Carbon Fibre Strengthening of Precast Reinforced Concrete Inverted U Bridge Beams

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SYNOPSIS

Main Roads Western Australia has a number of bridges constructed from precast reinforced concrete inverted U beams (approx 6400 mm long, 940 mm wide and 410 mm deep). These bridges were designed and constructed in the 1950's with plain bar reinforcement for trucks with a single rear axle, and concern has been expressed about their shear capacity and moment capacity at the location of curtailment of the main flexural reinforcement, particularly with loads from dual and tri axle groups.

Recently three bridges were removed from service and replaced with new construction. This provided the opportunity to test the ultimate capacity of the removed beams. A series of laboratory tests was conducted as the initial phase of research into the strength characteristics of these beams. The testing utilised 13 standard internal beams, 7 non-standard internal beams with external strengthening by 100 mm by 10 mm steel plate, and six standard internal beams which had 80 mm by 1.2 mm carbon fibre straps applied to the bottom of both webs as part of the testing program.

The strengthening with carbon fibre straps increased the stiffness of the beams and resulted in strength increases of between 20% to 30% when compared with the standard beams.

1 INTRODUCTION

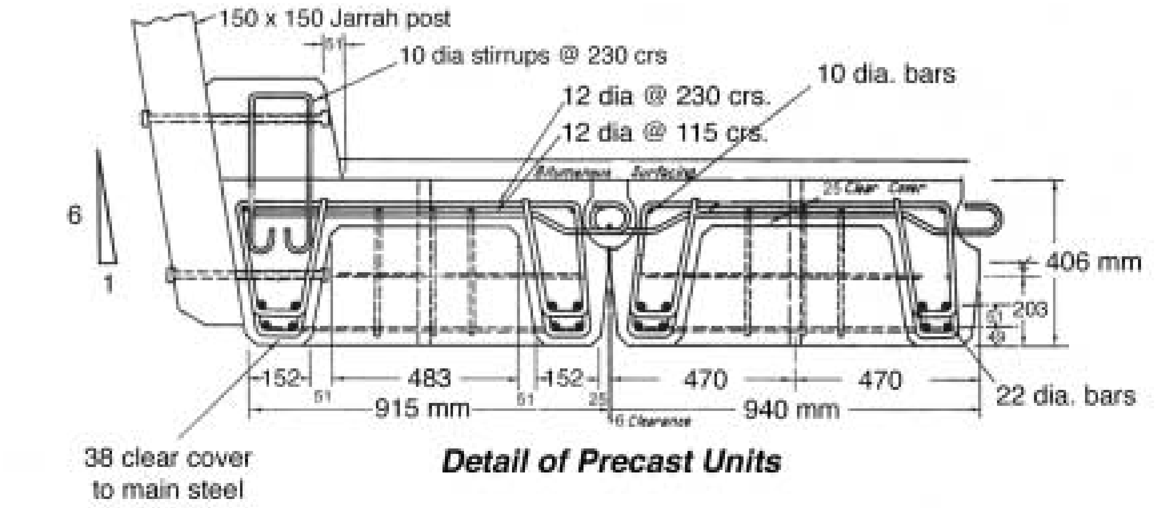
Main Roads Western Australia has a number of bridges constructed from precast reinforced concrete inverted U beams (approx 6400 mm long, 940 mm wide and 410 mm deep). These bridges were designed and constructed in the 1950's with plain bar reinforcement for trucks with a single rear axle, and concern has been expressed about their shear capacity and moment capacity at the location of curtailment of the main flexural reinforcement, particularly with loads from dual and tri axle groups.

Recently three bridges were removed from service and replaced with new construction. This provided the opportunity to test the ultimate capacity of the removed beams. A series of laboratory tests was conducted as the initial phase of research into the strength characteristics of these beams. The testing utilised 7 standard internal beams, 7 non-standard internal beams with external strengthening by 100 mm by 10 mm steel plate and six standard internal beams which had 80 mm by 1.2 mm carbon fibre straps applied to the bottom of both webs as part of the testing program.
The paper will present key details of the testing program, and present and discuss the results obtained particularly for the carbon fibre strengthened beams. Additional details of the testing program and strengthening with steel plates can be found in a previous paper by the author,

Chandler (1). Others, such as Oehlers (2), De Lorenzis et al (3) and Hassanen and Raoof (4) have also examined externally bonded strengthening of concrete beams.

