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An investigation of anchorage to the edge of steel plates bonded to RC structures

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Abstract. This paper presents the results of an experimental study investigating the effects of anchorage systems used in externally bonded steel plates on the strength and ductility of reinforced concrete structures. In the literature, diagonal steel plates bonded to frames were designed to be more flexible than the connections to eliminate the possible effect of the connection flexibility. However, to better evaluate the performance of the strengthened structures, the strength and behavior of connections should also be considered. The purpose of this study was to experimentally investigate the effects of different connection types of steel plates bonded to the frame using anchors on the strengthened RC structures. For this purpose, eleven specimens were designed to simulate the interior and exterior connection behavior. Two of these were used as the control beams and remaining nine for the investigation of the functionality of the end steel plates. Experimental results show that the load carrying capacity of the strengthened beams is directly related to the connection types of the steel plates. For the interior connections, L-shaped end plates that were strengthened using steel anchors must have adequate stiffness to prevent its shape. While, for the exterior connections, the connection with three anchors carried more load than the other exterior connections.

Keywords: anchorage; reinforced concrete; strengthening; externally bonded steel plates

1. Introduction

ver a period of use, reinforced concrete (RC) structures may lose their mechanical properties due to some detrimental effects. Not only their performance needs to be maintained based on their purpose of design but they are also expected to meet new performance criteria suggested by contemporary seismic design codes. In such cases, existing RC structures need to be repaired and strengthened. Over the last 50 years, various methods have been proposed by engineers and researches for the repair and strengthening of these structures and the efficiency of these methods has been discussed in the literature (Swamy 1989, Macdonald and Calder 1982, Hollaway and Leeming 2000, Kara and Bayat 2011).

Fiber Reinforced Polymer (FRP) and steel plates (sheets or strips) mounted to the existing internal or external masonry walls or RC shear walls is the most widely method used for the repair

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and strengthening of RC structures (Saadatmanesh *et al*. 1991, Almusallam and Al-Salloum 2001, Arslan *et al*. 2006, Altin *et al*. 2013, El-Hacha and Aly 2013, Cao *et al*. 2014, Anıl and Yımaz 2015). However, despite the advantages of FRP in terms of providing high strength, corrosion resistance and easy application, the potential non-ductile behavior of RC structures strengthened using FRP has made these plates less appealing. In addition, the cost of steel plates compared to that of FRPs is relatively low and the ductile structural performance achieved using the steel plates is much higher when compared to the use of FRP. Taghdi *et al*. (1998) and Altin *et al*. (2007) reported that the seismic load carrying capacity of RC frames significantly increased after strengthening with diagonal steel strips bonded to the infill walls of an RC frame; on the other hand, the relative horizontal floor displacements decreased to a certain extent. These studies showed that the RC structures strengthened/retrofitted using steel plates (Fig. 1(a)) failed due to either the buckling of steel plates or crushing of concrete at the connection between the steel strips and the concrete. The failure patterns of the RC frames due to the buckling of steel plates and the crushing of the concrete are presented in Figs. 2(a) and (b), respectively. This type of behavior leads to brittle failure (Jones *et al*. 1982, 1988, Swamy *et al*. 1989, Oehlers 1992, Hussain *et al*. 1995). These structures, therefore, do not meet the criteria of performance-based earthquake codes suggesting that all structure on the earthquake prone zone should be capable of accommodating substantial inelastic deformation without significant loss of strength on the loading capacity (TEC 2007).

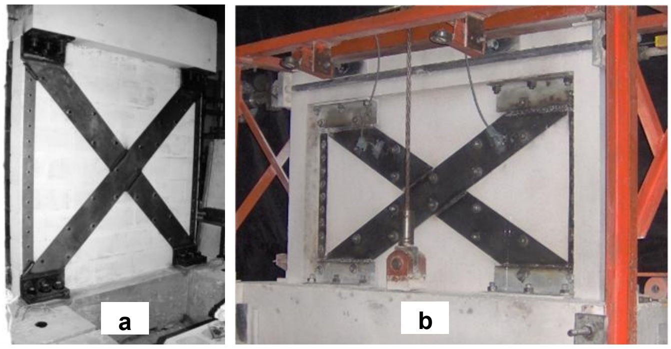


Fig. 1 A wall retrofitted with steel strips; (a) Taghdi *et al*. 1998; (b) Altın *et al*. 2007

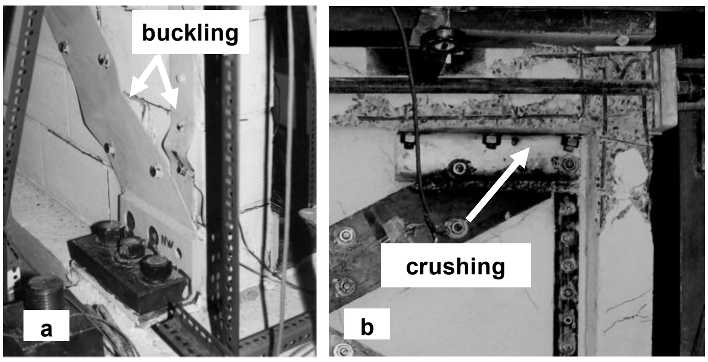
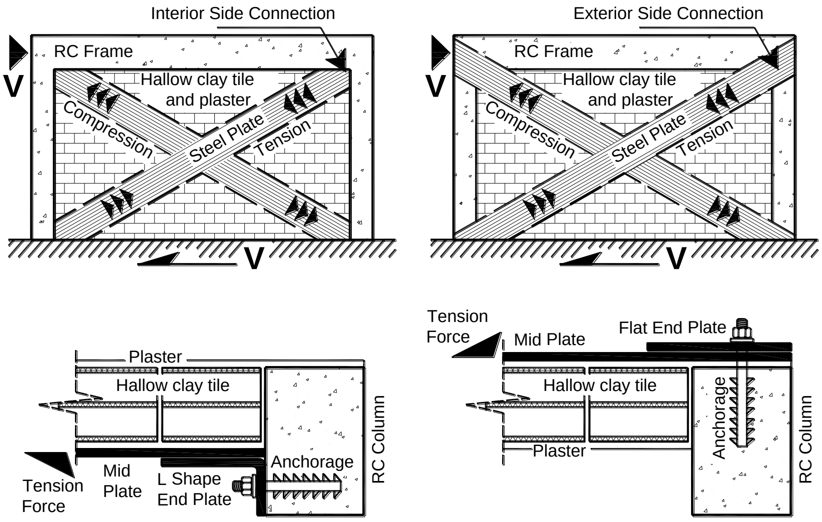


Fig. 2 The failure pattern of the RC wall retrofitted with steel strips;
(a) Taghdi 1998; and (b) Altin *et al*. 2007

