d are over strength and ductility factors respectively.

The building was modeled in SeismoStruct having a plan dimension 21 m × 21 m. The thickness of the couple shear wall is 650 mm. The wall thickness, the dimension of coupling beams is considered as per the minimum requirements of CSA A23.3-04 except the coupling beams are modelled with conventional reinforcement patterns.

In Seismostruct the concrete is modeled with Mander JB, et al. [11] non-linear concrete model and reinforcements are modeled with Menegotto Pinto steel model [10]. Dynamic time history analyses are performed with different wall length (L_w) 2 m, 2.5 m and 3 m and design parameters for that building.

In this study there are 06 coupling beams at different levels of the buildings are selected and investigated, those are CSH-223, CSH-255, CSH-2125, CSH-423, CSH-455 and CSH-4125;

Chord shear force, bending moments, inelastic rotations and curvatures with different conditions for those coupling beams are summarized in Table # 7 to 12; It has been summarized the geometric and material properties of 14 different models in Table # 5 and 6;

Analysis Results

Rotational capacity of coupling beam

The rotational capacity of coupling beams depends on various parameters. When the rotational demand is greater than the rotational capacity in a conventionally reinforced coupling beam, diagonally reinforcement could be provided as an alternative option and that depends also on Lb/db ratio of the coupling beam. In this research it has been conducted study with different Lb/db ratio and conventional reinforcement and compares the rotational demand with rotational capacities with different building codes. Various research works were performed by different researchers, some of those are summarized in below Tables 1-4. (Tables 7-12)

Table 6. Material properties and geometry considered in 14 different coupled shear walls.

No of Cases	Aspect Ratio (Coupled Wall)	Aspect ratio (Coupling Beam)	Wall Thickness	Rebar (Coupled Wall) (mm²)	Rebar (Coupling Beam) (mm²)	Rebar Grade
Model #01	2.5/43.2	0.8/2.0	0.65	29541.12	9650.79	GR-75
Model #02	2.5/43.2	0.8/2.0	0.65	25848.48	9650.79	GR-75
Model #03	2.5/43.2	0.8/2.0	0.65	25471.68	9650.79	GR-75
Model #04	2.5/43.2	0.8/2.0	0.65	25471.68	8000.72	GR-75
Model #05	2.5/43.2	0.8/2.0	0.65	25471.68	7693	GR-75
Model #06	2.5/43.2	0.75/2.0	0.65	25471.68	7693	GR-75
Model #07	2.5/43.2	0.70/2.0	0.65	25471.68	7693	GR-75
Model #08	3.0/43.2	1.0/2.0	0.65	62699.52	39564	GR-60
Model #09	3.0/43.2	0.8/2.0	0.65	36926	10173	Gr-75
Model #10	2.5/43.2	0.8/2.0	0.65	48230.4	28260	Gr-75
Model #11	2.5/43.2	0.8/2.0	0.65	36926.4	10173.6	Gr-75
Model #12	2.5/43.2	0.8/2.0	0.65	31839.6	8038.4	Gr-75
Model #13	2.5/43.2	0.8/2.0	0.65	31839.6	8038.4	Gr-75
Model #14	2.5/43.2	1.0/2.0	0.65	31839.6	8038.4	Gr-75

Table 7. Element. CSH-223 with conventional longitudinal reinforcement with conforming transverse reinforcement.

Coupling Beam (Type)	Coupling Beam Aspect Ratio (Lb/db)	Conditions. Shear/twLw√fc	Conditions. Shear/bwd √fc	Chord Shear (kN)	Chord Bending Moment (kN-m)	Rotations (Radians)	Curvature (Radians)
Case-1	2.5	0.25	0.32	2430	3585	0.00016	0.0006
Case-2	2.67	0.3	0.37	2859	4017	0.00016	0.0006
Case-3	2.85	0.25	0.31	2412	5643	0.00016	0.0006
Case-4	2.0	0.31	0.47	3579	6298	0.00015	0.0003
Case-5	2.0	0.30	0.38	2958	3982	0.0001	0.0006

Table 8. Element. CSH-255 with conventional longitudinal reinforcement with conforming transverse reinforcement.

Coupling Beam (Type)	Coupling Beam Aspect Ratio (Lb/db)	Conditions. Shear/twLw√fc	Conditions. Shear/bwd √fc	Chord Shear (kN)	Chord Bending Moment (kN-m)	Rotations (Radians)	Curvature (Radians)
Case-1	2.5	0.028	0.035	276	498	0.0013	0.003
Case-2	2.67	0.022	0.027	208	446	0.0014	0.003
Case-3	2.85	0.023	0.029	225	469	0.0014	0.003
Case-4	2.0	0.06	0.075	582	1117	0.001	0.0025
Case-5	2.0	0.056	0.07	539	838	0.0012	0.003

Table 9. Element. CSH-2125 with conventional longitudinal reinforcement with conforming transverse reinforcement.

Coupling Beam (Type)	Coupling Beam Aspect Ratio (Lb/db)	Conditions. Shear/twLw√fc	Conditions. Shear/bwd √fc	Chord Shear (kN)	Chord Bending Moment (kN-m)	Rotations (Radians)	Curvature (Radians)
Case-1	2.5	0.037	0.047	363	588	0.002	0.0037
Case-2	2.67	0.037	0.046	358	603	0.0022	0.0048
Case-3	2.85	0.035	0.044	337	711	0.001	0.003
Case-4	2.0	0.06	0.09	719	1374	0.001	0.003
Case-5	2.0	0.045	0.056	436	796	0.0011	0.0022

Table 10. Element: CSH-423 with conventional longitudinal reinforcement with conforming transverse reinforcement.

Coupling Beam (Type)	Coupling Beam Aspect Ratio (Lb/db)	Conditions. Shear/twLw√fc	Conditions. Shear/bwd √fc	Chord Shear (kN)	Chord Bending Moment (kN-m)	Rotations (Radians)	Curvature (Radians)
Case-1	2.5	0.25	0.32	2439	5425	0.00024	0.0007
Case-2	2.67	0.267	0.334	2573	5226	0.00024	0.0007
Case-3	2.85	0.334	0.417	3214	4699	0.00024	0.0007
Case-4	2.0	0.323	0.485	3735	7619	0.00013	0.0004
Case-5	2.0	0.26	0.326	2513	6401	0.00018	0.0006

Table 11. Element: CSH-455 with conventional longitudinal reinforcement with conforming transverse reinforcement.

Coupling Beam (Type)	Coupling Beam Aspect Ratio (Lb/db)	Conditions. Shear/twLw√fc	Conditions. Shear/bwd √fc	Chord Shear (kN)	Chord Bending Moment (kN-m)	Rotations (Radians)	Curvature (Radians)
Case-1	2.5	0.046	0.058	449	784	0.0027	0.004
Case-2	2.67	0.04	0.044	398	712	0.0027	0.005
Case-3	2.85	0.035	0.044	340	644	0.003	0.005
Case-4	2.0	0.08	0.122	941	1839	0.0015	0.0015
Case-5	2.0	0.065	0.08	629	1518	0.0023	0.0033

Table 12. Element. CSH-4125 with conventional longitudinal reinforcement with conforming transverse reinforcement.

Coupling Beam (Type)	Coupling Beam Aspect Ratio (Lb/db)	Conditions. Shear/twLw√fc	Conditions. Shear/bwd √fc	Chord Shear (kN)	Chord Bending Moment (kN-m)	Rotations (Radians)	Curvature (Radians)
Case-1	2.5	0.034	0.042	326	791	0.002	0.0037
Case-2	2.67	0.037	0.046	357	765	0.0028	0.0047
Case-3	2.85	0.039	0.049	377	718	0.0028	0.0056
Case-4	2.0	0.064	0.097	749	1483	0.001	0.0017
Case-5	2.0	0.042	0.052	407	938	0.0014	0.002
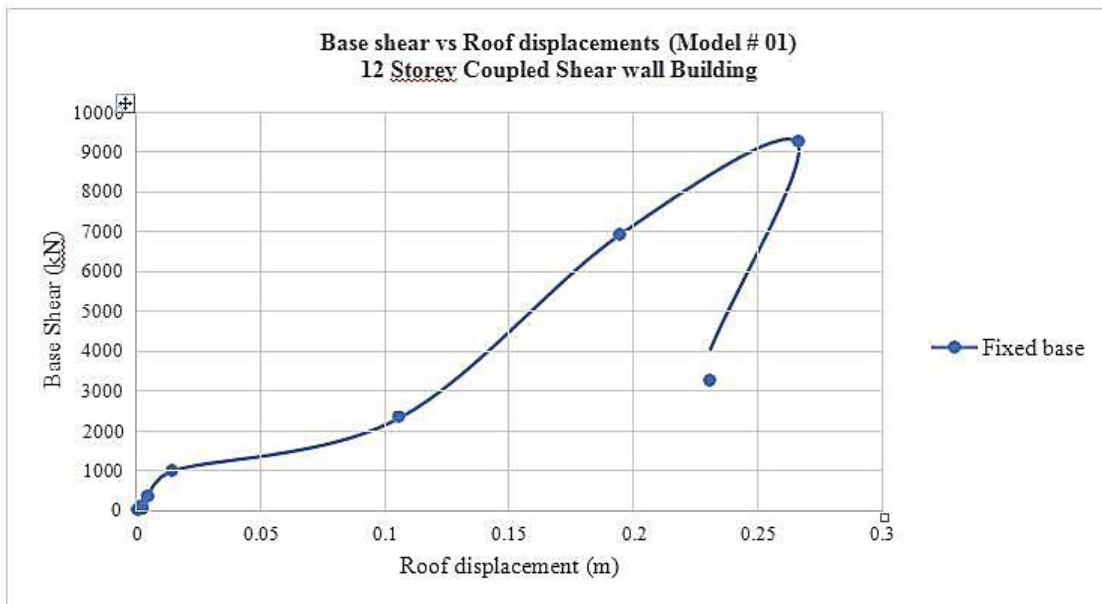
Figure 1. Model # 01 (Static analysis).

