2011

Rehabilitating destructed reinforced concrete T connections by steel straps

Muhammad N. S Hadi
University of Wollongong, mhadi@uow.edu.au

Publication Details
Hadi, M. N. (2011). Rehabilitating destructed reinforced concrete T connections by steel straps. Construction and Building Materials, 25 (2), 851-858. >

Rehabilitating destructed reinforced concrete T connections by steel straps

M.N.S. Hadi
School of Civil, Mining and Env Eng
University of Wollongong, Wollongong, NSW 2522, Australia
Tel: +61 2 4221 4762, Fax: +61 2 4221 3238
email: mhadi@uow.edu.au

ABSTRACT

The aim of this paper is to present results of testing a full scale reinforced concrete T connection by static loading. The connection represents a beam-column connection. The beam and column had a square cross section with a 300 mm dimension. The height of the column was 2.9 m and the clear beam length was 1.4 m. The connection was initially tested to failure. Galvanised steel straps were used to rehabilitate the connection. Epoxy resin was used to fix the steel straps to the concrete surface. The connection was tested after the rehabilitation. Results of testing the rehabilitated connection show that the yield and ultimate loads were 65 kN and 95 kN, respectively, compared with the original test results of 75 kN and 84 kN, respectively. Based on results of the tests, it can be concluded that the rehabilitating method used in this study was effective in increasing the ultimate strength of the T connection.

Keywords
rehabilitation; T connections; Steel straps; reinforced concrete connections

1 INTRODUCTION

Connections are defined as a common point of intersection of the columns and beams and provide resistance to applied external loads due to the bending moment encountered at the joint. Therefore, connections play an important part in structures. The loading on structures pass through the beam-column connections. Load paths are developed in the concrete members and this allows the transfer of the externally applied loads to the support structures. Connections are critical components of structures and they have to be designed so that the possible failure due to shear, torsion and moment are minimised or eliminated. Research studies have indicated that some of the factors that have an important influence on the beam-column RC connections are: concrete confinement, confinement of reinforcement, axial compression on columns and the panel geometry of the connection. Past events have shown that the collapses of structures are due mainly to the failure of the beam-column connections. Therefore, it is vital that beam-column connections are designed to the optimum possible ability. Research has been done to highlight the different factors that attribute to the failure of concrete connections and the methods used to counteract these failures.

This paper presents an investigation of testing a T connection. This T connection was originally cast and tested to failure in 2006. In 2007, the same connection was rehabilitated and tested to failure with the aim to test the viability of the strengthening technique. The rehabilitation technique composed of using galvanised steel straps with epoxy. Results of the test showed that the rehabilitation technique is an effective technique.

2 REVIEW OF LITERATURE

The Portland Cement Association conducted the first experimental tests on beam-column connections in the early 1960s (Hanson and Conner 1967). Since then other research studies have been done to provide applicable data for beam-column connection design problems. Some of these research studies are discussed below.

One such study was done to investigate the shear strength of reinforced concrete beam-column connections by Meinheit and Jirsa (1981). The objective of this investigation was to examine the methods to improve the shear strength and measure the basic shear strength characteristics of beam-column connections. Several reinforced concrete beam-column connections were developed and tested under cyclic loads. Meinheit and Jirsa (1981) found that the strength of the connection differed according to the axial load on the column, the presence of transverse beams and the amount of closed hoop reinforcement within the connection. Meinheit and Jirsa (1981) concluded that the shear capacity improved due to transverse reinforcement in the connection, unloaded transverse beams improved the shear capacity, the columns' axial load did not influence ultimate shear capacity of the connection, the connection geometry had no influence on the shear strength of the joint if the shear area of the connection remained constant and the increase in column longitudinal reinforcement did not result in an increase in shear strength.

Scott (1992) investigated the behaviour of reinforced concrete beam-column connections due to the different detailing methods of reinforcement. This study made detailed measurements occurring inside the connection specimen by using internally strain-gauged reinforcement. This was done to obtain detailed distributions of strain along the column and beam reinforcement bars. As such, the intrinsic mechanisms of the connection behaviour could be comprehended.

Adetifa and Polak (2005) presented a technique of using shear bolts to retrofit slab column interior connections. Binici and Bayrak (2005) used fibre reinforced polymers (FRP) for upgrading slab-column connections. Shannag and Alhassan (2005) used high-performance fibre reinforced concrete jackets to seismically upgrade interior beam-column subassemblies.

Harajli et al. (2006) presented a technique of using a combination of FRP sheets and steel bolts to strengthen interior slab-column connections. Engindeniz et al. (2005) presented a review of the state of the art of the repair and strengthening of reinforced concrete beam-column joints.

It is clear that many researchers have used novel materials to upgrade the performance of connections, for example FRP. Clearly connections are important components in structures. This paper shows the behaviour of a connection when it is tested to failure then rehabilitated using a combination of steel sheets, epoxy and steel straps; and then being tested to failure.

3 TESTING THE INITIAL CONNECTION

In 2006, a reinforced T connection was tested to failure. The dimensions for the beam-column connection and the testing geometry are shown in Table 1 and Figure 1, respectively.

