University of Southern Queensland
Faculty of Engineering and Surveying

Effect of Existing Cracks in Shear Strengthening of Concrete Girders with External Post-tensioning

A dissertation submitted by

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in fulfilment of the requirements of

Courses ENG4111 and 4112 Research Project

towards the degree of

Bachelor of Engineering (Civil)

Submitted: October, 2005

Abstract

There is a need to strengthen bridges structures due to increased traffic loading and aging of bridges. Full bridge replacement poses the problems of high cost and disruption to traffic, so a suitable method or repair and strengthening are required.

External post-tensioning is considered as one of the effective methods of to strengthen bridge structures. A common deterioration of bridges is shear cracking. Existing shear cracks can limit the effectiveness of external post-tensioning.

Epoxy injection is a method of structurally repairing cracks. The injection of epoxy resin could possibly repair the shear cracks. This paper is utilizes the rehabilitation technique of strengthening with external post-tension and crack repair with epoxy injection.

The effect of the existing shear crack in an externally post tensioned reinforced concrete beam is a complex function depends on a number of parameters including the nature of the crack, concrete strength, prestressing force and amount of shear reinforcement. The amount of shear reinforcement could significantly affect the amount of stress transfer across an existing shear crack.

The repair strength achieved for reinforce concrete beams was shown to be influenced by the condition of the major diagonal crack after initial damage. This conclusion was supported by surface-strain measurements, load deflection data and crack measurements.

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Certification

I certify that the ideas, design and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Signature

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Date

Acknowledgements

The author would like to express sincere gratitude to his supervisors Dr. Thiru Aravinthan for his guidance and support throughout the course of this project.

This research involved a large portion of experimental investigation, and so required a great deal of assistance throughout. Many thanks are directed to Dan Eising and Mohan Trada for their invaluable assistance during the construction and testing stages of this research.

The author would also like to thank his fellow peer, Kumaran Suntharavadivel for his kind assistance throughout the course of this project.

Mr Todd Whittaker

November, 2007

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Nomenclature

A_g = gross cross-sectional area

A_{pt} = cross-sectional area of prestressing steel

A_{sc} = cross-sectional area of compressive reinforcement

A_{st} = cross-sectional area of tensile reinforcement

A_{sv} = cross-sectional area of shear reinforcement

$A_{sv\text{-}min}$ = cross-sectional area of minimum shear reinforcement

$A_{sv\text{-}max}$ = cross-sectional area of maximum shear reinforcement

${}_v\alpha$ = distance from load point to nearest support

b = width of rectangular cross-section

b_{ef} = effective width of the compression face

b_v = effective width of web for shear (equal to b for rectangular cross-sections)

C_c = compressive force in the concrete

C_s = compressive force in the compressive reinforcement

D = depth of section

d = depth to resultant force in tensile steel at M_u

d_n = depth to neutral axis in a section

d_o = distance from the extreme compressive concrete fibre to the centroid of the outer most layer of tensile reinforcement

d_p = depth to the prestressing steel

d_{sc} = depth to centre of compressive reinforcement

d_{st} = depth to centre of tensile reinforcement

e = eccentricity of prestressing force from the centroidal axis of the section

E_s = Young's modulus of steel

f'_c = characteristic compressive cylinder strength of concrete at 28 days

f_{py} = yield strength of prestressing steel

f_{sy} = yield strength of reinforcing steel

$f_{sy.f}$ = yield strength of shear reinforcement

I_g = second moment of area of the uncracked section

k_u = ratio of neutral axis depth to outer most layer of tensile steel depth

L = distance between supports

M_{dec} = decompression moment

M_u = ultimate moment capacity 12

P = prestressing force

P_u = theoretical ultimate load capacity of a beam

$P_{u.f}$ = theoretical ultimate flexural capacity load of a beam

$P_{u.s}$ = theoretical ultimate shear capacity load of a beam

P_{ue} = experimental ultimate load capacity of a beam

P_v = vertical component of prestressing force

s = spacing of shear reinforcement

T_p = tensile force in prestressing steel

T_s = tensile force in tensile reinforcement

V_{dec} = shear force at decompression moment

V_u = ultimate shear strength

V_{uc} = concrete component of ultimate shear strength

V_{us} = shear reinforcement contribution to ultimate shear strength

$\beta_1\beta_2\beta_3$= multiplying factors for determining V_{uc}

γ= ratio of the depth of the assumed rectangular compressive stress block to d_n at M_u

γ_{xy} = shear strain

ε_{max} = maximum principal strain

ε_{min} = minimum principal strain

ε_{sc} = strain in the compressive reinforcement

γ_{st} = strain in the tensile reinforcement

ε_u = extreme compressive fibre strain at ultimate strength in pure bending

θ_v = angle between the concrete compression strut and the longitudinal axis of the beam

$\sigma_{p.ef}$ = effective stress in the prestressing steel

σ_{pu} = ultimate stress in the prestressing steel

CHAPTER 1

INTRODUCTION

1.1 Project Aim

This project aims to investigate the effectiveness of a technique for the repair of shear cracks of reinforced concrete beams. To investigate the effect of how shear reinforcement ratio affects the behaviour of this repair technique. The aim is experimentally investigated.

Furthermore this project aims to investigate the effect of shear reinforcement ratio in the behaviour of shear strengthening of reinforcement concrete beams using epoxy injection and horizontal external post-tensioning. This is an ongoing project which is an extension of the research done by Steven Luther and Paul Bolger. This project will further the research done Steven Luther. This research "*Effect of Existing Cracks in Shear Strengthening of Concrete Girders with External Post-tensioning*" Luther, USQ, 2005 will form the base of which this project will be conducted.

This will be predominately done though experimental tests conducted on four design beams. Experimental results will be used to make a comparison of existing design models will also be made.

To achieve these aims, the following objectives had to be met:

1. Research and review background information on the shear strengthening of concrete girders using epoxy injection and external post-tensioning.

2. Design model test beams for experimental investigations, taking into account previous test results.

3. Prepare model beams, and arrange testing devices.

4. Conduct tests on the model beams, and record observed results.

5. Evaluate and analyse the test results of the different model beams.

6. Arrive at a conclusion for the project, which will better explain the shear behaviour of rehabilitated girders using epoxy injection and external post tensioning.

1.2 Background

Most of the bridges on the road and motorway network were constructed in the 1960s and 1970s, and some bridges on the road network are very much older. Most were built in reinforced or pre-stressed concrete and have steel embedded within them. A combination of natural weathering, chemical attack, low quality construction materials, can cause structural deterioration. Over the years, traffic flows and the maximum permitted weight of heavy goods vehicles have both increased, and required standards of safety have improved. A combination of these factors and the deterioration of elements of a bridge create the need for strengthening.

In response to the need for a simple, efficient method to strengthen existing bridges, the use of external post-tensioning has gained wide acceptance. External post-tensioning is considered as one of the effective methods of to strengthen bridge structures. A common deterioration of bridges is shear cracking. Existing shear cracks can limit the effectiveness of external post-tensioning.

Epoxy injection is a method of structurally repairing cracks. The injection of epoxy resin could possibly repair the shear cracks. This paper is utilizes the rehabilitation technique of strengthening with external post-tension and crack repair with epoxy injection.

A structural rehabilitation of reinforced concrete beams in shear was studied, with particular attention paid to the effect of shear reinforcement ratio. A series of model beams, reinforced with a high shear reinforcement ratio, were loaded in the laboratory until a major diagonal shear crack developed on both shear plans. Individual beams were unloaded, repaired, again loaded to failure. Scaled size beams were tested. A conclusion is drawn form surface-strain measurements, load deflection data and crack measurements.

A laboratory study was conducted on a four reinforced concrete beams concrete beams loaded in four point flexural loading to understand the behavior repaired by the rehabilitation method. The influence on final repair strength from the amount of shear reinforcement was emphasized.

This is an on-going project aimed to experimentally investigate the effect of shear reinforcement ratio in a reinforced concrete beam strengthened by external post tensioning. A comparison of existing design models for shear strength predictions will also be made.

1.3 Summary

This project aims to investigate the effectiveness of a technique for the repair of shear cracks of reinforced concrete beams. Furthermore this project aims to investigate the effect of shear reinforcement ratio in the behaviour of shear strengthening of reinforcement concrete beams using epoxy injection and horizontal external post-tensioning.

CHAPTER 2

LITERARURE REVIEW

2.1 Introduction

Previous research has been done on "Horizontal post tensioning" by Luther (2005) and there was found to be a 58% increase in shear capacity when the "Horizontal post tensioning" technique was used in conjunction with the epoxy resin, while without the epoxy there was no effect of shear strengthening to be seen.

The research investigated by Luther (2005) was with minium shear reinforcement. Minium shear reinforcement has ligatures spacing of 250mm. This allowed 1 ligature in the shear zone of the experiment with 2 ligatures under the loading points. After pre loading this ligature may have significant failure. The ligature may have yielded or possibly broken proposing of a lower regain in shear strength.

This project will investigate the shear strength of the combined repair technique with an increase of shear reinforcement. The shear reinforcement will have a spacing of 180mm. This will allow 3 ligatures in the shear zone. The experiments will be conducted under the same conditions as Luther (2005) to allow a comparison in the regain in shear strength.

2.2 Need for Strengthening

In response to the demand for faster and more efficient transportation systems, there has been a steady increase in the weight and volume of traffic using national highway systems throughout the world. As well as increases in legal vehicle loads, the over-loading of vehicles is a common problem and this must also be considered when designing or assessing bridges. As a result, many bridges are now required to carry loads significantly greater than their original design loads.

