

STUDY ON THE INFLUENCE OF COUPLING BEAM DEPTH FOR COUPLED SHEAR WALL SYSTEMS

Radu VASILESCU -Assistant, PhD Student, Technical University of Civil Engineering, Faculty of Civil Engineering, Bucharest, Romania, e-mail: radu.vasilescu@phd.utcb.ro

Abstract: The geometrical dimensions of coupling beams can strongly alter the overall behavior of coupled shear walls systems. This study examines the differences in structural response of two reinforced concrete coupled shear walls systems with different coupling beam depths. Nonlinear static pushover analysis was conducted on both models, carried out up to a target overall rotation of 2%. The results are compared in terms of peak lateral force, energy dissipation and strength degradation. The deeper beam system proved stiffer and able to dissipate more energy than its counterpart. The shallower beam system offered greater deformation capacity. An important finding is represented by the shift of the strength degradation mechanism: the strong coupling effect generated by the deep coupling beams led to the eventual crushing of concrete in the compressed pier, while the shallower beams began to yield, causing a degradation of the overall coupling moment.

Keywords: nonlinear layered shell, pushover, nonlinear analysis, coupling effect.

1. Introduction

High-rise buildings in seismically active areas require a strong lateral force-resisting system in order to meet strength and rigidity demands. Reinforced concrete shear walls are proven to be one of the most effective such systems, in cases where the architectural layout permits them. Architectural requirements may sometimes make the use of normal shear walls impossible, as they can affect building functionality. In such cases, the shear walls needed to withstand lateral forces can be turned into a coupled shear wall system, which satisfies functionality demands while providing lateral force resistance. Coupled shear walls usually consist of two reinforced concrete wall piers interconnected with one or more coupling beams. The flexural response of the individual piers and the coupled axial force generated by the coupling beams' shears come together to resist the total overturning moment which the system is subjected to. Since the coupling effect plays an important role in the capacity of the structural system, the dimensions of the coupling beams can greatly alter the overall structural response. This variation in response based on the coupling ratio makes the object of this paper.

Massone *et al.*[1] study the structural response of reinforced concrete coupled shear walls subjected to cyclic loading, focusing on the distribution of cross-sectional curvature along the height of each wall pier. To this effect, four 1:35 scale specimens designed to replicate wall systems of 15 and 25 stories Chilean residential buildings were constructed and subjected to cyclic lateral loading. OpenSees fiber models are developed and validated against the experimental data. These validated models are further used to conduct a parametric study over 24 cases, covering different numbers of stories, varying wall lengths and coupling ratios. Their results show that shorter coupled shear wall systems exhibited a higher curvature demand, than their taller counterparts, due to the effect of the coupling ratio – which is defined as the proportion of the total overturning moment of the structural system resisted by the coupling action created by the shear force in the coupling beams.

Zhang *et al.*[2] present a numerical study on reinforced concrete coupled shear wall systems with the intent to address the gap between experimental studies on individual coupling beams and the response of coupling beam within coupled wall systems. A two-dimensional finite element model is created based on a fixed crack constitutive model and is validated against twelve coupling beam tests. To further verify the accuracy of the model, three more coupled shear wall specimens are simulated. Based on these findings, a parametric study of over twenty coupled shear wall specimens with varying coupling ratios and diagonal reinforcement ratios is conducted. The results showed that coupling beams in coupled shear walls systems with a coupling ratio greater than 66% exhibit significantly smaller maximum rotations than beams in systems with lesser coupling ratios.

In their paper, Mortazavi and Zaeimdar [3] present an investigation into the accuracy of macro-models, by creating two such models for a reinforced concrete wall with asymmetric openings and validating the results against a microscopic FEM model of the same wall. The wall macro-models are created using a series of vertical uniaxial spring elements connected between the beams to simulate flexural behavior and a horizontal spring placed at the rotation center to simulate shear, an approach which requires less computational time than the finite element model. The difference between the two models is in the rigidity of the connection between the coupling beam and the wall pier, as the first model assumes a fully rigid connection, while the second one allows relative vertical displacements at the connection point. Their findings show that the use of macro-models can reduce the computation time by a significant amount, but the produced results of two almost identical macro-models can vary greatly based on the stiffness of the beam-wall connection.

Papanikolaou *et al.*[4] present a nonlinear modelling approach, in the commercially available RAF software, using a multilayer shell element based on the MITC4 isoparametric elements, assigning each layer their own independent concrete or steel constitutive models. Their layer formulations were implemented as subroutines in the FORTRAN compiled programming language, with plans to map the section material model to MITC4 finite elements, as primary validation tests proved satisfactory.

In their study, Lu and Chen [5] propose a nonlinear macro-model for coupled shear wall systems, which consists of a multiple-vertical-line-element model for the wall piers, composed of multiple axial springs and a singular shear spring, and a composed model for the coupling beams which consists of three sub-elements: spread plastic, interface bond-slip and shear. Their model is validated against three 1:4 scale five-story specimens, differing only in coupling beam depth. Their experiment shows that the failure mode of the coupled shear walls system is influenced by the coupling beams' dimensions, as a transition from flexural to shear failure with the increase of the beams' depth is highlighted.

