A model for shear behavior of anchors in external shear walled frames

İffet Feyza Çırak[*1], Hasan Kaplan[2], Salih Yılmaz[3], Özlem Çalışkan Değirmenci[4], Nihat Çetinkaya[2]

Research Article

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Article Info	Abstract
	Retrofit of the existing seismically deficient buildings is a common need especially in earthquake prone regions. Chemical anchors are widely used to connect existing and newly added structural elements, such as shear walls. Therefore, modelling the behaviour of anchors which transfer axial and shear forces to the added members is important for design and analyses. There is no anchor model present in the current literature accounting shear behaviour. Therefore, a new model is established using results of a comprehensive experimental study conducted at Pamukkale University Earthquake and Construction Technology Research Laboratory. In this study, mentioned shear model is tested using two- story, one-bay RC frame specimens strengthened with external shear walls. In analyses of the models, SAP 2000 software is used and nonlinear shear behaviour of anchors is represented by NLLink elements. It is concluded that, suggested anchor shear model may be used for modelling external shear wall anchor behaviour.

1. Introduction

Most of the existing reinforced concrete residential buildings in many countries are seismically deficient because the load carrying system of these buildings contains deficiencies like soft stories, nonseismic reinforcement detailing and strong beam-weak column connections [1]. Safety and prevention of total collapse of a structure are the prime requirements of buildings. In order to meet these requirements, the structure should have adequate lateral strength, lateral stiffness and sufficient ductility [2]. Heavy damage and total collapse of RC buildings after the major earthquakes in the last three decades has initiated studies on strengthening techniques [3]. In strengthening reinforced concrete structures, two rehabilitation approaches are generally considered. One is global modification, the other is local modification. In local modification, deformation capacities of components are increased to the determined limit values, so damage occurrence is delayed. But, in this approach, there is no meaningful change in lateral load capacity. Adding concrete, steel, FRP (Fiber Reinforced Polymer) and reinforced concrete jackets for columns are possible methods for this approach [4]. In global modification, a general

modification is aimed. Reinforced concrete or steel shear walls are added. In Fig. 1, the effect of these two strengthening approaches on lateral load and deformation capacity are illustrated.
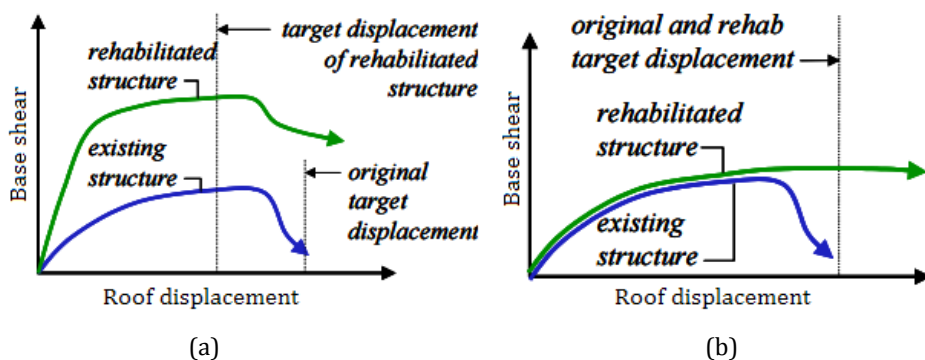

Fig. 1 Comparison of component (a) and system (b) strengthening [4]

In local modification, ductilities of components are usually increased for contributing ductility of the structural system. In global modification, load capacity of the structural system is increased by increasing the strength and stiffness of the structural system. In this way, structural system can fulfil the seismic forces with enough safety.

In global modification, adding shear walls is usually preferred. If addition of the shear wall is inside the structure, reinforced concrete shear wall is placed by removing existing infill wall. If the addition is outside of the building, shear wall can be added without removing existing infills. By rehabilitation of existing infills, the lateral load capacity of structure can be increased, as well. Research on this subject is in progress [1].

Recently, it is observed that lateral stiffness and strength of damaged and non-damaged reinforced concrete frames can be increased considerably by strengthening infill walls [5]. Besides, it was stated that infill walls can be used to control inter-story drift and out-of-plane failure [6].

It is concluded that, external shear wall application can provide system rehabilitation effectively and economically. Thus, in recent years this method is used commonly. Chemical anchors are widely used in external shear wall applications [7]. However, it is known that shear behaviour of anchors have an impact on capacity. Though, it is noticed that there is not any model concerning shear behaviour of anchors. In the referenced study [8], a model is suggested for shear behaviour of anchors by using multiple regression analysis. In this study, this shear model is used with the aim of modelling behaviour of external shear walls and compared with experimental results.

2. Material and Method

Chemical anchors are widely used to connect structural elements. They are embedded in the holes placed in hardened concrete. The diameter of the anchor hole which is drilled is at most 50% larger than that of the bar diameter. Chemical anchors have begun to be frequently used with the development of high resistance adhesives.

In the existing literature, it is observed that there are limited studies about shear behaviour of anchors. Besides that, there is not any model suggested for shear behaviour. In this study, two-story, one-bay RC frame specimens strengthened with external shear walls are modelled in SAP 2000 [9] and nonlinear static pushover analyses are performed. In analyses, anchor shear model established by using multiple regression method is used to

simulate shear behaviour of anchors [8]. Nonlinear static pushover analyses can be described as pushing until the stability of the structural system is lost. The applied lateral forces should represent behaviour of structure under lateral earthquake forces. The representative lateral load pattern is simultaneous lateral loading in experiments as pushing from floor levels. Scale of force between 1st and 2nd floor is considered as 1:2 (Fig. 2).
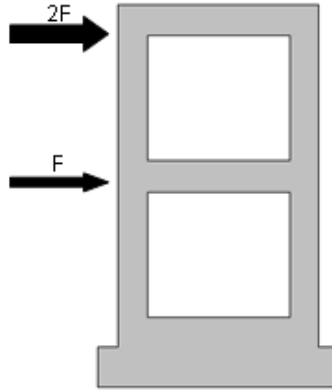