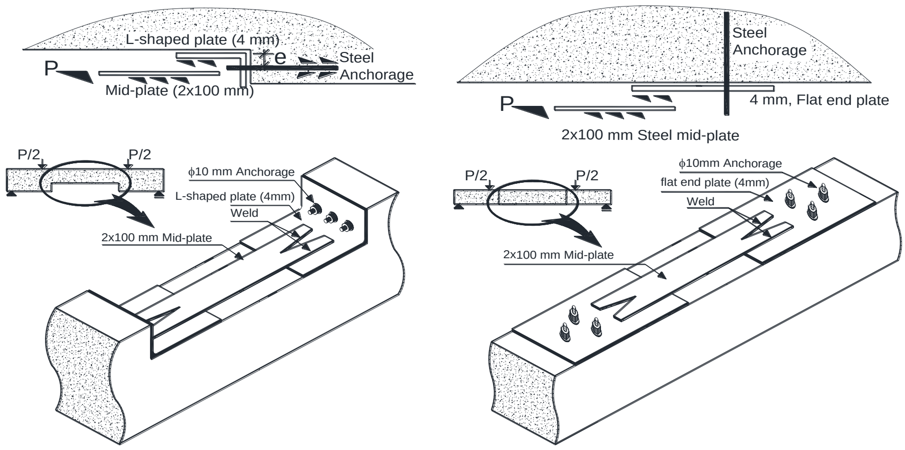


(a) Interior side connection (b) Exterior side connection

Fig. 3 The steel anchorage application for the construction of the RC frame

A commonly used method for the construction of an RC frame involves the use of a masonry brick wall as illustrated in Fig. 3(a). The details of the outer and inner connections used in the strengthening of the RC frame using diagonal steel plates are not identical due to the masonry walls not being symmetrically constructed with respect to the plane of bending. In these applications, the steel plates are generally constructed using three components; a mid-plate and two end plates. The end plates are flat on the outer side of the connection while the end plates on the inner side are in an L-shaped form. These two types of end plates coexist in the connection as shown in Fig. 3(b). The mechanical behavior of flat and L-shaped end plates may not be similar since they are bonded to the concrete using different types of anchorage. Anchors fastening the flat end plate to the concrete are subjected to shear force whereas those fastening the L-shaped end plate are under tension force.

Most studies conducted on the strengthening of RC structures using steel plates bonded to the infill walls have concluded that the success of the application heavily depends on the success of the mechanical fixing of the strengthening system. Therefore, the main purpose of the current experimental study was to investigate the effect of flat and L-shaped end plates on the behavior, strength and ductility of the RC frames. For this purpose, two different types of RC beams (notched and unnotched) were designed (Fig. 4) to simulate the different connection details of the inner and outer sides of the frame (Fig. 3). The notched beam represents the inner side connection whereas the unnotched beam is for the outer side connection. Nine different types of end plates that are generally used in the strengthening of frames were tested. Throughout the experiments, the dimensions and reinforcement of all beams were the same and the dimension of mid-plates was maintained. Eleven beam specimens were designed; two as the control beams and the remaining nine for the investigation of the functionality of the end plates. The beam specimens were tested under the influence of a four-point loading on an experimental set-up prepared for this study. This way, it was possible to explore the connection types that fulfilled the design purpose in terms of provide the yielding of the mid-plate.



(a) Simulation of interior connection (b) Simulation of exterior connection

Fig. 4 Anchorage application in the tested beams

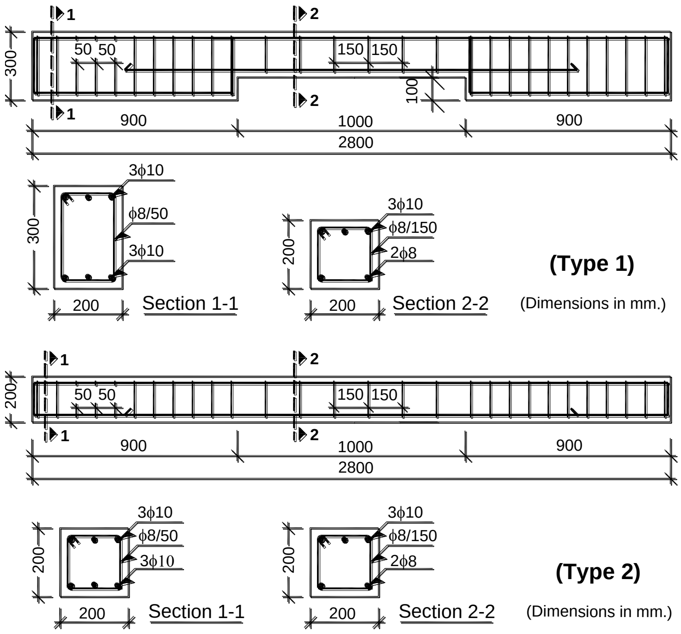


Fig. 5 The dimension and reinforcement details of the test beams

2. Experimental study

Eleven 2.8 m long RC beams with a 2.6 m center-to-center span between supports were constructed. Fig. 5 presents the geometrical properties and reinforcement details of all beams. There was a symmetrical notch of 100 mm depth and 1000 mm length in the middle of the Type 1 beams. The cross-section of all beams were 200 × 200 mm^2 in the notched section and 200 × 300 mm^2 in the unnotched section. There was no notch in Type 2 beams. Previous research conducted in Turkey on the RC buildings requiring repair and strengthening showed that the average value of in-situ concrete compressive strengths (8 MPa) is less than the values proposed in the relevant standards and regulations. In addition, the compressive strength of the concrete is not certainly known (Firat and Yucemen 2014). Therefore, in the present study, the average target compressive strength of the concrete was set approximately to 10 MPa for RC beams to be on the safe side.

In this experimental program, ribbed reinforcing steel bars with the diameters of 8 mm (ϕ8) and 10 mm (ϕ10) were used as reinforcement. The yield strengths (f_y) of the reinforcing steel bars were 572 MPa and 420 MPa, for ϕ8 and ϕ10 bars, respectively. In addition, the ultimate strength of steel bars (f_{ult}) with the diameters of ϕ8 mm and ϕ10 mm was obtained as 683 MPa and 686 MPa, respectively. Type 1 and Type 2 beams were reinforced with 2ϕ8 bars at the tension zone and 3 ϕ10 bars at the compression zone in the middle part of the beams.

In the experimental set-up, the notched section of the beam was planned to be under a constant positive moment zone. In the experiments, the horizontal leg of the L-shaped end plates was welded to the end of the mid-plate. The end of the mid-plate was formed in the shape of V to increase the welding length. The vertical leg of the L-shaped plates was drilled to allow the threated steel rod to pass through the leg.

In this study, different anchorage geometry of end plates with and without stiffener were also tested. The following subsections present the mechanical properties of anchors, steel plates and epoxy, the description of the test specimens and the bonding procedure.

2.1 Steel plates

The material properties of steel plates were determined with a special apparatus shown in Fig. 6(a). Using this apparatus, the millivolt (mV) strain gauge measurements were calibrated and the strains were converted. A steel specimen of a uniform cross section of 2 × 100 mm^2 and a length of 400 mm were used in the tests. The cross-section of the central portion of the specimen was reduced precisely to 2 × 20 mm^2 with a 250 mm gauge length in order to obtain a high stress region. Five strain gauges were positioned in the central part of the specimen as shown in Fig. 6(a). The elongation of the specimen was measured by two traditional linear vertical displacement transducers (LVDT) located on the two sides of the specimen. This way, it was possible to compute the strain dividing the elongation measured at the end of the test by the initial gauge length. The stress-strain curve obtained from the strain measurements are given in Fig. 7(a). The yielding