Most of these bridges have either flexural or shear cracks that need to be repaired to protect the reinforcement from corrosion, and reduce deflections to below serviceable limits. There is a need to strengthen bridges structures due to increased traffic loading and aging of bridges. Full bridge replacement poses the problems of high cost and disruption to traffic, so a suitable method or repair and strengthening are required.

There has been extensive research into the flexural strengthening of concrete bridges using external post tensioning, but little on shear strengthening. The flexural behaviour of new and existing bridge members by external post-tensioning has been studied in detail by many researchers. However, there have been relatively limited investigations on the shear strengthening and the effect of shear in externally post-tensioned members (Tan & Ng 1998).

From the previous studies on flexural strengthen-ing (Aravinthan, Sabonchy & Heldt 2004; Harajli 1993), it is proven that the flexural cracks were al-most or completely closed by the application of external post-tensioning. Hence, they have no influence on the capacity of the concrete beams. While a number of experimental studies attempted to study the effect of existing shear cracks in the reinforced concrete members (Khaloo 2000; Teng et al. 1996), the effect of the existing shear cracks in externally post-tensioned member has not been investigated adequately.

2.3 Selection of Appropriate Strengthening Technique

The selection of ani appropriate method for strengthening a particular bridge depends on a number of factors. The type of structure, the magnitude of the strength increase required and the associated costs are the main parameters to be considered. Many strengthening schemes are applicable to particular structural types and have limits on the extent to which strength can be increased. Strengthening costs would certainly be lower than bridge replacement, but the selection of a particular method of strengthening would need to be justified on economic grounds. It is important to consider, not only the initial capital costs of the strengthening project, but also the maintenance costs associated with the future in-service behaviour. The condition of the existing bridge is an important consideration. If the bridge is in bad condition, then future maintenance and safety problems might override the benefits of the reduced capital costs of strengthening and provide justification for bridge replacement. The strength and condition of the substructure must not be ignored and strengthening should not proceed without giving due consideration to the capacity of the bridge piers, abutments and foundations. The difficulties associated with traffic management and the costs arising from traffic delays should be considered in the economic justification. In some cases, this may limit the use of certain methods of strengthening.

Many strengthening techniques have general applicability, but some may be specific to particular bridge types and configurations. The decision to adopt a particular scheme is based on the consideration of a wide range of parameters. The remainder of this paper is concerned with external post-tensioning for bridge strengthening. The general principles, advantages and disadvantages are described in the following sections.

2.4 External Post-tensioning

The use of external prestressing as a means of strengthening or rehabilitating existing bridges has been used in many countries and has been found to provide an efficient and economic solution for a wide range of bridge types and conditions. The technique is growing in popularity because of the speed of installation and the minimal disruption to traffic flow which can, in many cases, be the critical factor in decisions regarding strengthening. In spite of its obvious advantages, there is a lack of general information on how it can be applied and there are no specific guidelines available on this method of strengthening.

External post-tensioning refers to the method of post-tensioning in which the strengthening system is installed outside the structural element. This strengthening technique is designed similarly to the unbounded prestressing systems. External post-tensioning requires access to the sides and sometimes the ends of the member. Tendons are connected to the structure at the anchor points, typically located at the member ends.

Pisani (1999) identifies that external post-tensioning appears to be the most promising form of rehabilitation or strengthening of statically determinant structures, particularly bridges. Pisani (1999) found that applying post tensioning to a beam without shear cracking, increases the shear capacity of the beam. This points towards post tensioning increasing a damaged member's shear capacity, if the shear cracks are successfully repaired with epoxy.

AS3600 Clause 8.1.6 states that for a beam with a span-to-depth ratio of 35 or less, the stress in a tendon not yet bonded at ultimate strength, shall be determined from

$$\sigma_{pu} = \sigma_{p.ef} + 70 + \left(\frac{f'c.b_{ef}.d_{ef}}{100.A_{pt}} \right) \leq \sigma_{p.ef} + 400$$

and for a beam with a span-to-depth ratio of greater than 35

$$\sigma_{pu} = \sigma_{p.ef} + 70 + \left(\frac{f'c.b_{ef}.d_{ef}}{300.A_{pt}} \right) \leq \sigma_{p.ef} + 200$$

Where,

$\sigma_{p.ef}$ = effective prestress

$f'c$ = 28 day compressive strength

b_{ef} = effective width of the compression face

d_p = depth to prestress tendons

A_{pt} = Area of prestressing tendon

2.5 Epoxy injection

Epoxy injection can be used as means of restoring shear cracks. This repair method involves the process by which resins are injected in a controllable manner to fill or treat a crack, thereby restoring the structure to its original design capability.

It is good for structural repair, as it has tensile and compressive strengths greater than concrete. According to Epoxysystems (2001), epoxy used for bonding concrete cracks has a tensile strength of 34-55 MPa, and a compressive strength of 70-80 MPa. Due to the strong bond formed with concrete, and the high strength characteristics of epoxy, cracked members repaired with epoxy injection should regain their original strength.

2.6 Epoxy Injection Combined with External Post-tensioning

Woods (2004) conducted model testing of a bridge headstock repaired by epoxy injection and external post-tensioning. He found that by repairing the existing shear cracks with epoxy injection and post-tensioning, significant increases in ultimate capacity and stiffness were achieved. He also strengthened one of the cracked model headstocks with just post-tensioning, with a slight increase in capacity found.

2.7 Shear Capacity Predictions

AS3600 Clause 8.2.2 states the design shear strength of a beam shall be taken as _Vu
Where,

$Vu \quad = V_{uc} + V_{us}$

$\varphi \quad = 0.7$ ((strength reduction factor used for shear strength in limit state design)

Here,

$V_{uc} \quad$ = Shear resisted by concrete and longitudinal bars

V_{us} = Shear resisted by ligatures

The shear force resisted by the ligatures is the same for both reinforced beams and post tensioned beams. For perpendicular shear reinforcement, Clause 8.2.10 of AS3600 states

$$V_{us} = \left(\frac{A_{sv} . f_{sy.f} . d_0}{s} \right) \cot \theta_v$$

Where,

s = centre to centre spacing of shear reinforcement

θ_v = angle between the axis of the concrete compression strut and the longitudinal axis of the member, taken as varying linearly from 30o when $V^* = \varphi V_{u.\min}$ to 45^{o} when $V^* = \varphi V_{u.\max}$

A_{sv} = cross sectional area of shear reinforcement

$f_{sy.f}$ = yield strength of shear reinforcement

2.8 Reinforced Concrete Beam

For a reinforced concrete beam, clause 8.2.7.1 of AS3600 states

$$V_{uc} = \beta_1 . \beta_2 . \beta_3 . b_v . d_o . \left(\frac{A_{st} . f'c}{b_v . d_o} \right)^{\frac{1}{3}}$$

Where,

$\beta_1 = 1.1 \times \left(1.6 - \frac{d_0}{1000}\right) \geq 1.1$

β_2 = 1, generally for members without significant axial force; or

$= 1 - \left(\frac{N^*}{3.5A_g}\right) \geq 0$, for members subject to significant axial tension;

or

$= 1 - \left(\frac{N*}{14A_g}\right)$ for members subject to significant axial compression.

This factor illustrates the effect of axial force on the propagation of shear cracks. A compressive force reduces crack propagation, and therefore increases the shear resisted by the concrete. Conversely, axial tension encourages the shear cracks to form.

β_3 = 1; or may be taken as –

$= 2 \times \frac{d_o}{\alpha_v}$ but not greater than 2, provided that the applied loads and the support are orientated so as to create diagonal compression over the length α_v

b_v = width of the section

d_o = distance from top edge to centre of bottom longitudinal reinforcement

$f'c$ = 28 day concrete compressive strength

A_{st} = cross sectional area of longitudinal reinforcement

A_g = gross cross sectional area

N^* = design axial force

α_v = shear span, the distance from the section being considered to the face of the nearest support.

2.9 Externally Post-tensioned Concrete Beam

For a post tensioned beam, clause 8.2.7.2 (a) of AS3600 states that for flexure-shear cracking

$$V_{uc} = \beta_1 .\beta_2 .\beta_3 .b_v .d_o .\left(\frac{(A_{st} + A_{pt}) f'c}{b_v .d_o} \right)^{\frac{1}{3}} + V_o + P_v$$

Where,

$\beta_1, \beta_2, \beta_3, b_v, d_o f'c$ and A_{pt} are the same as for the reinforced beam

P_v = 0, as the post-tensioning rods are horizontal

V_o = the shear force which would occur at the section when the bending moment at the section was equal to the decompression moment (Mo)

$$= \frac{M_0}{\left(\frac{M^*}{V^*} \right)}$$ for simply supported conditions, where M^* and V^* are the

bending moment and shear force respectively, due to the same design loading

$$M_o = \left(\frac{P}{A_g} + \frac{P.e.y_b}{I_g} \right) \frac{I_g}{y_b}$$

Where,

P = post-tensioning force

= 150kN

A_g= gross cross-sectional area

= b.D

= 150 x 300

= 45000mm2

e = eccentricity from the centroid of the section

= 50mm

y_b = distance from the centroid to bottom edge

= 150mm

I_g = Second moment of area of the uncracked section

For web-shear cracking, AS3600 clause 8.2.7.2 (b) states:

$$V_{uc} = V_t + P_v$$

Where,

V_t =the shear force, which in combination with the prestressing force and other action effects at the section, would produce a principle tensile stress of $0.33\sqrt{f'c}$ at either the centroidal axis or the intersection of flange and web, whichever is more critical.