2 DETAILS OF THE BEAMS

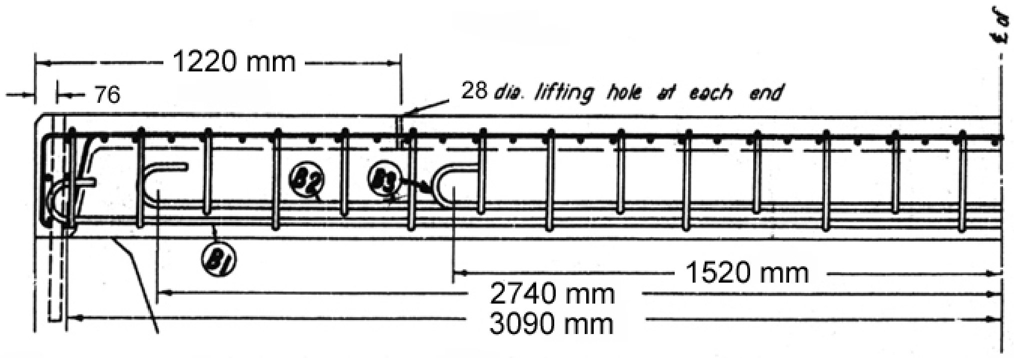
The inverted U bridge beams were 6400 mm long and of two cross sections – one for the edge kerb unit and the second for the symmetrical standard internal unit - both of which are shown in Figure 1. The design strength of the concrete for the precast beams was 24 MPa. The beams were connected by a shear key, which was filled onsite with a small-sized aggregate site-mixed concrete.



Note: Measurements hard converted to SI units from original drawings

***Figure 1:** Details of Cross Section for Both the Internal Standard and Edge Kerb Beams*

The main reinforcement for the beams was shown on the drawings as four plain mild steel 22 diameter bars which were progressively curtailed along the length of the beam. The curtailment points with hooks for end anchorage are shown in Figure 2. The reduction from four to three bars per web occurs at approximately 1500 mm from the end face of the beam and assessment of the adequacy of the bending moment capacity at this point was one aim of this research.



Note: Measurements hard converted to SI units from original drawings

***Figure 2:** Details of Main Reinforcement Curtailment*

3 DETAILS OF TESTING

3.1 Test Arrangement

The beams were set up on roller supports over a span of 6200 mm. The load system was two hydraulic jacks 1200 mm apart, which were secured to the reaction frames via steel swivel joints. The jacks transferred load to the beam via a ball joint onto steel plates representing the Austroads (5) wheel patch (400 mm by 200 mm). Loading was central across the width of the beam.

Two dial gauges were used to monitor deflection during the initial stages of the loading and were set under the web of the beam directly below the load points.

Several Linearly Variable Differential Transformers (LVDT's) were used as the primary method of deflection measurement. Typically these were placed on both sides of the top surface of the beam at 600 mm centres longitudinally, set out from the load position. The readings from the LVDT's were continuously logged with samples taken 10 times per second throughout the test.

3.2 Testing Procedure

Once the setup was complete, the beam and system was bedded using a line pressure of 4 or 5 MPa (equivalent to a total applied load of 30 to 40 kN) prior to recording datum readings.

Loading was usually in 2MPa increments of line pressure (equivalent to a load increment of 12 to 20 kN depending upon which jacks were used) and beams were unloaded once initial cracking was observed, generally around a load of 200 kN. New datum readings were recorded and the beam reloaded.

Throughout the loading the dial gauge readings were entered into a spreadsheet and a load-deflection plot was displayed on a laptop computer to assist in monitoring behaviour and the onset of cracking.

Once the beam was deflecting and cracking significantly the load was applied steadily and the peak load recorded as failure. In a few cases the tests were stopped due to the deflection exceeding the travel of the jacks or loss of the ball from the ball joint (due to large rotation at load point).

4 RESULTS OF TESTING

4.1 Results: - Flexural Tests

The layout of reinforcement in the beams makes it possible for loading near the first curtailment point (approximately 1500 mm from end face of the beam) to be critical in determining the flexural performance of the beam. Theoretical calculation of the bending moment and shear for various loading arrangements was conducted and two load arrangements produced similar predictions of failure load. These load arrangements were either with the two wheel loads 1200 apart near the midspan or with one wheel load 1500 mm from the end support (and the other 1200 mm further along). Both loading arrangements were evaluated during the flexural testing of five standard beams.

