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An experimental investigation for external RC shear wall applications

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Abstract. The strength and rigidity of most reinforced concrete (RC) buildings in Turkey, which are frequently hit by destructive earthquakes, is not at a sufficient level. Therefore, the result of earthquakes is a significant loss of life and property. The strengthening method most commonly preferred for these type of RC buildings is the application of RC infilled walls (shear walls) in the frame openings of the building. However, since the whole building has to be emptied and additional heavy costs arise during this type of strengthening, users prefer not to strengthen their buildings despite the heavy risk they are exposed to. Therefore, it is necessary to develop easier-to-apply and more effective methods for the rapid strengthening of housing and the heavily-used public buildings which cannot be emptied during the strengthening process (such as hospitals and schools). This study empirically analyses the different methods of a new system which can meet this need. In this new system, named "external shear wall application", RC shear walls are applied on the external surface of the building, along the frame plane rather than in the building. To this end, 7 test samples in 1/2 and 1/3 geometrical scale were designed to analyse the efficiency of the strengthening technique where the shear wall leans on the frame from outside of the building (external shear wall application) and of the strengthening technique where a specific space is left between the frame and the external shear wall by using a coupling beam to connect elements (application of external shear wall with coupling beam). Test results showed that the maximum lateral load capacity, initial rigidity and energy dissipation behaviours of the samples strengthened with external shear wall were much better than those of the bare frames.

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1 Introduction

RC shear wall application is the most preferred method in the strengthening process of the RC buildings having low earthquake behaviour, which is a process that started with the 1992 Erzincan earthquake and intensified after the 1999 Marmara earthquake. This method is applied in various alternating ways. These alternatives can be summarized as the turning of a partial or complete axle into a shear wall; the application of welded wire fabric and concrete on the non-load-bearing brick wall to create a shear wall; or the external shear wall application performed by applying an additional shear wall from the outside of the building.

When factors such as existing RC building stock, magnitude of earthquakes, damage and loss of life and property recorded in the settlements close to earthquake hypocentre are considered, it can be clearly concluded that there is a great need for the infrastructure required for effective and appropriate strengthening. In this scope, huge investments and research have been made in Turkey in the last two decades.

The tests performed in the scope of the present study analysed the changes in the structural performance outputs created as a result of external RC shear wall application onto the plane frames (reflecting the existing building stock in Turkey which 1) have low-quality material characteristics, 2) fail to meet section and strengthening requirements, and 3) have geometrical scales of 1/2 or 1/3). Moreover, external shear wall-frame connection status was analysed in these tests. In the tests carried out on the plane frame model, out-of-plane behaviours were prevented or ignored. Earthquake-simulating reversed-cyclic loading was applied either from the top storey or, at different rates (triangular load distribution), from the storey levels representing the real earthquake behaviour of the building (Sonuvar, 2004; Zhao, 2004; Erdem, 2006; Canbay, 2003, 2004; Chan, 2000; Ozcebe, 2003; Kamanli, 2010). In these tests, during which particularly the

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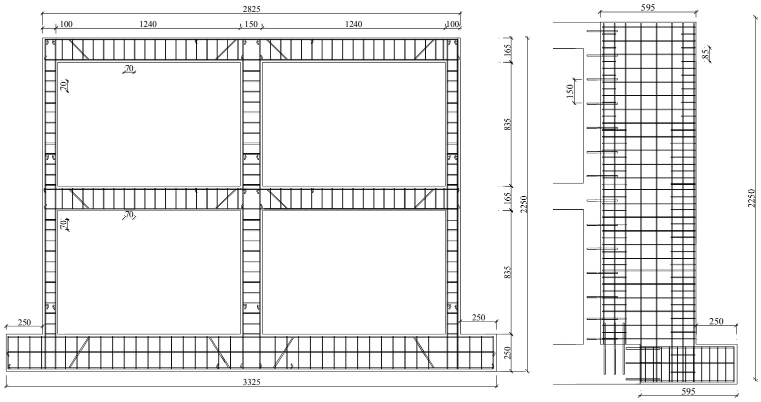


Fig. 1. Dimensions and reinforcement layouts of RC frame and external shear wall for the 1st group tests (Arslan, 2007; Kaltakci, 2007).

quasi-static loadings were used, load was applied as load-controlled until the yield displacement of the frame system and, afterwards, as displacement-controlled. Researchers who conducted the tests agreed that RC shear walls provided bare frame systems with sufficient rigidity and strength (Anil, 2007, 2008; Ozcebe, 2003; Erdem, 2006; Zhao, 2004).

Another important issue to be considered at this point is that strengthening is not only considered as a method that solely increases system performance. As a matter of fact, the feasibility of strengthening, its economic analysis, its impacts on the architectural design factors, user satisfaction and post-strengthening changes in the building functions are as important as the performance outputs of the strengthening method. Particularly the external shear wall method, which claims to be applied without the need to enter into the building, has a special place among shear wall application methods.

This study analyses the impacts of external shear wall application, the most-discussed experimental method, on the behaviours of the plane frame system. To this end, RC specimens designed in 1/2 or 1/3 geometrical scale and tested under different loading systems were analysed in terms of the changes observed in their earthquake behaviours. As well as the study findings and factors that should be considered in the strengthening process; the effects of dimensions, loading system and similar variables on the test results were discussed.

2 Experimental study

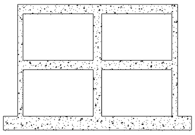
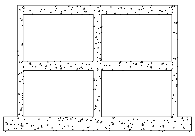
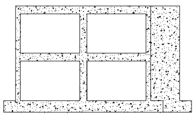
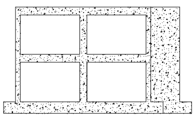
In the experimental study, two different test groups and their corresponding results were analysed. Despite the difference in the scale and loading system of these two groups, the main objective was to analyse the pros and cons of external shear wall application. In the tests conducted in the 1^{st} group, external shear wall was placed adjacent to the frame system. In the second group, on the other hand, external shear wall was connected to the frame system by using different coupling beams.