Fig. 2 Load pattern used in analyses

After determining load pattern, by controlling displacement of the roof or a certain node, lateral forces are applied monotonically step-by-step. In every step, relationship between the base shear force and the displacement of the roof is recorded. Also, it is controlled that, if predetermined hinge zones are reached their capacity. In plastic hinge zones, structural component is divided into two and hinge zone is transformed to a node. A rotation redor is defined for this node that represents rotation stiffness and analysis goes on. Analysis continues until the structure loses its stability. Thus, capacity curve (lateral load-roof displacement) is obtained (Fig. 3). In this study, multi pointed idealization is conducted with NLLink elements (nonlinear connection) rather than classic plastic hinges.
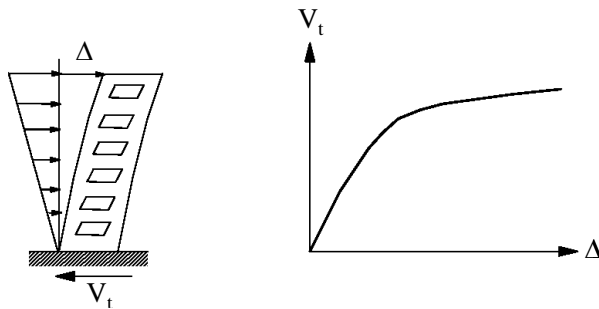

Fig. 3 Load pattern and base shear-roof displacement [10]

Suggested anchor shear model (Eqn.1) represents the shear behaviour of anchors that connect external shear wall and existing frames (Fig. 4).
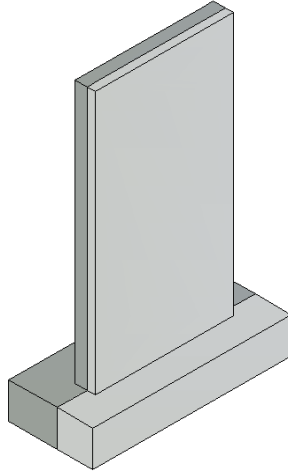
Fig. 4 Frame strengthened with external shear wall

$$V_a = 10.44 \cdot (L \cdot D)^{f_c} + 2 \cdot (D \cdot L \cdot F_y)^{0.5} - 3.55 \cdot L^{0.5} \qquad (1)$$

Here, V_a: Anchor shear load (kN); f_c: Concrete compressive strength (kN/mm²); f_y: Steel yielding strength (kN/mm²); D: Steel diameter (mm); L: Embedment depth (mm).

In this equation, anchor shear value is dependent on anchor diameter (D), embedment depth (L), compressive strength of concrete (f_c) and steel yield strength (f_y). In Fig. 5, parameters of suggested anchor shear model formula are given.
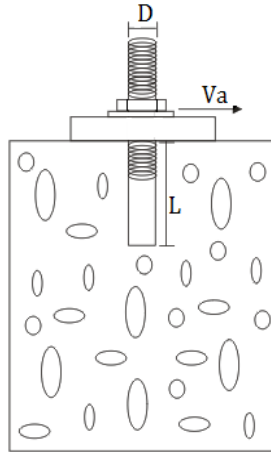

Fig. 5 Some parameters of suggested anchor shear model formula.

Nonlinear load-displacement features are calculated by using suggested model for anchors having different diameter (D) and embedment depth (L) as given in Table 1. For all anchors, embedment depth (L) is Φ10. Namely, for Φ6, embedment depth is 60 mm, for Φ8, 80 mm, for Φ10, 100 mm. Experimentally determined mean yield strength value is used as yield point of steel. Here, C1, C2, C3 represent the concrete compressive strength of specimens, 5.7 MPa, 9.1 MPa and 19 MPa, respectively. ⊡ is the displacement of the NLLink element representing the anchor.

Table1 Suggested model strength at different displacements and for different concrete grades for S420a steel

Displ. Δ(mm)	Anchor shear load V_a (kN)								
	$\Phi6$			$\Phi8$			$\Phi10$		
	C1	C2	C3	C1	C2	C3	C1	C2	C3
0	0	0	0	0	0	0	0	0	0
2	11.52	11.62	11.94	14.63	14.74	15.08	18.96	19.07	19.42
4	8.83	9.04	9.68	12.53	12.76	13.43	17.69	17.92	18.61
6	6.31	6.63	7.59	10.41	10.74	11.76	16.12	16.47	17.51
8	4.06	4.48	5.77	8.49	8.92	10.27	14.62	15.08	16.47
10	2.06	2.59	4.20	6.74	7.29	8.98	13.24	13.81	15.56
12	0.29	0.92	2.85	5.19	5.86	7.88	11.99	12.68	14.78
14	-	-	1.69	3.81	4.58	6.94	10.88	11.69	14.13
16	-	-	0.71	2.57	3.46	6.16	9.89	10.81	13.61
18	-	-	-	1.49	2.48	5.51	9.02	10.05	13.20
20	-	-	-	0.51	1.61	4.99	8.25	9.40	12.90
22	-	-	-	-	0.86	4.57	7.59	8.85	12.69
24	-	-	-	-	-	-	-	4.26	12.58

In Fig. 6, comparison of shear capacities of anchors by suggested shear model and ACI 318 [11] formulation are given for all anchor experiments.

Suggested anchor shear model is established with the results obtained from the tests of the anchors distant from the edges. Thereby, all of the anchors reached their capacity with steel damage. However, if the anchor is close to an edge, there may be critical changes in anchor behaviour. Suggested anchor model may lead to errors if the anchors are close to the edges. For this reason, it is suggested that more accurate results may be get for the anchors that reaches ultimate capacity by concrete failure by reducing the strength values by Eqn. 1 with respect to the ACI 318 [11].

In ACI 318 [11], three collapse modes are identified. These are concrete pryout failure, concrete breakout failure and steel failure (Fig. 7).