P_v = vertical component of prestress force

2.10 Summary

This section has given an overview of the problem that exists in deteriorating bridge structures, and background on research conducted on epoxy repairing of cracks and post-tensioning. With only limited research having been conducted on combining epoxy injection with external post-tensioning to shear strengthen concrete members, both repair methods were also looked at individually. The AS3600 prediction equations for the shear capacities of reinforced and post-tensioned beams were also looked at.

CHAPTER 3

DESIGN METHODOLOGY

3.1 Introduction

The effect of shear reinforcement in the technique described in Chapter 2 will be investigated experimentally on 4 model beams. The follow is a description of what each beam will be used for in this research.

Specimen B1– This will be used to determine the strength of a new reinforced concrete beam with no external post tensioning

Specimen B2- The beam will be preloaded to induce shear cracks, the beam will be strengthened with external post tensioning.

Specimen B3 - The beam will be preloaded to induce shear cracks, the beam will have it cracks repaired with epoxy injection it will also be strengthened with external post tensioning.

Specimen B4- This will be used to determine the strength of a new reinforced concrete beam with external post tensioning.

Specimen B1 will be loaded to it ultimate shear capacity. Specimen B2 and B3 will be loaded to approximately 90% of Specimens B1 ultimate shear capacity in the preloading to induce shear cracks. Specimen B1 and B4 will be used as control beam to make comparisons of the repaired beams against new beams.

The model beams were designed to facilitate testing of scaled sized proto-types. The section chosen was 300mm high by 150mm wide. The selection of loading position and span were made to ensure shear failure over flexural failure. Four point loading was chosen to cause shear failure in both ends of the beam. This type of loading also causes a lower design moment compared to midspan loading, which will encourage shear failure over flexural failure (Luther, 2005, p. 17). The specimen size and loading points are shown in Figure 3.1

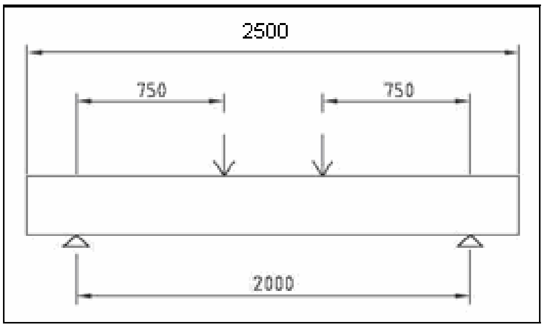

Figure 3.1: Specimen Size and Loading Position

(Figure adopted form Luther, 2005)

The project aims to investigate the effect of high shear reinforcement and its effects on the rehabilitation technique. To ensure shear failure, the beam was designed to have at a higher flexural failure load than its shear failure load. This was done through the spacing and size of the shear ligatures. R6 (round) bars were chosen for the ligatures. The Shear ligatures were chosen at a spacing of 180 mm. This was determined as highest amount of shear reinforcement while still under acceptable limits of a lower shear capacity over flexural capacity.

3.2 Preliminary Design

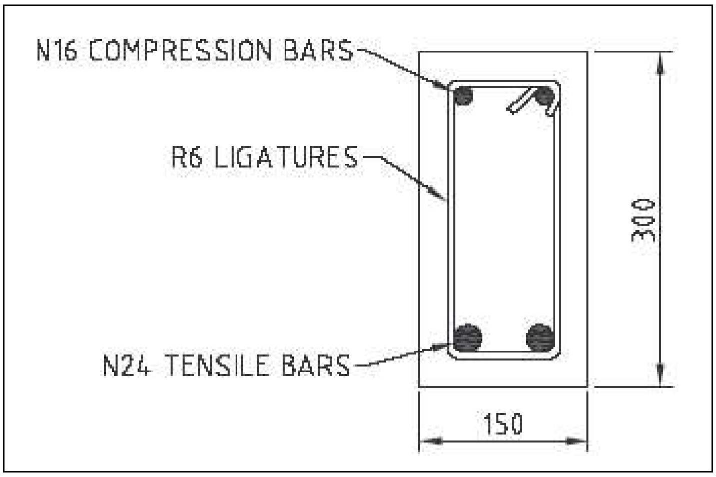

Figure 3.3: Cross Section of the Design Specimen
(Figure adopted form Luther, 2005)

Reinforcement details are shown in Figure 3.3. R6 bars were used for the ligatures, with a spacing of 180 mm. Three shear ligatures were present in the within the shear spans tested

For this testing, the model beams needed to be designed as both a reinforced beam, and a post-tensioned beam. AS 3600 prediction equations have been used in determining the flexural and shear strength of both the reinforced and post-tensioned concrete beam.

3.2.1 Design of Reinforced Beam

The ultimate moment capacity, Mu, and the shear capacity, Vu, for the design specimen before post-tensioning are calculated in this section.

3.2.2 Flexural Capacity

The flexural capacity of the specimens is calculated to ensure the mode is shear failure rather than flexural. The calculations below were used to determine the flexural capacity of the member.

In a doubly reinforced section at ultimate moment capacity, the resultant tensile force in the bottom steel, Ts, is equal to the compressive force in the concrete, Cc, plus the compressive force in the top steel, Cs (Luther 2005, p 20). Once the forces and their points of action are known, the moment capacity can be found by taking moments about the bottom tensile steel (Luther 2005, p 20). The internal strains, stresses, and forces in the section are shown in Figure 4.4, and the calculations to find the specimen's flexural capacity are shown below.

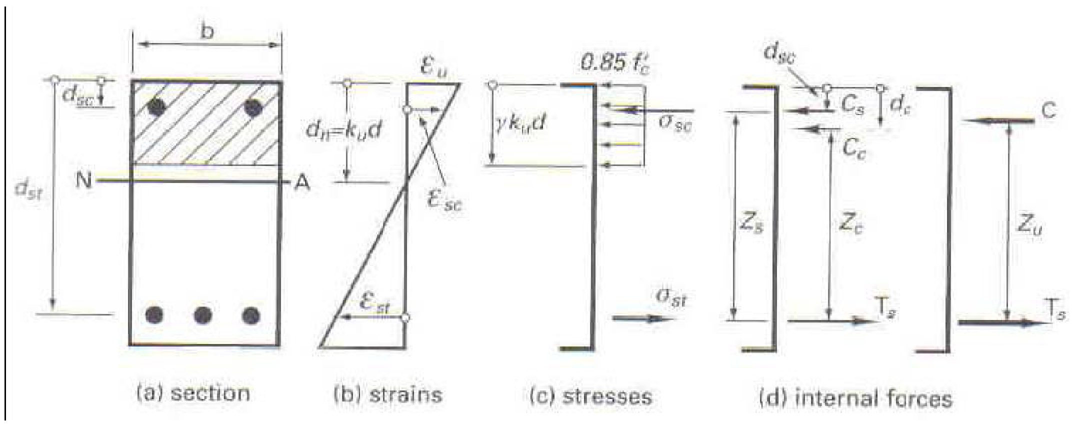

Figure 3.3: Doubly Reinforced Section at Ultimate Moment

(Source: Warner et al, 1998)

Section Properties

$Pf'c = 32$ MPa

$$\gamma = 0.85 - 0.007(f'c - 28)$$
$$= 0.85 - 0.007(32 - 28)$$
$$= 0.822$$

Reinforcing Properties

$$fsy = 500MPa$$

Depth to compression steel:

$$dsc = 25 + 6 + \frac{16}{2} = 39mm$$

Depth to tensile steel:

$$d = 300 - 25 - 6 - \frac{24}{2} = 257mm$$

Area of compression steel:

$$Asc = 400mm2$$

Area of tensile steel:

$$Ast = 900mm2$$

Initially assume all reinforcement yields before Mu, therefore;

Tensile steel force:

$$Ts = fsy.Ast$$
$$= 500 \times 900$$
$$= 450 \times 10^3 N$$

Compression steel force:

$$Cs = fsy.Ast$$

$$= 500 \times 400$$

$$= 200 \times 10^3$$

Concrete compressive force:

$$\text{Cc} = 0.85.f'c.b.d_o.\gamma$$

$$= 0.85 \times 32 \times 0.822 \times d_o$$

$$= 3353.76\,d_o$$

As the sum of the forces equals zero:

$$3353.76 d_n + 200 \times 10^3 = 450 \times 10^3$$

$$d_n = 74.5mm$$

Checking the compressive reinforcement has yielded:

$$\varepsilon_{st} = \varepsilon_c \left(\frac{d_n - d_{sc}}{d_n} \right)$$

$$= 0.003 \times \left(\frac{74.5 - 39}{74.5} \right)$$

$$= 0.0014$$

As εsc < 0.0025, the assumption that the compressive reinforcement had yielded is incorrect. The compressive forces will be recalculated knowing the compressive reinforcement is in the elastic range, with:

$$C_s = E_s.\varepsilon_u\left(\frac{k_u.d_{st} - d_{dc}}{k_u.d_{st}}\right)A_{sc}$$

By equating the sum of the forces to zero, the neutral axis depth is found using a quadratic equation to find ku:

$$k_u^{\ 2} + u_1.k_u - u_2 = 0$$

Where,

$$u_1 = \frac{\varepsilon_n.E_s.A_{sc} - f_{sy}.A_{st}}{0.85f'c.\gamma.b.d_{st}}$$

$$u_2 = \frac{\varepsilon_n.d_{dc}.E_s.A_{st}}{0.85f'c.\gamma.b.d_{st}^{\ 2}}$$

Therefore,

$$u_1 = \frac{0.003\times200\times10^3\times400 - 500\times900}{0.85\times32\times0.822\times150\times257}$$

$= -0.245$

$$u_2 = \frac{0.003\times39\times200\times10^3\times400}{0.85\times32\times0.822\times150\times257^2}$$

$= 0.0425$

This gives the quadratic equation:

$$k_u^{\ 2} - 0.245 \times k_u - 0.0425 = 0$$

Solving the quadratic equation:

$$k_u = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$k_u \frac{0.245 \pm \sqrt{(-0.245)^2 - 4 \times 1 \times 0.0425}}{2 \times 1}$$

$= 0.362$ or -0.117

Taking the positive value:

$ku = 0.362$

Therefore, the neutral axis depth:

$$d_n = k_u.d$$

$$d_n = 0.362 \times 257$$

$$d_n = 93.1mm$$

The force in the tensile steel is the same as previously, but the compressive forces in the concrete and compressive reinforcement need to be recalculated.