Base shear *vs.* roof displacements in different models (12 storey buildings)

Non-linear dynamic analyses are performed and seismic base shear are calculated on 14 different models of the 12-storey building as described in methodology sections. Those base shear are plotted with respect to roof displacement, those are plotted in below Figures 1-27.

Calculation of ductility

The objectives of wall's ductility provisions in earthquake resistant design are to ensure the curvature capacity ϕ_c should be greater than curvature demand ϕ_d imposed by any earthquake on it.

$$\phi_c > \phi_d \qquad \text{Equation \# 01}$$

In ductility approach, total curvature demand is calculated from curvature ductility demand $\mu_{\phi d}$ and yield curvature ϕ_y.

$$\phi_d = \mu_{\phi d} * \phi_y \qquad \text{Equation\#02}$$

The total displacement demand, which is calculated from an elastic portion referred as yield displacement Δ_y and an inelastic portion Δ_{id}. It can be estimated similarly from displacement ductility demand $\mu_{\Delta d}$ and yield displacements Δ_y.

$$\Delta_d = \Delta_{y+}\Delta_{id} = \mu_{\Delta d}\Delta_y \qquad \text{Equation \# 03}$$

According to Paulay T and Uzumeri SM [7] the yield displacements at the top of the wall can be calculated from the yield curvature at the base of the wall assuming a first mode of curvature distribution.

$$\Delta_y = 0.28\phi_y h_{w^2} \qquad \text{Equation \# 04}$$

The total curvature demand ϕ_d is equal to summation of an elastic portion referred as yield curvature ϕ_y and inelastic curvature demand ϕ_{id}.

$$\phi_d = \phi_y + \phi_{id} \qquad \text{Equation \# 05}$$

The inelastic curvature demand ϕ_{id} can be directly calculated from the inelastic displacement demand at the top of the wall Δid considering the displacement approach

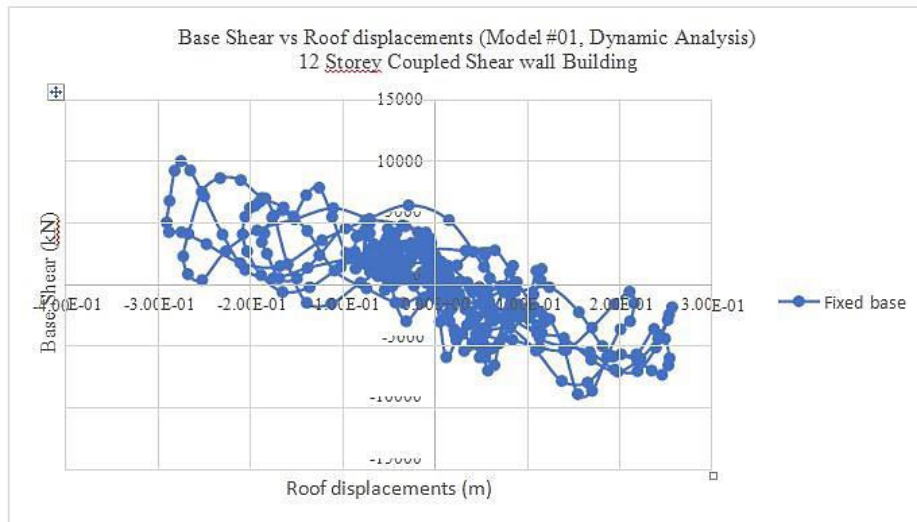$$\phi_{id} = \frac{\phi_{id}}{lp} = \frac{\Delta id}{lp(hw - 0.5lp)} \qquad \text{Equation \# 06}$$
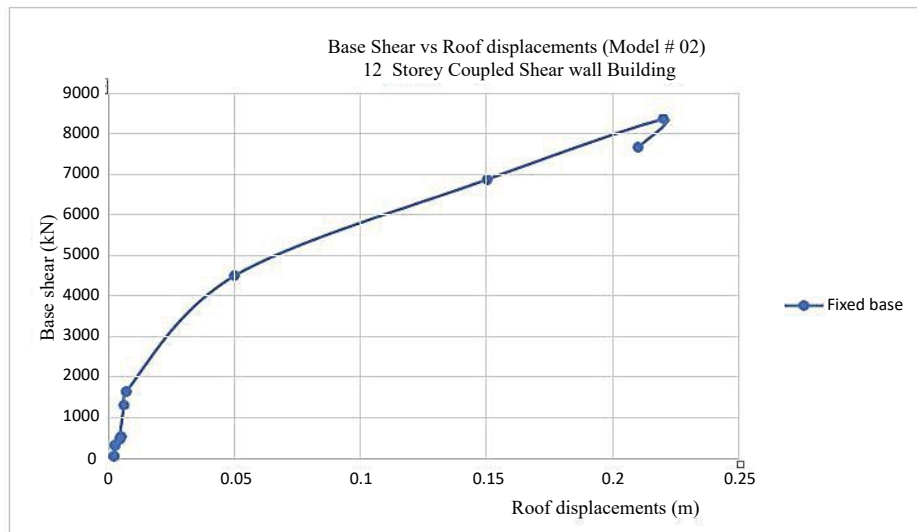
Figure 2. Model # 01 (Dynamic analysis).
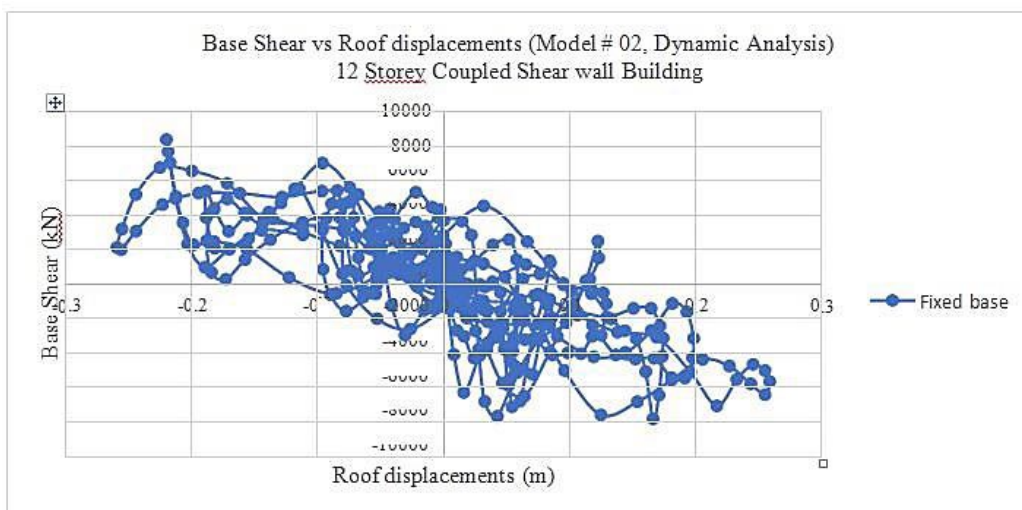

Figure 3. Model # 02 (Static analysis).

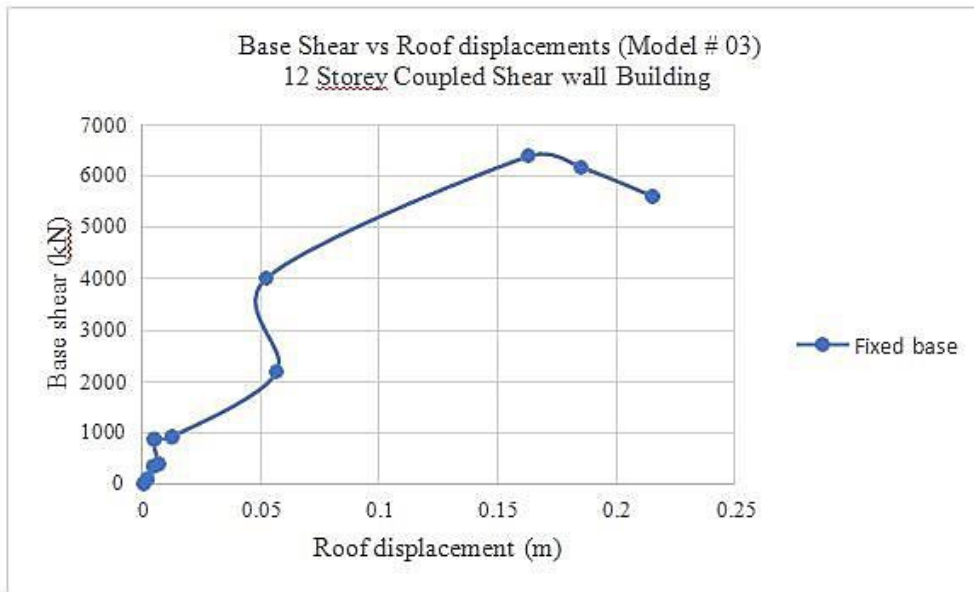**Figure 4.** Model # 02 (Dynamic analysis).