First of all, the RC frames have defects commonly encountered in buildings in Turkey, such as, (a) low concrete strength (12∼13 MPa), (b) lack of (frequently located) sufficient stirrup volumetric ratio at the beam and column ends, (c) column ties not extending into beam-column joinings, (d) use of vertical-hook type binders, insufficient debonding length of beam bars, (e) construction of along-the-column longitudinal bars as splices lapped insufficiently at the storey and base levels, (f) strong beam – weak column was analysed by strengthening it with an undamaged and non-defective RC shear wall (placed adjacent to the frame or connected to the frame via coupling beams) constructed in accordance with the applicable earthquake regulations of the same frame (Arslan, 2007; Bal, 2007; Bruneau, 2002; Dogangün, 2004; Hakan Arslan, 2009; Sezen, 2003).

In the first test group; behavioural changes recorded in the load-bearing system, when the external shear wall was placed adjacent to the RC frame, were analysed. To this end, 4 identical RC frames having weak earthquake behaviour (two-opening, two-storey frames modelled on 1/3 geometrical scale) were produced (Arslan, 2007). These frames were equipped with the design and construction defects commonly encountered in the buildings in Turkey. While 2 of these 4 frames were strengthened via external shear wall application, no strengthening was performed for the remaining two frames. Then, all frames were tested under reversed-cyclic lateral loads simulating earthquake loads. Normal force was applied on the columns of the test elements at such a level to ensure tensile failure in columns ($N = 0.1 \times A_c \times f_c$). At the end of the tests, behavioural characteristics of the specimens were determined and their lateral load carrying capacity and shear wall efficiency were analysed comparatively. Dimensions and reinforcement layouts of the specimens are presented in Fig. 1. In addition, Table 1 lists the general characteristics of the specimens.

As can be seen in Table 1, concrete used in frames had a compressive strength of approximately 13 MPa and the concrete used in shear walls of 29 MPa. Side column dimensions were 85×100 mm and mid column dimensions

Table 1. Characteristics of the specimens in the 1st test program.

Test No	Frame type	Axial load level of columns (N_o/N_r)	Longitudinal bar ratio of columns (ρ)	Average concrete compressive strength of frames and shear walls (MPa)		Yield and tensile strength of the bars used in frames and shear walls (MPa) (yield/tensile)	
				Frame	Shear Wall	Frame	Shear Wall
G_1-S_1		0.1	0.013	12.80	–	$\frac{387}{505}$	–
G_1-S_2		0.1	0.023	13.10	–	$\frac{387}{505}$	–
G_1-S_3		0.1	0.013	13.30	29.75	$\frac{387}{505}$	$\frac{590}{730}$
G_1-S_4		0.1	0.023	14.05	28.05	$\frac{387}{505}$	$\frac{590}{730}$

were 85×150 mm. While 4Φ6 longitudinal plain bars were used in the side columns and 6Φ6 plain bars in the mid columns of G_1-S_1 and G_1-S_3; 6Φ6 longitudinal plain bars were used in the side columns and 6Φ8 plain bars in the mid columns of G_1-S_2 and G_1-S_4. Beam sizes were 85×165 mm and, 3Φ8 tensile bars and 2Φ8 assembly bars were used in the beams. Plain bars with a diameter of 4 mm spaced at 70 mm were used as closed ties in columns and in beams.

To ensure the simultaneous movement of the frame and the shear wall used to strengthen it, deformed bars (8 mm in diameter) were used for anchorage. These anchorage bars were located at 150 mm intervals, starting at 100 mm from the base. Holes (each being 80 mm in diameter) were opened on the side column, complying with 150-mm interval. After the holes were cleaned, dowels were inserted using Sika Anchor fix-2 anchoring adhesive. Length of the dowel part inserted in the shear wall was 20ϕ. Section dimensions of the external shear wall placed for strengthening were 85×595 mm. As per TEC-2007 (Turkish Earthquake Code, 2007) requirements, broad shear wall edges were formed (in the part from base level up to the height of the shear wall) and winded firmly to prevent plastic hinge effects. Properties of the anchorage dowels is given in Fig. 2.

All test specimens were tested under reversed-cyclic lateral load effects, by using 500 kN-capacity rigid steel loading frame. Lateral and vertical load measurements were made via Loadcell and displacements via LVDT (Linear Variable Displacement Transducer) and Dial-gage. Tests of the specimens were conducted as load-controlled till the nominal yield load of the system and, afterwards, as displacement-controlled.

Fig. 2. Properties of anchor dowels (Arslan et al., 2010).

