2008

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Publication Details

Hadi, M. N. (2008). Strengthening reinforced concrete T connections by steel straps. Proceedings of the First International Symposium on Life-Cycle Civil Engineering (pp. 565-570). London, UK: Taylor Francis Group.

Strengthening reinforced concrete T connections by steel straps

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ABSTRACT: The aim of this paper is to present results of testing a full scale reinforced concrete T connection by static loading. The connection is a T connection representing 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 strengthen 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.

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 and have to be designed so that 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 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 a beam-column connection. 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 shear capacity improved due to transverse reinforcement in the connection, unloaded transverse beams improved the shear capacity, column 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 the increase in shear strength.

Scott (1992) investigated the behaviour of reinforced concrete beam-column connections due to the different detailing methods of reinforcement. This research 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.

Scott (1992) used three detailing arrangements for the reinforcement and three beam tension steel percentages in this research. They were: bending beam tension bars down into the column, bending beam tension bars up into the column and 'U' bars, in which the lower legs formed the bottom beam reinforcement. The beam tension steel percentage depended on the size of the steel bar used. This comprised of 1.0% and 1.9% respectively in a 12 mm or 16 mm diameter steel bar for shallow beam specimens and 1.3% in a 16 mm diameter steel bar for deep beam specimens. Several specimens were developed and tested in a purpose built testing rig. A full column load of 50 kN or 275 kN was used in increments of 25 kN. The load was held as the beam was loaded downwards in 1kN increments till failure. Strain measurements of the steel reinforcement bars were measured together with the concrete surface strains.

Scott (1992) found that specimens with 1.0% beam tension reinforcement bent down into the column or bent into the 'U' bar failed due to development of a plastic hinge on the beam at the face of the column when a column load of 275 kN was used. Gross yield of the reinforcement beam bars resulted in high reinforcement strains. However, when a column load of 50 kN was used on similar specimens, failure due to extensive joint cracking and strains was recorded. Other specimens failed due to extensive joint cracking and the strains were lower occasionally in the elastic range. The load transfer in the three beam details was mainly due to the development of bond stresses at the bend up to the point of cracking. Upon cracking, the loss of bond in bars bent down and the 'U' bars was provided for by bond development stresses over their length. This enabled a large load increment between joint cracking and failure. In contrast, the bars bent up detail failed to account for the loss of bond and resulted in a brittle failure. Scott (1992) concluded that the bars bent down and the 'U' bar details performed better than the bars bent up detail and recommended the use of the bars bent down detail if ductility was of main importance.

3 TESTING THE INITIAL CONNECTION

In 2006, a helically 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.

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

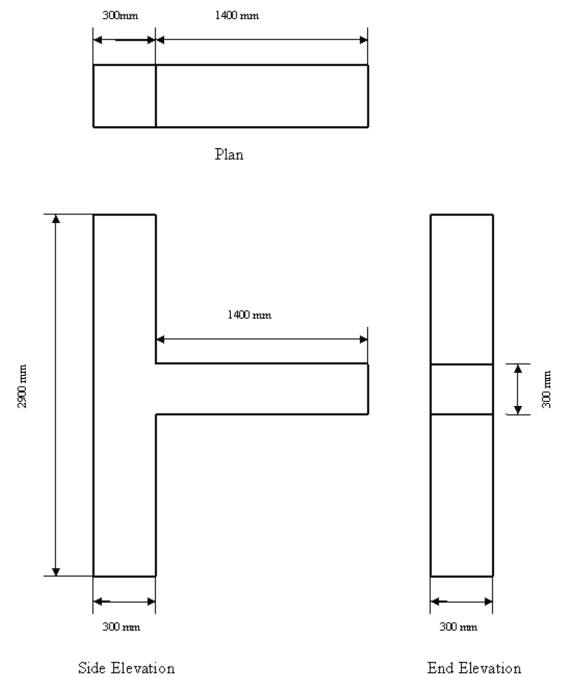

Figure 1. Dimensions of the beam-column connection.

3.1 *Materials Used*

A grade 32 NSC with a compressive strength of 32MPa was used to build the beam-column connection. The average concrete strength after 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 was above the specified value of 500 MPa.