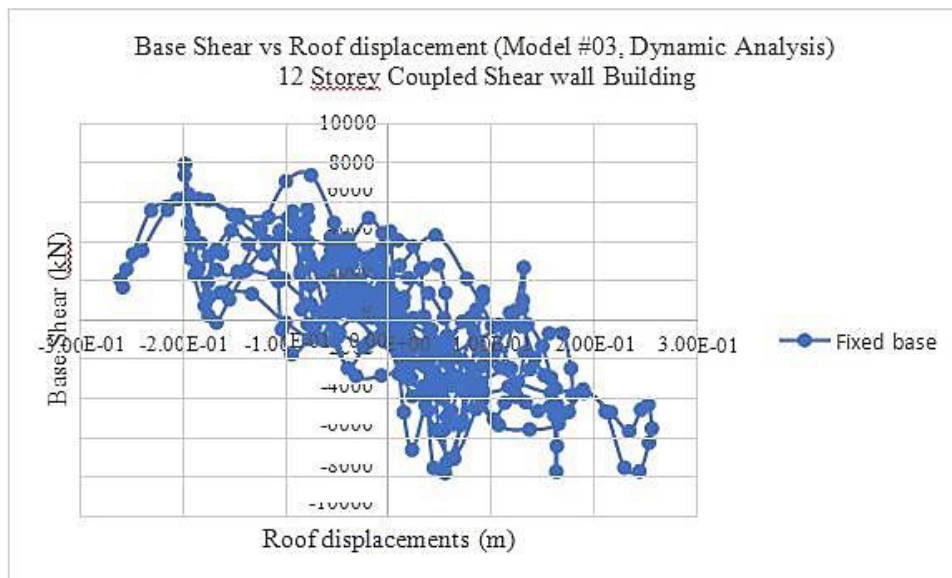**Figure 5.** Model # 03 (Static analysis).

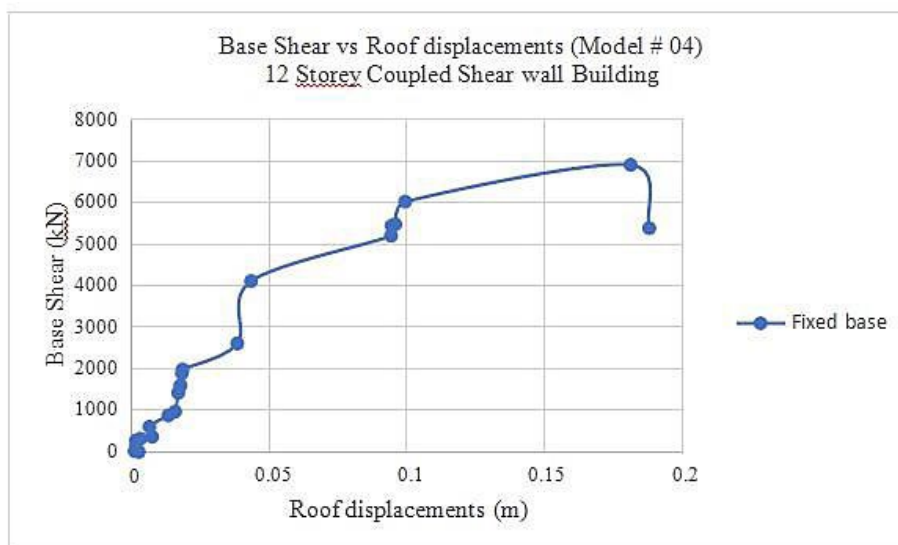**Figure 6.** Model # 03 (Dynamic analysis).

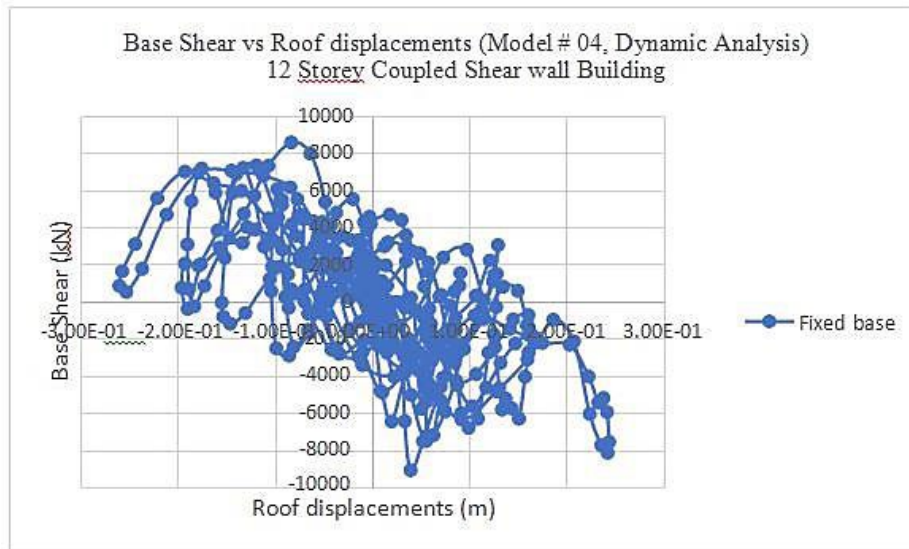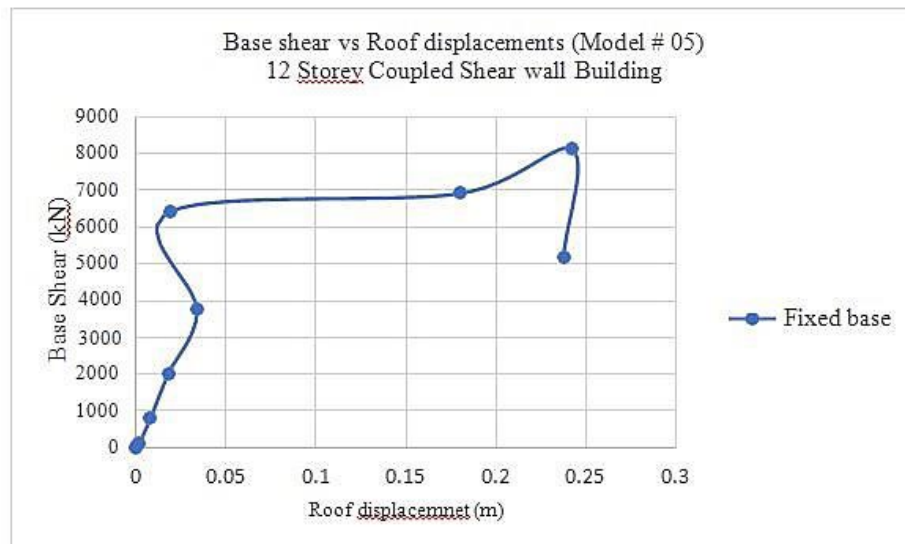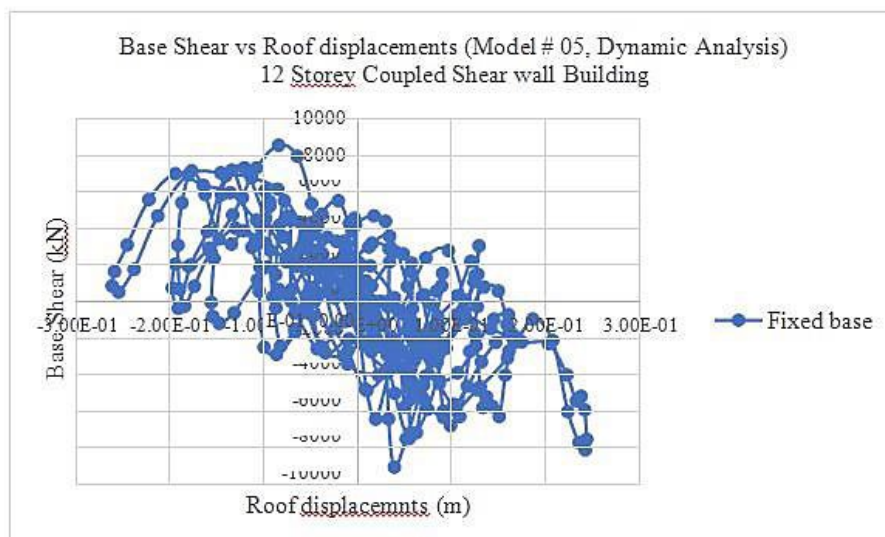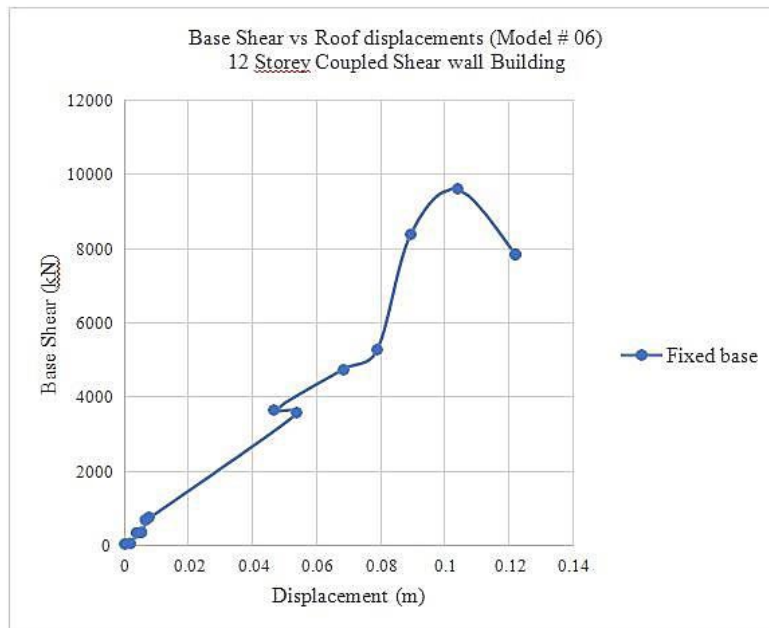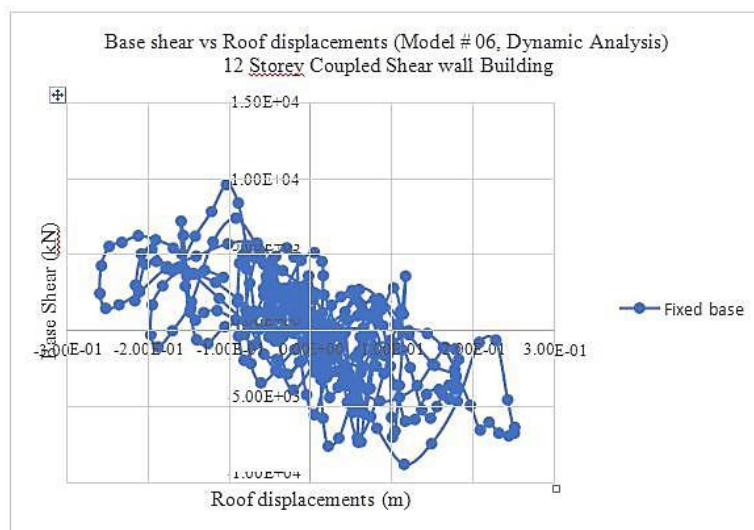**Figure 7.** Model # 04 (Static analysis).