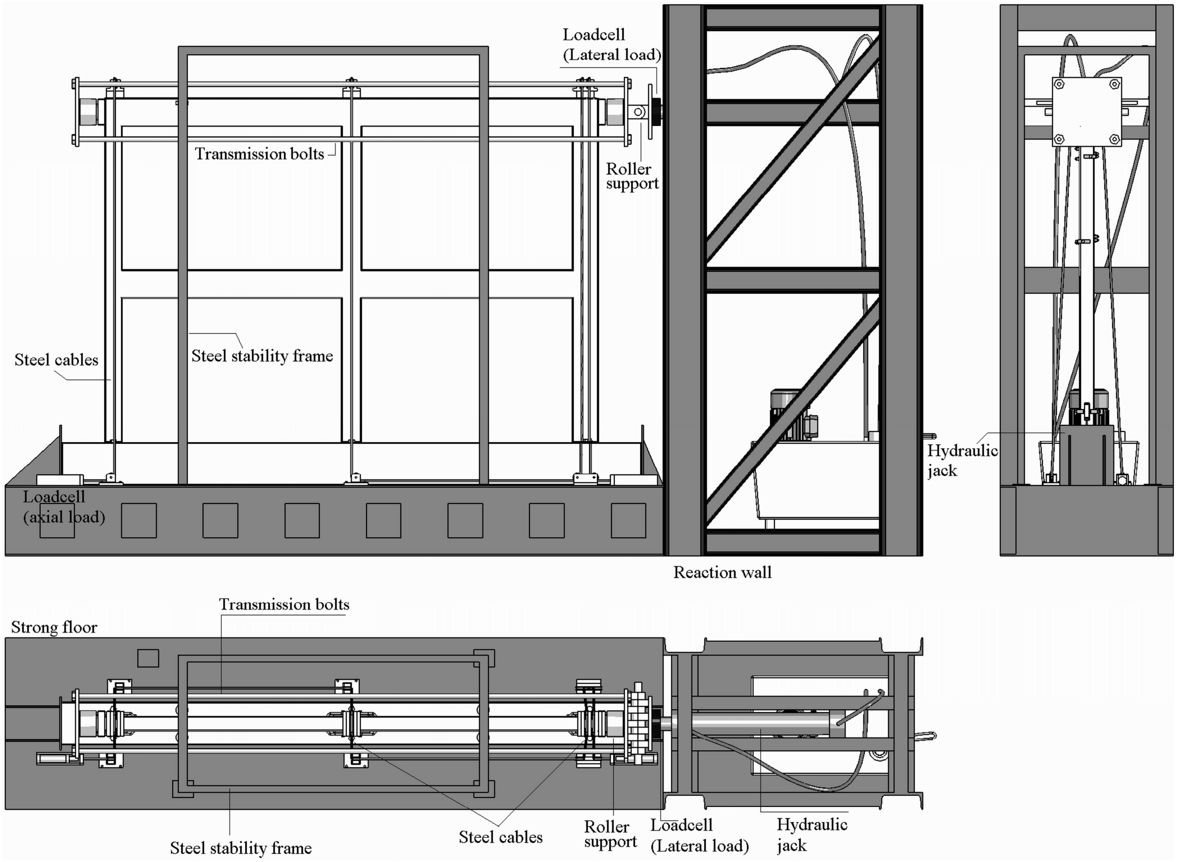


Fig. 3. Loading system of the 1st group tests (Arslan, 2007; Kaltakci, 2008).

A rigid loading frame, used in the tests, was constructed from different steel profiles and was designed in such a way as to enable the application of lateral load at upper storey level. The loading frame was designed as a rigid frame to keep the value of the horizontal and vertical movements and rotation of the loading mechanism (constituted by the pump and load cells) at a value which is close to zero and does not affect test measurements. Axial load was applied on the frame columns by using a roller system made up of steel cables. A loading platform and wall was designed as a rigid floor plate enabling a fixed support of the test specimens. Floor system, designed to be quite rigid when compared to the superstructure, was fixed to the loading platform via buttons placed at certain intervals. In this way, the floor was prevented from rotating due to horizontal or vertical shift. To prevent out-of-plane movement of the specimen, a secondary frame was constructed and fixed to the test specimen via sliding wheels. The test and measurement mechanism is presented in Fig. 3.

Lateral load-displacement ratio curves and cumulative energy dissipated-displacement ratio curves of the test specimens are listed in Fig. 4. These curves show that maximum loads, possibly carried by the bare frames, were 33 kN for G_1-S_1 and 45 kN for G_1-S_2. Maximum load was measured at 125 kN for G_1-S_3 and 170 kN for G_1-S_4. Accordingly, load carrying capacity of the system was increased to approximately 3.78 folds after the frames had been strengthened. Undoubtedly, a building can stand after an earthquake as long as it can dissipate sufficient amount of energy during the earthquake. As can be understood from the energy graphics, when compared to bare frames, the specimens strengthened with external shear wall (G_1-S_3, G_1-S_4) dissipated 3.63 to 4.55 times more energy. The energy dissipation was determined by calculating the areas inside the hysteretic load-displacement loops for each cycle. The cumulative energy dissipated was calculated as the sum of area enclosed by all previous hysteretic loops.

When G_1-S_1, G_1-S_2 are considered in terms of hinge formation time, 1st storey column-base connections of both frames were observed as the first parts to develop hinges. In G_1-S_3, G_1-S_4 specimens, on the other hand, frame damages started being observed in further cycles. The first cracks in G_1-S_3 were recorded at 60 kN on the support where the beams rest on the side columns and at 80 kN in G_1-S_4. Significant shear cracks developed on the column-beam connections of G_1-S_1, G_1-S_2. These cracks were recorded at 20 kN in G_1-S_1 and at 25 kN in G_1-S_2. These cracks were observed in G_1-S_3, G_1-S_4 specimens in the last cycles and after the