Tensile steel force:

$Ts = 450´10^3$ N

Compression steel force:

$$C_s = 200 \times 10^3 \times 0.003 \left(\frac{0.362 \times 257 - 39}{0.362 \times 257} \right) 400$$

= 140191 N

Concrete compressive force:

$$C_c = 3353.76 d_o$$

= 312498 N

Therefore, the ultimate moment capacity of the beam:

$$M_u = C_s \left(d_{st} - d_{sc} \right) + C_c \left(d_{st} - 0.5 \gamma . k_u . d_{st} \right)$$

$$= 140191 \times (257 - 39) + 312198 \times (257 - 0.5 \times 0.822 \times 0.362 \times 257)$$

=98.85 kN.m

The force required to produce the ultimate moment, Mu, is found from:

$$P_1 = \frac{M_u}{\alpha v}$$

Where,

P_1 = Distance between the support and loading point

α = Load from one loading point

Therefore,

$$P = \frac{98.85}{0.75}$$

= 131.8kN

As four point loading is used, the ultimate flexural load capacity of the beam, Pu.f, is calculated as:

$$P_{u.f} = 2 \times P_1$$

= 2*131.8

= 263.8kN

3.2.3 Shear Capacity

The ultimate shear capacity, Vu, of the reinforced concrete beam is determined in this section.

$$Vu = Vuc + V\,s$$

The ultimate shear strength of the concrete, Vuc, is:

$$V_{uc} = \beta_1 . \beta_2 . \beta_3 . b_v . d_o . \left(\frac{A_{st} . f'c}{b_v . d_o} \right)^{\frac{1}{3}}$$

Where,

$$\beta_1 = 1.1 \times \left(1.6 - \frac{d_0}{1000} \right) \geq 1.1$$

$$= 1.1 \times \left(1.6 - \frac{257}{1000} \right) \geq 1.1$$

= 1.48

$\beta_1 = 1$ (as there is no axial load present)

$\beta_3 = 1$

Therefore,

$$V_{uc} = 1.48 \times 1 \times 1 \times 150 \times 257 \times \left(\frac{900 \times 32}{150 \times 257} \right)^{\frac{1}{3}}$$

= 51.77 kN

The ultimate shear strength of the shear reinforcement, Vus, is:

$$V_{us} = \left(\frac{A_{sv}.f_{sy.f}.d_0}{s} \right) \cot \theta_v$$

Where,

$$\theta_v = 30^o + 15^o \left(\frac{A_{sv} - A_{sv.\min}}{A_{sv.\max} - A_{sv.\min}} \right)$$

Where,

A_{sv} = cross sectional area of shear reinforcement

$= 2 \times \pi \times 3^2$

$= 56.5\text{mm}^2$

$A_{sv.\min}$ = cross sectional area of minimum shear reinforcement

$$= \frac{0.35 b_v .s}{f_{sy.f}}$$

$$= \frac{0.35 \times 150 \times 150}{52.5}$$

= 42mm^2

$Asv.$.max = cross sectional area o f maximum shear reinforcement

$$= \frac{b_v .s \left(0.2 f'c - \frac{V_{uc}}{b_v .d_{st}} \right)}{f_{sy.f}}$$

$$= \frac{150 \times 150 \times \left(0.2 \times 32 - \frac{51770}{150 \times 257} \right)}{250}$$

= 606.9mm^2

Therefore,

$$\theta_v = 30^o + 15^o \left(\frac{56.5 - 42}{606.9 - 42} \right)$$

= 30.04^{o}

Therefore, the ultimate shear strength of the shear reinforcement, Vus, is:

$$V_{us} = \left(\frac{56.5 \times 250 \times 257}{150} \right) \cot(30.04)$$

$= 30.1\text{kN}$

Calculating the reinforced concrete beam's ultimate shear capacity:

$$V_u = V_{uc} + V_{us}$$

$=51.77 + 34.87$

$= 86.64$ kN

As four point loading is used, the ultimate shear capacity load, Pu.s, is calculated as:

$$P_{u.s} = 2 \times 82.8$$

$= 173.28$ kN

The beam's shear capacity load, $P_{u.s}$ (173.28kN) is lower than the beam's flexural capacity load $P_{u.f}$(263.6kN), so the beam should fail in shear.

3.3 Design of Externally Post-tensioned Beam

The ultimate moment capacity, Mu, and the shear capacity, Vu, for the design specimen after post-tensioning are calculated in this section.

3.3.1 Selection of Post-tension Force

The post-tension force to be used on the beams has been selected as 150kN, with an eccentricity of 50mm towards the bottom. The post tensioning force will increase the specimens ultimate shear capacity. The specimen is still to fail under shear and the following section will used AS3600 prediction calculations to make sure shear failure will occur.

The eccentricity of the force was chosen to maximise the effectiveness of the post-tensioning, but was kept within the middle third of the section to ensure no tensile stresses were induced on the top face of the beams due to the post-tensioning (Luther 2005, p 27). The positioning of the post-tensioning rods can be seen in Figure 3.5.

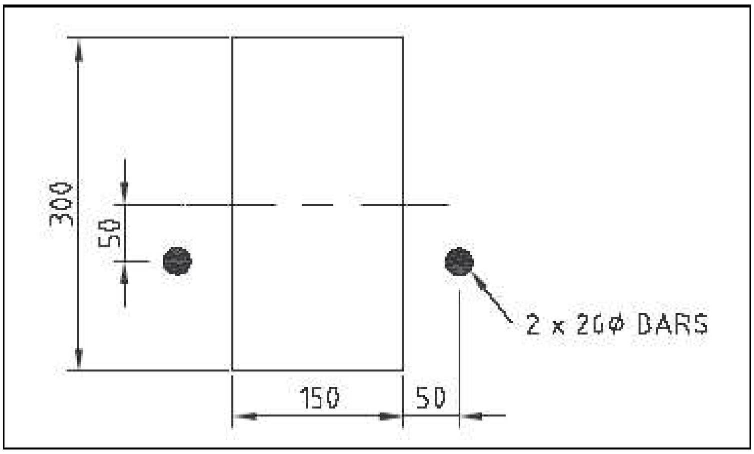

Figure 3.4: Positioning of Post-tensioning Bars
(Figure adopted form Luther, 2005)

The post tensioning rods were sourced form USQ. The available post-tensioning rods were 26mm high tensile Maceloy bars. The post tensioning rods were connected to the specimen by 150C10 sections. The 150C10 end plates transfered the post-tension force to the beams.

3.3.2 Flexural Capacity

The flexural capacity of the beam after post-tensioning needs to be less than the beam's shear capacity after post-tensioning. The calculations for the beam's flexural capacity after post-tensioning are shown below. The parameters of the initial post-tensioning are:
Post-tensioning force:

$F = 150kN$

Depth to post-tensioning steel:

$$d_o = 200mm$$

Area of post-tensioning steel:

$$A_{pt} = 2 \times \frac{\pi D^2}{4}$$

$$= 2 \times \frac{\pi \times 26^2}{4}$$

= 1061.9mm^2

Effective post-tensioning stress:

$$\sigma_{p.ef} = \frac{F}{A_{pt}}$$

$$= \frac{150 \times 10^3}{1061.9}$$

= 141.3MPa

To find the stress in the post-tensioning rods, Clause 8.1.6 of AS3600 is used. For a beam with a span-to-depth ratio of less than 35 (2000/300 = 6.67), the ultimate stress in the rods is:

$$\sigma_{pu} = \sigma_{p.ef} + 70 + \left(\frac{f'c.b_{ef}.d_p}{100.A_{pt}} \right) \leq \sigma_{p.ef} + 400$$

$$= 141.3 + 70 + \left(\frac{32 \times 150 \times 200}{100 \times 1061.9} \right) \leq 141.3 + 400$$

=220.3 MPa

The flexural capacity of the post-tensioned beam is calculated in a similar way as for the reinforced beam, except the tensile force of the post-tensioning rods is added to the balancing equation.