Several modelling approaches, ranging from multiple-vertical-line-element macro-models to fiber-based models, have been used in the literature presented above to investigate the nonlinear behavior of coupled shear walls systems. These approaches require specialized research software and substantial modelling proficiency that cannot always be readily available in standard structural design practice. Thus, validation of the use of a straightforward modelling approach in a commercially available structural analysis software, which is easy to integrate in a typical design workflow, can provide a practical tool for assessing nonlinear behavior of coupled shear wall systems. This paper presents the process of modelling two coupled shear walls systems, with different coupling beam depths, in the finite element analysis software ETABS, using only shell layered elements, and compares the results following a pushover analysis

on both models, to validate if the chosen modelling approach is able to capture the coupling effect and its influence on the overall behavior of the structural system.

2. The Coupled Shear Walls Systems

As stated before, the dimensions of the coupling beams can greatly impact the response of coupled shear wall systems, as any change in depth will affect the coupling ratio, defined earlier as proportion of the total overturning moment resisted by the coupling action, of the two piers and thus alter one of the main components of the combined structural response. For the purposes of this study, two different coupling shear walls systems were created, both with identical wall piers but different coupling beam depths.

The coupled shear walls systems have a height of 31.50 m and a length of 16.00 m. The individual wall piers have a length of 6.95 m and are 40 cm thick. They are interconnected with nine coupling beams along their height, geometrically similar to the structural layout of a typical nine stories building. The coupling beams have the same thickness as the wall piers and their span is 210 cm. The beam depth for each coupled shear walls system is as follows: 95 cm and 210 cm. From this point forward, the coupled shear walls systems will be referred to by their clear span (l_{cl}) over the beam depth (h) ratio. Thus, the two systems become: $l_{cl}/h = 2.20$ and $l_{cl}/h = 1.00$. To differentiate between the wall piers of each system, the wall piers will be identified as P1 and P2. The geometry of each coupled shear wall system can be seen in Fig.1.

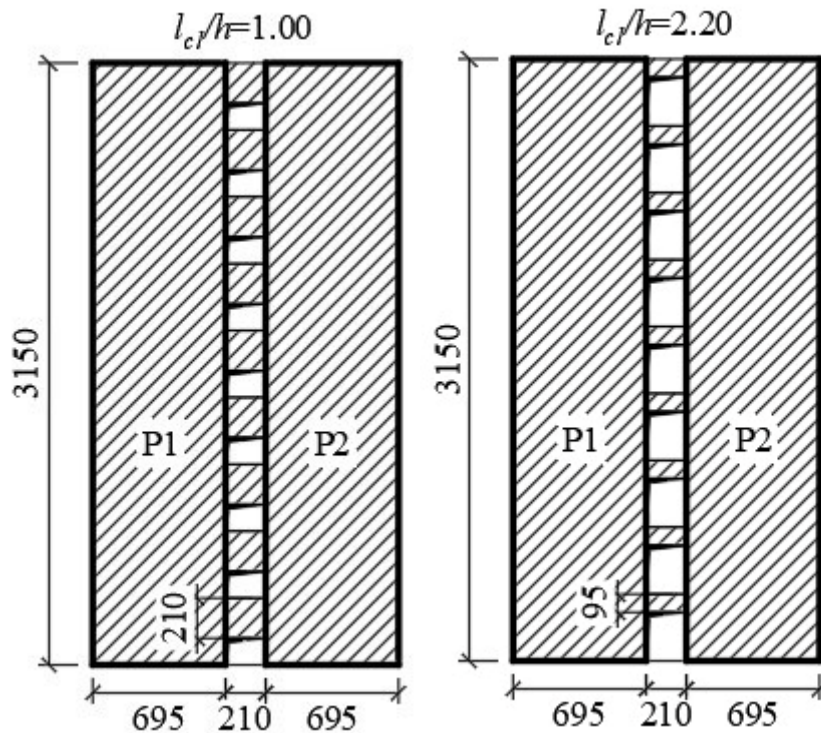


Fig. 1 – Dimensions of the two coupled shear walls systems

The materials considered for all coupled walls systems were concrete grade C30/37 and reinforcement bars BST500C. The longitudinal reinforcement area in both the wall piers and coupling beams was calculated using the minimum rebar ratios in CR2-1-1.1/2022 [6]. Thus, for the wall piers the longitudinal reinforcement ratio chosen was $\rho_l = 0.005$, while for the coupling beams the ratio was $\rho_l = 0.0025$. The coupling beam

transversal reinforcement was calculated using the minimum ductility requirements, while it was not considered for the wall pier since its behavior is flexure controlled.

For the vertical load, a 12 kN/m² dead load was considered, typical to what would be found in a residential building, with a tributary area of 128 m² (equivalent to an 8 m long bay).

3. The Finite Element Analysis Model

A two-dimensional finite element analysis model was constructed for each coupled shear walls system in the commercially available software ETABS. Shell layered finite elements were used to model both the wall piers and coupling beams. The shell layered section consists of multiple layers which represent the concrete and steel materials found within the element. Each layer has its own individual thickness and material stress-strain curve. The thickness of the concrete layers is 400 mm, equal to the thickness of the element, while the reinforcement layer's thickness is calculated as the smeared reinforcement area along the length of the element. No transversal reinforcement was considered for the wall piers as their behavior was flexure controlled. Two different shell-layered sections were created, one for the wall piers and one for the coupling beams. The configuration of each shell layered section can be seen in Tables 1-2. The coupling beam section has an additional layer, compared to its wall pier counterpart, to represent the transversal reinforcement, as both flexure and shear control the beam response, while the wall pier is primarily flexure controlled.