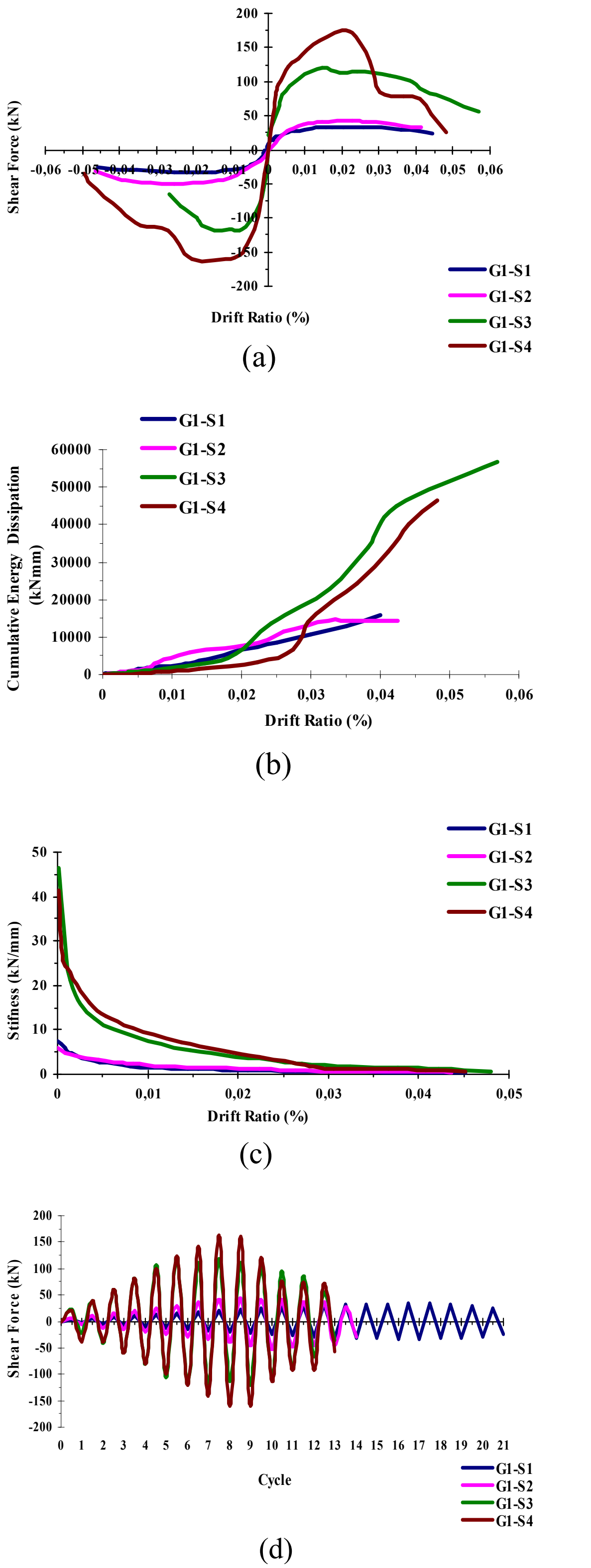


Fig. 4. Results of the 1st group tests - **(a)** load-displacement envelope curve of specimens, **(b)** energy dissipation capacity graphs of specimens, **(c)** stiffness characteristics of specimens, **(d)** load histories of specimens.

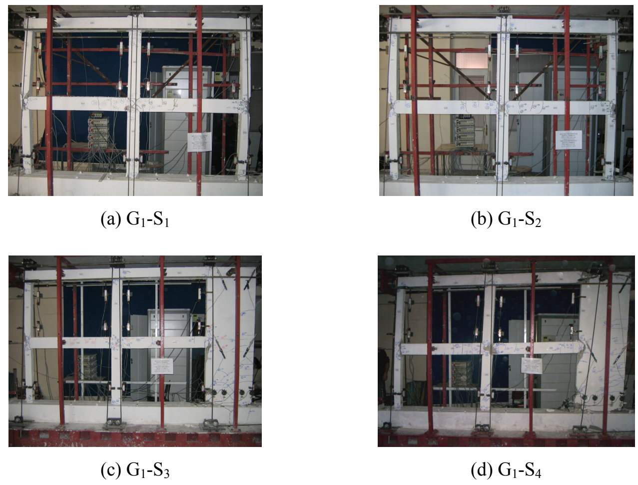
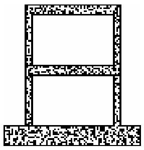
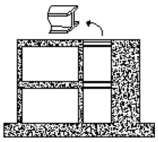
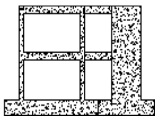


Fig. 5. Post-test views of the 1st group test specimens (Arslan, 2007).

shear reinforcements broke. Examination of the test specimens' behaviours shows that hinges developed on the beam ends and column bases and that vertical cracks developed on the pile fracture regions of particularly the first storey beams (160∼170 mm inside the support). After the tests, shear wall damages were recorded as flexural damages, which concentrated mainly within critical shear wall height (Fig. 5). No separation was observed in the anchorage-type joint between the frame system and shear wall (bases included) until the end of the tests.

A different type of shear wall application was analysed in the 2nd test serial. When it is impossible to ensure full connection between the external shear wall and the existing frame due to architectural reasons (the need to allocate openings for human passage as the ground floor is used for commercial purposes) or static (the need to lay a new foundation for the external shear wall), inter-element load transfer can be achieved via RC or a steel coupling beam, by leaving some space between the shear wall and frame. To make a behavioural analysis of the systems strengthened via this method (which is named "external shear wall application via coupling beam") under earthquake effect, 3 specimens at 1/2 geometrical scale were tested under reversed-cyclic lateral load effect (Ozturk, 2010). Among these three specimens, the first one was not strengthened and the second one was strengthened via an external shear wall equipped with a steel coupling beam. In the third test, on the other hand, system behaviour was analysed when the coupling beam (ensuring load transfer) was a RC beam. One-opening, two-storey test frame was a RC frame having the same design defects and failures as the one used in the 1st group tests (strong beam – weak column, low concrete strength, lack of (frequently located) sufficient binders, etc.). A schematic view of the tests and material characteristics are listed in Table 2.

Table 2. Characteristics of the specimens in the 2nd test program.

Test No	Frame type	Axial load level of columns (N_o/N_r)	Longitudinal bar ratio of columns (ρ)	Average concrete compressive strength of frames and shear walls (MPa)		Yield and tensile strength of the bars used in frames and shear walls Y(MPa) (yield/tensile)	
				Frame	Shear Wall	Frame	Shear Wall
G_2-S_1		0.1	0.013	13.56	–	$\frac{393}{492}$	–
G_2-S_2		0.1	0.013	13.56	26.60	$\frac{393}{492}$	$\frac{472}{573}$
G_2-S_3		0.1	0.013	13.56	26.60	$\frac{393}{492}$	$\frac{472}{573}$

Columns and beams were constructed with dimensions 160×240 mm and 240×240 mm, respectively, for the specimens. In the fourth column, 12 mm diameter plain bars were used as longitudinal reinforcement. Six plain bars with a diameter of 12 mm were used as longitudinal reinforcement in beams. Plain bars with a diameter of 8 mm spaced at 150 mm were used as closed ties in columns and in beams. No piles were formed on the beams, therefore, one specific bar ratio was selected for both support and opening sections. Frame concrete was produced from low-quality concrete to reflect the existing building stock of Turkey.