3.1 *Materials Used*

The concrete used in the experimental programme was provided by a local supplier. The average concrete strength at 29 days was found to be 46.78 MPa.

D500N deformed steel bars were used in building the beam-column connection. The steel bar had a specified yield stress of 500 MPa and had normal ductility. R10 plain steel bars were used for the stirrups, having a specified yield stress of 250 MPa and normal ductility. Three samples 300 mm long were tested in the Instron testing machine. The steel bars were found to have an average tensile strength of 538.6 MPa. This tensile strength was above the specified value of 500 MPa.

3.2 *Reinforcement*

The connection specimen was reinforced with N20 (20 mm diameter deformed bars with 500 MPa nominal tensile strength and normal ductility) and N16 (16 mm diameter deformed bars with 500 MPa nominal tensile strength and normal ductility) bars as shown in Table 2 and Figure 2.

3.3 *Testing the specimen*

The testing frame shown in Figure 3 was used to test the specimen both the initial specimen in 2006 and the rehabilitated specimen in 2007. The loading regime was chosen in line with the capabilities of the frame and the loading jack. An increasing single load was adopted.

The hydraulic jack applied a downward vertical load onto the beam to create a large turning moment within the concrete connection. The load was applied at a distance of 1100 mm from the column beam interface while the column was held securely in place.

The hydraulic jack applied a constantly increasing point load at the end of the beam until the beam reached ultimate failure. The loading rate was determined by the increase or decrease in pressure applied to the hydraulic jack by the hydraulic pump. The hydraulic pressure supplied to the jack was adjusted by using the turning knob on the hydraulic pump and was constantly increased to keep the deflection rate of around 2.5 to 5 mm per minute until the beam yielded, at which point the applied pressure was kept constant as the beam continued to deflect at approximately 3 to 5 mm per minute. The pressure began to decrease as the beam reached ultimate failure and the internal tensile steel ruptured.

The beam was loaded with a 550 kN universal hydraulic jack from 0 kN to the ultimate load point whilst deflection and rotation measurements were taken throughout the test. All measurements were attached via a data logger into the computer for a constant readout of the performance of the beam logged at around 5 Hz, five readings per second. An 111.5 kN load cell that was connected above the hydraulic jack during testing measured the applied load. A LVDT measured the deflection directly above the loading point of the beam in millimetres.

The rotation of the beam during the test indicates exactly the rotational capacity of the connection. Two inclinometers were used to measure the rotation of the beam and column during testing. The inclinometers logged the rotation in degrees during the entire test. The overall rotation of the joint was calculated as the difference between the rotation of the inclinometer on the beam and the inclinometer on the column.

4 REHABILITATION OF THE SPECIMEN

4.1 *Preliminary Testing*

Four main materials were used in rehabilitating the specimen, viz. epoxy (EP40 and EP10), galvanised steel and steel straps.

Epoxy is an organic compound which is commercially available in wide variety. This experimental program utilised super low viscosity (SLV) gravity-fed epoxy resin. The epoxy resin type used is the Conbextra EP. The Conbextra EP10, supplied with the Nitofill LV injection system, was used to fill the tensile cracks ranging from 0.2 mm to 0.01 mm. The Conbextra EP40 was used to fill cracks ranging from 10 mm to 40 mm.

Specimens from Conbextra EP40 and EP10 were tested. Two EP40 and three EP10 test specimens were prepared to test for their tensile and compressive strength, respectively. Both sets of specimens were tested on the same day as when the RC T-Connection was tested. These test were done to obtain the exact working strength of the epoxy when the actual testing of the connection was done.

The EP40 specimen samples were tested for their compressive strength using the AVERY Compressive machine in the civil engineering laboratory of the University of Wollongong. The samples were prepared in two cylindrical containers with a height of 100 mm and a diameter of 55 mm. The cylindrical containers were cut from a PVC pipe. The corners of the containers were smoothened out using a grinding machine in the laboratory. As such, the average heights and diameters of the samples were taken after curing to obtain the realistic dimension values.

The epoxy based resin and hardener were mixed in the ratio 1 to 4 in a container. The inside of the sample containers were greased to aid the removal of the sample after curing. The sample containers were placed on a level wooden board and the edges were sealed with a silicone sealant and strapped with a buckle to prevent leakage of the epoxy. The epoxy was then poured into the cylindrical sample containers. The sample containers were patted with a steel rod, while the epoxy was being poured, to create vibration so as to prevent the creation of air bubbles. Hence, homogeneous samples were achieved.

The samples were removed from the containers and their heights and diameters were measured. The diameter measurements were taken on the top, middle and bottom parts of the sample. The height measurements were taken at three different sides of the samples. The average diameter and height were then computed to achieve realistic dimension values.

The samples were tested on the same day as the testing on the connection was done. These tests were done to achieve the exact working strength of the epoxy acting in the connection on the day of the testing. Table 3 shows the average dimensions and results of the two samples. The table reveals that the average compressive strength of EP40 specimens was 99.6 MPa.