Table 1 - Shell-layered configuration for the wall piers

Layer	Distance (mm)	Thickness (mm)	Modelling type	Material	Material Angle(°)	Material Behavior	Material S11	Material S22	Material S12
Concrete	0	400	Membrane	C30/37	0	Directional	Linear	Nonlinear	Linear
Rebar	0	2	Membrane	BST500C	90	Directional	Nonlinear	Inactive	Linear

Table 2 - Shell-layered configuration for the coupling beams

Layer	Distance (mm)	Thickness (mm)	Modelling type	Material	Material Angle(°)	Material Behavior	Material S11	Material S22	Material S12
Concrete	0	400	Membrane	C30/37	0	Directional	Nonlinear	Nonlinear	Linear
Rebar	0	1	Membrane	BST500C	90	Directional	Nonlinear	Inactive	Linear
Rebar	0	1.6	Membrane	BST500C	0	Directional	Nonlinear	Inactive	Linear

An elastic perfectly plastic stress-strain relation was used for the BST500C reinforcement steel material, with a yield strength equal to $f_y=500$ MPa. For the grade C30/37 concrete material, the stress-strain relation for nonlinear structural analysis found in EN 1992-1-1: Eurocode 2 [7] was used, with the mean concrete compressive strength taken as $f_{cm}=38$ MPa and with modified $\epsilon_{c1}=0.006$ and $\epsilon_{cu1}=0.0175$ parameters to account for the confinement which would be provided by the typical

transversal reinforcement. The stress-strain curves for both materials can be seen in Fig. 2-3.

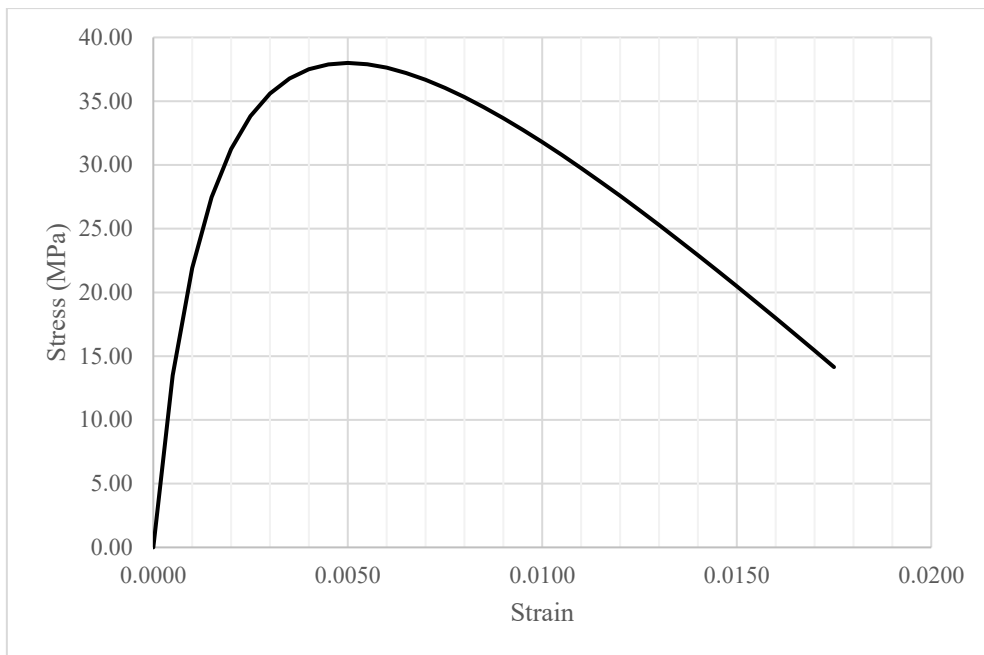


Fig. 2 – The stress-strain curve of the concrete material.