An external shear wall, to be connected to the frame via a RC or steel coupling beam, was casted horizontally according to TEC-2007 (Turkish Earthquake Code, 2007) specifications and, afterwards, was lifted and placed near the frame. Finally, different coupling beam details were applied between the shear wall and frame.

Additional, the external shear wall was 150×1050 mm in section dimension and RC coupling beam in 150×240 mm section dimension in G_2-S_3. Longitudinal deformed bars of 16Φ8 mm and horizontal web bars of Φ8/150 mm were used in the RC shear wall. Within the critical shear wall height (the area starting from the base up to the length (l_w) of the shear wall), special edges were formed and detailed. Longitudinal deformed bars of 6Φ12 mm were used in the RC coupling beam. Deformed bars with a diameter of 8 mm spaced at 70 mm were used as closed ties in RC coupling beam. To ensure load transfer between the existing frame and RC coupling beam, quadro group-anchorage was applied on the frame by using deformed bars (14 mm in diameter) at the level of each storey beam. Since the anchorage would be applied on a restricted area, the depth to be chosen had to minimize side distance effect. To this end, suggestions made in the literature on this issue were complied with and an anchorage depth was determined as 9 inches (230 mm) (Cannon, 1995). The dimension and reinforcement details of G_2-S_3 are shown in Fig. 6. Locations of the anchorages (applied to the existing frame) on the connection region are shown in the figure. The only difference in G_2-S_2 test was that the coupling beam connecting the frame to the external shear wall was constructed not of RC but of IPE 240 steel profile. The cross-section of the used profile is given in Fig. 6.

To simulate earthquake load in a reverse-cyclic manner and by taking into consideration actual earthquake behaviour to be shown by the building, test specimens were loaded and tested with the help of a special mechanism in such a way as to apply 2 units of load on the upper storey and 1 unit of load on the lower storey. Success of the strengthening via external shear wall equipped with a coupling beam depends on the effectiveness of the anchorage bars connecting the frame and strengthening elements. Therefore, tests required a loading method which could reveal debonding effects on the anchorage bars. To this end, the plates, fixed on the grooved irons placed on the existing frame before the concrete pouring process, were connected to the main loading system. Reloading was performed with the help of the tensile forces applied to the centre of the frame beams.

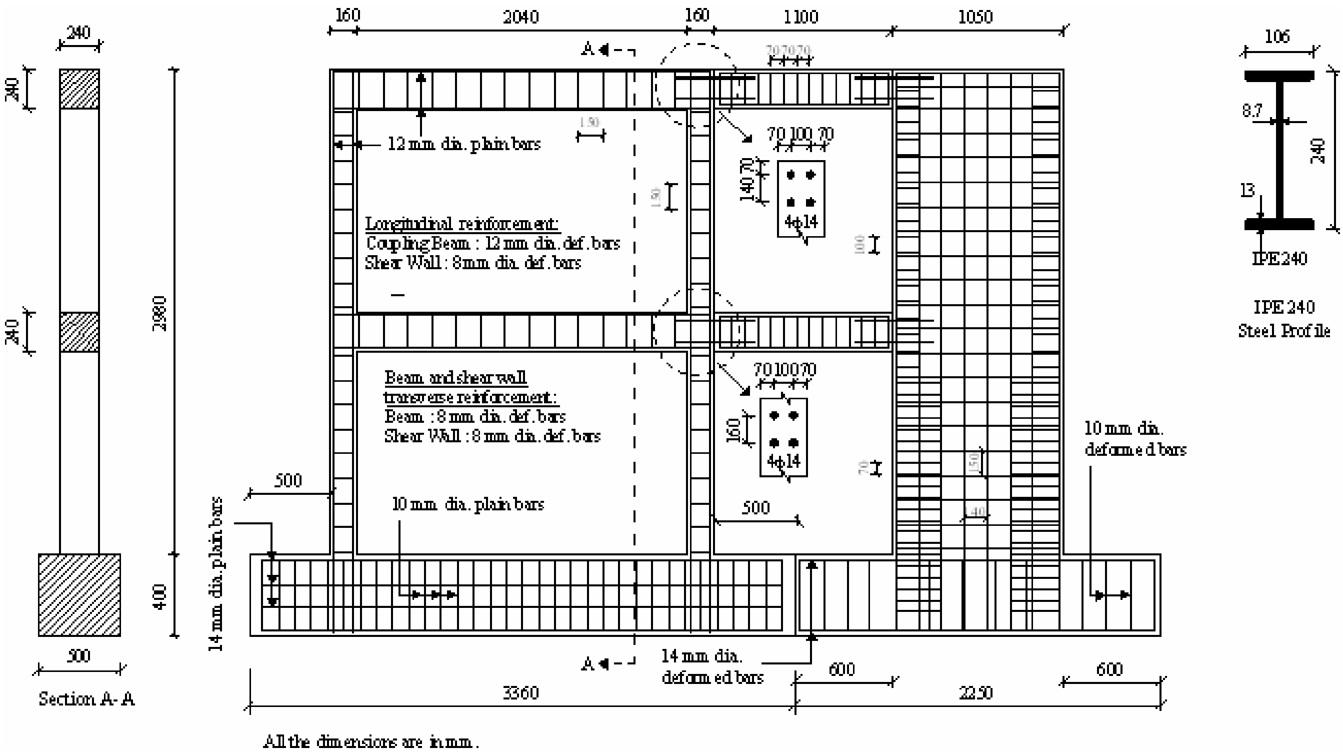


Fig. 6. Dimensions and reinforcement schemes of the RC frame and external shear wall of the 2nd group tests.