The EP10 samples were tested for their tensile strength using the Instron machine in the civil engineering laboratory at the University of Wollongong. The EP10 specimens were tested for tensile strength as EP10 would be subject to tension when the load is applied onto the connection. Two EP10 samples were prepared for the tensile test. The samples were cast as single units in a prefabricated mould. The mould was made using a timber board with a 15 mm high steel plates screwed onto the sides. The sides of the board were sealed with silicone sealant to prevent leakage of the epoxy. The shape of the specimens was dog bone 360 mm long and 20 mm by 6.5 mm nominal cross sectional area in the test region.

The mould was placed on a level surface and a water bubble leveller was used to ensure that the mould was level before the epoxy was poured into the mould. The mould was greased before pouring the epoxy. EP10 was then poured into the mould and the sample was left to cure for 13 days. The sample was then removed from the mould and two samples were cut. The average tensile strength of the EP10 epoxy specimens was 46.2 MPa.

Galvanised steel sheets were used to externally reinforce the connection. The galvanised steel sheet supplied was 2440 mm long, 1220 mm wide and 0.55 mm thick. A steel jacket was fabricated and used to externally reinforce the connection by fitting and strapping it with steel straps. The steel straps provide a confining pressure.

The galvanised steel and the steel straps were tested for their tensile strength using the Instron tensile testing machine in the civil engineering laboratory of the University of Wollongong. The total length of the specimen was 360 mm and 80 mm in the test region. The width of the specimen was 20 mm in the test region. The thickness of the specimen was 0.55 mm.

Table 4 shows the test results of testing the galvanised steel samples. The average tensile strength of the galvanised steel specimens was 358.4 MPa.

Two steel strapping samples were tested in the Instron machine for their tensile strength. The samples were cut off from the bulk roll of steel straps and were 270 mm long. Table 5 shows the test results. The average tensile strength of the steel straps was calculated to be 711.5 MPa.

4.2 *Preparing the Specimen*

The RC T-Connection with longitudinal reinforcement was tested in 2006 using a single increasing load to simulate progressive collapse. The load was applied downward at the edge of the beam through a hydraulic jack at a distance of 1100 mm from the beam-column interface. The damage caused to the connection due to the testing is depicted in Figure 4.

Three major tensile cracks formed and there was a major crushing of the concrete in the compression zone of the connection. Two of the major tensile cracks propagated right through the beam cross section while the other tensile crack did not. Some minor cracks formed on the surface of the beam and column.

The contact surfaces had to be cleaned before the application of epoxy into the cracks. Loose concrete, oil, grease, free standing water and dust had to be removed. The contact surfaces were free of oil, grease and free standing water. Loose concrete and dust were removed by hand and air blasting. Air blasting was done by using an air compressor.

A problem was encountered while trying to lift the beam. The whole structure moved while an upward force was applied through a hydraulic jack to lift the beam. Apparently, there was not enough restraint on the top end of the column. The self weight of the column could not resist the applied load. As a result the force from the hydraulic jack caused the whole structure to move. Therefore, a restraint had to be applied on the top end of the column before the beam could be lifted.

The beam was lifted through the application of an upward force by the hydraulic jack after the column was restrained. There was no movement of the structure and the beam was lifted to its horizontal position. After the beam was put to its horizontal position it was supported by a wooden prop to prevent the beam from falling against its own weight.

A designed to fit formwork was constructed to prevent the leakage of epoxy on the underside and the sides of the gap. The formwork was made of plywood and screwed using threaded rods and bolts to hold it together. The formwork was installed after the beam was aligned to its horizontal position as shown in Figure 5.

The formwork was sealed along its edges using Bostik Silicone. This sealing was done to prevent any leaking of the epoxy.

As shown in Figure 4, there was a large spalling of concrete in the underside of the beam. For this particular region, injection of epoxy was not practical. Hence, a different grade of epoxy was used to patch it up. The Conbextra EP40 was used to fill the gap. The epoxy based resin and hardener were mixed in the ratio 1 to 4 respectively. Care was taken while handling the epoxy because of its corrosive nature. Safety goggles, mouth mask and rubber gloves were worn at all times when handling the epoxy.

A hole had been drilled into the formwork prior to installation to create an opening for the pouring of epoxy. The epoxy was poured through a funnel that was connected to a hose fitted into the opening in the formwork.

Epoxy was injected into the tensile cracks after the removal of loose concrete and dust. The Conbextra EP10 was used to fill the tensile cracks that ranged from 0.2 mm to 0.01 mm. Holes were meant to be driven to inject the epoxy if needed. However, it was not needed in this case as the epoxy was very viscous. In fact the epoxy flowed just like water. Hence, the epoxy flowed very well into the cracks. The epoxy was then left to cure for 7 days to gain its specified strengths.

A galvanised steel jacket was fabricated from galvanised steel. The steel jacket was measured, cut and bent to the required shape. The dimensions of the fabricated steel jacket are shown in Figure 6.