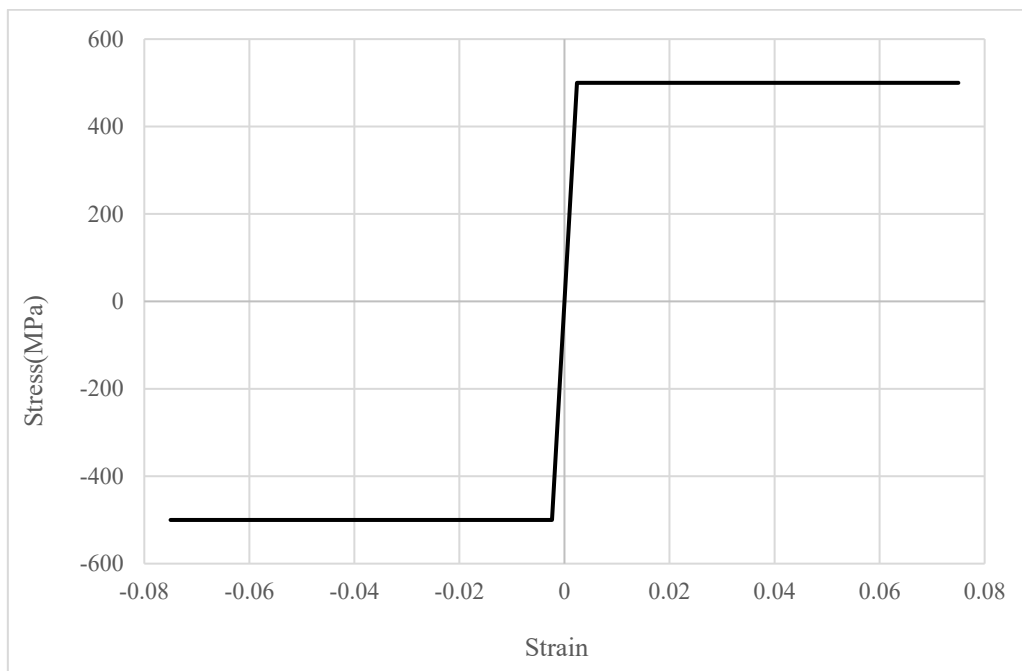


Fig. 3 – The stress-strain curve of the reinforcement steel material.

A nonlinear static pushover analysis, in the model's positive X direction, was conducted on both coupled shear walls systems models. The models were pushed until the top target displacement reached a value corresponding to an overall rotation equal to 2%. The results are presented in the next chapter, while the finite element models for both coupled shear walls system can be seen in Fig.4.

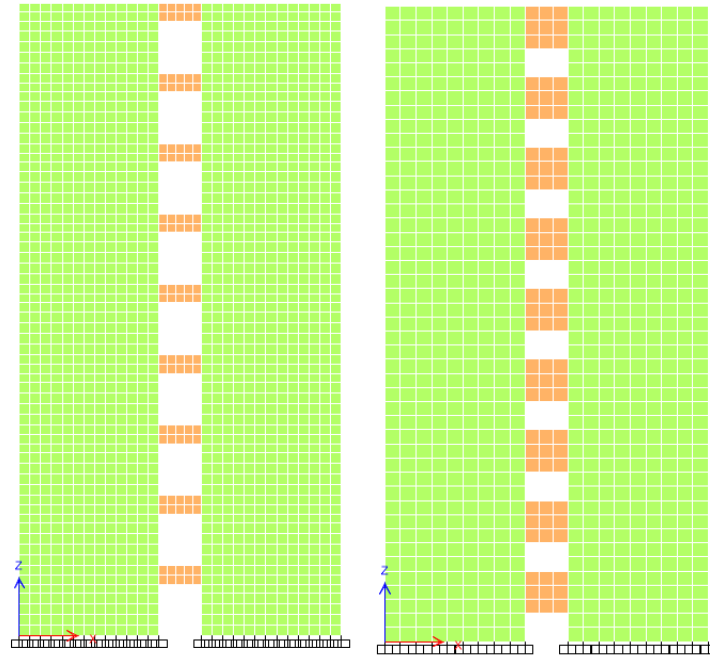


Fig. 4 – The finite element models of the two coupled shear walls systems.

4. Numerical Results

After the nonlinear static analysis was conducted, the two pushover curves were obtained. A comparison between the two curves, plotted as base shear force against overall rotation, can be seen in Fig.5.

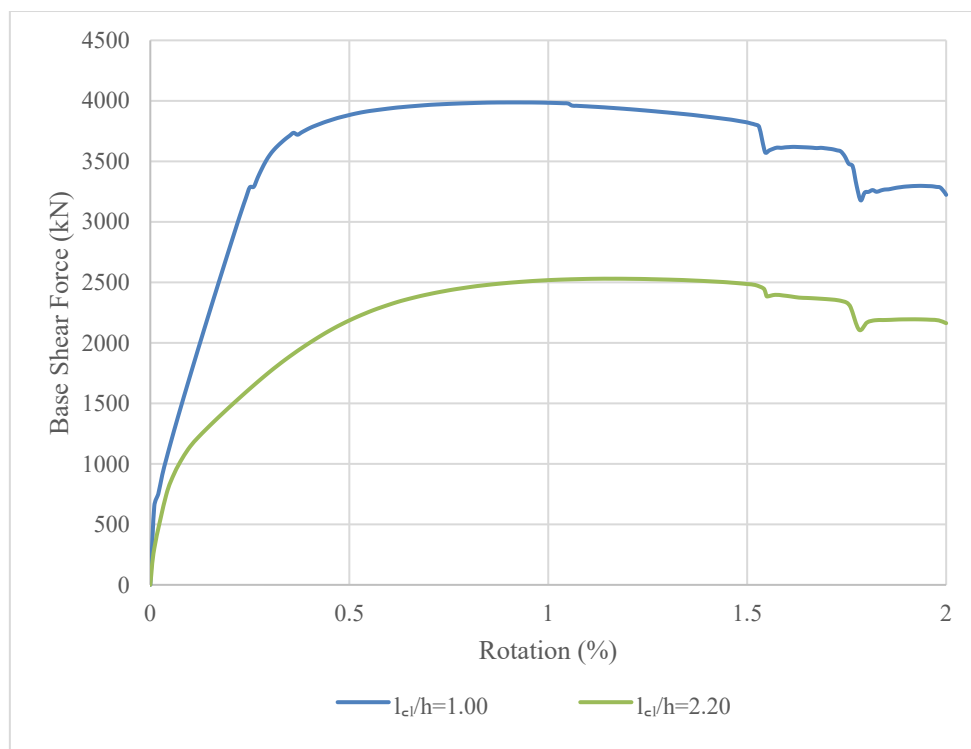


Fig. 5 – Resulting pushover curves.