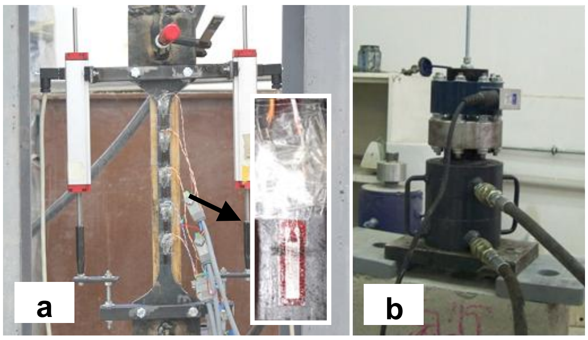
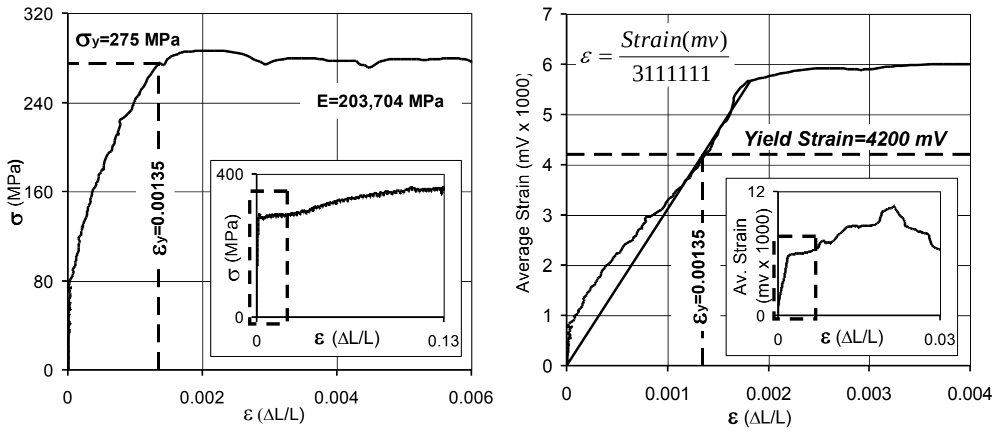


Fig. 6 Material test: (a) The steel tensile test for the strain calibration; (b) The pull-out test for anchors



(a) Stress-strain curve (b) Strain gauge - strain calibration curve

Fig. 7 The steel plate stress-strain ($\sigma - \varepsilon$) and the strain gauge calibration graphs

stress of the specimen was found to be 275 MPa, the strain corresponding to the yielding stress was 0.00135 mm/mm and the modulus of elasticity was 203.7 GPa. The specimen was stretched until the strain reached 1.3%, at the point of which the stress was less than the rupturing strength.

Five strain gauges of 10 mm length and 120 ohm resistance were mounted to the steel mid-plate to measure the strain. The average of the five strain measurements in terms of millivolt was calculated, then matched with the strains computed in accordance with the LVDT measurements. Fig. 7(b) presents the results. In this figure, the straight line deviates from the linear behavior after the average strain of 5600 mV and the average elongation of 0.0018 mm/mm. This is due to either the specimen section elongating more than the other parts or some strain gauges not being properly mounted to the surface of the specimen. However, the yield strength is clearly seen in Fig. 7(a). The strain corresponding yielding was observed in the section, where the measurements matched the linear behavior, as given in Fig. 7(b). Consequently, it was determined that the specimen under the tension force yielded a strain value of 4200 mV and yield strain of 0.00135. A linear relation between the strain obtained by the LVTDs and the strain obtained from the strain gauge was assumed to calculate the strain values less than the yielding strain. The stress on the plate was computed using this linear relation, dividing the millivolt measurements of the elongation by the value of 3.11×10^6 based on the slope of the line. As a result of the uniaxial tensile test, the yielding and the ultimate strength of the steel plate were determined as 275 MPa and 360 MPa, respectively.

2.2 Anchors

The flat and L-shaped steel end plates were bonded to the concrete using steel-threaded anchors. Pull-out tests were performed to determine the required anchor diameter and development length (Fig. 4(b)). Different tests were carried out for bolts with the outer diameters of 8, 10 and 12 mm and different embedment lengths. Based on the test results, it can be concluded that anchors with a 10 mm outer diameter (8 mm thread diameter) should be embedded 150 mm into the holes with a diameter of 14 mm. The yielding load and yielding strength of 10 mm anchors were determined experimentally as 34 kN and 677 MPa, respectively.

2.3 Epoxy

Adhesives such as epoxy are generally used to provide a shear connection between similar or dissimilar materials, enabling the components being bonded to act as a composite structural unit (Halloway and Leeming 2000). A commercial two-component epoxy, YKS Concresive 1406 (YKS-Concresive 1406), was used to bond the steel end plates to the beam bottom surface and fix the steel bolts anchors to the concrete. The mechanical properties of the epoxy supplied by the manufacturer were as follows; compressive strength = 75 MPa, flexural strength = 25 MPa, bonding strength to steel = 6.5 MPa, and bonding strength to concrete = 3.5 MPa.

2.4 Beam specimens

As mentioned before, two different series of beam specimens were constructed to simulate inner and outer connections. In the first series (Type 1), the behavior of the L-shaped end steel plate used in the notched beams was investigated. The geometry of the connection detail of the end plates and the number of anchors were determined as the experimental parameters. Seven test specimens given in Fig. 8 were constructed for Type 1 specimens.

Specimen-1 was the control specimen with no strengthening and was used to interpret the behavior of the other test specimens that were strengthened using steel plates. In Specimen-2, the horizontal leg of the L-shaped end plates was welded to the middle plate. The vertical leg of the end plate was connected to the beam with three steel anchors of ϕ10 mm. The number of anchors was determined based on the results of the pull-out tests. Three bolts with a ϕ10 mm diameter are supposed to withstand the axial tensile force due to the bending moment without the failure of the anchors. Anchors in Specimen-2 (Fig. 4(a)) were drilled parallel to the beam axis with a 15 mm eccentricity due to the application position of the drilling machine. Specimen-3 was different from Specimen-2 only in terms of the positioning of the anchors. To eliminate the eccentricity problem experienced in Specimen-2, the anchors in Specimen-3 were mounted to the corner of the L-shaped end plate at a 45^o angle to the beam axis. While performing the experiments on Specimen-2 and Specimen-3, large deformations occurred in the L-shaped end plates. Therefore, the L-shaped end plates of the test Specimens 4 and 5 were reinforced using stiffeners. In Specimen-4, the vertical leg of the L-shaped end plate was further reinforced using an additional header plate with a thickness of 10 mm, a height of 96 mm and a width of 200 mm. In Specimen-5, however, the L-shaped end plates were strengthened using two stiffener plates in the form of a triangular with a height of 96 mm, a thickness of 4 mm and a width of 150 mm to keep the 90° angle between the two legs of the L-shaped end plates.

For the strengthening of Specimens 2 to 5, the following procedure was employed. First, the L-shape end plates and the $2 \times 100\ \mathrm{mm}^2$ middle steel plate were bonded to the notched section of the specimens using epoxy with a thickness of 2 mm and 4 mm, respectively. Then, the two ends of the mid-plates were welded to the horizontal legs of the L-shaped end plates. The horizontal legs of the L-shaped end plate were placed on the top of the end of the mid-plates. The common practice is; first bonding the middle steel plate to concrete then placing the end plate on it. The connection mechanism was similar in Specimens 4, 6 and 7. The difference in Specimen-6 was that the L-shaped end plate was placed on the steel mid-plate. The horizontal leg of the L-shaped end plates was formed in a V shape to provide a sufficient length for welding. The difference in Specimen-7 when compared to Specimen 4 was the number of anchors that were used. In Specimen-7, two anchors were used rather than three.