The dimensions of the steel jacket were marked onto the steel sheet, the sheet was then clamped with G-clamps onto a large working table and finally the steel sheet was cut. The steel sheet was cut to the required dimensions. Subsequently it was bent along the dotted lines (Figure 6) to achieve its final form and was then fitted into the connection. The galvanised steel sheet was bent to the required shape.

The steel jacket was then placed into the connection. Steel straps were used to hold the steel jacket in place and apply a confining pressure. The straps were spaced at 20 mm intervals. 22 steel straps and 19 steel straps were clamped onto the column and beam, respectively. A special band-it tool was used to apply a confining pressure to the steel straps and clamp it in place.

The steel straps were provided in a bulk roll. The steel straps were first cut into lengths of 1800 mm to ease the application of the straps onto the connection using the band-it tool. The above mentioned procedure was necessary as the accessibility was restricted due to the testing frame. A buckle was used to lock the steel straps in place.

Figure 7 shows the completed strapping of the galvanised steel sheet onto the connection with the steel straps.

4.3 *Testing the Rehabilitated Specimen*

A hydraulic jack applied a downward vertical load onto the beam to create a turning moment within the connection. The load was applied at a distance of 1100 mm from the column-beam interface.

The hydraulic jack applied an increasing point load until ultimate failure of the beam was reached. The loading rate was set to keep the deflection rate between 2.5 mm and 5 mm per minute and was determined by increasing or decreasing the pressure applied by the hydraulic pump to the hydraulic jack.

An increasing single load was applied onto the beam with a 550 kN universal hydraulic jack from 0 kN to ultimate failure load. The deflections of the beam and the rotation measurements were taken throughout the testing period. The measurements were attached to a computer data logger to obtain a constant readout of the performance throughout the test.

The applied load was measured via an 111.5 kN load cell which was connected on top of the hydraulic jack. The load cell was calibrated before it was fixed on top of the hydraulic jack.

A LVDT was placed to measure the beam deflection and was placed on the edge of the beam at a distance of 1100 mm from the column-beam interface. Two inclinometers were used to measure the rotation of the column and the beam. The inclinometers logged the rotation in degrees for the whole test. One inclinometer was placed on the column and the other on the beam. The overall rotation of the joint is equal to the rotation of the beam minus the rotation of the column.

During the initial test, three major tensile cracks were formed and there was a major crushing of the concrete in the compression zone of the connection. Two of the major tensile cracks propagated right through the beam cross section while the other tensile crack did not. Some minor cracks formed on the surface of the beam and column.

5 TEST RESULTS

The performance of the rehabilitated specimen was determined by comparing its results with the results obtained in 2006. The overall comparison of the results is summarised in Table 6.

From Table 6 several conclusions can be made:

- The yield load for the rehabilitated specimen was lower than the original specimen.
- The ultimate load for the rehabilitated specimen was higher than the original specimen.
- The rehabilitated specimen did not increase the joint rotation.
- The rehabilitated specimen reached ultimate failure.

Figure 8 shows the load-deflection curves of the initial and rehabilitated specimen. Likewise Figure 9 shows bending moment-rotation of the initial and rehabilitated specimen.

The initial specimen had a crack pattern as depicted in Figure 10. It shows three major tensile cracks on the beam and crushing of concrete in the compression zone. Tiny cracks were also observed on the concrete surface of the column and beam near the joint.

Figure 11 shows a single major tensile crack in the rehabilitated specimen after testing. There was also no crushing of epoxy in the compression zone of the beam. The epoxy applied to fill the cracks in the tensile cracks is very viscous and transparent. As such the initial crack lines are vividly visible. Therefore, these cracks should not be mistaken for new cracks forming in the epoxy repaired region.

It can also be seen from Figure 11 that the new major tensile crack is propagating into the epoxy in the compression zone. However, at this point, the beam reached ultimate failure and visual inspection showed that the two primary tensile reinforcement bars had fully ruptured.

The external reinforcement using galvanised steel and steel straps provided confinement to the concrete and epoxy. Figure 12 shows the extent of the damage on the original specimen after the test. Concrete crushing was observed in the compression zone of the connection. Three major tensile cracks and tiny surface cracks formed as well.

It was observed that the galvanised steel buckled during the testing as shown in Figure 13. This buckling was due to the tensile forces acting in the tensile zone of the beam and the compressive forces acting in the compression zone.

It can be seen in Figure 13 that the steel straps near the beam-column interface moved slightly outwards from their original positions. This movement was due to the tensile force acting on the galvanised steel in the tensile zone. It is evident that the galvanised steel and steel straps added strength to the connection together with the epoxy adhesive. More importantly, most of the added strength in the specimen comes from the epoxy as there was no reopening of the cracks in the repaired region. However, it is difficult to gauge exactly how much strength was added by the galvanised steel and the steel straps.

Figure 14 depicts the damage of the rehabilitated specimen after it was tested. A major tensile crack formed and slowly propagated into the compression zone. The tensile crack was ripped off the column due to the rupturing of the two primary reinforcement bars.