Figure 8. Model # 04 (Dynamic analysis).

Figure 9. Model # 05 (Static analysis).

Figure 10. Model # 05 (Dynamic analysis).

Figure 11. Model # 06 (Static analysis).

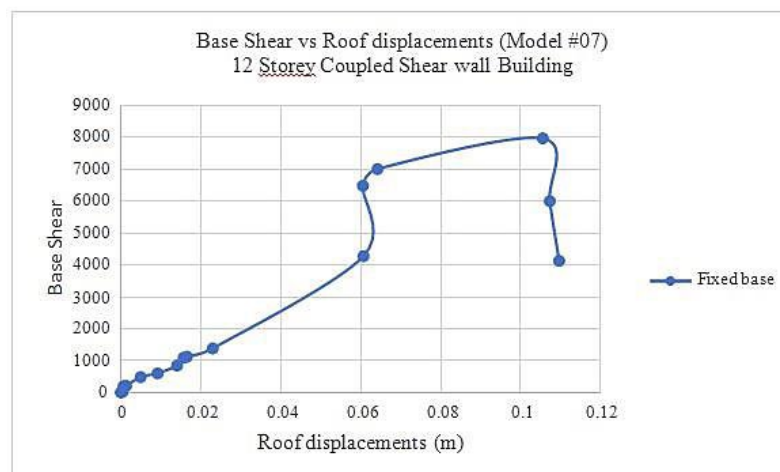**Figure 12.** Model # 06 (Dynamic analysis).

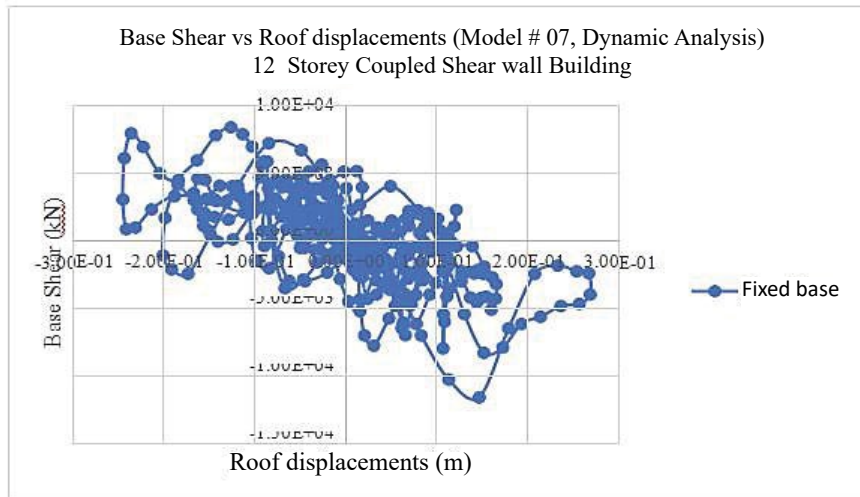**Figure 13.** Model # 07 (Static analysis).

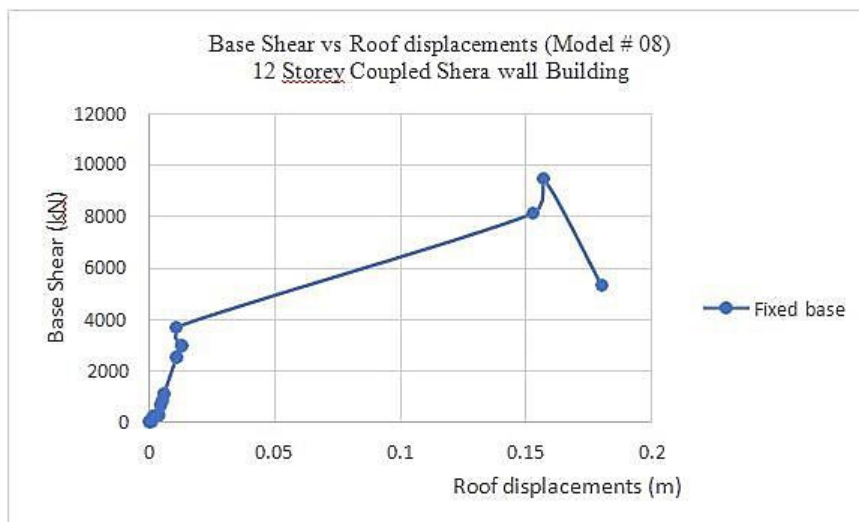**Figure 14** .Model # 07 (Dynamic analysis).

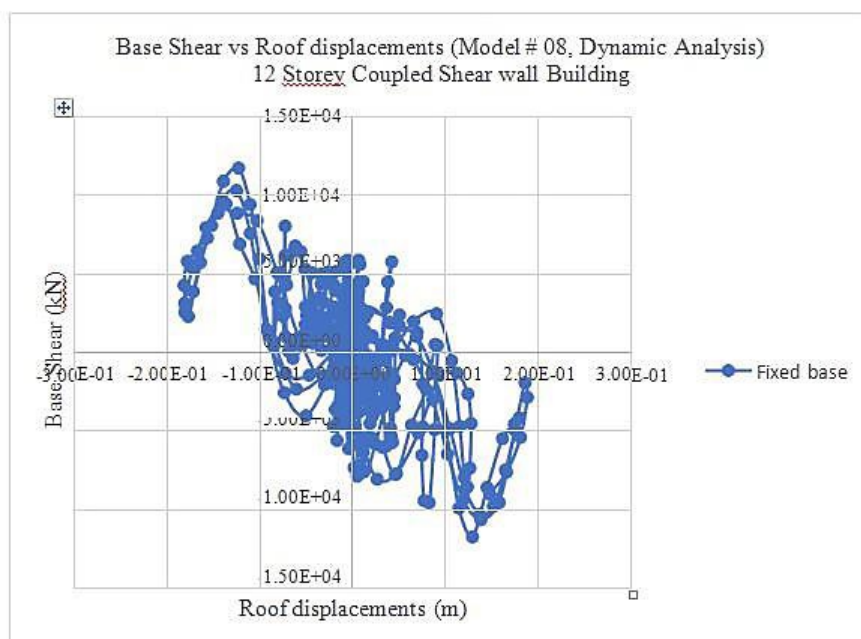**Figure 15.** Model # 08 (Static analysis).