The analysis reached the target displacement of 630 mm (value which corresponds to an overall rotation of 2%) for both models.

As expected, the stiffest lateral response was obtained by the $l_{cl}/h = 1.00$ model, for which the deep coupling beam acts as a stronger and more rigid link between the two wall piers, while the other coupled walls system exhibited a more flexible response. Computing the elastic stiffness of the two models, by determining the slope of the line from the origin to the yield point on the pushover curve, reveals that the deep beam model response is 1.97 times stiffer than its shallower counterpart. All models experience a plateau after the base shear force reaches its peak, due to the longitudinal reinforcement layer in the wall piers yielding, and a sudden strength degradation at the end of the plateau. The strength degradation is caused by the crushing of the compressed concrete for both models, while the shallow beam model also experiences yielding of the coupling beams. These phenomena begin at an overall rotation of 1.53% for the $l_{cl}/h = 1.00$ model and 1.55% for the $l_{cl}/h = 2.20$ model.

Another deformability parameter that highlights the post-elastic behavior of the coupled shear walls systems is the yield overall rotation (θ_y), which is the measured overall rotation of the system when the recorded lateral force begins to plateau. The highest yield rotation was recorded for the $l_{cl}/h = 2.20$ model, with the $l_{cl}/h = 1.00$ model coming in second. As the more flexible of the two models, the $l_{cl}/h = 2.20$ coupled shear walls system reaches its yield rotation later than its more rigid analogue.

Similar to the stiffness response, the $l_{cl}/h = 1.00$ model reaches the highest peak lateral force value (V_{max}) of 3987.10 kN at a displacement equal to 289.8 mm (0.92% overall rotation), while the $l_{cl}/h = 2.20$ model exhibits a lower recorded peak base shear force of 2530.20 kN at a displacement equal to 359.15 mm (1.14% overall rotation), 36.5% less than its counterpart. Both values are reached after the yield rotation of the model because, despite the fact that the lateral force begins to plateau, small variations of its value occur.

Integrating the area under each pushover curve gives us the total energy dissipated during the nonlinear analysis. The deep beam model dissipates the highest amount of energy at 2230.7 kJ, while the shallow beam model dissipates 1378.2 kJ. To create a meaningful comparison between the two models, the total dissipated energy can be normalized by the recorded peak lateral force times its associated displacement. Thus, a 1.93 normalized energy ratio is obtained for the deep beam model, while for the shallow beam model a 1.51 ratio is obtained, approximately 21.4% lower.

All calculated parameters can be seen in Table 3.

Table 3 - Resulting stiffness, energy dissipation, strengths and deformability of models

l_{cl}/h	Elastic stiffness (kN/mm)	E (kJ)	V_{max} (kN)	θ_y (%)	Normalized Energy Ratio
1.00	15.80	2230.70	3987.10	0.80	1.93
2.20	7.99	1378.20	2530.20	1.00	1.51

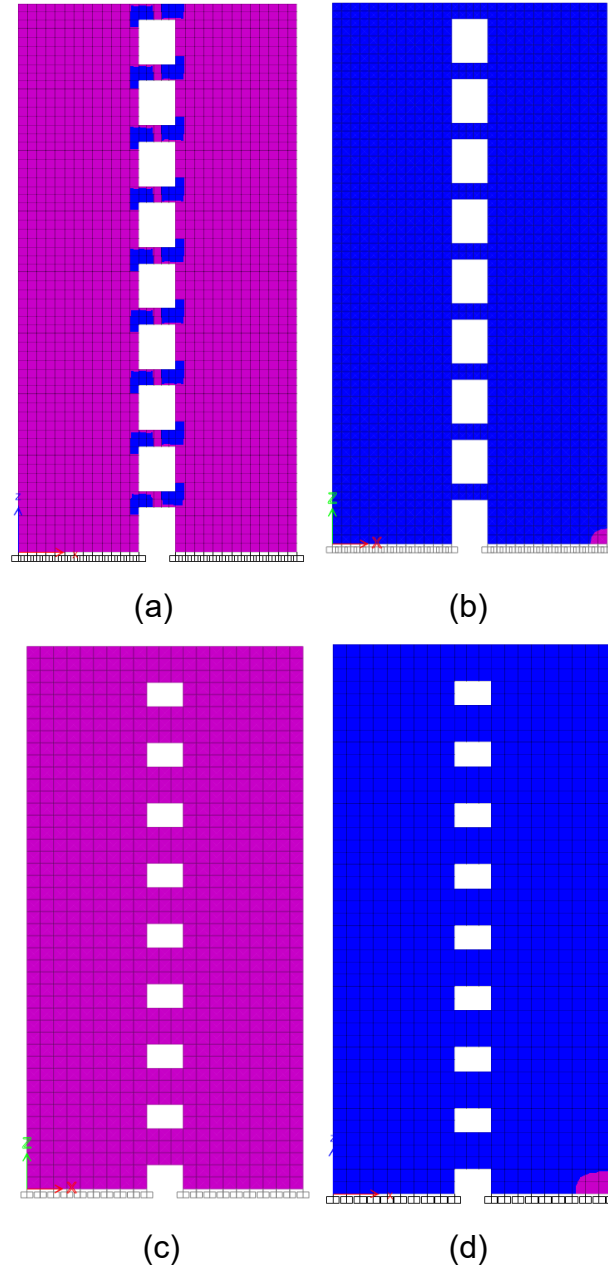


Fig. 6 – (a) Longitudinal strains past yield strain in the coupling beam steel layer for the $l_c/h = 2.20$ model.