Initially assume all reinforcement yields before Mu, therefore;

Tensile steel force:

$$
\begin{aligned}
Ts &= fsy.Ast \\
&= 500 \times 900 \\
&= 450 \times 10^{3}\, N
\end{aligned}
$$

Compression steel force:

$$
\begin{aligned}
Cs &= fsy.Ast \\
&= 500 \times 400 \\
&= 200 \times 10^{3}
\end{aligned}
$$

Concrete compressive force:

$$
\begin{aligned}
\mathrm{Cc} &= 0.85.f'c.b.d_{n}.\gamma \\
&= 0.85 \times 32 \times 0.822 \times d_{n} \\
&= 3353.76\, d_{n}
\end{aligned}
$$

Post-tensioning steel tensile force:

$$T_{p} = \sigma_{pu}.A_{pt}$$

$$= 220.3 \times 1061.9$$

$$=22393\text{N}$$

As the sum of the forces equals zero:

$$3353.76 d_n + 200 \times 10^3 = 150 \times 10^3 + 233930$$

$$d_n = 114.3mm$$

Checking the compressive reinforcement has yielded:

$$\varepsilon_{sc} = 0.003 \times \left(\frac{144.3 - 39}{144.3} \right)$$

$$=0.0022$$

As ε_{sc} < 0.0025, the assumption that the compressive reinforcement had yielded is incorrect. The compressive forces will be recalculated knowing the compressive reinforcement is in the elastic range.

Checking the tensile reinforcement has yielded:

$$\varepsilon_{st} = \varepsilon_c \left(\frac{d_{st} - d_n}{d_n} \right)$$

$$= 0.003 \times \left(\frac{257 - 144.3}{144.3} \right)$$

$$=0.00234$$

As ε_{st} < 0.0025, the assumption that the tensile reinforcement had yielded is incorrect. The tensile forces will be recalculated knowing the tensile reinforcement is in the elastic range:

$$T_p = E_s.\varepsilon_{st}.A_{st}$$

As ε_{st} is dependent on the neutral axis depth, *dn*, the forces will be solved by trial and error knowing both the compressive and tensile reinforcement are in the elastic range.

From the trial and error, $dn = 143.98mm$

The strains equal:

$$\varepsilon_{sc} = 0.003 \times \left(\frac{143.98 - 39}{143.98} \right)$$

$$=0.002187$$

$$\varepsilon_{st} = 0.003 \times \left(\frac{257 - 143.98}{143.98} \right)$$

$$=0.002355$$

The forces equal:

$$C_s = E_s.\varepsilon_{sc}.A_{sc}$$

$$= 200 \times 10^3 \times 0.002187 \times 400$$

$= 174960 \text{ N}$

$C_c = 0.85 f'c.\gamma.b.d_n$

$= 0.85 \times 32 \times 0.822 \times 150 \times 143.98$

$=482874\text{N}$

$T_s = E_s.\varepsilon_{st}.A_{st}$

$= 200 \times 10^3 \times 0.002355 \times 900$

$= 423900 \text{ N}$

$T_p = \sigma_{pu}.A_{pt}$

$= 220.3 \times 1061.9$

$= 233930 N$

Therefore, taking moments about the tensile reinforcement, the ultimate moment capacity is:

$$M = C_s(d_{st} - d_{dc}) + C_c(d_{st} = 0.5\gamma' d_n) - T_p(d_{st} - d_p)$$

$$= 174960 \times (257 - 39) + 482874 \times (257 - 0.5 \times 0.822 \times 143.98) - 233930 \times (257 - 200)$$

$= 120.33\text{kN.m}$

The force required to produce the ultimate moment, Mu, is:

$$Pu.f = 2 \times \frac{120.33}{0.75}$$

=320.9kN.m

3.3.3 Shear Capacity

The ultimate shear capacity, Vu, of the post-tensioned beam is again determined from:

Vu = Vuc +Vus

The ultimate shear strength of the shear reinforcement, Vus, is as for the reinforced concrete beam, but the ultimate shear strength of the concrete, Vuc, is:

$$V_{uc} = \beta_1 .\beta_2 .\beta_3 .b_v .d_o .\left(\frac{(A_{st} + A_{pt}) f'c}{b_v .d_o} \right)^{\frac{1}{3}} + V_o + P_v$$

Where,

$\beta_1, \beta_2, \beta_3, b_v, d_o f'c$ and *Ast* are the same as for the reinforced beam

Pv = 0, as the post-tensioning rods are horizontal

Vo = the shear force which would occur at the section when the bending moment at the section was equal to the decompression moment (Mo)

$= \dfrac{M_0}{\left(\dfrac{M^*}{V^*} \right)}$ for simply supported conditions, where *M** and *V** are the

bending moment and shear force respectively, due to the same design loading

$$M_o = \left(\frac{P}{A_g} + \frac{P.e.y_b}{I_g}\right)\frac{I_g}{y_b}$$

Where,

P = post-tensioning force

= 150kN

Ag= gross cross-sectional area

= b.D

= 150 x 300

= 45000mm2

e = eccentricity from the centroid of the section

= 50mm

yb = distance from the centroid to bottom edge

= 150mm

Ig = Second moment of area of the uncracked section

$$= \left(\frac{bD^3}{12}\right)$$

$$= \left(\frac{150 \times 300^3}{12}\right)$$

$$= 3.375 \times 10^8\, mm^4$$

Therefore, the decompression moment is:

$$M_o = \left(\frac{150 \times 10^3}{45000} + \frac{150 \times 10^3 \times 50 \times 150}{3.375 \times 10^8}\right) \times \frac{3.375 \times 10^8}{150}$$

= 15kN.m

$$\frac{M^*}{V^*} = \left(\frac{\frac{Dis\tan ceBetweenLoadandSupport \times Load}{2}}{\frac{Load}{2}} \right)$$

=0.75

Therefore, the shear force where decompression occurs is:

$$V_0 = \frac{15}{0.75}$$

=20 kN

Therefore, the shear force where decompression occurs is:

$$V_{uc} = \beta_1 .\beta_2 .\beta_3 .b_v .d_o .\left(\frac{(A_{st} + A_{pt})f'c}{b_v .d_o} \right)^{\frac{1}{3}} + V_o + P_v$$

$$= 1.477 \times 1 \times 1 \times 150 \times 257 \times \left(\frac{(900 + 1061.9) \times 32}{150 \times 257} \right)^{\frac{1}{3}} + 40 \times 10^3 + 0$$

= 106.9kN

Therefore, the ultimate shear capacity of the post-tensioned beam is:

$$V_u = 106.98 + 34.48$$
$$= 141.46kN$$

Therefore, the ultimate shear capacity load, Pu.s, is calculated as:

$P_{u.s} = 2 \times 141.46$

$= 282.92\text{kN}$

The post-tensioned beam's shear capacity load, Pu.s (282.9.kN) is lower than the post-tensioned beam's flexural capacity load Pu.f (320.9kN), so the beam again should fail in shear.

3.4 Design Summary

The design of the specimens to be used for this testing is shown in Figure 3.8.

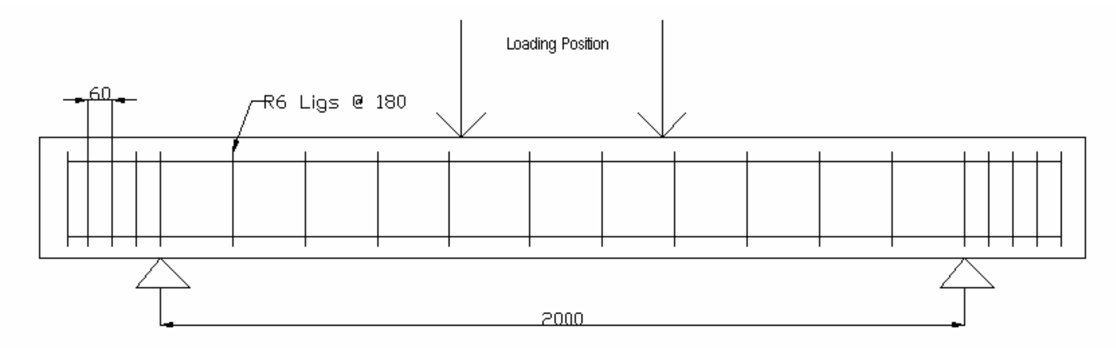

Figure 3.5: Specimen Design

The model beams have been designed to ensure they fail in shear over flexure. This has been done for both the reinforced control beam, and the post-tensioned beams.
A summary of the design capacities can be seen in Table 3.1.

	Shear Capacity Load, Pu.s (kN)	**Flexural Capacity Load, Pu.f (kN)**
Reinforced Beam	171.9	263.3
Post-tensioned Beam	293.3	319.6

Table 3.1: Summary of Design Capacities

CHAPTER 4

EXPERIMENTAL METHODOLOGY

4.1 Introduction

This chapter presents the details of the construction and testing processes that were used to undertake the research. A brief description of the materials utilised along with the method in which the specimens were built will be presented in the construction section. The testing section will cover the work that was undertaken in order to obtain the results.

Note: *This section of the report has been largely taken form the report of Steven Luther as the methodologies are the same for both research projects.*

4.2.1 Formwork

The formwork was constructed by university staff for a previous student research, Steven Luther, and this will be used for this work. The formwork was constructed to have the four beams side by side to minimise the material used (Luther, 2005, p. 36). Three separator boards were used to stabilise the middle formwork ply, until the concrete had been filled on both sides of the ply. The formwork was greased and the edges sealed with silicone before the beams were cast, to ensure they could easily be removed (Luther, 2005, p. 36).

4.2.2 Reinforcement

Mostly the reinforcement used in the steel fixing was sourced at the university; however the N24 bars were order pre-cut and bent due to their size and the universities restrictions on bending that specific sized material.