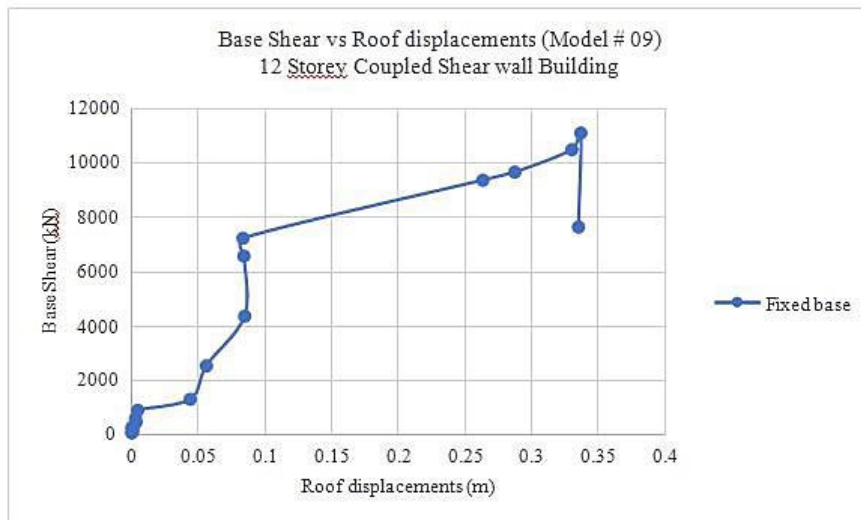**Figure 16.** Model # 08 (Dynamic analysis).

Figure 17. Model #09 (Static analysis).

Figure 18. Model # 09 (Dynamic analysis).

Figure 19. Model #10 (Static analysis).

Figure 20. Model # 10 (Dynamic analysis).

Figure 21. Model #11 (Static analysis).

Figure 22. Model # 11 (Dynamic analysis).

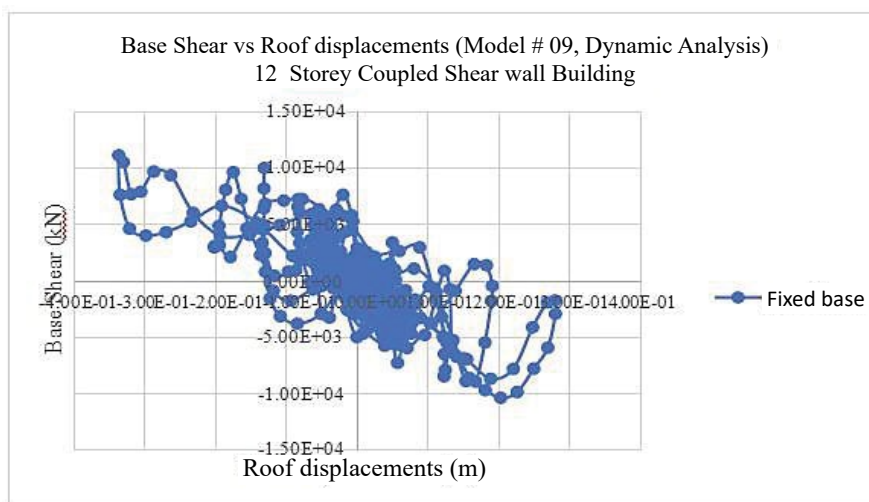**Figure 23.** Model # 12 (Static analysis).

Figure 24. Model # 12 (Dynamic analysis).