– (b) Longitudinal strains past peak concrete stress strain in the wall pier concrete layer for the $l_c/h = 2.20$ model.

– (c) Longitudinal strains past yield strain in the coupling beam steel layer for the $l_c/h = 1.00$ model.

– (d) Longitudinal strains past peak concrete stress strain in the wall pier concrete layer for the $l_c/h = 1.00$ model.

Strain results at the onset of strength degradation can reveal the failure mechanisms of each model. As seen in Fig.6 (a), the coupling beams of the $l_c/h = 2.20$ model begin to fully plastify when the overall rotation of the model reaches 1.55%. This inevitably leads to a decrease in the overall coupling effect created by the presence of the coupling beams, thus reducing the lateral resistance of the coupled shear wall system. Fig. 6 (b) shows that at the same time as the coupling beams fully yield, the concrete in the compressed wall pier is just beginning to crush, accelerating the strength degradation of the system. Fig.6 (d) reveals that the concrete strain in the compressed

wall pier of the $l_c/h = 1.00$ model is past the associated peak stress strain at an overall rotation of 1.53%. This leads to strength degradation by way of the compressed concrete getting crushed. The coupling beams of the deep beam model never begin to fully plastify, as can be seen in Fig. 6 (c).

Studying the effective axial force in each wall pier can supplement the understanding of the failure modes of each coupled shear wall system and the influence of the coupling effect. In Fig.7-8, the axial force in each pier plotted against the overall rotation of the model can be seen.

At the beginning of the nonlinear analysis all wall piers across both models are compressed as the pushover analysis starts after the nonlinear dead load case. As the coupling effect begins to take form, the P1 wall pier across both models starts to experience tension, while the compressive axial force in the P2 wall pier steadily increases. For the $l_c/h = 1.00$ model, as the coupling beams never plastify, the axial force in both piers plateaus after reaching its peak throughout the analysis. In the $l_c/h = 2.20$ model, as the coupling effect begins to degrade, the axial forces acting on both wall piers begin to decrease, a consequence of the yielded coupling beams.

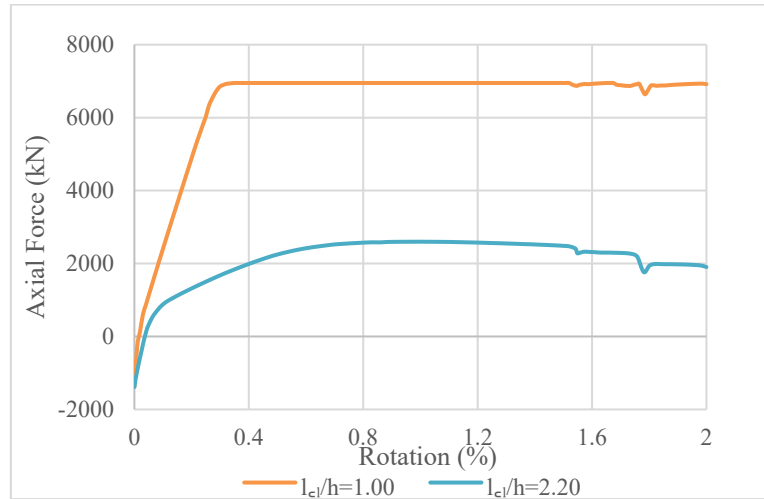


Fig. 7 - Axial force in the P1 wall pier across all models plotted against overall rotation.

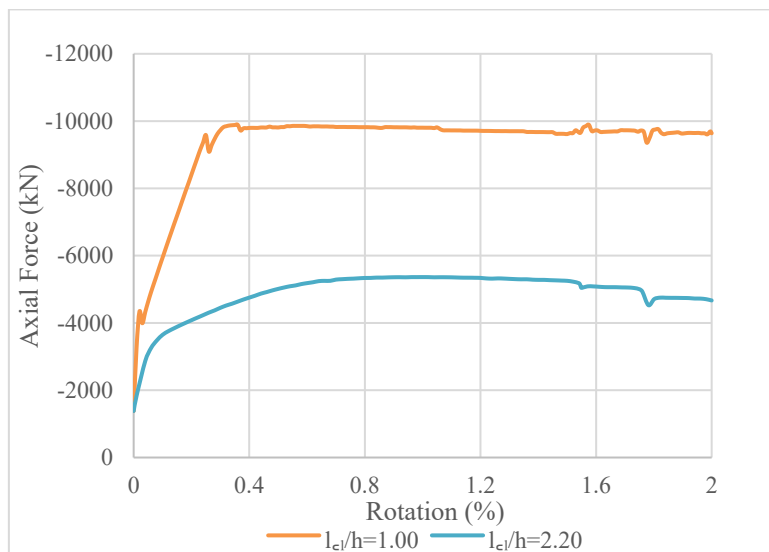


Fig. 8– Axial force in the P2 wall pier across all wall piers plotted against overall rotation.