Five of the reinforced concrete inverted U bridge beams were tested in flexure, one loaded at the curtailment point and the other four loaded near midspan to establish the ultimate flexural strength of these sections. All beams failed in flexure due to crushing of the concrete in the compression zone, and the details of failure for these beams have been set out in Table 1. The maximum moment at failure is consistent with the calculated ultimate strength bending capacity using a stress of approximately 300 – 320 MPa for the steel reinforcement. This stress range would be quite reasonable as tests on the steel reinforcement show (see details in section 4.5.2).

All four beams loaded at midspan had very similar load-deflection responses throughout the loading. Typically the beams deflected with an initial linear response up to a deflection of approximately 8 mm at a load of 100 kN. This load approximated the turning point in stiffness from the uncracked section to the cracked section with a second linear deflection response to load up to approximately 20 mm at 200 kN. Once the load exceeded 210 kN the deflection increased dramatically with large cracks opening and yielding of the reinforcement prior to failure of the compression zone concrete. Deflections exceeding 200 mm occurred prior to failure of each beam.

Table 1: Summary of Failure Details for Flexural Tests

Bridge Id	Beam Id	Load Positions (Relative to N end in mm)	Details of Failure		
			Total Load (kN)	Max Moment (kNm)	Corresponding Shear (kN)
4658	2-1	1500, 2700	250	300, 250[2]	165
4658	3-1	2500, 3700	235[1]	295	120
413	2	2500, 3700	250	310	128
4658	2-2	2500, 3700	250	310	128
4658	3-2	2500, 3700	235	295	120

1 *estimate of failure load – jacks reached full ram extension at a load of 230 kN*
2 *Moment of 250 kNm is in the curtailed section with only three reinforcing bars*

4.2 Results: - Flexural Tests with External Strengthening by Carbon Fibre Straps

Six precast reinforced concrete inverted U bridge beams had 80 mm by 1.2 mm low-modulus carbon fibre straps applied to the bottom of both webs. The aim of these tests was to compare performance of the beams with carbon fibre strap repair to those with steel plate repair.

The carbon fibre strap was donated by MBT (Australia) Pty Ltd and the preparation and application of the strap to the beams was conducted by their approved installer in WA, Structural Systems Pty Ltd. Preparation and installation was conducted in a manner similar to what could be expected at a bridge site.

The key results from testing of the beams with external strengthening by carbon fibre straps are shown in Table 2. The beams have been grouped according to three different testing configurations and the details of the first two groups of tests are discussed further in the following sections.

Table 2: Summary of Failure Details for External Strengthening with Carbon Fibre Straps

Bridge	Beam	Total Load at Failure (kN)	Max Moment(s) at Failure (kNm)	Corresponding Shear at Failure (kN)
Loaded 1500mm and 2700 mm from End Face of Beam				
4658	2-3	315	366, 299	213
413	6	310	360, 290	210
Loaded 1500mm and 2700 mm from End Face of Beam – Ends of Straps Clamped				
4658	1-2	320	377, 308	220
413	4	325	372, 303	217
Loaded Near Midspan - 2500mm, 3700 mm				
4658	1-3	300	377	155

4.2.1 Results for Beams with Carbon Fibre Straps Loaded 1500 mm from End

Beam 2-3 from bridge 4658 was tested with loads at 1500 mm and 2700 mm from the north end face to assess behaviour when the bending moment just beyond the main curtailment point for the reinforcement was a maximum. Flexural cracks were first noted at a load of 130 kN, although the load-deflection plots show that a change in stiffness consistent with cracking started at a load of 100kN, slightly more than the standard section without strengthening.

The initial loading was to 200 kN, and when unloaded there was approximately 2 mm of permanent set in the deflection at the south load point.

During the reloading, very little cracking was evident until a load of 250 kN when some significant new flexural cracks opened. At a load of 300 kN some cracking noises associated with the carbon fibre strap were heard, followed by two loud sharp bangs and the debonding of the west side strap. It was not possible to determine which section of the strap debonded first due to the very rapid failure (very large displacement of the strap took place within one frame of the video record = 0.04 of a second). Almost immediately the east side strap debonded and the middle of the strap moved down hitting the floor and rebounding. As this strap released, a significant flexural crack opened and a “fist sized” piece of concrete fell with the strap. Once again it was not clear whether the amount of cracking and curvature under the north load initiated the debonding and peeling propagated from this location, or whether the peeling commenced from the end of the strap and the loss of strap at the load position allowed the flexural crack to open. Failure of the beam followed with rapid increase in deflection and significant curvature at the north load point. The maximum sustained load was 315 kN.