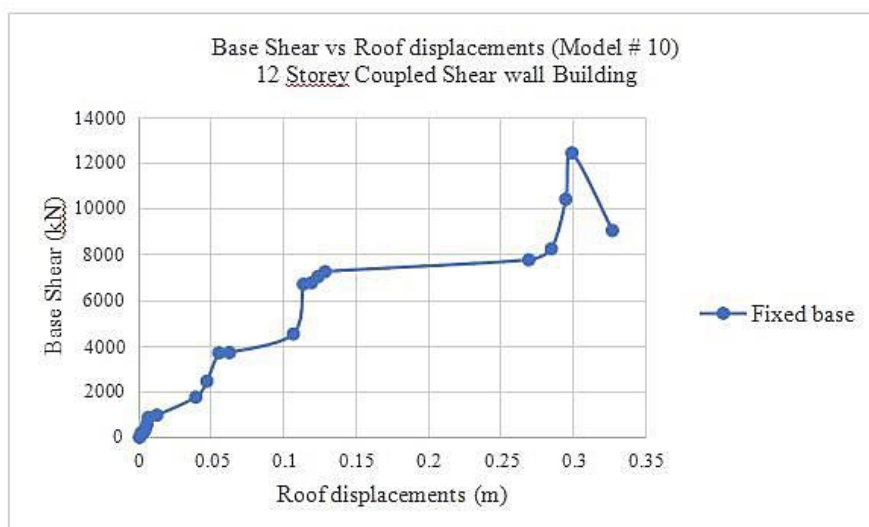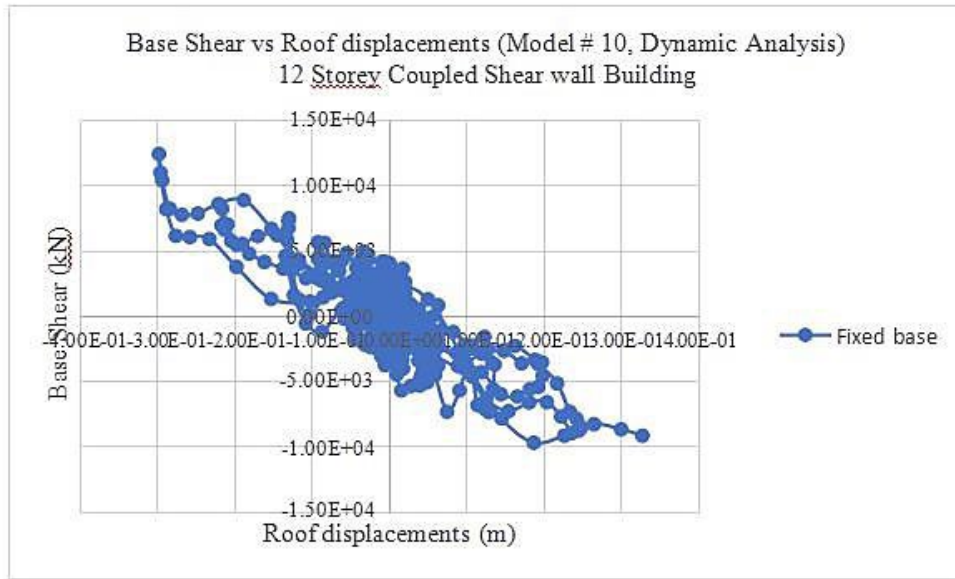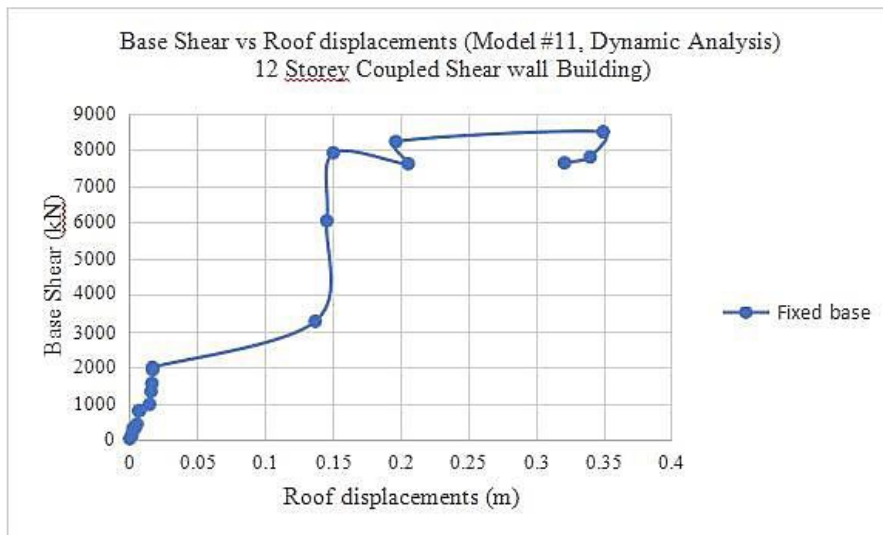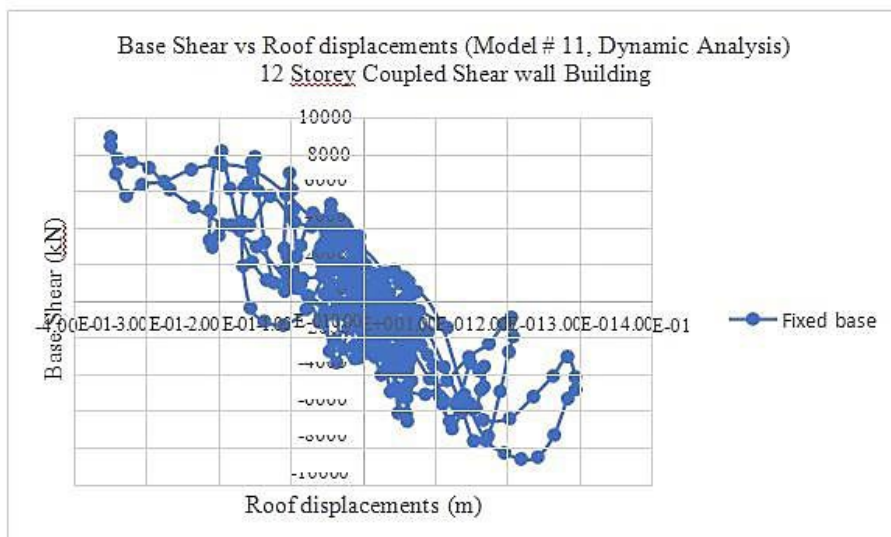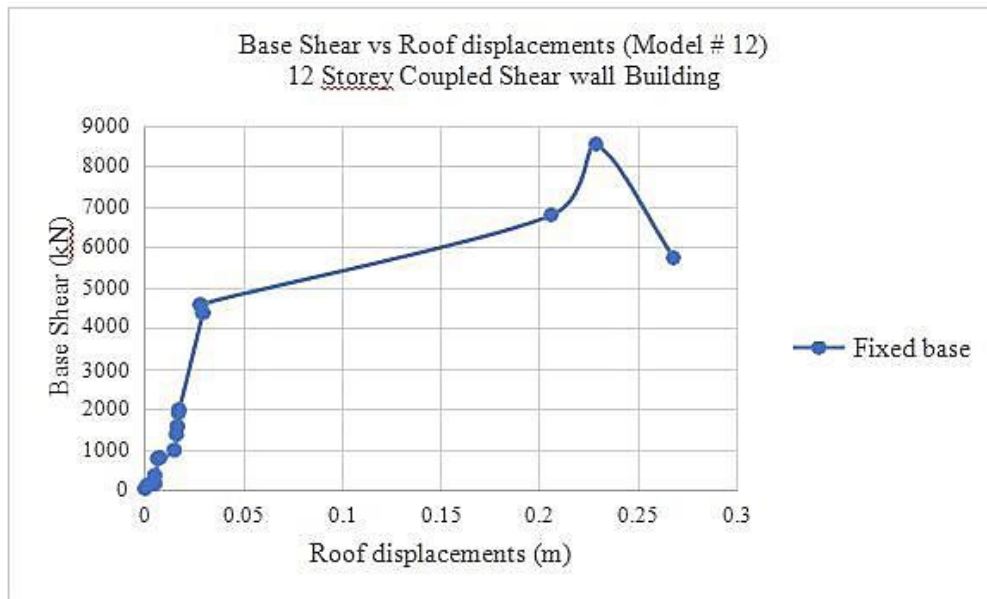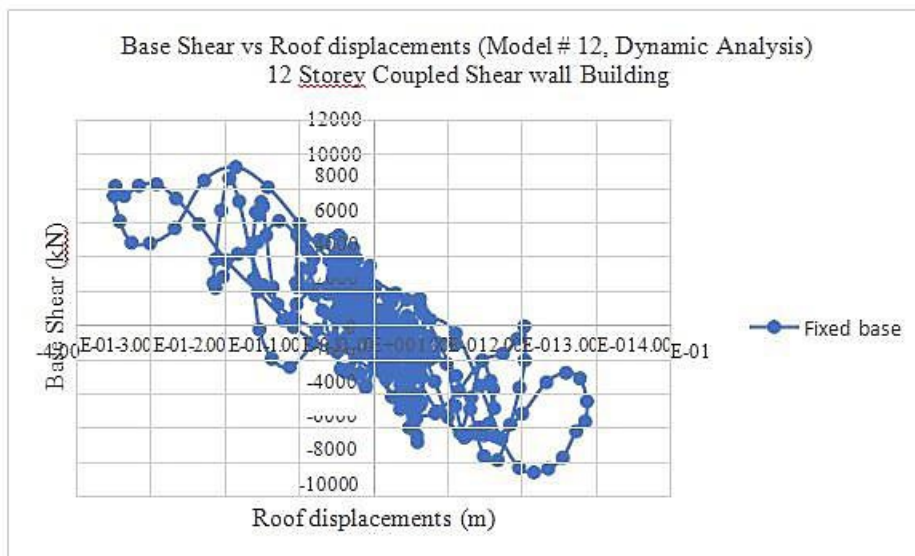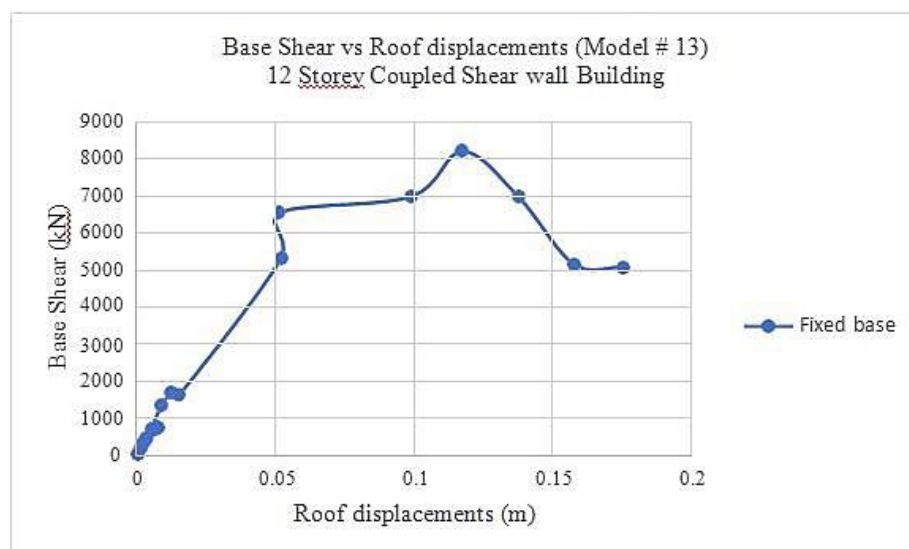**Figure 25.** Model #13 (Static analysis).

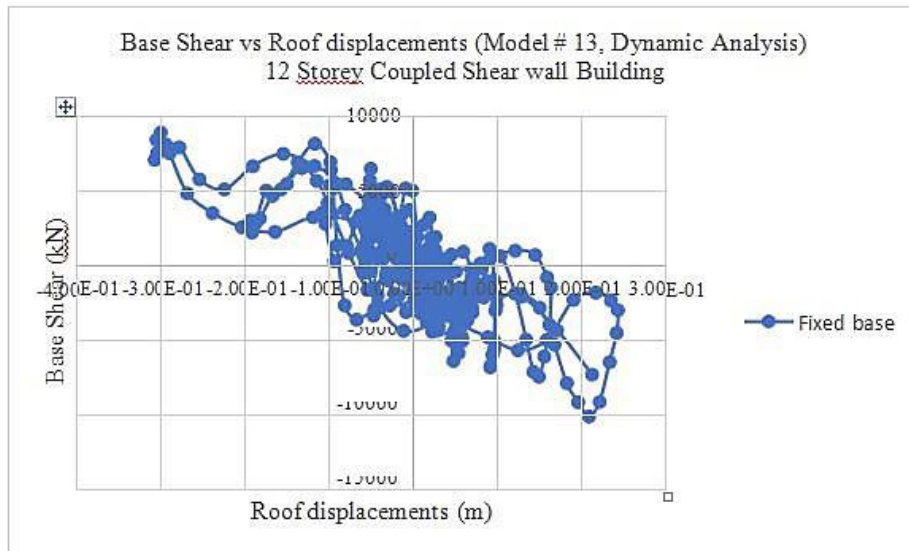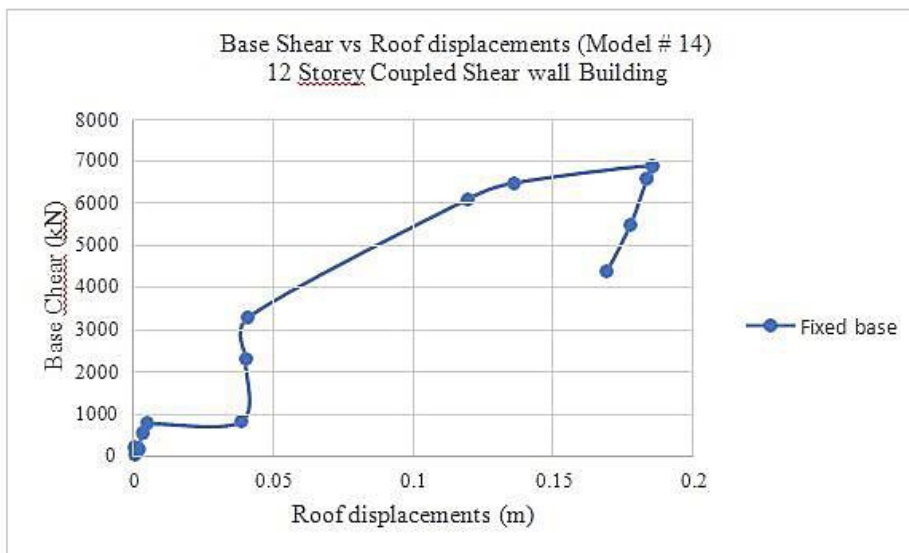**Figure 26.** Model # 13 (Dynamic analysis).
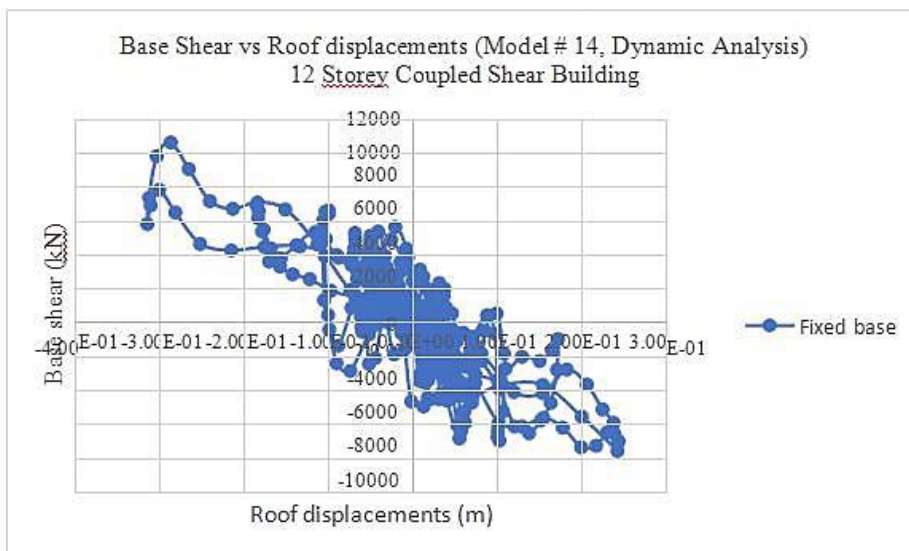