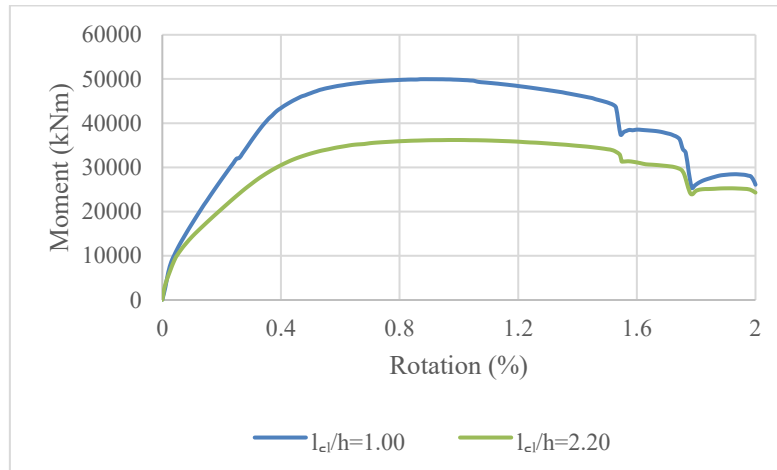


Fig. 9 - Comparison between the moments recorded at the base of the P2 wall pier for both models.

Comparing the bending moment generated at the base of the compressed piers, makes the effect of the axial compressive load generated by the coupling effect evident, as the $l_c/h = 1.00$ model pier is able to develop a higher peak value due to its stronger coupling degree granted by its deeper beams.

Taking the global overturning moment and subtracting from it the bending moments that develop at the base of the individual piers reveals the coupling effect moment that is generated by the coupled shear walls systems. Plotting it for every step of the analysis, alongside the moments recorded in the individual piers highlights the influence of the coupling effect in the overall lateral resistance of the system.

As can be seen in Fig.10 for the $l_c/h = 1.00$ model, P1 wall pier generates almost no bending moment, as it is fully tension throughout the analysis, thus having its capacity reduced. The coupling moment reaches a plateau quickly at around 0.3% overall rotation and maintains until the end of the pushover case. The P2 wall pier bending moment curve displays a shape similar to that of the model's pushover curve, with its value degrading at a 1.53% overall rotation, corresponding to the crushing of the compressed concrete.

Fig. 11 displays a completely different behavior for the $l_c/h = 2.20$ model. The coupling moment and the moment generated by the P2 wall pier follow a close path. They both start to significantly degrade at an overall rotation of 1.55%, which is when the coupling beams begin to plastify and thus the coupling effect starts to decrease. The loss of the axial compressive load generated by the coupling effect on the P2 wall pier causes the bending moment to decrease. Compared to the previous model, the P1 wall pier is able to generate moment at its base, due to the fact that the coupling effect generated by the shallower beam is not strong enough to fully suppress the wall's flexural capacity.

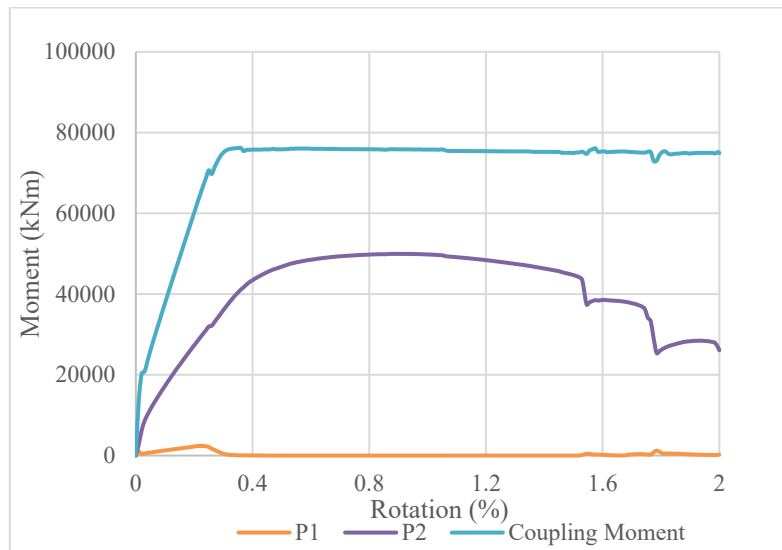


Fig. 10 – Moment plotted against overall rotation for the $l_c/h = 1.00$ model.

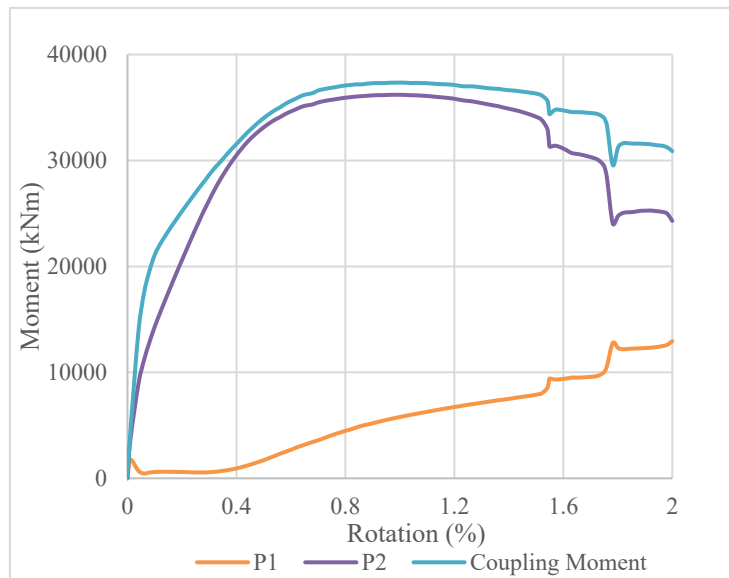


Fig. 11 - Moment plotted against overall rotation for the $l_c/h = 2.20$ model.