The second beam with this loading arrangement was beam 6 of bridge 413, and it behaved in a similar manner to the first of these tests. Cracking started at a load of 100 kN, and there was another change in slope of the load-deflection curve at 200 kN with numerous flexural cracks on both webs extending to about half the overall depth. The maximum deflection at this load was 13.6 mm.

Loading continued and crackling noises of carbon fibre debonding were heard at 290 kN, and they increased in intensity until debonding of the straps occurred at a load of 305 kN – first the east side then the west. Immediately following the debonding on the east side a large flexural crack opened near the north load position. Maximum load was 310 kN.

4.2.2 Results for Beams with Clamped Carbon Fibre Straps Loaded 1500 mm from End

As the beams were failing due to the sudden debonding of the carbon fibre strap, it was decided to secure the ends of the straps with a mechanical anchorage to investigate improved performance. Due to the narrow web and the fact that the strap was already attached, the option adopted was to apply a steel plate bolted to the concrete through the strap. The plate was 250 mm long by 100 mm wide by 10 mm thick with two Hilti high tensile 18 mm diameter fasteners installed between the main reinforcing bars and to avoid the stirrups.

Installation of the plate was achieved by firstly coring the straps with an oversized hole, then using a masonry drill, creating the holes in the concrete, applying epoxy to the plate, inserting the bolts into the concrete and tightening. The epoxy was allowed to cure, and just prior to testing all bolts were tightened to a specific torque (100 Nm for beam 1-2 and 120 Nm for beam 4).

The testing procedure was similar to the beams without the clamping plates with the beams loaded to 200 kN, unloaded and reloaded. The permanent set upon unloading was 1.9 mm for beam 1-2 of bridge 4658 and 1.5 mm beam 4 of bridge 413. Several flexural cracks were visible along the middle section of the beams and these had extended up the web to the underside of the shear key.

With beam 1-2 of bridge 4658, noises of carbon fibre debonding began at 300 kN, and as the load increased there were many sharp cracking noises followed by release of the middle strip of the carbon fibre strap on the west side. This middle section had torn away at the clamp bolt and separated by longitudinal splits from the remainder of the strap, which although debonded and slipping had still been retained beneath the end clamping plates. The maximum load was 320 kN, and a dominant flexural crack opened at the north load position.

Beam 4 of bridge 413 failed in a similar manner with the noises starting at 300 kN and debonding and slip of the straps occurring before longitudinal splitting of the strap and large flexural cracks opening. The maximum load was 325 kN.

4.3 Results: - Shear Tests

Several trial beams were used to investigate the shear capacity of the inverted U beams and these early trials indicated the shear strength was substantial. A brief summary of the key test has been included here for comparison with the beams strengthened with carbon fibre.

As a result of the trial testing, it was decided to test the key beams with a single point load 600 mm from the end face over a span of 6200 mm. The main details of the failures are summarised in Table 3 and description of a typical test follows.

Table 3: Summary of Failure Details for Shear Tests

Bridge	Beam	Load at Failure (kN)	Shear at Failure (kN)	Max Moment at Failure (kNm)
413	5	470*	430	215
4658	4-2	435	400	200
4658	4-3	450	415	205

Note: * Loading was discontinued as maximum capacity of jacking system had been reached

4.3.2 Description of Typical Shear Test

Beam 5 from bridge 413 was loaded using the single 50 tonne jack with its centre 600 millimetres in from the end face with the full span from centre of supports of 6200 mm.

Some initial cracks were observed at a load of 220 kN and by 285 kN the flexure cracks directly beneath the load had extended up into the shear key. At 325 kN audible cracking was noted and a flexure-shear crack had opened up along with several equidistant vertical flexural cracks approximately 250 mm apart.

The cracks slowly extended as the load was increased until 450 kN, at which point the first shear cracks appeared in the direct line from inside face of support to the load point. The maximum load reached was 470 kN (capacity of the jacking system) and significant noises were heard as the shear crack extended to the top of the shear key. The final web-shear crack was quite fine. With a greater capacity load system it is expected that a slightly larger failure load would have been achieved. The fine shear cracking on the east side of the beam occurred approximately 300 to 350 millimetres from the end of the beam near the soffit and was steeply inclined until into the shear key where it flattened and continued under the loading position. In this way it cracked around the hook anchorage, crossing two longitudinal bars instead of three, and the second stirrup along the beam. The shear crack on the west side developed closer to the support, also crossing two longitudinal bars and the second stirrup.

4.3.2 Additional Shear Tests

Several other tests were conducted to investigate the shear capacity and in particular the influence of different load positions which varied the shear span to depth ratio.