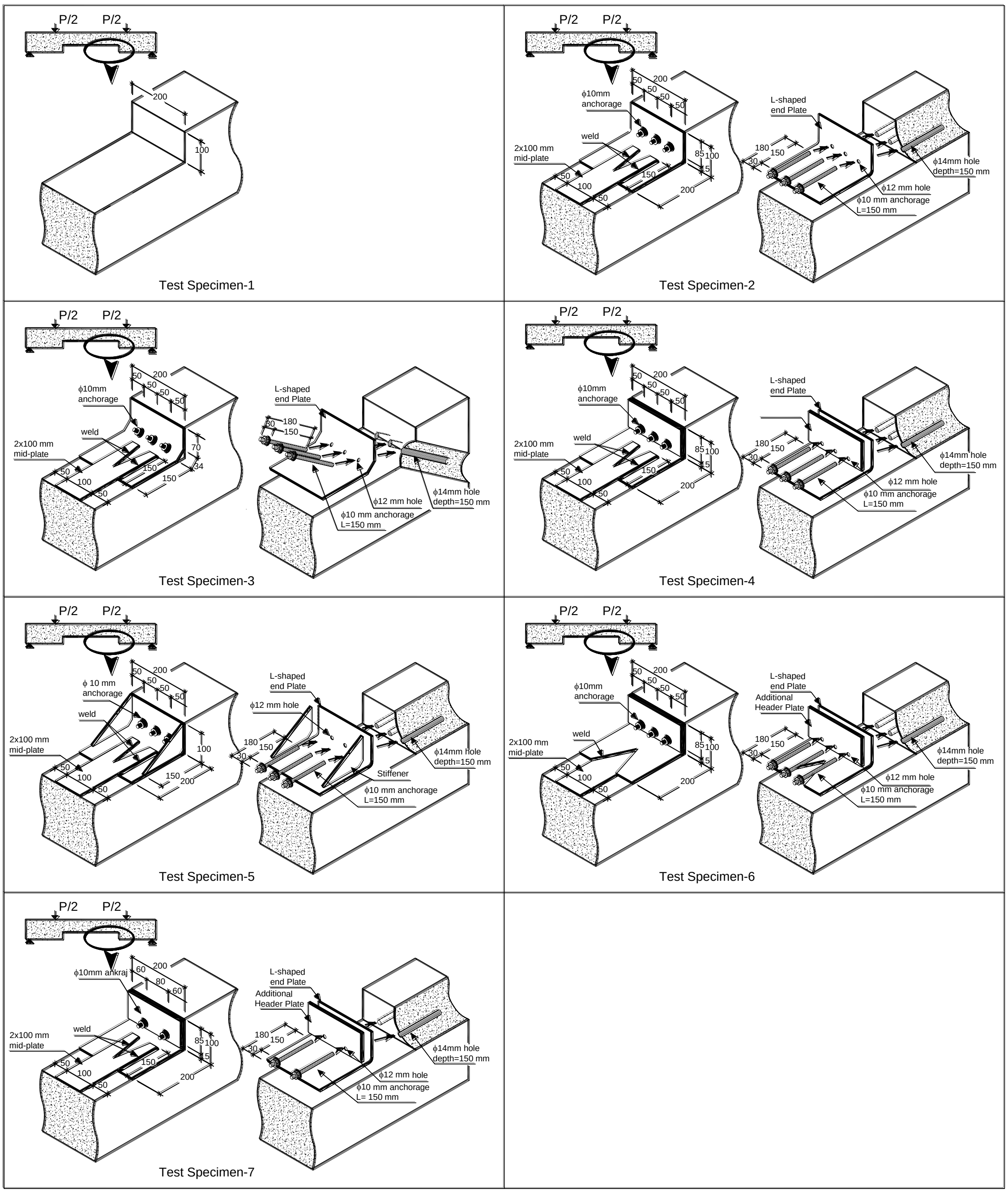


Fig. 8 Details of the Type 1 test specimens

In the Type 2 beams that were constructed to simulate outer connections, the mid-plates were fastened to the bottom surface of the unnotched beams for bonding to the outer side of the infill walls of the frames. The connection details used in this test specimen are given in Fig. 9. Here, the axis of anchors is perpendicular to the axis of the beam. The Specimen-8 is a control specimen without the strengthening plate. In Specimen-9, the mid-steel plate without the flat end plates was directly fastened using only one anchor. In contrast, in Specimen-10, two anchors were used. In Specimen-11, a mid-plate was welded to the two flat end plates with a $4\ \mathrm{mm}$ thickness, $200\ \mathrm{mm}$ width and $350\ \mathrm{mm}$ length. The properties of all test specimens are summarized in Table 1.

2.5 Bonding procedure

All the test specimens were strengthened based on the following procedure. The beam specimens were cured under laboratory conditions stored for 28 days to ensure that the concrete had a sufficient strength. First, holes with a diameter of $14\ \mathrm{mm}$ were drilled on the beam samples for the placement of the anchors. Then, the concrete surface was mechanically roughened by sandblasting until the aggregates were formed. Following this process, the entire surface of the beams was cleaned of all loose materials and dust using compressed air to ensure good bonding between the concrete and the steel plates.

Epoxy was applied to the bottom surface of the beams in a uniform thickness, and the holes that had been drilled for anchors were filled and the steel plates were bonded to the surface. The heads of the steel anchors were wrapped with tape to prevent smearing. Then, the anchors were placed in the holes. A certain amount of pressure was applied to the steel plate in order to ensure that the epoxy resin was uniformly distributed for full bonding with the steel plates and the concrete surface. The beam specimens were left to cure for 7 days and then welding was performed. During the welding process, the welding temperature was continually observed for the epoxy not to be adversely affected by the temperature. After the completion of the welding application, the protection tapes covering the end of the steel anchors were removed and anchors were installed using bolt nuts. The bonding procedure for Type 1 and Type 2 specimens are given in Fig. 10.