6 CONCLUSIONS

The performance level of the rehabilitated specimen was determined by comparing the results obtained with the results of the original specimen. It was found that the rehabilitated specimen had no increase in the rotational capacity of the joint. The rehabilitated specimen had a yield load of 65.37 kN. This value was lower than that of the initial specimen which had a yield load of 75.83 kN. However, the rehabilitated specimen achieved a higher ultimate load of 95.48 kN compared to an ultimate load of 84 kN achieved by the initial specimen. This was an approximate increase of 13.67% in ultimate load.

It was found that the rehabilitated specimen had a brittle failure as opposed to a ductile failure of the initial specimen. In addition, a single major tensile crack developed in the rehabilitated specimen. This crack was ripped off the column. This failure pattern was totally different from the original specimen in which case multiple tensile cracks had occurred in the beam. Therefore, it shows that the stresses were not evenly spread along the beam for the rehabilitated specimen resulting in a brittle failure.

It was also observed that the region repaired with epoxy did not reopen. The epoxy, Conbextra EP10, managed to withstand the tensile load applied. There was no epoxy crushing in the compression zone of the beam as opposed to a large crushing of concrete in the original specimen. This behaviour shows that the epoxy, Conbextra EP40, managed to withstand the applied compressive load. The two primary tensile reinforcement bars were totally ruptured upon ultimate failure.

The epoxy performed satisfactorily. However, the contribution of the external steel reinforcement on the performance of the structure was difficult to gauge.

Finally, although the reinforcing steel did yield during the initial test, the rehabilitated specimen proved to be capable of carrying considerable loads before failure.

REFERENCES

Adetifa, B. and Polak, M.A. 2005. Retrofit of slab column interior connections using shear bolts. *ACI Structural Journal*. 102(2):268- 274.

Binici, B. and Bayrak, O. 2005. Upgrading of slab-column connections using fiber reinforced polymers. *Engineering Structures*. 27(1):97-107.

Engindeniz, M., Kahn, L.F. and Zureick, A.H. 2005. Repair and strengthening of reinforced concrete beam-column joints: State of the art. *ACI Structural Journal*. 102(2):187-197.

Hanson, N.W. and Conner, H.W. 1967. *Seismic resistance of reinforced concrete beam-column joints* - Portland Cement Association, Research and Development.

Harajli, M.H., Soudki, K.A. and Kudsi, T. 2006. Strengthening of interior slab-column connections using a combination of FRP sheets and steel bolts. *Journal of Composites for Construction*. 10(5):399-409.

Meinheit, F.D and Jirsa, O.J. 1981. Shear strength of RC beam-column connections. *ASCE* 107(ST11): pp.43-45.

Scott R.H. 1992. The effects of detailing on RC beam/column connection behaviour. *The Structural Engineer* 70(18): 318-324.

Shannag, M.J. and Alhassan, M.A. 2005. Seismic upgrade of interior beam-column subassemblages with high-performance fiber-reinforced concrete jackets. *ACI Structural Journal*. 102(1):131-138.

LIST OF FIGURES

LIST OF TABLES

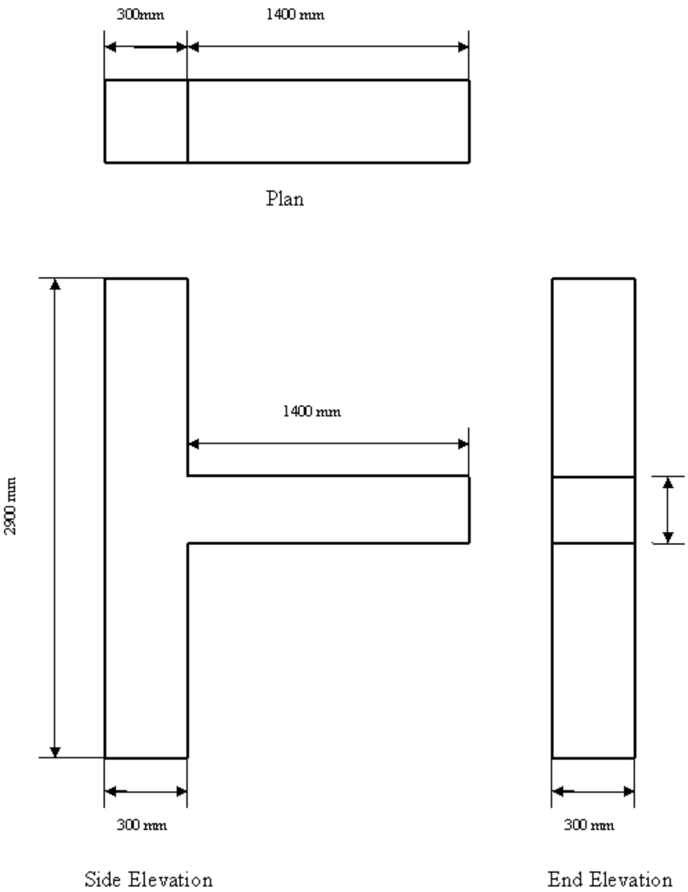
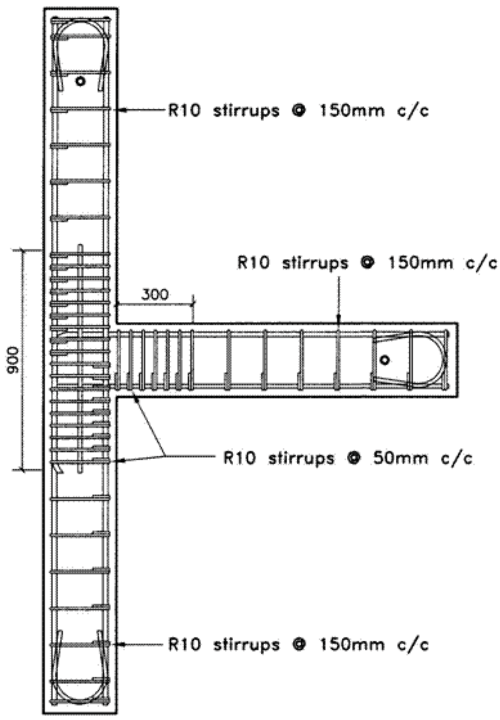
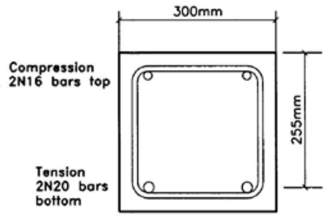
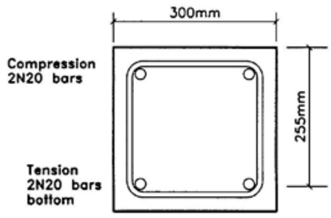