Beam 3 from bridge 413 was chosen to explore behaviour of the full span beam under a single point load near the end. Similar tests were conducted for beams strengthened with carbon fibre. The first test was with a single point load 1000 mm from the end and a span of 6200 mm. Initial flexural cracks were observed at 120 kN, significant extension of cracks occurred from 260 kN and final failure by flexure was at 320 kN. The second test on this beam was on a shortened span of 4200 mm with the single load at 1200 mm from end face. As the beam had already been tested in flexure the initial cracking observed was at 100 kN with cracks from the first loading reopening. The cracks had extended to the shear key by 260 kN and initial indications of flexural failure were apparent at 330 kN. Final flexural failure was at a load of 380 kN. A summary of key details has been set out in Table 4.

Table 4: Summary of Failure Details for Shear Tests on Beam 3 from Bridge 413

Bridge	Beam	Load position (mm from end)	Span (mm)	Load at Failure (kN)	Shear at Failure (kN)	Max Moment at Failure (kNm)
413	3 A	1000	6200	320	275	245
413	3 B	1200	4200	380	280	310

4.4 Results: - Shear Tests with External Strengthening by Carbon Fibre Straps

Beam 3-3 of bridge 4658 was tested twice, initially, with a single load point 1000 mm from the end face over a span of 6200 mm (test A), and secondly, with the load at 1200 mm from end face and the span reduced to 4200 mm to avoid the failed end section of the beam from the initial test (test B). The carbon fibre straps were cut at midspan to ensure failure would not precipitate complete debonding of the straps, thus enabling the two tests to be conducted on the one beam. The key results from testing of this beam are shown in Table 5.

Table 5: Summary of Failure Details for Shear Tests on Beams with External Strengthening by Carbon Fibre Straps

Bridge	Beam	Load at Failure (kN)	Shear at Failure (kN)	Max Moment at Failure (kNm)
Loaded with Single Load Near One End of Beam (1000,6200 span & 1200,4200 span)				
4658	3-3A	400	342	308
4658	3-3B	400	295	325

For test A, a change in the slope of the load-deflection plot indicated that cracking initially occurred at 150 kN and fine flexural cracks near the load were visible at a load of 190 kN. A significant flexural crack opened on the east side approximately 700 mm from end face at a load of 270 kN causing a small change in stiffness (visible in load-deflection plots). As loading continued some fine cracks opened including some web shear cracks between the load and support and as the load reached 375 kN some cracking noises suggesting the carbon fibre strap was debonding were heard. The maximum deflection was approximately 18 mm at this load. At 390 kN a load sharp noise was heard and soon after the north east strap violently debonded and a large flexural shear crack formed from around 600 mm from end at the bottom of the web up to the load point.

Failure on the west side followed with a slow peeling failure of the strap with large pieces of concrete remaining attached to the end of the strap (see Figure 3). The dominant shear crack that formed on this side was closer to the end face and formed just outside the region where the carbon fibre strap was attached to the bottom surface. The long dowel formed by the main reinforcement and the lack of effective stirrups crossing the cracks can also be seen clearly in Figure 3.

In test B, the initial cracking was at a load of 140 kN with a deflection of approximately 2.5 mm. The cracks extended and at 200 kN additional flexural cracks opened. At 260 kN a loud cracking noise was heard that suggested the carbon fibre strap may have debonded but the load was increased and it was not until a load of 360 kN that further noises were heard. At approximately 390 kN there was a single loud noise of carbon fibre debonding, and a significant flexural crack was observed under the load position on east side. Soon after the strap on the east web rapidly debonded from the centre of the beam scattering pieces of epoxy, but the last 300 mm of the strap remained attached to the concrete. Almost immediately following this the west web strap debonded in a similar fashion. The flexural cracks opened rapidly with large deflections (greater than 100 mm) occurring, and the maximum load sustained was 400 kN.

Figure 5: Failure Showing Extensive Cracks And Peeling of Carbon Fibre Strap on West Side for Beam 3-3 from Bridge 4658

Note that the visible stirrup loops around the upper layer of main reinforcement which terminates at this crack

4.5 Results: - Material Properties

4.5.1 Concrete Compressive Strength

The strength of the concrete was determined from 12 concrete cores taken from the web of 7 different beams. The mean strength was 62.1 MPa, the standard deviation 5.1 MPa and the characteristic strength 59.5 MPa. However, three of these core samples had some steel reinforcement in the core. Discarding these results, the total number of valid samples is reduced to nine, the mean reduces to 60.7 MPa, and the characteristic strength becomes 57.7 MPa.