Steel failure can be determined with Eqn. 2:

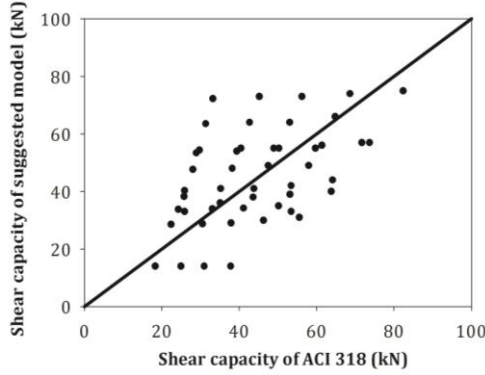$$V_{sa} = 0.6 \cdot n \cdot A_{se} \cdot f_{uta} \qquad (2)$$

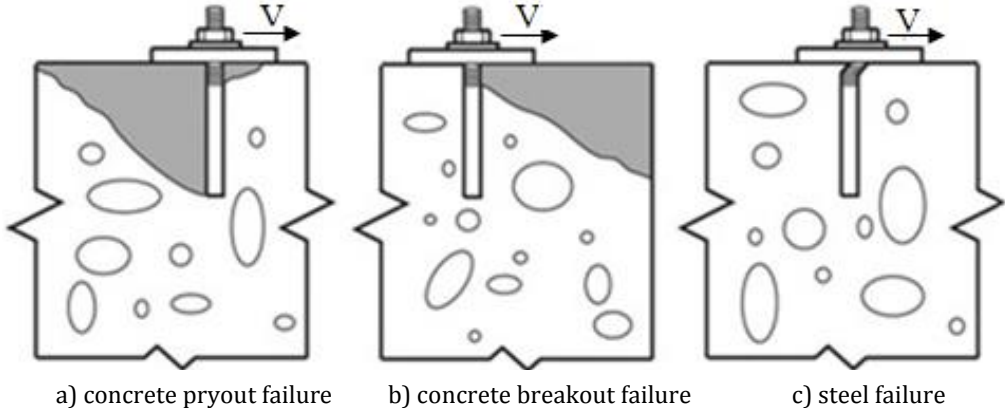Fig. 6 Comparison of shear capacities of anchors by suggested shear model and ACI 318 [11] formulation

a) concrete pryout failure   b) concrete breakout failure   c) steel failure

Fig. 7 Anchor failure modes under shear loading in ACI 318 [11]

Here, n is number of anchors in group, A_{se} is cross sectional area of an anchor (mm²), f_{uta} is, tensile strength of the anchor steel (N/mm²).

In concrete edge failure, breakout capacity for single anchor (V_b) can be computed by Eqn. 3.

$$V_b = 0.6(\frac{l_e}{d_0})^{0.2} \sqrt{d_0} \sqrt{f_c'} (c_{a1})^{1.5} \tag{3}$$

Here, l_e (mm) is effective anchor depth, d_0 (mm) is bar diameter , c_{a1} (mm) is distance to free edge taken in the direction of the applied shear and f_c' (MPa) is specified concrete strength. For the pryout collapse of concrete, ACI 318 [11] presents Eqn. 4 and Eqn. 5 for single and group anchors respectively.

$$V_{cp} = k_{cp} \cdot N_{cb} \quad \text{(for single anchors)} \tag{4}$$
$$V_{cpg} = k_{cp} \cdot N_{cbg} \quad \text{(for group anchors)} \tag{5}$$

Here, V_{cp} (N) and V_{cpg} (N) are nominal concrete pryout strength of single and group anchors; N_{cb} (N) and N_{cbg} (N) are nominal concrete breakout strength in tension of single and group anchors, respectively. k_{cp} is a constant that changes with embedment depth of anchors. For shallow anchors ($h_{ef} < 65$ mm), k_{cp} is 1.0. For other effective depth values ($h_{ef} \geq 65$), k_{cp} is 2.0.

In ACI 318 [11], no relation between anchor displacement and corresponding strength is given. As all the tests used to develop the given model contain anchor far from edges, ACI 318 strength values are used as an upper bound for the obtained strength values. However, please note that the displacement-strength relationship is still used as suggested by the proposed model. Fig. 8 illustrates the modification for the anchors that may reach capacity by concrete failure.
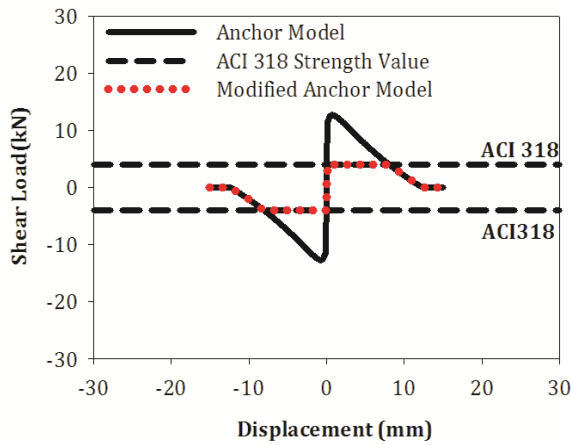

Fig. 8 Modification for the anchors that are close to edges using ACI 318 strenght as upper bound

Lateral load-displacement relationship related with NLLink components are as given in Figs. 9-11 for different anchor diameters and concrete types. Strengths obtained according to suggested model and reduced values of these strengths which are obtained by using formulas in ACI 318 [11] as an upper bound.
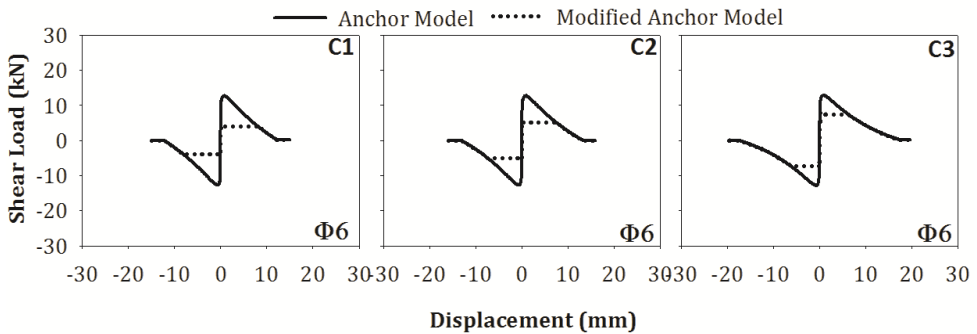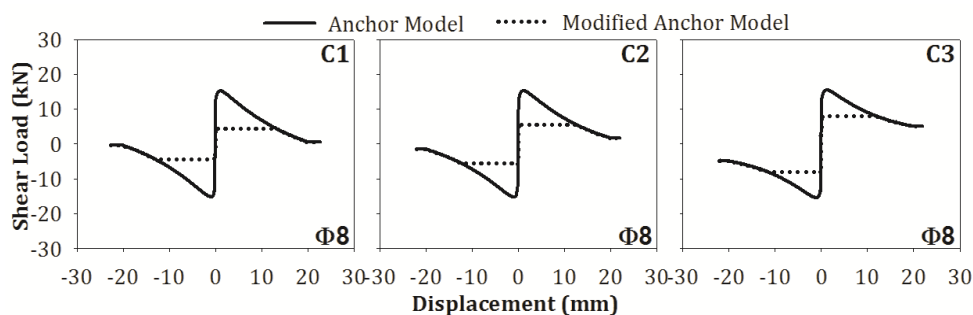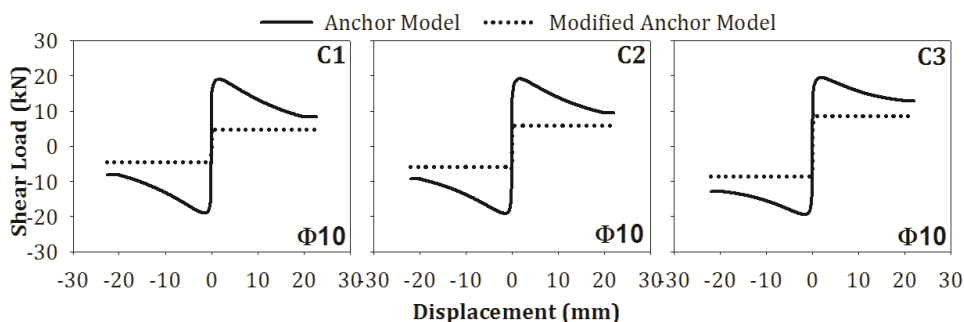