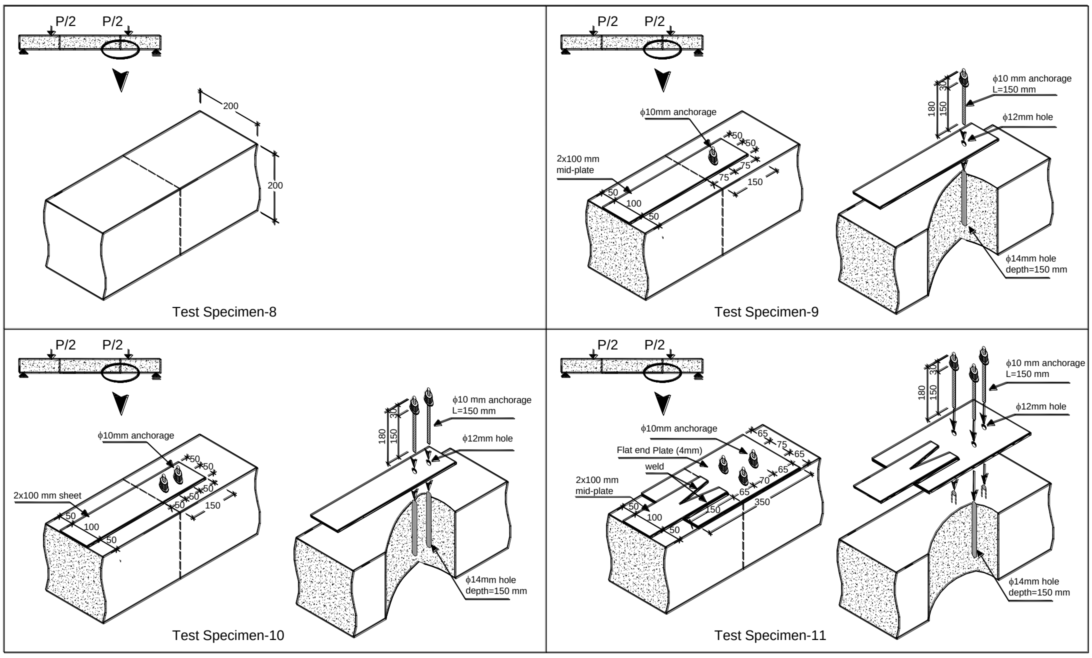


Fig. 9 The details of the Type 2 test specimens

Table 1 The properties of test specimens

Specimen No.	100×2 mm steel mid-plate	Notch (100 mm)	Number of Anchors	Angle of the Anchors	End Plate Thickness = 4 mm	Stiffener Plate Thickness = 4 mm	Additional Plate Thickness = 10 mm	Concrete *fc*, (MPa)
1 (Ref.)	-	Yes	-	-		-	-	11
2	Yes	Yes	3	0	Yes	-	-	10
3	Yes	Yes	3	45	Yes	-	-	12
4	Yes	Yes	3	0	Yes	-	Yes	10
5	Yes	Yes	3	0	Yes	Yes	-	12
6	Yes	Yes	3	0	Yes	-	Yes	11
7	Yes	Yes	2	0	Yes	-	Yes	13
8 (Ref.)	-	-	-	-	-	-	-	10
9	Yes	-	1	90	-	-	-	12
10	Yes	-	2	90	-	-	-	10
11	Yes	-	3	90	Yes	-	-	13

Fig. 10 Bonding procedure of test specimens

2.6 Experimental setup

Fig. 11 presents the experimental set-up and instrumentation. Using a reaction beam with a distance of 1300 mm between the supports, loading was performed on two points. A hydraulic jack (500 kN capacity) and a hand-operated hydraulic pump. The applied load was measured using a load cell with the capacity of up to 100 kN. Specimens were tested under cyclic loading. Load cycles were applied to specimens under load control up to the maximum load capacity and the displacement control was applied for further cycles. Each cycle was repeated once and a 5 kN

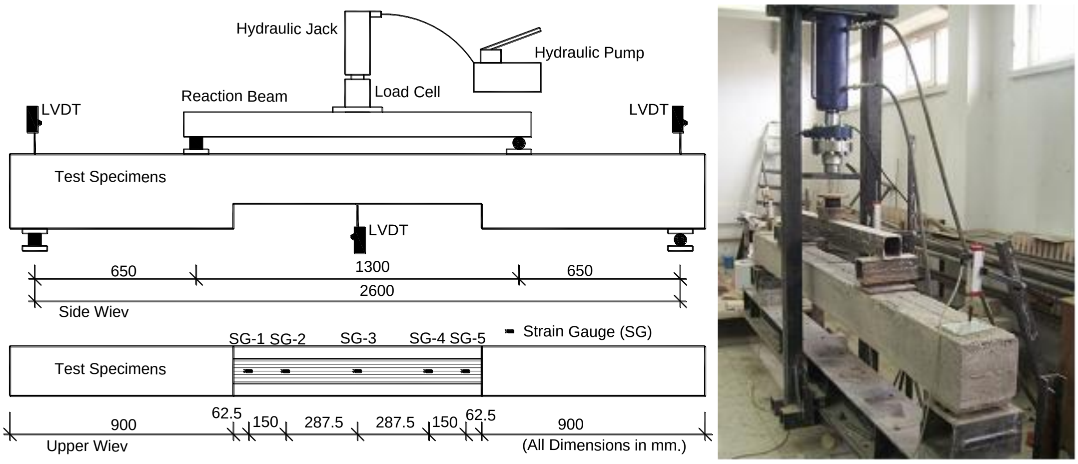


Fig. 11 The experimental setup and instrumentation

load increment was applied for the load controlled cycles. LVDTs were used to measure the displacements. Five strain gauges were placed on the potential locations of failure. The location of the strain-gages is shown in Fig. 11. All data obtained from these gages were recorded using data loggers.

3. Experimental results and discussion

3.1 Behavior of the test specimens and failure modes

Table 2 presents the strain and stress on the steel mid-plates and the failure mode of the beams. Figs. 12 and 13 also give the variation of the point load that was applied to the reaction beam with the maximum beam deflection measured using the LVDT for Type 1 and Type 2 specimens, respectively. Fig. 14 presents the photos of the beams taken after the experiment. The most successful result regarding the yielding of the steel plate without any damage at the end of the connection was investigated.

The test results of Type 1 and Type 2 specimens were separately evaluated. The first specimen of Type 1, Specimen-1, carried a 24 kN load and exhibited a ductile behavior as expected, then it failed with the crushing of the concrete in the compression zone after the 70 mm deflection. Specimen-2 had two L-shaped end plates with three anchors. The load carrying capacity of this test specimen was found to be 54 kN. When the applied load exceeded 40 kN, the rigidity of the specimen considerably decreased. At this load level, the vertical leg of the L-shape end plate bent due to the eccentricity of 17 mm as shown in Fig. 14. The steel mid-plate did not yield and the measured maximum stress on the mid-plate reached 66% of its yield tensile strength. In addition, it was observed that the thickness of the L-shaped end plate was not adequate in terms of rigidity.

The anchors in Specimen-3 were placed on the corner of the L-shaped end plate with a slope of 45° to prevent eccentricity, which results in a significant deformation on the vertical leg of the L-shaped end plate. As seen in the graph (Fig. 12) of load and displacement of Specimen-3, after reaching its maximum load carrying capacity of 43 kN, the stiffness of this test specimen gradually decreased . The main reason for this behavior is that the axis of the anchors did not carry the load on a slope of 45° since the concrete was not strong enough. Steel anchors did not carry the expected load in the direction of its axis. The steel plate did not yield and the total load induced to the mid-plate was about 47% of its yielding strength.

Due to the fact that deformation of vertical leg of shaped end plate in Specimen-2 has been observed, an additional header plate with the thickness of 10 mm was placed on the vertical leg of the L-shape end plate in Specimen-4. Since an additional header plate was used, the load carrying capacity was increased to 58 kN. The mid-plate started yielding at this load and no considerable change was observed in terms of the stiffness of Specimen-4 until the maximum load was reached. In Specimen-5, the L-shaped end plates were strengthened using two triangle stiffeners and the stiffness of the beam kept on slightly increasing. The load-displacement curve indicates that yielding started at 59 kN. After this point, the vertical leg of the L-shaped end plate was not able to maintain its form and the mid-plate also yielded.

In Specimen-6, the mid-plate was placed below the horizontal leg of the L-shaped end plate. This type of connection is commonly used. The process of strengthening in Specimen-6 was very similar to that of Specimen-4. The only difference between these two specimens was regarding the

Table 2 Test results

Specimen No.	Max. load (kN)	Max strain	Yield strain	Stress calculated (MPa)	Yield stress (MPa)	$\frac{\sigma}{\sigma_y}$	*Failure modes (Steel sheet)
	P	ε	ε_{sy} = 0.135%	σ	$\sigma_y = 275$	%	
1 (Ref.)	24						
2	54	0.00089	$< \varepsilon_{sy}$	182	Not yielded	66	Deformation of vertical leg of shaped end plate
3	43	0.00064	$< \varepsilon_{sy}$	129	Not yielded	47	Local failure on concrete
4	58	0.00301	$> \varepsilon_{sy}$	**275**	**Yielded**	100	Yielding of mid-plate
5	59	0.00296	$> \varepsilon_{sy}$	**275**	**Yielded**	100	Yielding of mid-plate
6	60	0.00293	$> \varepsilon_{sy}$	**275**	**Yielded**	100	Yielding of mid-plate
7	55	0.00101	$< \varepsilon_{sy}$	206	Not yielded	75	Stripping of the thread of the anchors
8 (Ref.)	22	-	-	-	-	-	-
9	41	0.00099	$< \varepsilon_{sy}$	201	Not yielded	73	Local failure on concrete
10	48	0.00103	$< \varepsilon_{sy}$	209	Not yielded	76	Local failure on concrete
11	50	0.00149	$> \varepsilon_{sy}$	**275**	**Yielded**	100	Local failure on concrete and yielding of the mid-plate

* At all specimen, tensile reinforcement is yielded, in notch. Failure modes is flexure

location of the mid-plate. With the use of this technique, eccentricity increased by 4 mm resulting in an additional moment. However, there was no significant change in the load carrying capacity of Specimen-4 and Specimen-6, in which the mid-plates of both specimens were observed to yield. The load displacement graphs (Fig. 12) show that Specimen-6 carried the same load as Specimen-4 with a little more displacement.