Figure 27. Model #14 (Static analysis).

Figure 28. Model # 14 (Dynamic analysis).

Combining equations 1-5 and rearranging to in order to solve curvature ductility demand the following equation can be found that is a function of displacement ductility demand $\mu_{\Delta d}$

$$\mu\phi d = 1 + \frac{(\mu\Delta d - 1)0.28hw2}{lp(hw - 0.5lp)} \quad \text{Equation \# 07}$$

Calculation of displacement ductility demand ($\mu\Delta_d$)

It has been considered the bi-linear relationship approach as shown in below Figure 29, Bhunia D, et al. [12] that was prepared based on the concept given
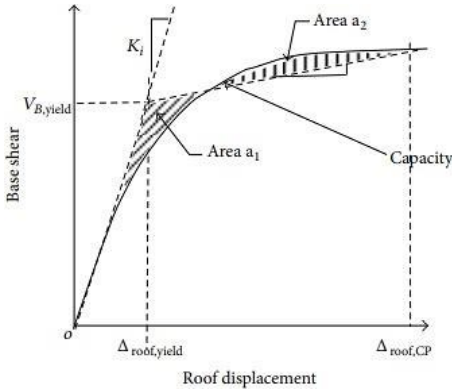

Figure 29. Bilinear representation of the capacity curve (Bhunia D, et al.) [12].

in ATC 40; It is observed from the below Figure that the bilinear relationship approach is considered due to the initial tangent stiffness and equal energies concept, i.e., Area a_1 equal to Area a_2;

Hence, the ductility of the coupled shear can be calculated:

$$\mu_{\Delta d} = \frac{\Delta roof, CP}{\Delta roof, yield}$$

$\mu_{\Delta d}$ has been calculated from 14 different model and populated in Table 13. It has been considered fixed base boundary condition for modelling of all 14 different cantilever coupled shear walls.

Curvature ductility demands $\mu\phi_d$ are calculated with 14 different models considering different equations for plastic hinge length (lp) with respect to ACI-318, 19, CSA A23.3-04, CSA A23.3- 14 and authors formula and populated in Table 14.

Total curvature demands are calculated using the equation # 2,

$$\phi_d = \mu_{\phi d} * \phi_y$$

It has been populated in Table 15.

Calculation of curvature capacity

In a ductile shear wall (R/γ=4) the maximum compression zone length, c is permitted as per CSA A23.3-14 is equal to 0.14Lw, where Lw is length of the shear wall. It has been calculated the curvature capacity considering this maximum compression zone length criteria and populated in Tables 16 and 17.

Table 13. Displacement ductility demands ($\mu\Delta d$) of 14 different models of coupled shear walls.

Ductility Chart	
	Fixed Base
Model # 1	2.5
Model # 2	2.67
Model # 3	1.89
Model # 4	2.33
Model # 5	2.5
Model # 6	1.4
Model # 7	1.83
Model # 8	3.71
Model # 09	2.78
Model # 10	2.08
Model # 11	1.67
Model # 12	1.75
Model # 13	2.4
Model # 14	1.75

Table 14. Calculations of curvature ductility demand with different building codes and comparison with author's formula.

	μΔd	Lw (m)	Lp=1.0*Lw	Lp=1.5*Lw	Lp=2.0*Lw	Lp=0.5*Lw+0.1*hw	hw (m)	ACI-318 2019 μɸd (Lp=1.0*Lw)	CSA A23.3 2004 μɸd (Lp=1.5*Lw)	Author's Formula μɸd (Lp=2.0*Lw)	CSA A23.3 2014 μɸd (Lp=0.5*Lw +0.1*hw)
Model #01	2.5	2.5	2.5	3.75	5	6.82	43.2	8.473857	6.057928	4.851699	3.888408
Model #02	2.67	2.5	2.5	3.75	5	6.82	43.2	9.320895	6.63116	5.288225	4.21576
Model #03	1.89	2.5	2.5	3.75	5	6.82	43.2	5.434489	4.001037	3.285342	2.713788
Model #04	2.33	2.5	2.5	3.75	5	6.82	43.2	7.62682	5.484696	4.415173	3.561055
Model #05	2.5	2.5	2.5	3.75	5	6.82	43.2	8.473857	6.057928	4.851699	3.888408
Model #06	1.4	2.5	2.5	3.75	5	6.82	43.2	2.993029	2.348781	2.02712	1.770242
Model #07	1.83	2.5	2.5	3.75	5	6.82	43.2	5.135534	3.79872	3.131274	2.598252
Model #08	3.71	3	3	4.5	6	7.32	43.2	12.31977	8.684726	6.871073	5.892683
Model #09	2.78	3	3	4.5	6	7.32	43.2	8.435124	6.047532	4.856277	4.213644
Model #10	2.08	2.5	2.5	3.75	5	6.82	43.2	6.381177	4.641708	3.773223	3.079653
Model #11	1.67	2.5	2.5	3.75	5	6.82	43.2	4.338323	3.259208	2.720426	2.290155
Model #12	1.75	2.5	2.5	3.75	5	6.82	43.2	4.736929	3.528964	2.92585	2.444204
Model #13	2.4	2.5	2.5	3.75	5	6.82	43.2	7.9756	5.720733	4.594919	3.695847
Model #14	1.75	2.5	2.5	3.75	5	6.82	43.2	4.736929	3.528964	2.92585	2.444204

In Table 16, it is considered plastic hinge length equation as per CSA A23.3-04 and in Table 17, it is considered the plastic hinge length equation as per CSA A23.3-14.

It is checked also at yielding consideration by considering c = 0.25*Lw, since c = 0.14* Lw is the maximum limit for ductile shear wall as per CSA A23.3-14, c =0.25*Lw is the reasonable assumption when the first reinforcement will be yielding. This curvature capacity are populated in Table 18. Basically, this is representing the yield curvature.

Research Findings

(1) Conventional reinforcements in coupling beams: In this research all the coupling beams are modelled with conventional reinforcements instead of diagonal reinforcements as recommended in CSA A23.4 04. It is observed on the selected coupling beams in this research that rotational demands are within the limit of the rotational capacities.

(2) It is observed by comparing curvature demands, those are shown in Table # 15 with the curvature capacities, those are populated in Table # 16 to 21. The followings observations are made. In a ductile (R/r$_w$ = 4).

shear wall the compression zone length maximum limit assumption (c = 0.14*Lw) as per Canadian code CSA A23.3 04 is not working in coupled shear wall in both of the cases where plastic hinge length (L$_p$) is considered individual length of the compression or tension wall as per (CSA A23.3 04) [13] and also where the plastic hinge length is considered overall length of the couple shear wall as per (CSA A23.3 14) [14];

(3) Coupled shear wall design approach: Considering ductility approach calculations in coupled shear wall design, it can be commented that separate formula needs to be developed to calculate rotational demand vs rotational capacity and curvature demand vs curvature capacity and also the plastic hinge length for cantilever shear wall and coupled shear wall both.

(4) It is observed from Table # 19 that the maximum compression zone length (c = 0.11* L$_w$) formula working as per CSA A23.3 04, where the plastic hinge length (L$_p$) calculation allows to consider individual length of the walls but it does not work as per CSA A23.3 14, where the calculation of plastic hinge length allows to consider overall length of the coupled shear wall.