The N16 compression bars were cut to a length of 2500 mm, to allow 50 mm of space at either end for the ferrules, which were needed to position the end anchorage for the post-tensioning (Luther, 2005, p. 40). The R6 ligatures were cut using bolt cutters, and bent using a jig to suit the required cage. The cage was designed to have 25 mm cover from the outside of the ligatures (Luther, 2005, p. 40).

To ensure a cover of 25 mm, the reinforcement cage was positioned on mortar blocks. These were made five days before casting the beams from a water, cement and sand mix. Each block had a tie wire cast in, so the blocks could also be tied to the cages and used for lateral positioning (Luther, 2005, p. 41).

Four ferrules were secured to the formwork using M10 bolts on either end of each beam, to be used to fasten the end plates during post-tensioning. (Luther, 2005, p. 42).

4.2.3 Pouring the Concrete

The concrete used to construct the test beams was obtained from a local supplier (Wagners). The concrete that was ordered was 32MPa strength, 80mm slump and 20mm nominal aggregate size. The actual properties of the concrete were similar to this, and are shown in Chapter 5.

The concrete was poured straight from the truck, using the slide, into the formwork and manoeuvred using shovels. A vibrator was then used to compact the concrete and to expel any air pockets inside the box (Luther, 2005, p. 41).

The concrete was also lightly watered day to day to prevent shrinkage cracks from appearing. Figure 4.5.1 shows the beams after casting.

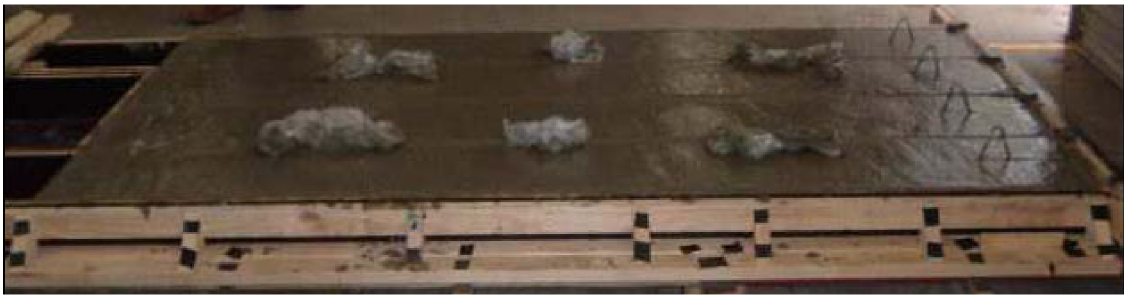

Figure 4.1: Beams after casting

(Figure adopted form Luther, 2005)

A number of test cylinders were also cast during this stage so as to ascertain the compressive and indirect tensile strengths at particular stages of the concrete's life. Figure 4.5.2 shows a sample of these cylinders.

Figure 4.2: Test cylinders

(Figure adopted form Luther, 2005)

4.2.4 Stripping, curing and movement

The formwork was removed from the beams 4 days after casting. Initially one side of the formwork was removed so as to move the beams one at a time using a fork lift and a chain attached to the lifting hooks at either end of the beam. This process was repeated for all four beams. Then beams were left to cure for the required 28 days. The cast test cylinders were also removed from their moulds and placed outside next to the beams so as to cure in the same environment

After the 28 day minimum curing period the beams transported to the Instron testing facility located in the Z4 building of the university.

4.2.5 Post-tensioning

This section will explain about the post-tensioning system used for the test beams. It will include the elements involved in the setup, and the process used to tension the beams.

4.2.5.1 Prestressing Rods

The prestressing rods used for the post-tensioning were 26 mm high tensile threaded rods. The two rods were tensioned to 75 kN each using a hollow core hydraulic jack. The rods were tensioned by jacking the system between the end anchorage plate, and a nut and plate positioned behind the jack. A housing arrangement was used around the nut against the anchorage plate, to allow the nut to be tightened once the jack had tensioned the rod. As only one jack was available for the post tensioning, each tendon was stressed in increments of 20 to 25 kN, to ensure both rods were carrying approximately the same load. This was done to ensure the tensile stress in the end anchorage bolts was not excessive, which could have caused the end plates to be pulled off. After each increment of tensioning, the nut in front of the jack was tightened to hold the increased force. The load applied in each rod was measured using a hollow load cell which was positioned at the end of each rod. The load cells were connected to the data logger, where the force applied during tensioning was monitored from (Luther, 2005, p. 48).

4.2.5.2 End Anchorage

End anchorage was used to transfer the post-tensioned force in the rods to the beam.. The end anchorage was made up of an 10x150mm C-section, with a 15mm thick high strength steel plate behind it, as shown in Figure 4.11. These bearing plates had small lugs welded on the top and bottom to hold them in position. The C-sections were held in position with four M10 bolts which were screwed into four ferrules that had been cast in each end of the beams. The force from the rods was transferred through high strength nuts to the bearing plates and C-section, then to the beam (Luther, 2005, p.49).

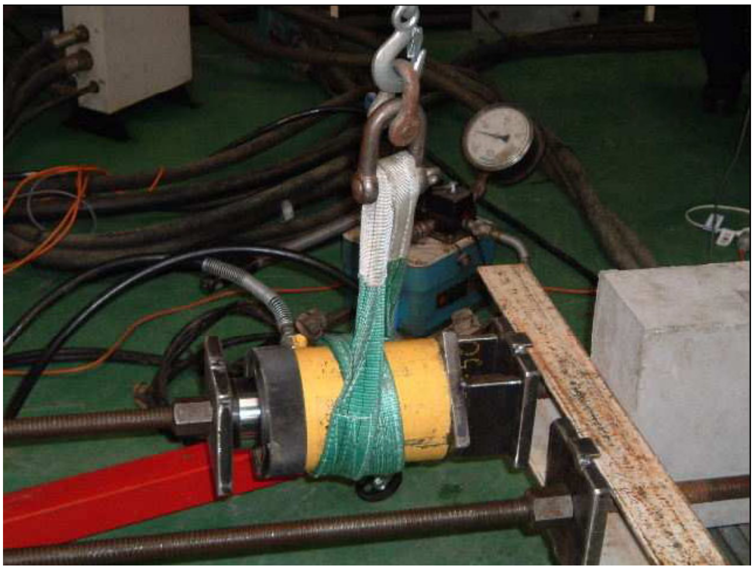

Figure 4.3: Stressing of Rods
(Figure adopted form Luther, 2005)

4.2.6 Epoxy Repairing

The crack repair of the beams involved pressure injecting a two part epoxy, Nitofill LV. As the epoxy was required to be pressure injected, an impermeable seal on the surface of the beam along the crack lines was required. This was done using a two part epoxy crack sealant, Lokfix E. The application process for the Nitofill LV and Lokfix E are discussed below, and the data sheets relating to these products are shown in Appendix D (Luther, 2005, p. 50).

4.2.6.1 Lokfix E

Lokfix E is a two part epoxy sealant for use on concrete structures. It is used to give member impermeability, and reduce the chance of reinforcement corrosion. The Lokfix E was obtained in two cartridges connected together, for use with a double barrel corking gun. The cartridges were sized to automatically apply the correct proportions of the two parts of the epoxy. The two parts of the epoxy were mixed together when extruded, using a specially supplied mixing tube that was connected to the end of the cartridges (Luther, 2005, p.50).

Before the sealant was applied, the crack surface was cleaned using a wire brush, and any loose pieces of concrete were removed. Small holes were then drilled along the crack line, and injection nozzles for the Nitofill LV were glued to the concrete over the holes. The holes were needed to allow the Nitofill LV to freely enter the cracks. The Lokfix E was then extruded over the crack lines, and spread over a 5cm strip using a knife. Care was taken to ensure all cracks were covered and a tight seal was obtained around the flange of the nozzles. Figure 4.12 shows the positioning of the injection nozzles, and the sealing of the cracks using the Lokfix E. The sealant was left to dry for four days after it was applied before the Nitofill LV was injected (Luther, 2005, p. 50).

Before the Nitofill LV was injected, the impermeability of the sealant was tested by pressure injecting water into the cracks. Leakage occurred around some of the injection nozzles, so these were repatched with Lokfix E to form an impermeable seal. The beam was then left for a day before the Nitofill LV was injected, to ensure the crack surfaces had dried (Luther, 2005, p. 51).

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4.2.6.2 Nitofill LV

Nitofill LV is a two part epoxy resin designed to rebond cracked concrete surfaces. It has very low viscosity, and is therefore ideally suited to being pressure injected into fine cracks (Luther, 2005, p. 51).

The Nitofill LV was obtained in two cartridges connected together, for use with a double barrel corking gun. The cartridges were sized to automatically apply the correct proportions of the two parts of the epoxy. Like the Lokfix E, the two parts of the epoxy were mixed together when extruded, using a specially supplied mixing tube. The end of the tube was attached to the injection nozzles using a connector, and the resin was injected using the corking gun. The injection process started at the lowest most nozzle, with all the nozzles being left open. This was to show when the resin had reached all parts of the crack. As the resin began to flow out of a nozzle, it was then closed. Once only the nozzle being used for injection was open, and the pressure on the corking gun was noticeably higher, the final nozzle where the injection was occurring was closed. The increase in pressure indicated the entire crack had been filled. The Nitofill LV was then left to cure for seven days before the beam was reloaded (Luther, 2005, p. 52).

Before the beam was reloaded, the Lokfix E sealant was removed with a grinder, to allow the initial crack lines to be seen. This was so a comparison of the initial crack lines and the crack formed after repair could be made (Luther, 2005, p. 52).