Total load, and the first and second storey loads, were measured via 3 load-cells. Specimens were fixed on the rigid laboratory floor to prevent any movement. The level of the axial load applied on the frame columns was the same as the one applied on the 1st group tests ($N = 0.1 \times A_c \times f_c$). Axial load was produced by pushing the steel beam downwards (located on the columns via simple supports) with the help of the hydraulic jacks mounted on it and of the bolts connected to the floor via hinges. The measurement mechanisms of the loading system and strengthened system are shown in Fig. 7.

Lateral load-displacement ratio curves and cumulative energy dissipated-displacement ratio curves of the 2nd group test specimens are listed in Fig. 8. As shown by the curves, the application of external shear wall equipped with coupling beam significantly increased the lateral load capacity and energy dissipation capacity of the frame. While the maximum load carried by G_2-S_1 was 33 kN, it was 180 kN for G_2-S_2 and 176 kN for G_2-S_3. Accordingly, load carrying capacity of the system was increased by approximately 5.80 fold after the frames had been strengthened. As can be understood from energy graphics, when compared to bare frames, the specimens strengthened with external shear wall (equipped with coupling beam) dissipated 3.41 to 5.66 times more energy. G_2-S_3 dissipated the highest amount of energy, in turn, showed the most ductile behaviour (Ozturk, 2010).

Post-test views of the specimens are shown in Fig. 9. As seen from the views, damages were concentrated on the nodal points in G_2-S_1 and concentrated within the critical shear wall height and on the coupling beam-external shear wall connections in G_2-S_2 and G_2-S_3. No debonding was observed on the anchorages in G_2-S_2 and G_2-S_3, which is an indication that the quadro-anchorage detailing produced positive results. However, anchorages formed apparent fracture cones on the shear wall when trying to debond from concrete. The height of the fracture cones and their width (starting from anchorage) was nearly equal to anchorage depth. This is an important result for implementation since the distance of the anchorage bars in the upper storey to the upper surface of the shear wall was significantly less than anchorage depth. Although no debonding was observed in the present study, a sudden debonding may develop when fracture cone reaches the upper surface of the shear wall due to faulty anchorage workmanship. Therefore, the construction of a shear wall higher (as much as the anchorage depth) than the frame will prevent possible debonding.

3 Results and discussion

– Strengthened frames remained within the elasticity limits and no system damage was observed even at a load level which is two times higher than lateral load carrying capacity of the bare frames. Shear damages were recorded on the columns and nodal points (in collapse mode) of the bare frames.

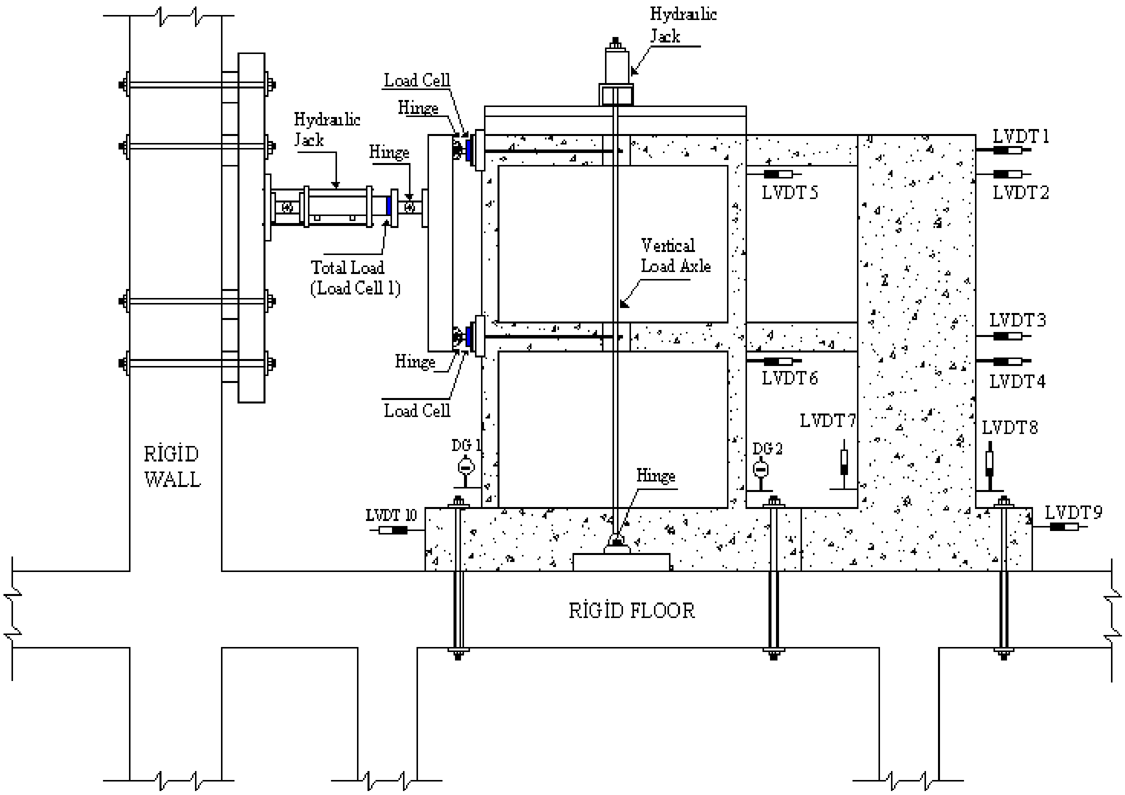


Fig. 7. Loading system of the 2nd group tests (G_2-S_3) (Ozturk, 2010).