The three bridges were constructed at three different times; therefore the bridges could be taken as three separate populations. However the data supports the hypothesis that all samples come from the same population, so it would be reasonable to use a characteristic value of 57.7 MPa for all beams.

4.5.2 Steel Reinforcement Tensile Strength

Sixteen plain round reinforcing bars were removed from four test beams to establish the tensile capacity of the reinforcement. Tests were conducted in accordance with AS1391 (6) in an Avery Denison Universal testing machine.

The characteristic tensile strength was calculated as 455 MPa, and the characteristic yield strength was calculated as 280 MPa.

4.5.3 Pull-off Strength of Epoxy Used for Carbon Fibre Straps

Mechanical pull-off tests were undertaken to confirm the bond strength of the epoxy used to attach the Carbon Fibre Straps. A Scheinck TREBEC machine was used for the pull-off with steel cylindrical discs (50 mm diameter) known as dollies attached to the concrete. A summary of the average results has been set out in Table 6. The tabulated results show that for all the valid samples (see notes above) the bond strength was in excess of the required design value of 1.5 MPa.

Table 6: Average Results for Dolly Pull-off Tests

Type of Test	Average Tensile Force (kN)	Average Tensile Stress (MPa)	No of Tests
Epoxy to U/side of beam – Steel Dolly	15.8	8.0	5
Epoxy to single laminate on U/side of web	8.3	4.2	2
Epoxy to double layer laminate on U/side of web	11.5	6.0	2
Epoxy to concrete on U/side of web	7.8	4.0	2

5 SUMMARY

The key results from this investigation are summarised in the following sections.

5.1 Materials

Both the concrete and steel were significantly stronger than expected and this contributed to the good flexural performance of the beams.

- The design strength of the concrete was 24 MPa, but cores taken during this testing program gave a characteristic strength of 58 MPa and a density of 2500 kg/m^3.
- The main reinforcement was 22 mm diameter plain bars, and tension tests indicated a characteristic yield point of 280 MPa and a characteristic tensile strength of 455 MPa.

5.2 Flexural Capacity

- When tested with two point loads 1200 mm apart (representing half of a dual axle loading), failure loads for the beams in bending ranged from 230 kN to 250 kN.
- Flexural failures were very ductile. The main reinforcement yielded and a plastic hinge developed, with the maximum deflection usually exceeding 200 mm before compression failure of the concrete.
- The beams deflected approximately 8 mm under a total load of 100 kN, and approximately 20 mm when the load was 200 kN. A permanent set of approximately 1.5 mm to 2 mm deflection occurred when beams loaded to 200 kN were unloaded.
- Loads of less than 100 kN (maximum likely load on a single beam from typical dual axle truck traffic) are unlikely to crack the beam or cause permanent deflection.

5.3 Shear Capacity

- The beams did not fail in shear with two point loads 1200 mm apart.
- Shear failures were produced with a single point load close to the end of the beam at loads ranging from 435 to 470 kN.
- Even with a single point load, flexural failure dominates when the load is approximately 900 mm from end of beam. With a single load 1000 mm from end, a flexure failure at a load of 320 kN occurred.
- All shear failures during testing gave some warning of failure, and showed large displacements at failure due to dowel action from main reinforcement.

5.4 External Strengthening by Carbon Fibre Straps

- When tested with two point loads 1200 mm apart, failure loads for the beams in bending ranged from 300 kN to 325 kN.
- The typical failure for all of the beams with external strengthening by carbon fibre straps was due to a sudden delamination of the strap at the epoxy- laminate boundary.
- External steel anchor plates were attached to the ends of two beams to clamp the straps. This provided a more controlled failure as the straps slipped without detaching completely. The load increase from these clamped beams was modest (approximately 5% greater than beams with unclamped carbon fibre - but the small number of tests limits validity of this comparison).

6 ACKNOWLEDGEMENT

The author gratefully acknowledges the support and sponsorship of this work by Main Roads, Western Australia. The work was conducted under contract through BGE Consultants. The author also thanks the many others who assisted with the project and in particular: - Timothy Kent, Clayton Lewis, Sebastian Miraudo, and Edmond Mo, who worked on this project as part of their final year undergraduate projects, Andrew Sarkady and Kim Boyd from MBT (Australia) Pty Ltd, Lloyd Margetts from MRWA, Raph Woon and Harry van Kleef from BGE Consultants.

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