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4.3 Testing Methodology

4.3.1 Introduction

This section outlines the experimental testing that was involved in this research. As previously stated, four beams were tested under varying conditions, which can be seen in Table 4.1.

Specimen	Preloaded	Post-tensioned	Epoxy Repaired
B1	No	No	No
B2	Yes	Yes	No
B3	Yes	Yes	Yes
B4	No	Yes	No

Table 4.1: Test Conditions for Specimens

4.3.2 Test Configuration

The test configuration used to load the specimens can be seen in Figure 4.13. This example shows the test setup for the post-tensioned beams. The setup for the beams without post-tensioning did not require the post-tensioning rods, end anchorage or post-tensioning load cells (Luther, 2005, p. 53).

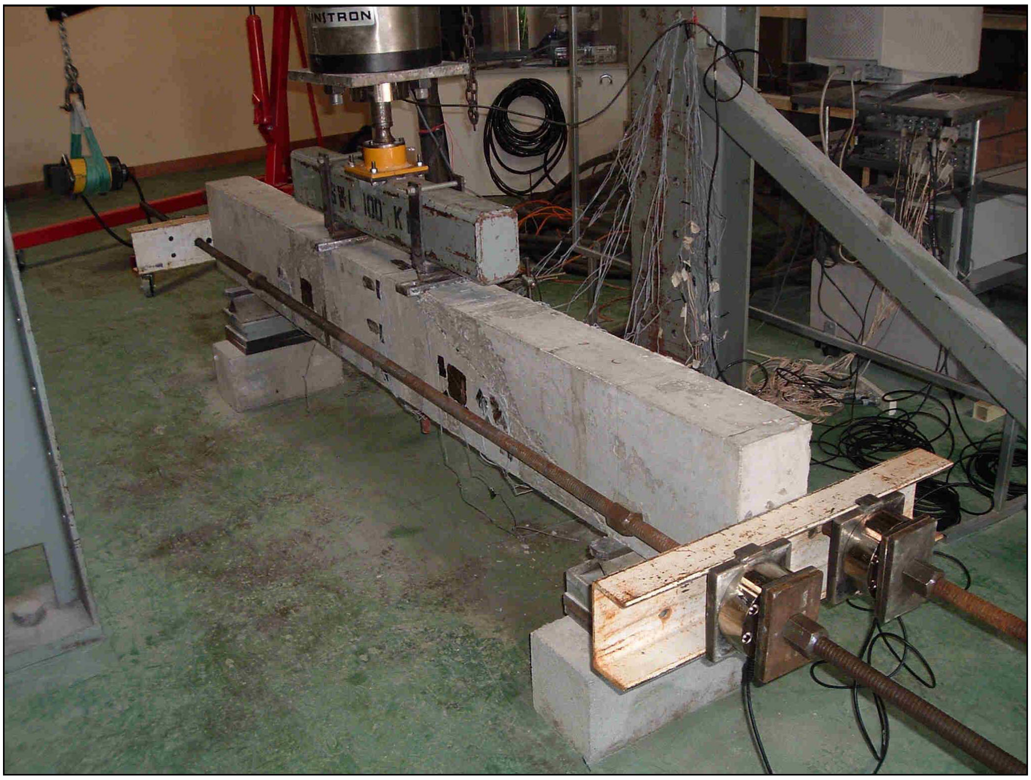

Figure 4.4: Test Configuration
(Figure adopted form Luther, 2005)

To simulate pin supports as the beams were modelled for, the supports for the beam were made up of triangular shape steel blocks. A 30mm wide steel plate was positioned on top of each triangular block to avoid local cracking around the support (Luther, 2005, p. 54).

Figure 4.13 show how the beams were tested with four point loading. A single 500kN Instron loading ram, with a maximum travel of 150mm, was used to apply the load. The loading ram was supported by the loading frame, which had been adjusted to the correct height for the test beam setup. The load was transferred evenly to the beam at two points via the spreader beam. The spreader beam was connected to the loading ram with a ball and socket joint, and the two loading points were set 500 mm apart (Luther, 2005, p. 55).

The force being applied from the loading ram was measured by a load cell that was positioned directly under the ram. Two load cells were also used to measure the force in the post-tensioning rods when they were being stressed, as well as when the load was being applied. The data gathered from the load cells was stored by the system 5000 data logger (Luther, 2005, p. 55).

The midspan deflection of the test beams when loaded was measured by a load variable string plot recorder. The data gathered from the string plot was stored by the system 5000 data logger(Luther, 2005, p. 55).

4.4 Data logging

All the data collected from the load cell, strain gauges and string plot during the testing process was recorded by the system 5000 data logger. To do this each of the measuring devices were allocated a channel on the system 5000 data logger, or in this case data loggers due to the amount of channels needed to record all the measuring devices. One channel used for the load cell, one for the string plot. During testing a reading was taken from each of these devices every second (Luther, 2005, p. 56).

4.5 Loading

All of the test beams were loaded at a constant rate of 1 mm per minute using the Instron loading ram. This load rate was achieved by the loading ram being computer controlled (Luther, 2005, p. 56).

4.6 Concrete Compressive Strength Tests

To find the compressive strength of the concrete, 20 cylinders, 100mm diameter by 200mm high, were cast when the beams were poured. These were left to cure in exactly the same conditions as the test beams. On the day of testing for each of the beams, five of the test cylinders were compression tested. The results from these tests can be seen in Chapter 5 (Luther, 2005, p. 57).

CHAPTER 5

RESULTS AND DUSSCUSSION

5.1. Introduction

This chapter analyses the data that was collected from the experimental work. It provides an overview, of the experimental results that were observed, and attempts to explain the findings. Finally a comparison between the theoretical and experimental capacities can be found at the conclusion of this chapter.

5.2. Material Tests

5.2.1. Concrete Slump Analysis

A slump test was conducted on the mix to gain some indication of the wet concrete's properties. A slump of 120 mm was determined for the batch. It has been noted that the concrete had been ordered with a slump of 80mm and strength of 32MPa. From slump results it is obvious that the concrete delivered was somewhat different to what was ordered. This concrete was still utilised, however the strengths of the batches were decreased due to the high moisture content.

5.2.2. Concrete Compressive Strengths

The strength of the specimen is a function of the concrete strength. Several compressive tests were conducted to determine the strength of the concrete. It is essential that this is undertaken because as concrete ages, its strength increases (Table 5.1 to 5.3). Through examining the f'c from each test in the table, it becomes apparent that during test of the control beam and the initial cracking stages of testing the concrete strength was lower than the ordered 32 MPa. The testing of the repaired beam occurred at a later stage and hence the concrete strength has increased.

Test No	Cylinder No.	Maximum Load (kN)	Compressive strength *f'c* (MPa)	Average f'c (MPa)
1 Control Beam	1	220	27.6	29.3
	2	240	30.5	
	3	230	29.1	
	4	235	30.0	

Table 5.1 Concrete compression test for the control beam

Beam No.	Cylinder No.	Maximum Load (kN)	Compressive strength f'c (Mpa)	Average f'c (MPa)
2 Initial Cracks	1	215	27.5	29.8
	2	255	31.8	
	3	235	30.0	

Table 5.2 Concrete Compression test for the initial cracks

Beam No.	Cylinder No.	Maximum Load (kN)	Compressive strength f'c (Mpa)	Average f'c (MPa)
3 Repaired Beams	1	265	33.7	34.5
	2	270	34.5	
	3	275	35.1	
	4	265	33.9	

Table 5.3 Concrete Compression test for the repaired beams

For the theoretical analysis in Chapter 3, a concrete strength of 32MPa was used, however this is different to the experimental concrete strengths, and therefore the strength of the specimens may be slightly different. This will be examined towards the end of the chapter.

5.3 Crack Observation

5.3.1 Specimen B1 (control beam)

The cracking in the control beam formed with a major shear crack propagating from the support to the loading point. The crack started at approximately half-way between the loading points at 108 kN. As the load increased a second shear crack formed above the main crack. The second shear crack appeared at a load of 125 kN. The main crack developed to a spacing of 4 mm while the second stopped at 2 mm. The ultimate shear load was 198 kN for the control beam. Figure 6.4.1 depicts the crack propagation of the control beam.

Figure 5.1 Crack pattern for B1

5.3.2 Specimen B2

Specimen B2 was used to determine the strength that could be regained to a beam with shear cracks strengthened with post-tensioning alone. Specimen B2 was preloaded to form a shear cracks at both ends of the beam. Loading was ceased when the crack had a maximum width of approximately 4 mm. A load of 203 kN was required to form this crack. The initial shear cracking in the specimen formed in the same way as Specimen B1, with the first shear cracks forming at approximately 108 kN. It was also noted that a second shear crack developed in the shear zone. The initial crack formed in Specimen B2 can be seen in Figure 5.3.

Figure 5.3 Initial shear crack pattern in B2

After preloading Specimen 2 a post-tensioning force was applied without the use of epoxy resin to repair the existing cracks. The applied post-tensioning force was set at 150 kN. Once loading was reapplied the existing cracks began to reopen. At the maximum load of 173 kN, the maximum crack width was 4 mm. The failure crack of the repaired specimen 6 mm can be seen in Figure 6.4.4.