3.2 *Reinforcement*

The specimen was reinforced with N20 and N16 bars as shown in Table 2 and Figure 2.

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

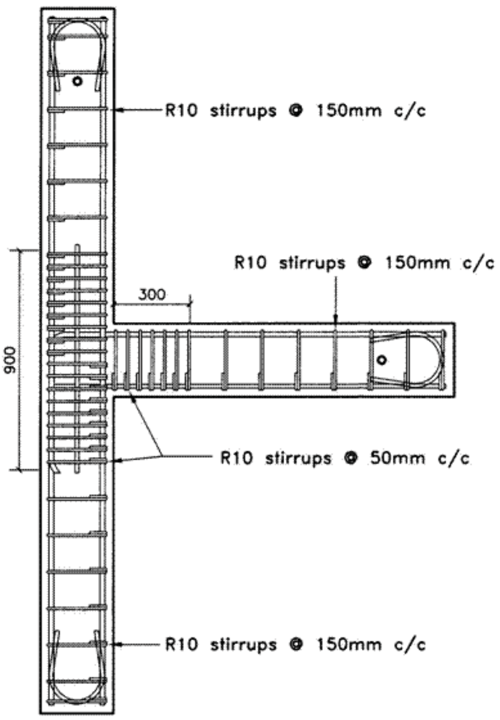

Figure 2. Reinforcement details.

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 the 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 yields, at which 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 ruptures.

Figure 3. Testing frame.

The beam was loaded with a 550 kN universal hydraulic jack from 0 kN to the ultimate load point whilst deflections, strain readings 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. The 111.5 kN load cell was calibrated on the INSTRON by technical staff. An LVDT measured the deflection directly above the loading point of the beam in millimetres.

Steel reinforcement strains within the beam were read by the change in resistance of the strain gauges that were logged onto the computer. The concrete tensile strains were measured using a concrete embedment gauge developed at the University of Wollongong. The embedment gauge consists of a normal steel strain gauge that is embedded within an epoxy resin shape that can bend and stretch to measure the strain of the concrete. Several strain gauges during testing stopped reading after yield of the beam occurred as concrete movement can destroy the small strain gauge wires or scratch the gauge surface.

The rotation of the beam during the test indicates exactly the rotational capacity of the connection. Two inclinometers were used to measure the degree of 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 will be equal to the rotation of the inclinometer on the beam minus the inclinometer on the column. Table 3 shows results of testing the original specimen.

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. Two specimens (cylinder with a nominal diameter of 50 mm and nominal height of 90 mm) of EP40 were tested for compressive strength which yielded an average compressive strength of 99.6 MPa. Two specimens of EP10 epoxy specimens were tested for tensile strength. 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 average tensile strength of the EP10 epoxy specimens was 46.2 MPa. Three specimens of galvanised steel were tested. The specimens were dog bone in shape and had an overall length of 360 and cross sectional nominal dimensions of 20 mm by 0.5 mm. 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. The nominal cross sectional dimensions of the steel strap specimens was 0.5 mm by 19 mm. The average tensile strength of the steel straps was calculated to be 711.5 MPa.

4.2 *Preparing the Specimen*

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.

The following procedures were undertaken to repair and retrofit the damaged T connection. Each procedure is explained below.

The contact surfaces had to be cleaned before the application of epoxy into the cracks. Loose concrete, oil, grease, free standing water and dust were 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 blasted. Air blasting was done through the operation of 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 load applied. 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.

A chain block was used to restraint the column on the top end to counter the problem. A hole had been drilled on the top of the column to aid its transportation previously. A steel rod was placed through the hole and the chain block was anchored.

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

There was a large removal 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. Three samples were made to test for the compressive strength. The samples were cast on the same day as the epoxy was applied. 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.

Epoxy was then applied into the underside of the beam. 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 0.01mm. 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 injected using the Nitofill LV injection system. The cartridge containing the epoxy was inserted into the injection gun and a static mixer hose was fitted onto the cartridge. 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 4.

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 using an electric cutter. Subsequently it was bent along the dotted lines to achieve its final form and was then fitted into the connection. The galvanised steel sheet was bent to the required shape using the bending machine. 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 them in place. The tool consists of a cutting handle, a grip lock and a turning handle.