(5) It is observed the curvature capacities from Table # 20 and 21 and comparing with the curvature demand Table # 15, the maximum compression zone length (c = 0.09*L$_w$) works for all three codes CSA A23.3 04, CSA A23.3

Table 15. Calculation of total curvature demand for 14 different models of couple shear wall buildings.

		Total Curvature Demand			
		Yield curvature (φy).	φd =μφd*φy		
		0.004/Lw	μφd (Lp=1.0*Lw)	μφd (Lp=1.5*Lw)	μφd (Lp=2.0*Lw)	μφd (Lp=0.5*Lw +0.1*hw)
	Lw (m)		ACI-318-209	CSA A23.3 2004	Author's Formula	CSA A23.3 2014
Model #01	2.5	0.0016	0.013558172	0.009692685	0.007762719	0.006221452
Model #02	2.5	0.0016	0.014913431	0.010609856	0.00846116	0.006745217
Model #03	2.5	0.0016	0.008695182	0.00640166	0.005256547	0.004342062
Model #04	2.5	0.0016	0.012202912	0.008775514	0.007064277	0.005697687
Model #05	2.5	0.0016	0.013558172	0.009692685	0.007762719	0.006221452
Model #06	2.5	0.0016	0.004788846	0.003758049	0.003243392	0.002832387
Model #07	2.5	0.0016	0.008216855	0.006077952	0.005010038	0.004157203
Model #08	3	0.001333	0.016426357	0.011579635	0.009161431	0.007856911
Model #09	3	0.001333	0.011246832	0.008063376	0.006475036	0.005618192
Model #10	2.5	0.0016	0.010209884	0.007426733	0.006037158	0.004927445
Model #11	2.5	0.0016	0.006941317	0.005214733	0.004352681	0.003664249
Model #12	2.5	0.0016	0.007579086	0.005646343	0.004681359	0.003910726
Model #13	2.5	0.0016	0.01276096	0.009153173	0.007351871	0.005913355
Model #14	2.5	0.0016	0.007579086	0.005646343	0.004681359	0.003910726

Table 16. Calculation of curvature capacity (Lp as per CSA A23.3-04).

Calculation of Curvature Capacity Φ$_c$ =0.0035/c			
Considering the Maximum Limit for Ductile Wall (CSA-A23.3 2004) c= 0.14*Lw			
	Lw (m)	c (m)	φc
Model #01	2.5	0.35	0.01
Model #02	2.5	0.35	0.01
Model #03	2.5	0.35	0.01
Model #04	2.5	0.35	0.01
Model #05	2.5	0.35	0.01
Model #06	2.5	0.35	0.01
Model #07	2.5	0.35	0.01
Model #08	2.5	0.35	0.01
Model #09	3	0.42	0.0083333
Model #10	3	0.42	0.0083333
Model #11	2.5	0.35	0.01
Model #12	2.5	0.35	0.01
Model #13	2.5	0.35	0.01
Model #14	2.5	0.35	0.01

Note: Lw = Length of the longest coupled shear wall

Table 17. Calculation of curvature capacity (Lp as per CSA A23.3-14).

Calculation of Curvature Capacity ϕ_c =0.0035/c

Considering the Maximum Limit for Ductile Wall (CSA-A23.3 2014) c= 0.14*Lw

	Lw (m)	c (m)	φc
Model #01	2.5	0.7	0.005
Model #02	2.5	0.7	0.005
Model #03	2.5	0.7	0.005
Model #04	2.5	0.7	0.005
Model #05	2.5	0.7	0.005
Model #06	2.5	0.7	0.005
Model #07	2.5	0.7	0.005
Model #08	2.5	0.7	0.005
Model #09	3	0.84	0.004167
Model #10	3	0.84	0.004167
Model #11	2.5	0.7	0.005
Model #12	2.5	0.7	0.005
Model #13	2.5	0.7	0.005
Model #14	2.5	0.7	0.005

Note: Lw = Overall length of the coupled shear wall.

Table 18. Calculation of curvature capacity at yielding.

Calculation of Curvature Capacity. ϕ_c =0.0035/c

Considering the Maximum Limit for Ductile Wall (CSA-A23.3 2004) c= 0.25*Lw

	Lw (m)	c (m)	φc
Model #01	2.5	0.625	0.0056
Model #02	2.5	0.625	0.0056
Model #03	2.5	0.625	0.0056
Model #04	2.5	0.625	0.0056
Model #05	2.5	0.625	0.0056
Model #06	2.5	0.625	0.0056
Model #07	2.5	0.625	0.0056
Model #08	2.5	0.625	0.0056
Model #09	3	0.75	0.0046667
Model #10	3	0.75	0.0046667
Model #11	2.5	0.625	0.0056
Model #12	2.5	0.625	0.0056
Model #13	2.5	0.625	0.0056
Model #14	2.5	0.625	0.0056

Table 19. Calculation of curvature capacity (Lp as per CSA A23.3-04).

Calculation of Curvature Capacity. ϕ_c =0.0035/c

Considering the Maximum Limit for Ductile Wall (CSA-A23.3 2004) c= 0.11* Lw

	Lw (m)	c (m)	φc
Model #01	2.5	0.275	0.012727273
Model #02	2.5	0.275	0.012727273
Model #03	2.5	0.275	0.012727273
Model #04	2.5	0.275	0.012727273
Model #05	2.5	0.275	0.012727273
Model #06	2.5	0.275	0.012727273
Model #07	2.5	0.275	0.012727273
Model #08	2.5	0.275	0.012727273
Model #09	3	0.33	0.010606061
Model #10	3	0.33	0.010606061
Model #11	2.5	0.275	0.012727273
Model #12	2.5	0.275	0.012727273
Model #13	2.5	0.275	0.012727273
Model #14	2.5	0.275	0.012727273

Table 20. Calculation of curvature capacity (Lp as per CSA A23.3-14).

Calculation of Curvature Capacity. $\phi_c = 0.0035/c$			
Considering the Maximum Limit for Ductile Wall (CSA-A23.3 2014) c= 0.09* Lw			
	Lw (m)	c (m)	ϕc
Model #01	2.5	0.45	0.007777778
Model #02	2.5	0.45	0.007777778
Model #03	2.5	0.45	0.007777778
Model #04	2.5	0.45	0.007777778
Model #05	2.5	0.45	0.007777778
Model #06	2.5	0.45	0.007777778
Model #07	2.5	0.45	0.007777778
Model #08	2.5	0.45	0.007777778
Model #09	3	0.54	0.006481481
Model #10	3	0.54	0.006481481
Model #11	2.5	0.45	0.007777778
Model #12	2.5	0.45	0.007777778
Model #13	2.5	0.45	0.007777778
Model #14	2.5	0.45	0.007777778

Table 21. Calculation of curvature capacity (Lp as per CSA A23.3-04).

Calculation of Curvature Capacity. $\phi_c = 0.0035/c$			
Considering the Maximum Limit for Ductile Wall (CSA-A23.3 2014) c= 0.09* Lw			
	Lw (m)	c (m)	ϕc
Model #01	2.5	0.225	0.015555556
Model #02	2.5	0.225	0.015555556
Model #03	2.5	0.225	0.015555556
Model #04	2.5	0.225	0.015555556
Model #05	2.5	0.225	0.015555556
Model #06	2.5	0.225	0.015555556
Model #07	2.5	0.225	0.015555556
Model #08	2.5	0.225	0.015555556
Model #10	3	0.27	0.012962963
Model #11	3	0.27	0.012962963
Model #12	2.5	0.225	0.015555556
Model #13	2.5	0.225	0.015555556
Model #14	2.5	0.225	0.015555556
Model #15	2.5	0.225	0.015555556

14 and ACI-318 19, where it has been considered the plastic hinge length (L_p) calculations as per the corresponding building codes.

Conclusion

It can be commented that it is not properly justified to model and treat the coupled shear wall as a solid cantilever wall with opening since mass and stiffness distribution are not same for cantilever and coupled shear walls. In order to get accurate prediction of failure mechanism of coupled shear wall and coupling beams, it is very important to model and analyze it as a ductility approach. In this research it was investigated with non-linear dynamic analysis of 14 different models of a twelve-story coupled shear wall buildings as per CSA A23.3 14 except the coupling beams are modeled with conventional reinforcements. Study shows that the maximum flexural compression zone length without confinement reinforcement as per (CSA A23.3 1994) is not working with slender coupled shear walls (tension or compression wall), while considering the plastic hinge length as per CSA-A23.3 04 or CSA-A23.3 14; Separate formula is proposed for the maximum compression zone length limitation by analyzing the coupled shear wall in ductility approach and investigation shows that the proposed formula represents the accurate behavior of the coupled shear wall maximum compression zone length limitation that works while the plastic hinge length calculations done by CSA A23.3 04, CSA A23.3 14 and ACI 318 19. Since, wall overstrength (γ_w) has little influence on

the maximum flexural compression zone length of the coupled shear wall, it has not been investigated its influence on this research.

As this research is based on seismicity of Vancouver and National Building Code of Canada on the coupled shear wall, similar work can be performed considering different seismicity and other National Building codes.

Acknowledgement

Conflict of Interest

The authors declare that there is no conflict of interest.

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