In Specimen-7, two anchors were used to fix the L-shaped plate to the concrete. Despite having less anchors than Specimen 4, Specimen-7 exhibited a behavior similar to that of Specimen-4 up to the load of 50 kN. On the other hand, in the subsequent cycles, the thread of anchors was damaged and the Specimen-7 failed. The stress at the maximum load level was 78% of the yielding strength of the mid-plate.

Specimen-8 classified in Type 2 was constructed as the control specimen. This specimen exhibited ductile behavior, had large displacement after the yielding of tensile reinforcement, and failure occurred due to the cracking of the concrete in the compression zone. In Specimen-9, the steel plate was directly connected to the beam using one anchor. The specimen exhibiting the strength of 41 kN collapsed at the level of maximum load, where the anchorage cracked the concrete along a direction perpendicular to its axis. The steel plate did not yield.

Specimen-10, in which two anchors were used in the connection, exhibited similar behavior compared to Specimen-9 with one anchor. Fig. 11 shows that the failure occurred on the left end where the anchorage was placed. After conducting tests on these two specimens, three anchors were used in Specimen-11, An additional header plate was also used to place these three anchors. When the loading level reached 50 kN, the steel plate yielded. At the end of the experiment, the end anchor cracked the concrete and the stiffness of the specimen significantly decreased.

3.2 Comparison of strains

The aim of this experimental research was to conduct an extensive investigation into the load carrying capacity of beams that were strengthened by steel plates bonded to the concrete at the end section. Therefore, the measurements of the strains of the test specimens were of key importance. The measurements in milivolt were calibrated to strain values based on the test results of the material. As explained in Section 2.1, the strain gauge measurement of the 4200 mV, which corresponded to the strains of 0.00135 mm/mm, was the yielding strain of the steel plates. Some representative load-strain curves measured on the mid-plates of different specimens were given in Fig. 16. The locations of the strain gauges labeled from SG-1 to SG-5 can be seen in Fig. 11. During the experimental work, 55 load-strain reading were obtained. But the lack of space, only four of them selected randomly were presented herein. Fig. 16(a) shows the variation of the concentrated load applied to Specimen-2 with the strain measured with SG-2. The reason of selection of this gauge location is that the maximum strain Specimen-2 reached its load carrying capacity (54 kN) without the yielding of the mid-plates. On the other hand, the curves in Fig. 16(b) and 16(c) demonstrate that Specimens 5 and 6 reached their load carrying capacity before the strain gauges detected any yielding of the mid-plate. As the deformation increased, the mid-plates yielded at the point where the strain gages, SG-3 and SG-5, were mounted. The curve given in Fig. 16(d) shows that the middle plate yielded and then due to one of the anchorage failure, strain and applied load could not be increased.

As mentioned before, five strain gauges were mounted to each mid-plate. Fig. 17 presents a comparison of the maximum strains of the steel plates obtained using these gauges. In this figure, the locations of the strain gauges and the maximum strain measurements are given in the *x*-axis

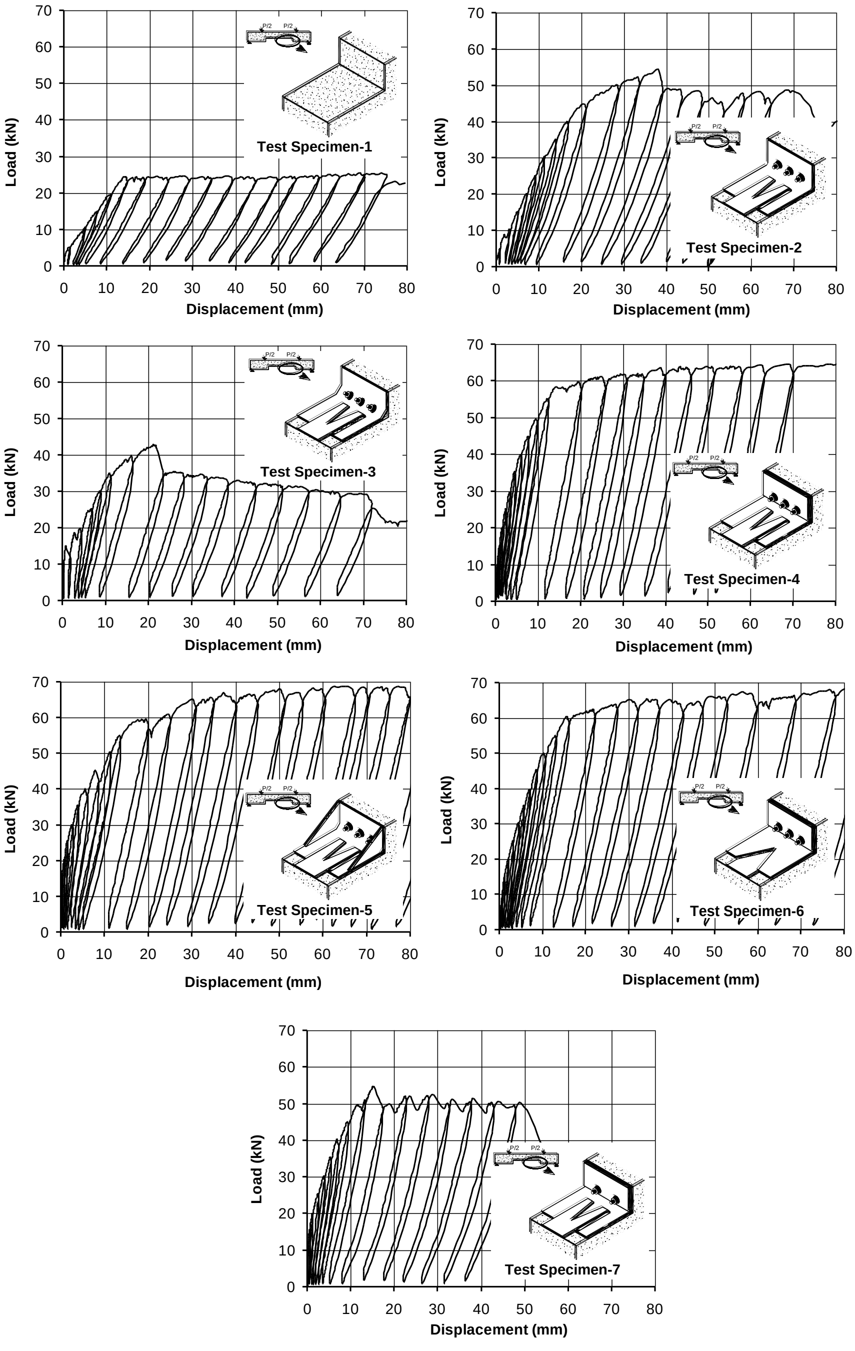


Fig. 12 The load-displacement curves of the Type 1 specimens

and *y*-axis, respectively. Fig. 17(a) shows that the steel plates in Specimens 2 and 3 did not yield but the connection failure lead to beam failure. The strains in Specimens 4 and 6 were above the yielding strain (Fig. 17(b)). This means that the anchorages fulfilled their design purpose. The