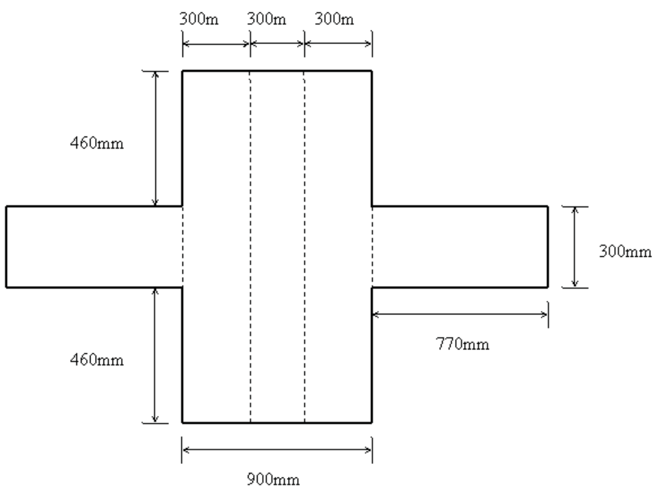

Figure 5. Dimensions of steel jacket.

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

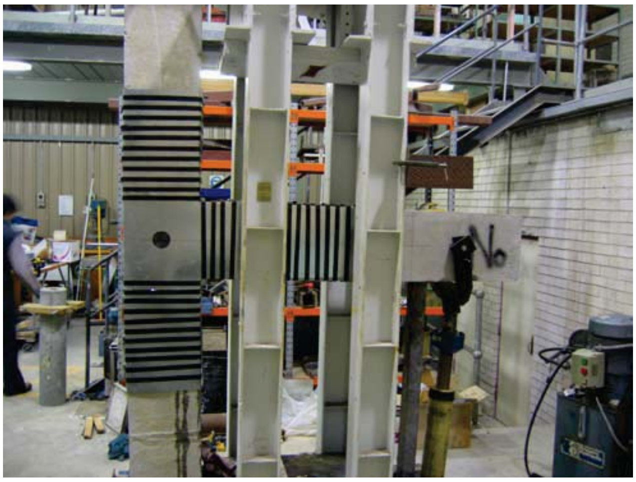

Figure 5. T-Connection completed with steel straps.

5 TESTING PROCEDURE

An increasing single load was applied to simulate progressive collapse loading as done during testing the initial 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 a 111.5 kN load cell which was connected on top of the hydraulic jack. The load cell was calibrated by technical staff 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 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. The position of the inclinometers is shown in Figure 6.

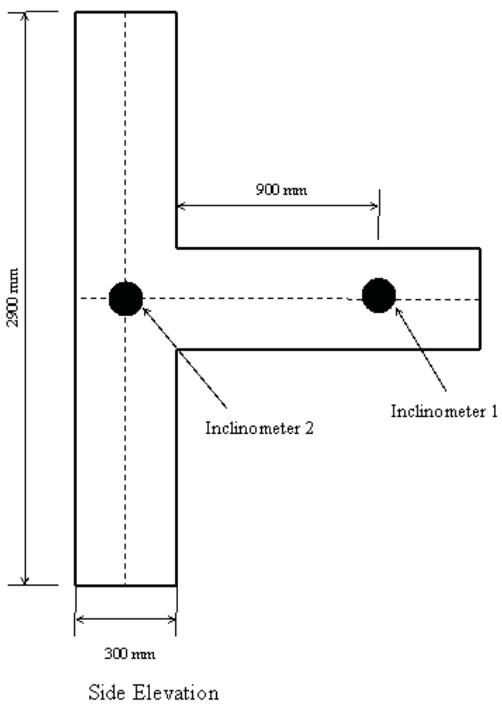

Figure 6. Position of Inclinometers.

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 3.

Table 3. 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

From Table 3 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.

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 original standard 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 original standard specimen. This was an approximate increase of 13.67% in the ultimate load.

It was found that the rehabilitated specimen had a brittle failure as opposed to a ductile failure of the original 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 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.

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