Fig. 9 Properties of NLLink components modelling Φ6 anchors

Fig. 10 Properties of NLLink components modelling Φ8 anchors

Fig. 11 Properties of NLLink components modelling Φ10 anchors

Using suggested methodology, shear strength of anchors, which are modelled with NLLink components, are determined with respect to the displacement level. Established NLLink components can also carry load outside of given displacement limits. Load values outside of given displacement limits are determined by continuing last slope line. For that reason, a permanent force value in NLLink components are formed outside of displacement limits reflecting the residual capacity. This zone can be seen as constant force zone in the end of graphics.

3. Features of external shear wall

In the experiments of external shear wall, it was observed that TS500 [12] formula can provide a certain safety coefficient in anchor components with a small diameter. By using anchor shear forces determined with this way, sixteen specimens are tested under repetitive and reversible loads. Specimens are produced with three different types of concretes, C1, C2, C3 as explained above. Connection with external shear wall and existing frame is provided with anchors that are embedded to the columns and beams of these specimens. For representing the deficiencies of the structures in Turkey, the specimens are produced with low compressive strength concrete.

Anchors in connections of shear wall and columns and beams in the specimens are varied. The cases are named as C1, C2 and C3 according to concrete strength of frames and beam-traverse anchor ratios. These frames are strengthened with shear walls with concrete compressive strength of 30 MPa.

In Fig. 12, two- story, one-bay RC frame dimensions; in Fig. 13, steel detailing of columns and beams are given. Anchorage amounts which were used in frame tests of reference study can be seen in Table 2.
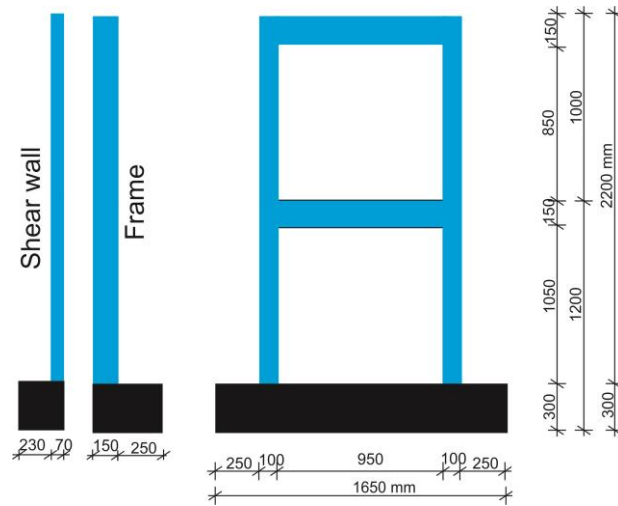
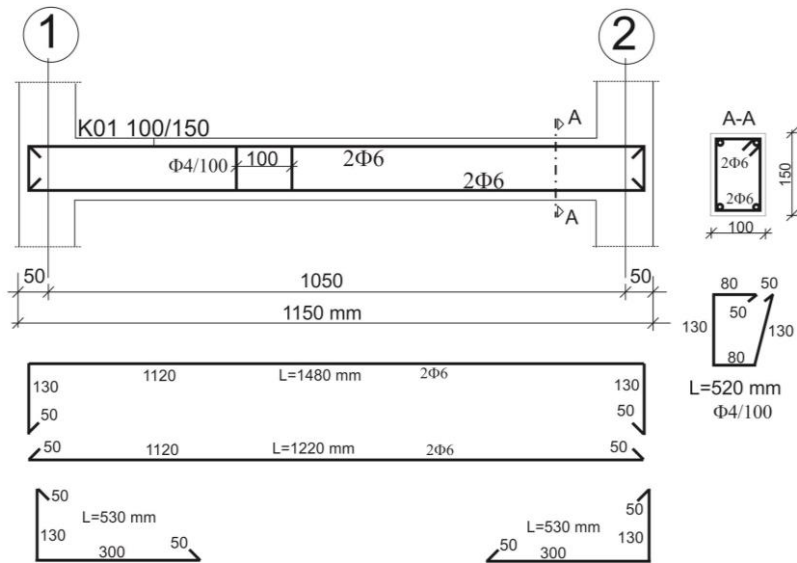Fig. 12 Dimensions of two- story, one-bay RC frame specimens