Figure 1. Dimensions of the beam-column connection.





Beam Reinforcement



Column Reinforcement

Figure 2. Reinforcement details.

Figure 3. Testing frame.

Figure 4 Cracks in damaged RC T-connection

Figure 5 Installed Formwork

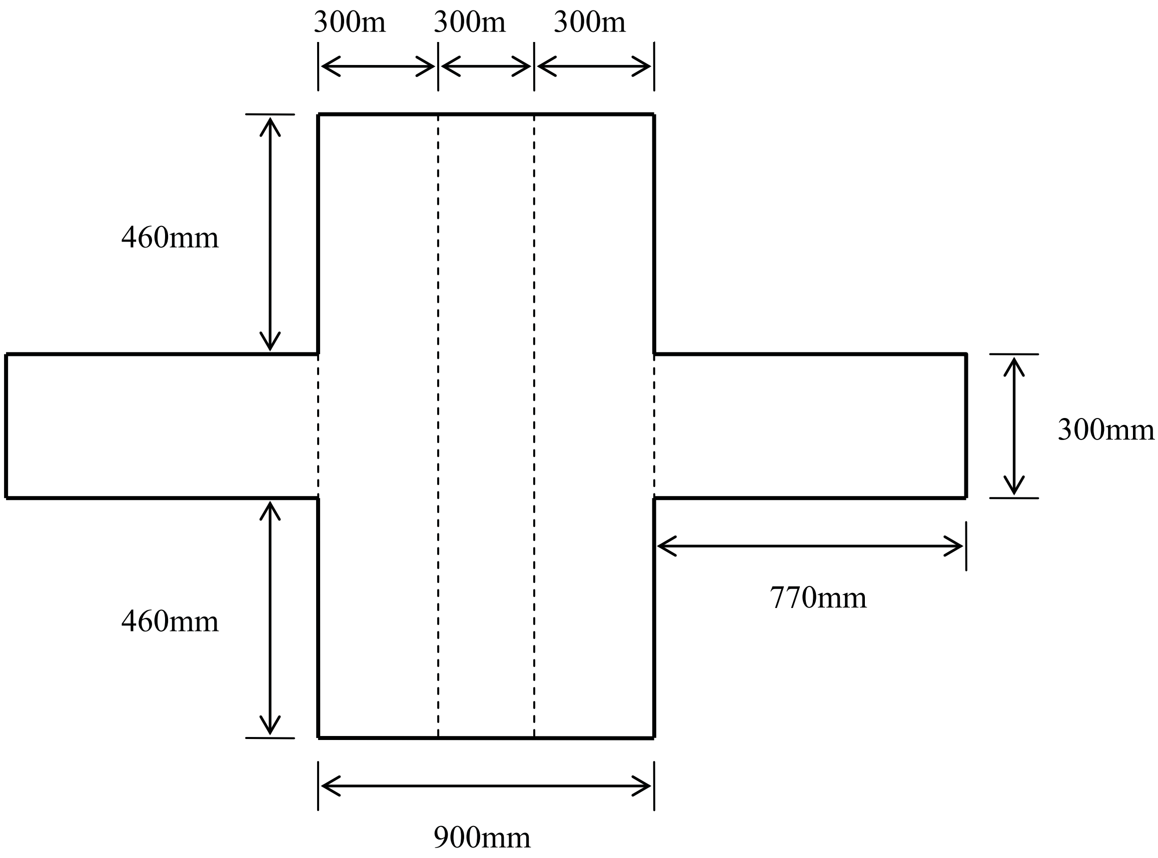


Figure 6 Dimensions of the steel jacket