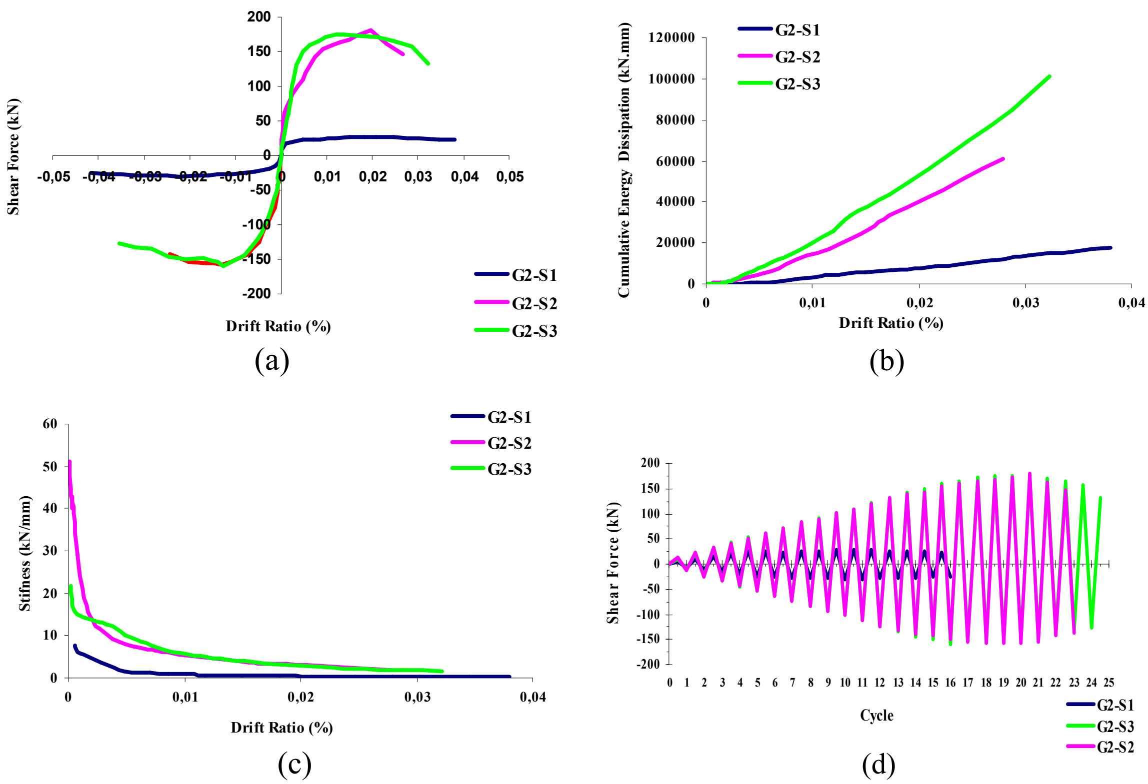


Fig. 8. Results of the 2nd group tests - **(a)** load-displacement envelope curve of specimens, **(b)** energy dissipation capacity graphs of specimens, **(c)** stiffness characteristics of specimens, **(d)** load histories of specimens.

(a) G2-S1 (b) G2-S2 (c) G2-S3

Fig. 9. Post-test views of the 2nd group test specimens (Ozturk, 2010).

– Lateral load carrying capacity increased after strengthening by approximately 3.78 fold in the 1st group tests and 5.80 fold in the 2nd group tests. A similar result was obtained in an energy dissipation capacity. When compared to the 1st group tests, post-strengthening energy dissipation was recorded as higher in the 2nd group tests.

– Load carrying capacity of the bare frames started to decrease in both groups when a maximum of 2% displacement ratio was exceeded. This shows that strengthening proves to be ineffective after a 2% global displacement. Many literature tests were completed at a maximum of 2% horizontal drift ratio (Kara, 2006; Sozen, 1987). In the tests carried out under the present study, on the other hand, loading continued to 4–5% displacement level, permitted by the measurement mechanism.

– The behaviour expected from the strengthened system was the ductile behaviour that was not as much as for the reference specimen. In the tests, the lateral load carrying capacity for the strengthened shear wall – frame systems (G_1-S_3, G_1-S_4) started to lose after an approximate 2% top displacement ratio whose basic reason was due to the anchorage reinforcement debonding as a result of insufficient lap splice length at the foundation – shear wall joint. Therefore, a considerable capacity loss was observed for a strengthening process applied by external shear wall after 2% cumulative displacement ratio.

– In the shear wall systems (where shear wall is adjacent to the frame), debonding was being observed particularly after 2% displacement ratio in the shear wall strengthening anchored to the ground. In the specimens (equipped with RC coupling beams), debonding was observed in the coupling beam-shear wall connections. Therefore, what actually limits shear wall performance is the anchorage serving as frame-wall connector.

– Although loading mechanisms were different, a general improvement was observed in the behaviours of both groups. Since the tests were conducted on the plane frame, special attention should be paid to the calculation of the actual torsion effect (estimated in the 3-D system) which will occur due to shear wall application.

– Shear wall height/length (h_w/l_w) ratio was taken as 3 in the tests. This ratio will be apparently much higher in a real RC building. In this case, shear wall will reach bending capacity faster than shear capacity, therefore, bending mode will be more determined by the behaviour.

– Both groups of strengthened systems can be applied on the non-adjacent RC buildings, the columns of which are located on the external axes. These systems are preferred in the 2nd group tests when there is passenger traffic particularly on the sidewalks and when the lower storeys are used for commercial purposes.

4 Conclusions

This study analysed the changes observed in the behaviours of the RC frames (equipped with the design and implementation defects commonly encountered in the buildings in Turkey and having weak poor earthquake behaviour) after they were strengthened with external shear walls with and without coupling beams. The tests give an overview of the structural behaviour of bare frames and strengthened frames with external RC-SW. The study also presents the results obtained at the end of the tests. Mixed system, established by applying external shear wall as one-side strengthening on the RC frame-type structures, significantly increases lateral load strength, rigidity and energy dissipation capacity of the bare frame. Test results showed that maximum lateral load capacity, initial rigidity and energy dissipation behaviours of the samples strengthened with external shear wall were much better than those of the bare frames. In addition to the structural parameters, the following should be considered in the strengthening of existing RC buildings;

– Usability of the building after strengthening.