Fig. 13 Detailing of columns and beams

Table 2 Anchorages used in frames retrofitted with outer shear wall

Test Frame Models	1st Floor Column	2nd Floor Column	1st Floor Beam	2nd Floor Beam
C1DE	3Φ6+2Φ8	1Φ6+2Φ8	1Φ8	3Φ6
C2ED	1Φ6+2Φ8	3Φ6	2Φ6	1Φ6+2Φ8
C2AA	4Φ10+2Φ8+3Φ6	2Φ6+5Φ8	3Φ8+1Φ6	7Φ8
C2BA	3Φ10+1Φ8+2Φ6	1Φ10+3Φ8	1Φ10+3Φ6	6Φ8+2Φ6
C2BB	5Φ8+3Φ6	4Φ8+1Φ6	2Φ8+1Φ6	2Φ10+1Φ8+2Φ6
C2DA+	1Φ10+2Φ8	2Φ8	4Φ10+2Φ8+1Φ6	4Φ10+2Φ8+1Φ6
C3HD	-	-	2Φ6	1Φ6+2Φ8
C3DH	3Φ6+2Φ8	1Φ6+2Φ8	-	-
C3DG	1Φ6+2Φ8	3Φ6+2Φ8	1Φ6	1Φ8
C3EE	1Φ6+2Φ8	3Φ6	1Φ8	3Φ6
C3AA	4Φ10+2Φ8+3Φ6	2Φ6+5Φ8	3Φ8+1Φ6	3Φ10+4Φ6
C3BA	3Φ10+2Φ8	2Φ10+2Φ6	3Φ8+1Φ6	3Φ10+4Φ6
C3BB	2Φ10+4Φ8+1Φ6	2Φ8+4Φ6	1Φ10+2Φ6	4Φ8+2Φ6
C3BB+	3Φ10+1Φ8+2Φ6	1Φ10+3Φ8	2Φ8+1Φ6	5Φ10+2Φ8
C3DA+	1Φ8+4Φ6	1Φ8+2Φ6	4Φ10+2Φ8+1Φ6	1Φ10+1Φ8+2Φ6
C3DD	3Φ6+2Φ8	1Φ6+2Φ8	2Φ6	1Φ6+2Φ8

4. Non Linear Analyses of Strengthened Frames

Two-story, one-bay RC frame specimens tested in [13] and [14] are modelled using SAP 2000 [9]. Behaviour of anchors connecting the frame and shear wall is reflected with NLLink elements using proposed model Nonlinear pushover analyses are performed. Capacity curves which was derived from analyses and envelope curves which was derived from test data is given in Figs. 17-18.

4.1 Structural Model Info Performed Using SAP 2000 [9] Software

For composing structural analyses, two dimensional models of test frames, as seen in Fig. 14, established in SAP 2000 [9] software. In these models, nonlinear behaviour that can be occur in shear walls, columns, beams and anchors were considered. Nonlinear pushover capacity curves obtained from analyses were compared with experimental values. In SAP 2000 [9] models, experimental values for material strengths for shear walls, columns, beams were used.
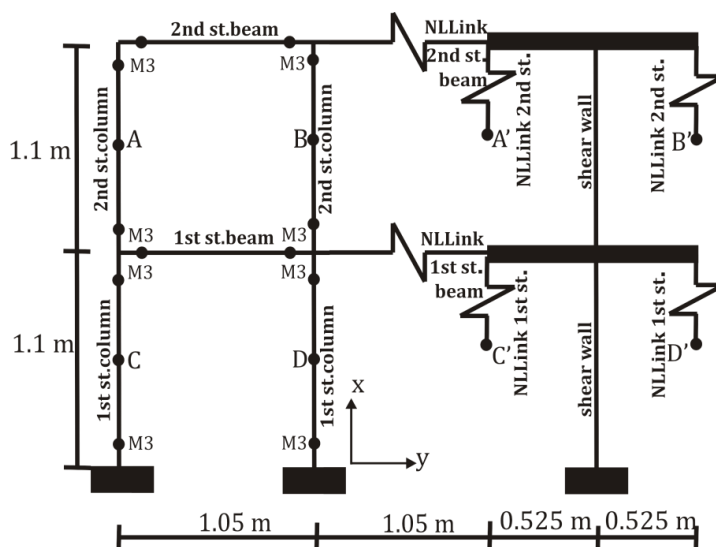
Fig. 14 Hinges and NL Links in SAP 2000 [9] models

Frame and shear wall were modelled side by side as given in Fig. 14. Infinitely rigid beams are connected to the shear wall in story levels. In this way, wide column analogy is considered for modelling of the shear wall behaviour. As it can be seen in Fig. 15, in wide column analogy, shear walls can be modelled as a column that is located at the axis of the shear wall. Through the B dimension of the shear wall, infinite flexural stiffness is considered (EI=∞) [15]. Wide column analogy is therefore a tool which is not only available to researchers but also to design engineers [16].
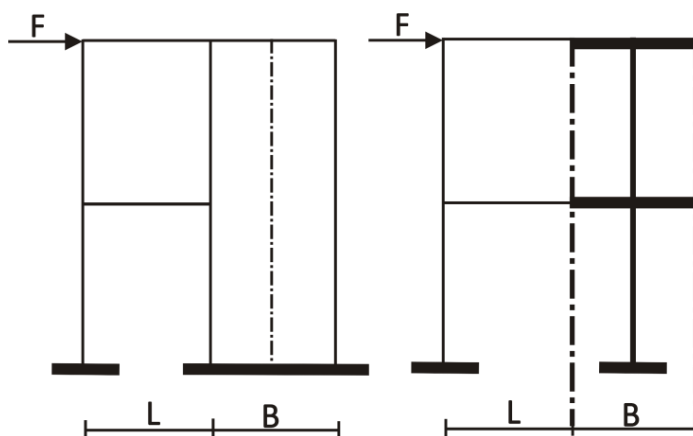

Fig. 15 Using wide-column analogy in modelling of a shear wall-frame system

Connections between shear wall and frame beams in story levels were provided by lateral NLLink components. Shear force-displacement behaviour of all anchorages between the shear wall and a story beam was provided by a single NLLink component. Load transfer of RC shear wall and columns were provided using NLLink components from the midpoint of the story levels. In the model, EQUAL components were assigned for providing rigidity between endpoints of NLLinks and RC shear walls.

EQUAL constraint makes sure that the located place in the chosen degree of freedom moves with same amount of displacement. A, B, C and D points define the mid points of the story levels. These points displace along with A', B', C', D', owing to EQUAL constraint.

Hinges were defined at the endpoints of beams and columns. Flexural capacity of shear wall component was calculated and is considered in nonlinear model. For representing hinging in the endpoint of shear wall, NLLink components were used. Moment-curvature relationship belonging to shear wall was obtained by using SEMAp [17] software which was developed for modelling nonlinear behaviour of reinforced concrete components.