Figure 7 T-Connection completed with steel straps

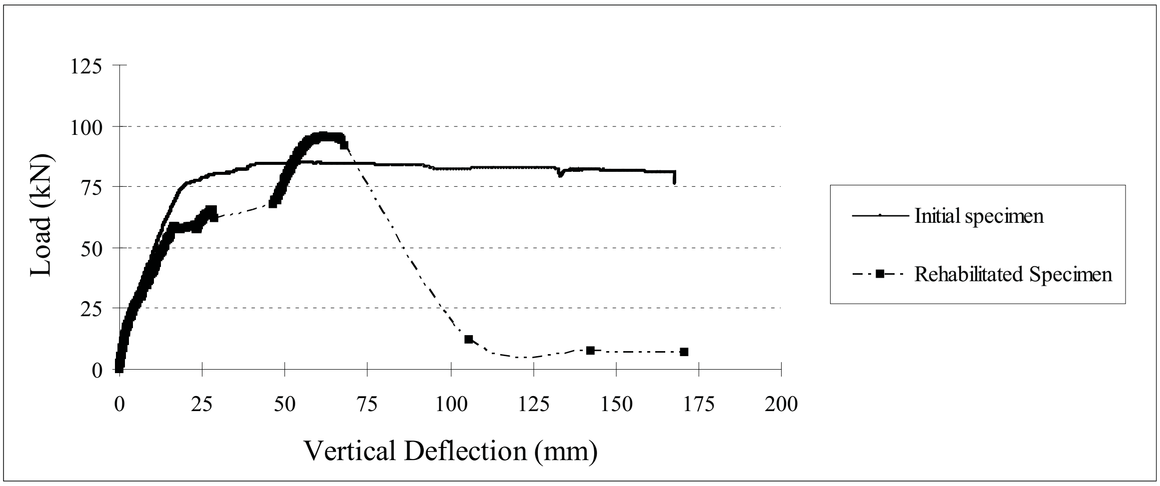


Figure 8 Load-deflection Curves of the Initial and the Rehabilitated Specimen

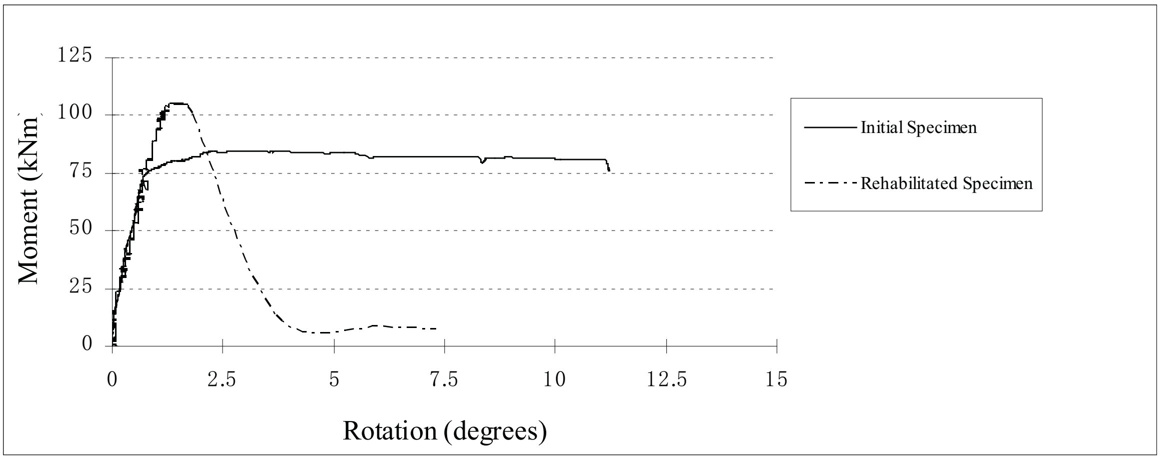


Figure 9 Bending Moment-Rotation Curves of the Initial and the Rehabilitated Specimen