5. Conclusions

Two coupled shear walls system with different coupling beam depths were modelled in the commercially available finite element analysis software ETABS using layered shell elements and subjected to a pushover analysis to determine the influence of the coupling beam depths on the overall structural response.

Both models had identical wall piers, with the only difference between them being the depth of the interconnecting coupling beams. The geometrical dimensions of the coupled shear walls systems were similar to ones that could be found in residential buildings. The reinforcement ratios given by the Romanian design code CR2-1-1.1/2022 [6] were used.

The wall piers and the coupling beams were modelled with the nonlinear shell layered element, by creating separate layers for every material used, each layer having its own thickness and associated stress-strain relation. An elastic perfectly plastic relation was used for the BST500C reinforcement steel material and EN 1992-1-1: Eurocode 2 [7]

relation for nonlinear structural analysis with modified parameters to account for confinement was used for the grade C30/37 concrete.

The $l_c/h = 1.00$ model, with its deeper coupling beams, displayed a higher initial stiffness, a higher recorded peak lateral force of 3987.1 kN and a greater overall normalized energy ratio of 1.93. Compared to it, the shallower beam model with the l_c/h ratio equal to 2.20 exhibited a more flexible initial response, reached a lower peak lateral force of 2530.2 kN and had a 21.4% lower normalized energy ratio of 1.51.

The two coupled shear walls systems experienced different failure mechanisms, highlighting the importance of the coupling ratio on the overall seismic response. The $l_c/h = 2.20$ model's coupling beams start to plastify at an overall rotation equal to 1.55%, leading to a progressive degradation of the coupling effect and a decrease of the axial forces acting on the individual wall piers. The $l_c/h = 1.00$ model's coupling beams remained in the elastic domain throughout the entire analysis, maintaining a stable coupling effect and increased axial load on the compressed pier, which eventually led to the onset of strength degradation at an overall rotation equal to 1.53% caused by the crushing of the compressed concrete.

The nonlinear analysis highlighted differences in behavior between the two coupled shear walls systems. The coupling moment of the deep beam model stabilized and remained nearly constant until the concrete began to crush, while in the shallow beam model the coupling moment closely correlated with the softening of the coupling beams, gradually degrading as the beams began to yield. Overall, the deep coupling beam model showed greater lateral strength and energy dissipation capacity than its counterpart, with an unfavorable failure mechanism. While the shallower beam model was outperformed in all measured metrics, except that of yield rotation, its failure mechanism is preferred when designing ductile structures. These findings confirm that coupling beam depth greatly alters the response of coupled shear walls systems, as changing this geometrical parameter influenced the strength and overall deformation capacity of both models. Further research should investigate the use of different types of finite elements, stress-strain relations and clear span over beam depth ratios.

References

- [1] Massone L M, Inga J, Alfaro J, Rojas F, Hube M, (2026). Experimental study and numerical modeling of coupled reinforced concrete walls, *Journal of Building Engineering*, 119 <https://doi.org/10.1016/j.jobbe.2025.114767>.
- [2] Zhang Z Y, Ding R, Fan J S, Tao M-X, Nie X, (2021). Numerical study of reinforced concrete coupled shear walls based on a two-dimensional finite element model, *Engineering Structures*, 244. <https://doi.org/10.1016/j.engstruct.2021.112792>
- [3] Mortazavi S M R, Zaeimdar B, (2021). Shear wall modeling with asymmetric openings by macro elements. *Structures*, 29, pp. 899–910. <https://doi.org/10.1016/j.istruc.2020.08.049>.
- [4] Papanikolaou V K, Karakostas C Z, Morfidis K E, Baboukas E., (2021). A multilayer shell element for nonlinear analysis of R/C shear walls, *COMPADYN 2021 - Proceedings of 8th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*, M. Papadrakakis, M. Fragiadakis (eds.), Athens, 28 - 30 June 2021, pp. 4294-4301, <https://doi.org/10.7712/120121.8786.19540>.
- [5] Lu X, Chen Y, (2005). Modeling of Coupled Shear Walls and Its Experimental Verification, *Journal of Structural Engineering*, 131(1), pp. 75–84. [https://doi.org/10.1061/\(asce\)0733-9445\(2005\)131:1\(75\)](https://doi.org/10.1061/(asce)0733-9445(2005)131:1(75)).
- [6] Cod de proiectare a construcțiilor cu pereți structurali de beton armat, Indicativ CR 2-1-1.1/2022.
- [7] EN 1992-1-1: Eurocode 2: Design of Concrete Structures - Part 1-1: General Rules and Rules for Buildings.2004.