Moment–rotation behaviour of external shear wall was modelled with an idealization as given in Fig. 16. Obtained moment-curvature relationship was transformed into moment–rotation behaviour. Elastic rotation during yielding and plastic rotation in hinge can be calculated respectively with Eqn.6 and Eqn 7.

$$\vartheta_y = \phi_y \cdot \ell_p \tag{6}$$
$$\vartheta_p = (\phi_u - \phi_y) \cdot \ell_p \tag{7}$$

In these equations, ϑ_y is yielding rotation; ϕ_y is yielding curvature; ϑ_p is rotation capacity of plastic hinge, ϕ_u is the ultimate curvature, ℓ_p is size of plastic hinge.
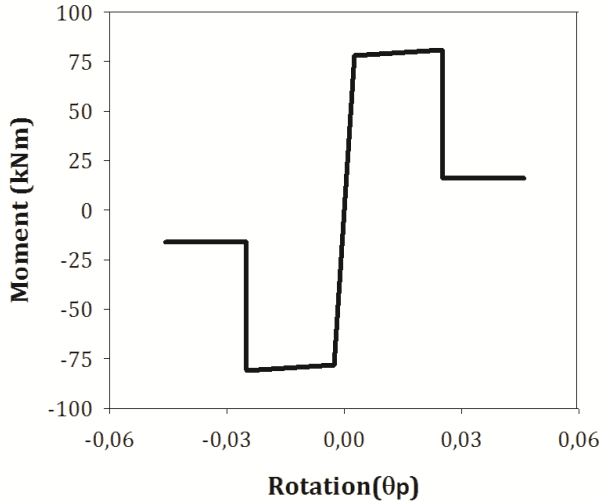

Fig. 16 Moment-rotation relationship which was calculated for shear wall component

In columns located on endpoint of shear walls, besides anchor damage, columns can fail under axial load and rupture of steel reinforcement by tensile forces is possible. For that reason, compressive and tensile strength capacities of columns were considered in nonlinear model.

4.2 Comparison of Experiment and Analyses Results

The suggested model and reduced values with ACI 318 [11] formulation for SAP2000 [9] results are given for comparison with experimental values in Figs. 17-18. Modified model with ACI 318 [11] capacity values, have lower capacity due to reductions made for possible concrete failure. An amount of decrease in capacity was observed in all models after reaching lateral load capacity. This decrease indicates that capacity of anchors have started

to decrease. In Figs. 17-18, the continuous line named as "Monolithic Behaviour" represents the case of shear wall with full capacity reflecting the monolithic behaviour with the frame. The reduction in capacity due to connection with anchors is obvious in figures. The red dotted lines in the figures represent the suggested model modified for concrete failure using ACI 318 strength values as upper bound.
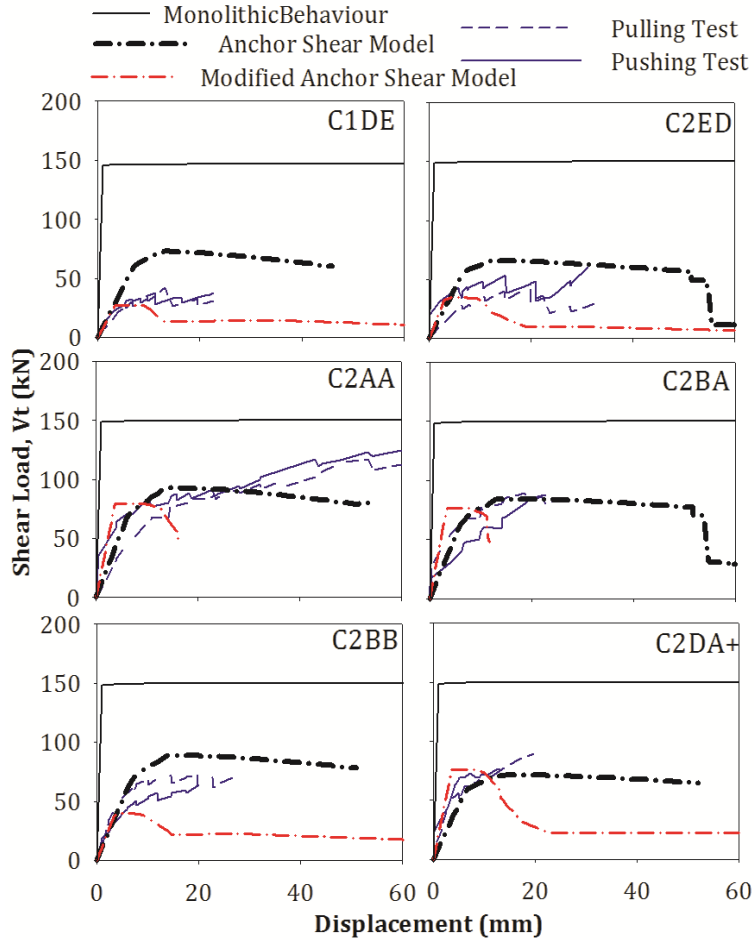

Fig. 17 Envelope curves of C1 and C2 test specimens and capacity curves of SAP 2000 [9]

Differences in lateral load-displacement curves which were obtained as a result of experiments and analytical studies can be related with applied cyclic load during experiments. In the analyses, the mentioned load pattern is applied in a monotonically increasing manner and the rigidity loss is not directly taken into account.