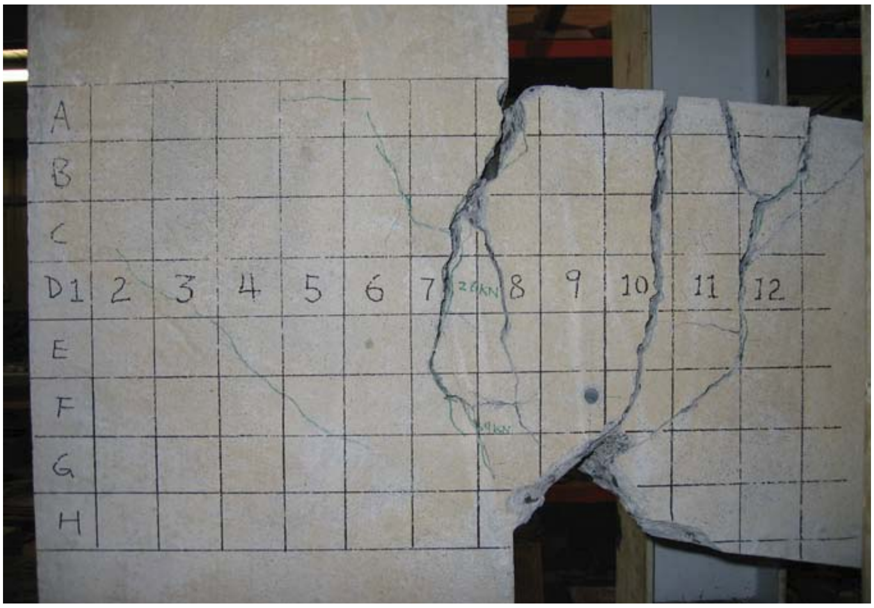


Figure 10 Tensile cracks and concrete crushing before rehabilitation

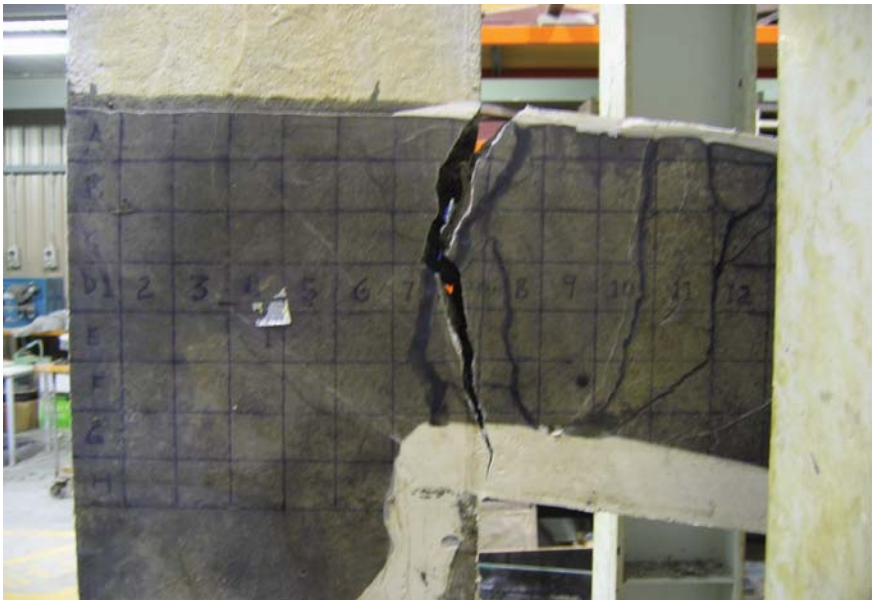


Figure 11 A single major tensile crack in rehabilitated specimen

Figure 12 Concrete crushing and tensile cracks in original specimen

Figure 13 Buckling of galvanised steel

Figure 14 Rehabilitated specimen after testing

Table 1. Dimensions of the structural elements.

Structural Element	Dimension (mm)
Column length	2900
Beam length	1400
Column cross-section	300×300
Beam cross section	300×300

Table 2. Specimen Reinforcement.

Member	Reinforcement location	Steel used
Beam	Tensile reinforcement	2N20
	Compressive reinforcement	2N16
	Stirrups – normal spacing	150 mm
	Stirrups – joint spacing	50 mm
Column	Tensile reinforcement	2N20
	Compressive reinforcement	2N16
	Stirrups – normal spacing	150 mm
	Stirrups – joint spacing	50 mm

Table 3 Test results of EP40 samples

Conbextra EP40	Sample 1	Sample 2
Average Diameter (mm)	53.81	53.65
Average Height (mm)	88.27	89.80
Cross Sectional Area (mm^2)	2274.13	2253.05
Maximum Compressive Load (kN)	235.5	215.5
Maximum Compressive Strength (MPa)	103.56	95.65
Average Compressive Strength (MPa)	99.6	

Table 4 Galvanised steel test results

Galvanised Steel	Sample 1	Sample 2	Sample 3
Gauge Length	80	80	80
Average Thickness (mm)	0.5	0.51	0.5
Average Width (mm)	19.69	20.05	19.91
Yield Stress (MPa)	330.14	326.33	330.17
Yield Strain (%)	5.39	9.14	7.96
Tensile Strength (MPa)	361.18	351.64	362.47
Ultimate Strain (%)	25.37	30.32	25.50

Table 5 Test results of steel straps

Steel Straps	Sample 1	Sample 2
Gauge Length (mm)	270	270
Average Thickness (mm)	0.54	0.55
Average Width (mm)	19.06	19.10
Yield Stress (MPa)	620.98	622.46
Yield Strain (%)	0.40	0.47
Tensile Strength (MPa)	704.51	718.50
Ultimate Strain (%)	1.12	1.11

Table 6. Comparison of results.

	Original	Rehabilitated
Yield load (kN)	75.83	65.37
Yield Deflection (mm)	20.15	28.2
Yield Rotation (degrees)	0.665	0.8
Ultimate Load (kN)	84.69	95.48
Ultimate deflection (mm)	167.89	170.8
Ultimate Rotation (degrees)	10.94	7.3