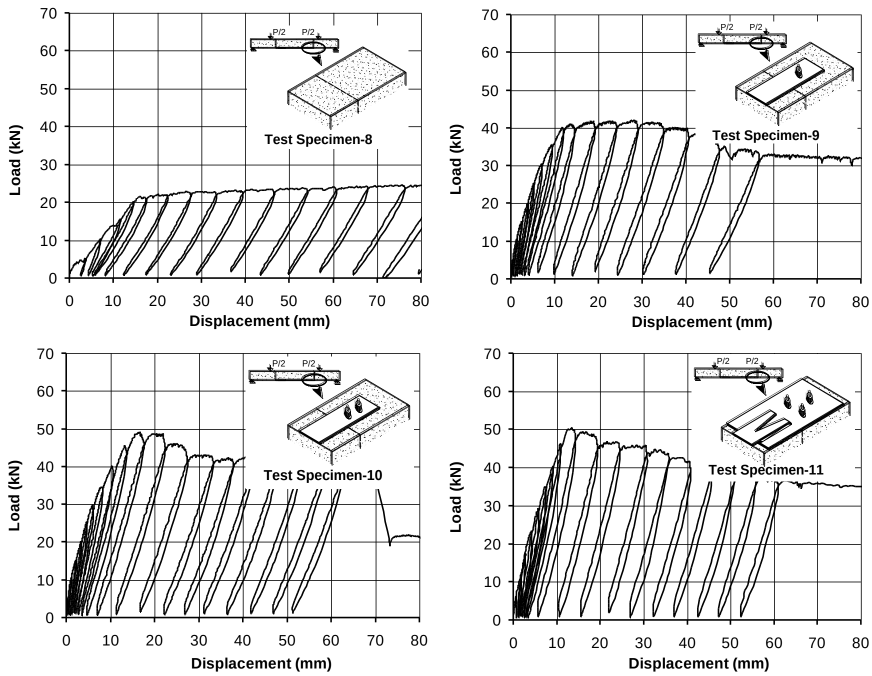


Fig. 13 The load-displacement curves of the Type 2 specimens

most successful connection types in this test program were found to be Specimens 4 and 6. In addition, even though the yield strength of Specimen-5 is relatively less than Specimens 4 and 6, Specimen 5 showed very good performance at the high strain levels.

In terms of the Type 2 specimens, only Specimen-11 achieved the yielding stress in the steel plate. However, failure of one anchorage which is near to the beam supports occurred in this specimen. The evaluation of the strains in Type 2 specimens indicates that the steel anchors were not functional against the forces in the direction perpendicular to the axes of the steel plates.

4. Conclusions

In the present study, the anchoring connections were used to strengthen the RC structures with steel plates. It can be concluded that the functionality of the strengthening of these steel plates integrated with the existing structures heavily depends on the connection detail between the existing structure and the strengthening material. In addition, the end anchoring of the steel plate bonded to the concrete was taken into consideration in the evaluation of the end connection based on the yielding of the steel plate. The results obtained by experiments can be summarized as follows:

- When the angle between the anchoring axis and the beam axis was about 45, the anchors were not able to carry the tension force they were expected to withstand axially. Although the initial rigidity of the beam that was strengthened using this type of connection was good, the strengthened plates were not able to carry the additional force when the anchoring failed due to the cracks once the bending moment started.

- The anchors placed perpendicularly to the beam axis were not able to carry the load proposed in the design. A local cracking was observed in all test specimens.
- It is crucial to ensure that the tension force due to the flexural effect of the bending moment is parallel to the axis of the anchors. In practice, this may not be possible due to the presence of reinforcement. Therefore, different anchoring angles should be used. In these cases, it is difficult to determine the ultimate capacity of anchors. When different angles are used for a variety of reasons, detailed material tests should be performed to consider the shear effects.
- In the notched beams, the thickness of the L-shaped end plates (4 mm) was not sufficient; hence, very large deformations took place in the vertical leg of the L-shaped end plate. To minimize this deformation, an additional plate with the thickness of 10 mm was placed on the surface of the leg of the L-shaped end plates that were strengthened using steel anchors. In the relevant studies, large size L profiles or very strong additional plates have been used for strengthening purposes. The use of an additional plate with a thickness of 10 mm was found to be sufficient to transfer the tension force from the anchors to the mid-plate.

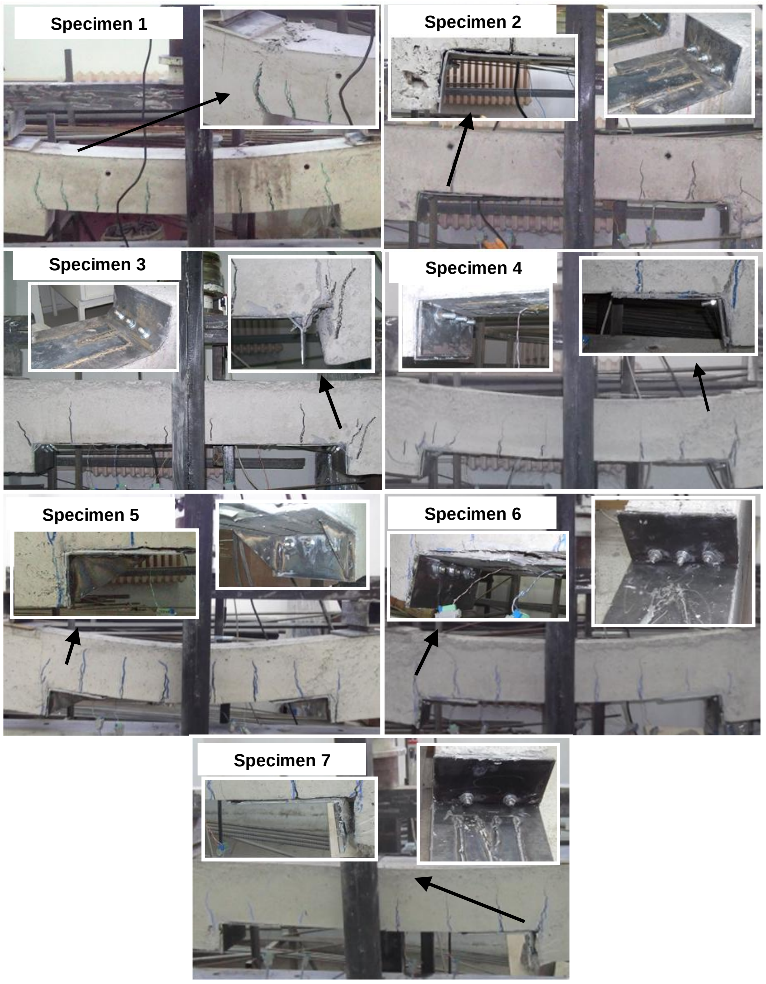


Fig. 14 The crack pattern after failure and the failure modes of Type 1 specimens