Maximum load and collapse damages obtained in SAP 2000 [9] models for all of the frames, strengthened with external shear wall, are seen in Figs. 19-30. The black dots represent heavy damage and failure at the corresponding points.
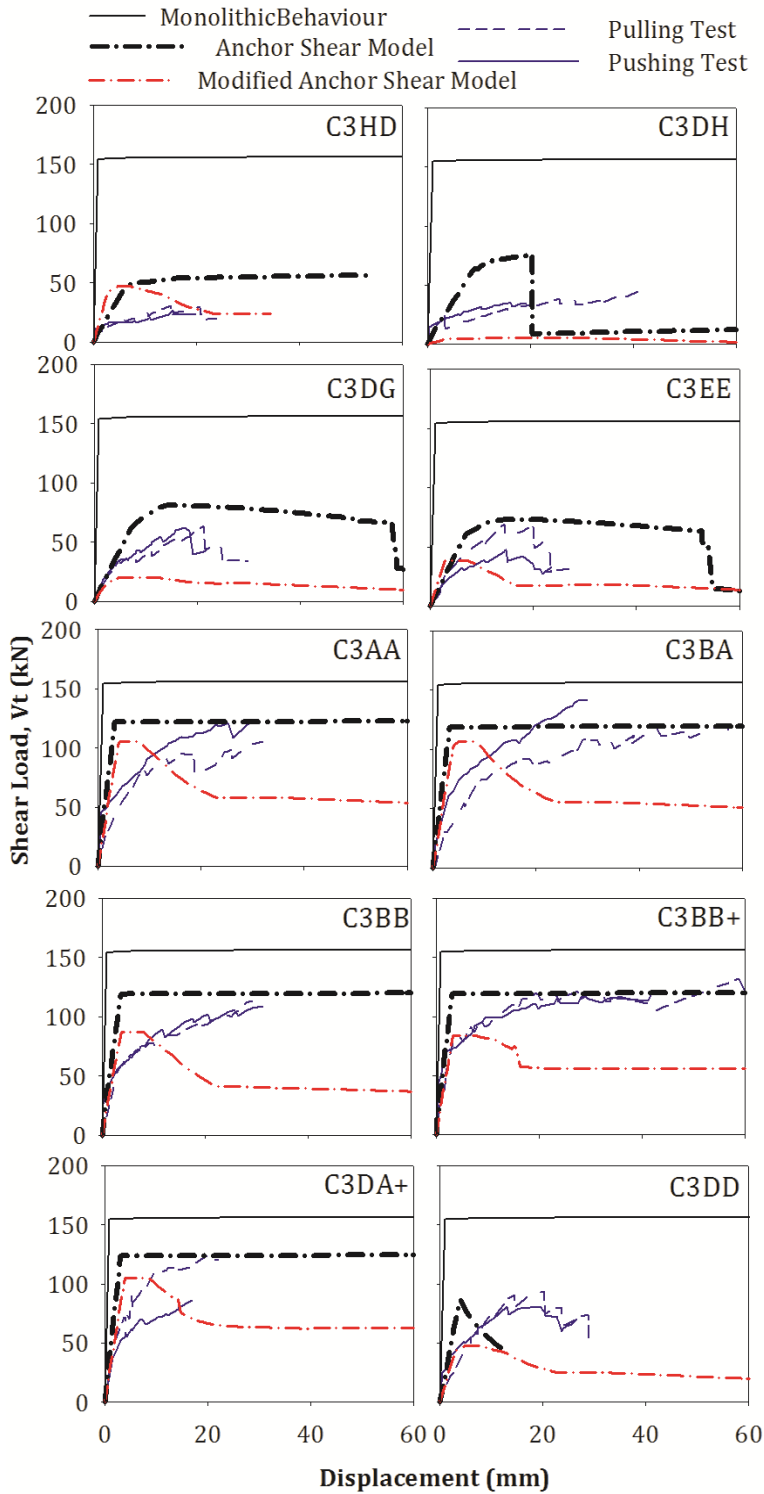
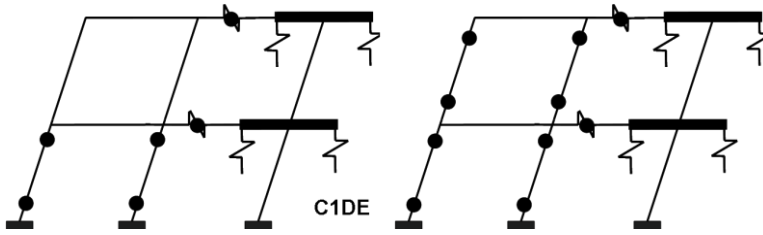Fig. 18 Envelope curves of C3 test specimens and capacity curves of SAP 2000 [9]

Fig. 19 Maximum load and component damages in case of collapsing for C1DE model
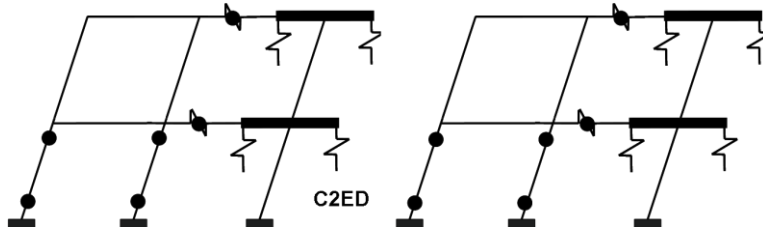

Fig. 20 Maximum load and component damages in case of collapsing for C2ED model
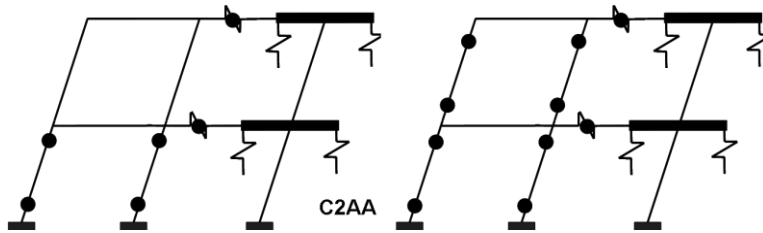

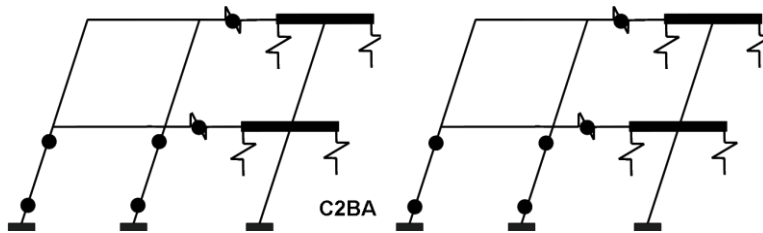Fig. 21 Maximum load and component damages in case of collapsing for C2AA model

Fig. 22 Maximum load and component damages in case of collapsing for C2BA model
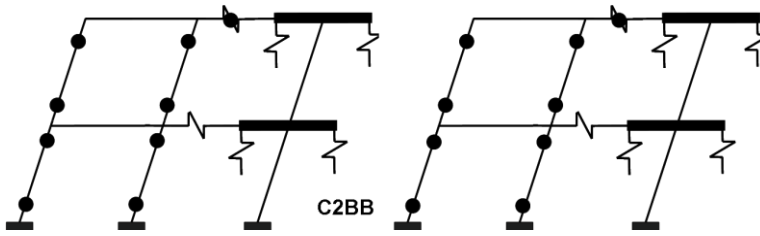
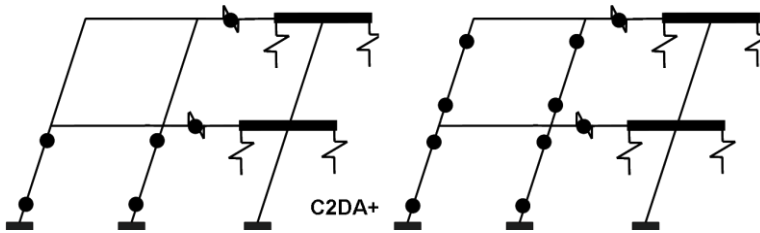Fig. 23 Maximum load and component damages in case of collapsing for C2BB model