– Extra internal and external damage costs to occur during strengthening.

– Difficulties to be experienced during strengthening application, necessity of the user to leave the building for a long time during strengthening.

– Duration of strengthening, etc.

This study showed that strengthening and system improvement of the RC buildings, via low-cost external shear wall application with and without coupling beam, not only brings about ease of construction and use but also provides the system with significant behavioural improvements, strength and rigidity.

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References

Anil, Ö. and Altin, S.: An Experimental study on reinforced concrete partially infilled frames, Eng. Struct., 29(3), 449–460, March 2007.

Anil, Ö., Altin, S., and Kara, M. E.: Strengthening of RC nonductile frames with RC infills-An experimental study, Cement Concrete Comp., 30(7), 612–621, August 2008.

Arslan, M. H. and Korkmaz, H. H.: What is to be learned from damage and failure of reinforced concrete structures during recent earthquakes in Turkey?, Eng. Fail. Anal., 14, 1–22, 2007.

Arslan, M. H.: Strengthening non-ductile reinforced concrete frames by columns failure in tension by appending external shear wall, Ph.D. thesis, Selcuk University, Konya, Turkey, 2007.

Arslan, M. H., Yuksel, I., and Kaltakci, M. Y.: An Investigation on global ductility of strengthened 2 story 2 bay RC frames, Struct. Build. J., 163(SB3), 177–194, 2010.

Bal, I. E., Crowly, H., Pinho, R., and Gülay, F. G.: Structural characteristics of Turkish RC building stock in Northern Marmara Region for loss assesment applications, IUSS Press, 2007.

Bruneau, M.: Building damage from the Marmara, Turkey Earthquake of August 1999, J. Seismol., 6, 357–77, 2002.

Canbay, E., Ersoy, U., and Ozcebe, G.: Contribution of reinforced concrete infills to seismic behaviour of structural systems, ACI Struct. J., 100(5), 637–643, September–October 2003.

Canbay, E., Ersoy, U., and Tankut, T.: A three component force transducer for reinforced concrete structural testing, Eng. Struct., 26, 257–265, 2004.

Cannon, R. W.: Straight talk about anchorage to concrete-Part 1, ACI Struct. J., 92(6), 580–586, 1995.

Chan, C. M., Ning, F., and Mickleborough, N. C.: Lateral stiffness characteristics of tall reinforced concrete buildings under service loading, Struct. Des. Tall Buil., 9, 365–383, 2000.

Dogangün, A.: Performance of reinforced concrete buildings during the May 1. 2003 Bingol Earthquake in Turkey, Eng. Struct., 26, 841–856, 2004.

Erdem, I., Akyuz, U., Ersoy, U., and Ozcebe, G.: An Experimental study on two different strengthening techniques for RC frames, Eng. Struct., 28(13), 1843–1851, November 2006.

Hakan Arslan, M.: Application of ANN to evaluate effective parameters affecting failure load and displacement of RC buildings, Nat. Hazards Earth Syst. Sci., 9, 967–977, doi:10.5194/nhess-9-967-2009, 2009.

Kamanli, M. and Balik, F. S.: The behaviour of roof gable walls under the effect of earthquake load, Nat. Hazards Earth Syst. Sci., 10, 251–263, doi:10.5194/nhess-10-251-2010, 2010.

Kara, M. E. and Altin, S.: Behavior of reinforced concrete frames with reinforced concrete partial infills, ACI Struct. J., 701–709, 2006.

Kaltakci, M. Y., Yavuz, G., and Arslan, M. H.: The effect of column cross section to frame ductility in RC frames having poor seismic performance, J. Eng. Appl. Sci., 2(10), 1524–1532, 2007.

Kaltakci, M. Y., Arslan, M. H., Yilmaz, U. S., and Arslan, H. D.: A new approach on the strengthening of primary school buildings in Turkey: An application of external shear wall, Build. Environ., 43, 983–990, 2008.

Ozcebe, E., Ersoy, U., Tankut, T., Erduran, E., Keskin, O., and Mertol, C. H.: Strengthening of brick infilled RC frames with CFRP, Technical Report, Middle East Technical University (METU), Structural Laboratory, TUBITAK-METU Press, 2003.

Ozturk, M.: Strengthening of reinforced concrete frames of insufficient earthquake resistance by applying external shear wall with coupling beam, Ph.D. thesis, Selcuk University, Konya, Turkey 2010.

Sezen, H., Whittaker, A. S., Elwood, K. J., and Mosalam, K. W.: Performance of reinforced concrete buildings during the August 17. 1999 Kocaeli Turkey Earthquake and the seismic design and construction practice in Turkey, Eng. Struct., 25, 103–14, 2003.

Sonuvar, M., Ozcebe, G., and Ersoy, U.: Rehabilitation of reinforced concrete frames with reinforced concrete infills, ACI Struct. J., 101(4), 401–500, July–August 2004.

Sozen, M. A.: Toward a behaviour based design of reinforced concrete frames to resist earthquakes, in: Proceedings of The Ninth Technical Conference of Turkish Society of Civil Engineering, V1, Ankara-Turkey, 1–47, 1987 (in Turkish).

TEC-2007: Turkish Earthquake Code, Regulations on structures constructed in disaster regions, Ministry of Public Works And Settlement, Ankara, 2007.

Zhao, Q. and Astaneh, A. A.: Cyclic behaviour of traditional and innovative composite shear walls, J. Struct. Eng.-ASCE, 130(2), 271–284, February 2004.