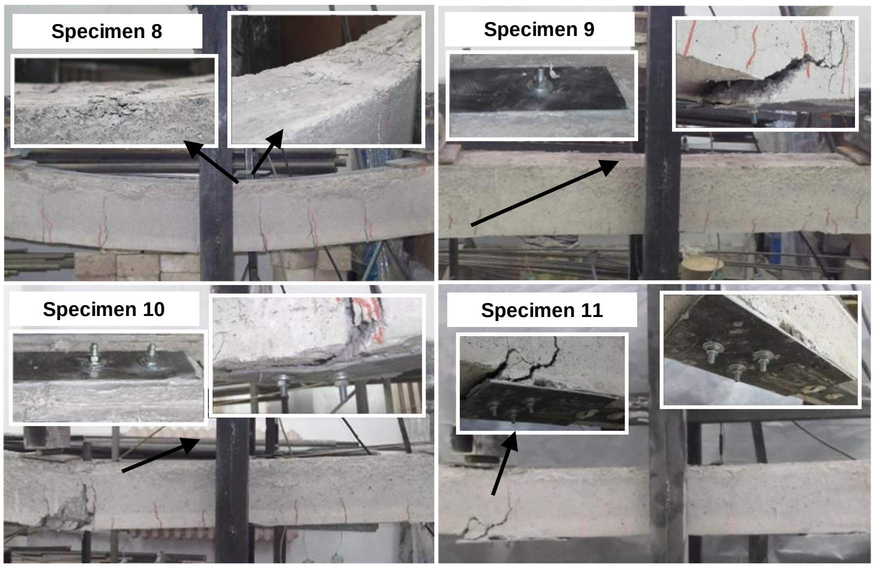


Fig. 15 The crack pattern after failure and the failure modes of Type 2 Specimens

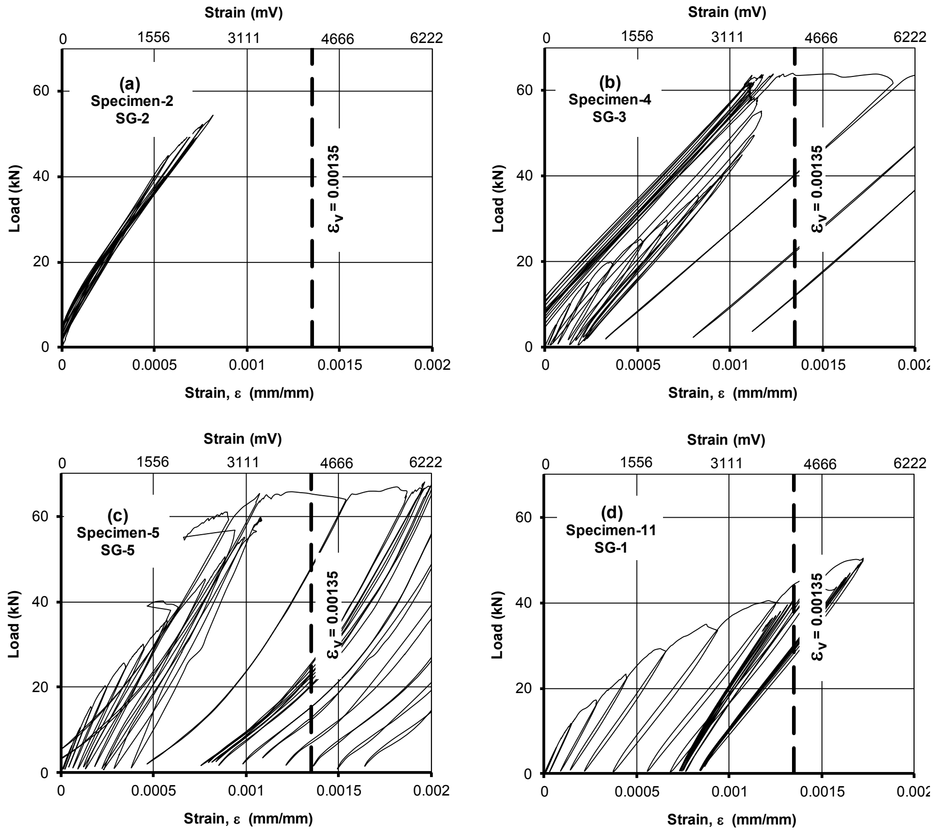


Fig. 16 The load-strain curves of some specimens

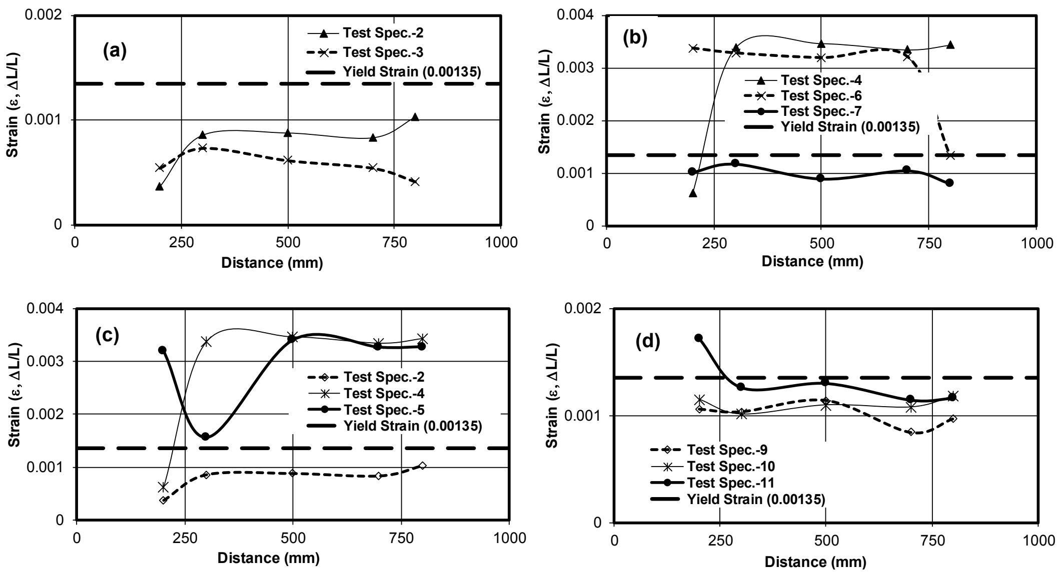


Fig. 17 The strain distribution of steel plates

- The use of two triangle stiffeners to strengthen the L-shaped end plates was only partially successful. Two stiffener plates were not sufficient to prevent deformation on the vertical leg of the L-shaped end plate. Despite reaching the yield strength of the mid-plate, a local failure occurred at the end of the steel plate. The implementation of a triangle stiffener was also quite difficult with regard to the workmanship.
- The horizontal leg of the L-shaped plate being below or above the mid-plate did not have a significant effect on the load carrying capacity of the beam. The mid-plate being below the end plate caused an increase in the bending moment due to eccentricity. This resulted in a decrease in the stiffness of the specimens.
- When one or two anchorages were perpendicularly connected to the beam axis to fix the strengthened steel plates, the thread of the steel bolts used in anchoring got damaged and could not transmit the load. On the other hand, when an end plate with the thickness of 10 mm was welded to the end of the middle plate and three anchors were used, the load carrying capacity of the beam increased compared with the connection using one or two anchors. It is noteworthy that as the number of cycling load increases, the stiffness of the beams tends to decrease.
- Considering all strengthening cases, the best interior connection was obtained from the L-shaped end plate using an additional head plate and three anchors. Among the exterior connections, the connection with three anchors carried more load than the other exterior connections.

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