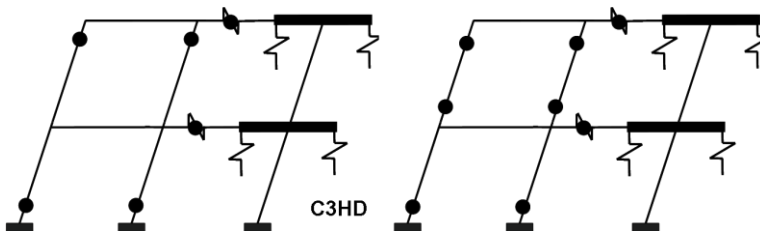Fig. 24 Maximum load and component damages in case of collapsing for C2DA+ model

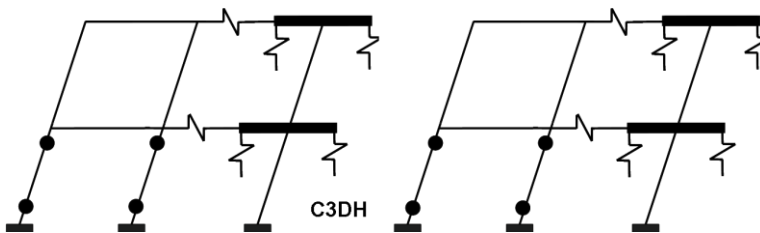Fig. 25 Maximum load and component damages in case of collapsing for C3HD model

Fig. 26 Maximum load and component damages in case of collapsing for C3DH model

Fig. 27 Maximum load and component damages in case of collapsing for C3DG model

Fig. 28 Maximum load and component damages in case of collapsing for C3EE model

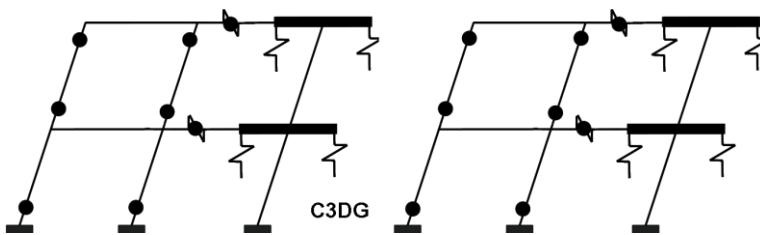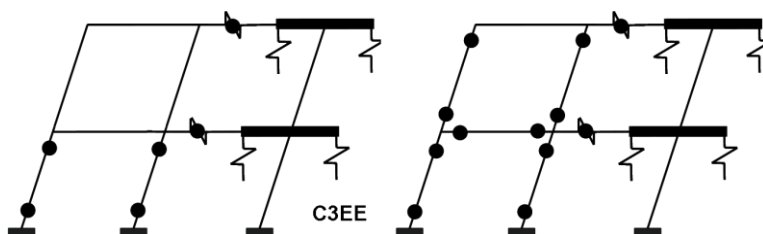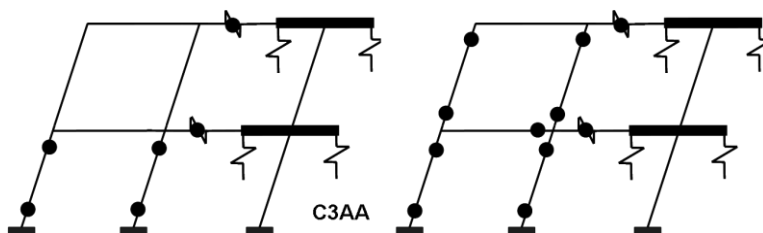

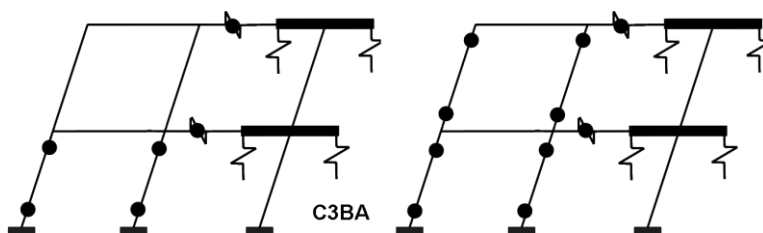Fig. 29 Maximum load and component damages in case of collapsing for C3AA model

Fig. 30 Maximum load and component damages in case of collapsing for C3BA model

5. Results and Discussion

In this study, finite element models of two-story, one-bay RC frame specimens strengthened with external shear walls in Pamukkale University Earthquake and Construction Technology Research Laboratory were performed using SAP 2000 [9] software. In these models; nonlinear behaviour of anchors connecting RC shear wall and existing frame is represented with suggested anchor shear model. Reliability of suggested anchor shear model was tested upon these models. It is observed that, experiment results and nonlinear static pushover results are in consistence. As a result of these comparisons, suggested anchor shear model may be used for modelling shear behaviour of anchors.

Authors emphasize that, in developing process of this model, experimental data of anchors which are sufficiently away from edge were used. But there may be some cases that, anchors embedded close to the free edges, defected concrete existence in the place where the anchors are embedded or anchors reach breakout or pryout failure capacity. All of these factors were considered and lateral load-displacement capacities obtained by using suggested anchor shear model were bounded by the ultimate loads obtained from the formulas in ACI 318 [11].

In case of a compressive strength higher than 20 MPa and with reasonable amount of anchors, it was seen that suggested model and single diecast shear wall are closer compared to others. But, in case of a compressive strength under 20 MPa, capacities obtained by considering both cases were nearly the half of single diecast shear wall.

Therefore, anchors reach their capacity with steel damage due to the increase of concrete compressive strength. Also, it can be stated that concrete compressive strength of existing structure is very important when calculating shear capacity. In specimens, the least value for concrete compressive strength is 5 MPa. Thus, use of this model in structures that has concrete compressive strength under 5MPa is not suggested because it is not in scope of the study.

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