Figure 5.4 Crack pattern in specimen B2

5.3.3 Specimen B3

Specimen B3 was used to determine the strength that could be regained to a beam with shear cracks being repaired with epoxy injection and strengthened with post-tensioning. Specimen B3 was preloaded to form a shear crack with a maximum width of approximately 4 mm. A load of 181 kN was required to form this crack. The initial shear cracking in the specimen formed in the same way as the previous specimens, with the first shear crack forming at 115 kN. The cracking began in the middle of the beam and propagated towards the loading point as the load was increased. The initial crack formed in Specimen B3 can be seen in Figure 5.5.

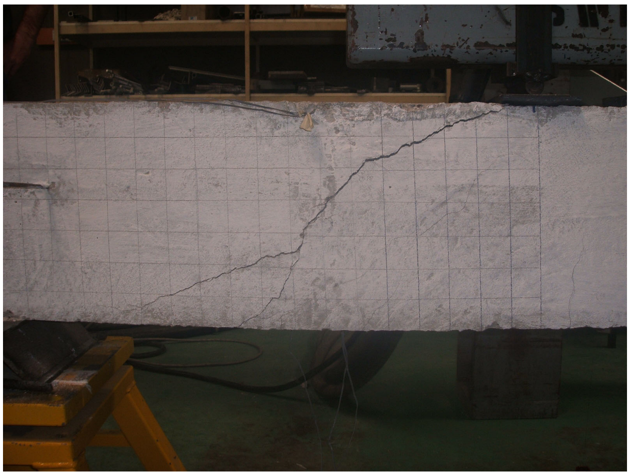

Figure 5.5. Initial crack pattern in specimen B3

After preloading, the shear cracks were repaired by epoxy injection, and the beam was post-tensioned. The initial cracks were completely repaired, as the new crack lines formed away from the initial cracks. Once the loading was reapplied, a new set of shear cracks began to form at 220 kN. The onset of the new shear cracks was at a much higher load than for the reinforced control beam, as the post-tensioning caused the beam to be in compression. The maximum load reached was of 293 kN, the maximum. It can be seen that 3 new shear cracks developed with in the shear zone. The angle of these cracks is has changed. The cracks opened flatter, more in the longitudinal direction of the beam. The maximum crack width at this point was 7 mm. The failure crack of Specimen B3 can be seen in Figure 5.6.

Figure 5.6 Crack pattern in specimen B3.

5.3.4 Specimen B4

Specimen B4 was used to determine the strength of a new beam strengthened with a post tensioning. The post tensioning force was set a 150 kN. The first shear crack formed at 180 kN. The crack then propagated towards the applied load, increasing in width as the load was increased. A maximum load of 287 kN with the maximum crack width of 2 mm. The maximum crack width after failure was 8 mm. The shear cracking of Specimen B4 can be seen in Figure 5.7.

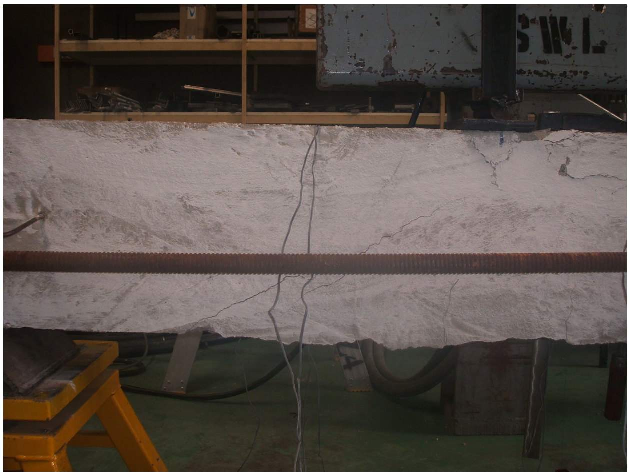

Figure 5.7 Crack pattern in specimen B4.

5.3.5 Comparison of Crack Patterns

In all specimens, the first crack always appeared as a flexural crack at the midpoint of the beam where the maximum bending moment is present. Under 60–70% of failure load, these cracks in the shear span were inclined and propagated toward the loading points. External post-tensioning, which were used to strengthen and repair the beams, successfully controlled the size of the cracks propagated in the beams, although with out epoxy injection, the major diagonal shear cracks reopened. Hence no new shear cracks. The use of epoxy injection successfully repaired the shear cracks. This is evident in specimen 3 where the cracks where repaired with epoxy resin. The development of new shear cracks appeared while the repaired cracks remained intact.

5.4 Load – Deflection Characteristics

From the data that was obtained during testing, a simple load-deflection relationship can be produced for each specimen. Initially the load is carried by the concrete in the linear region. Once the concrete begins to crack, the load is transferred to the reinforcement. As the load is increased, it is carried by the reinforcement and is distinguished by the change in stiffness, as can be seen by the different slope on the plot below. In this section yielding may occur and the reinforcement will then go on to fail if the load is increased. Conversely, if the concrete begins to fail before the steel yields, a brittle failure will be observed.

5.4.1. Specimen S1

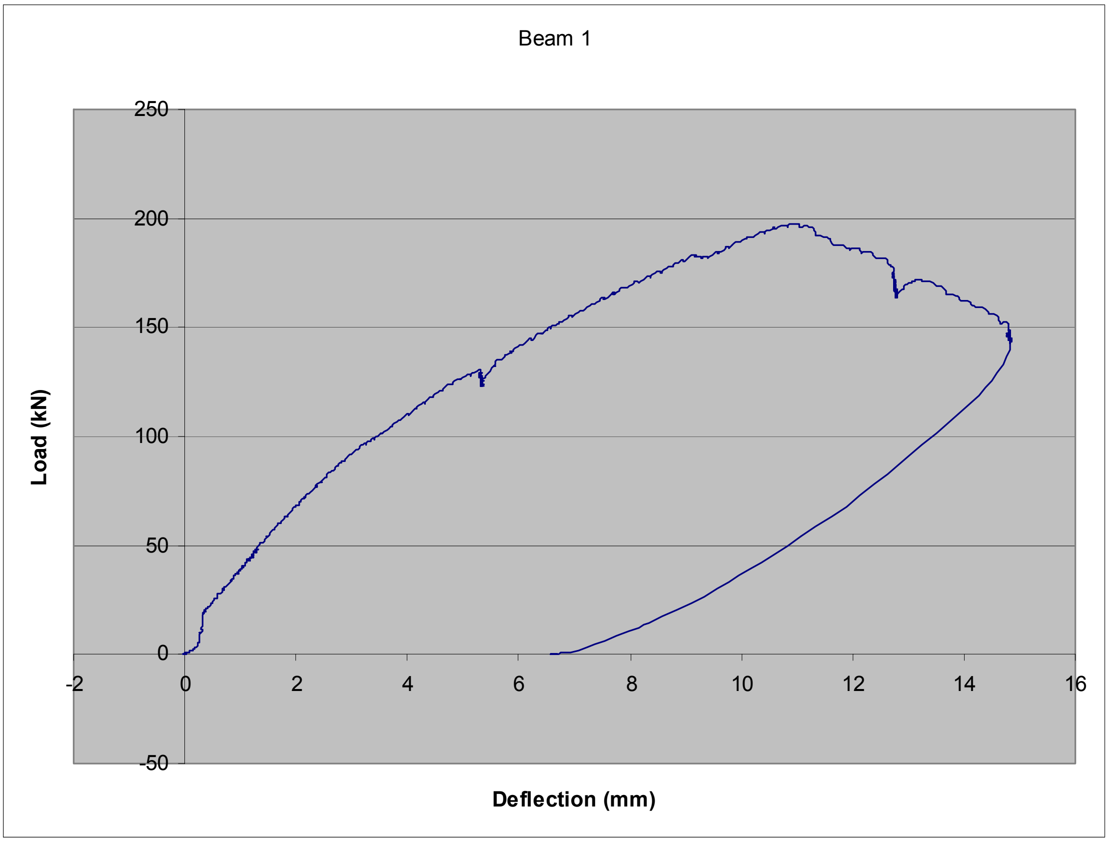

Figure 5.8 Load-deflection plot of Specimen B1

The plot in Figure 5.8 above represents the load-deflection relationship for specimen S1 (the control beam). It is a apparent that that on initial loading that a seating error has occurred, the line should continue in a linear from to zero point of the graph. However from the plot a linear section can be observed. A large dip occurs at a load of 129 kN. This corresponds to the stage at which major cracks began to appear and the load is being transferred to the reinforcement, hence the change in slope. Once the load reaches 198kN, the specimen no longer deforms with an increase in load. Once the cracks become large enough that the member cannot withstand the applied load, the specimen fails. In the plot above this occurs at approximately 189kN and 14.8 mm of deflection.

5.4.2 Specimen S2

Figure 5.9 Load-deflection plot of Specimen B2

Figure 5.9 shows the load versus deflection graph for the post-tensioned repair beam. The preloading of Specimen B2 shows similar behaviour to Specimen B1. The preloading was taken to 204 kN, with 12.85m deflection.

After post-tensioning, initial loading the beam displayed a similar behaviour to the preloading but ultimate failed at a much loser load the maximum shear load taken by the beam was 173 kN, compared to Specimen B1 which took a load of 198 kN. This equates to a 12% decrease in strength due to the post tensioning.

This has been explained by Luther, 2005 his research describes an important factor is the angle of the shear cracks relative to the post-tensioning force. As the cracks are on an angle of approximately 30 degrees, part of the post-tensioning force is actually causing the crack faces to slide against each other, instead of forcing them together. This reduces the concrete component of the shear strength for the beam, leaving the load to be predominantly taken by the shear ligatures.

This shows that post-tensioning alone will not increase a beam's shear strength if it has existing shear cracks. The loading was stopped at.12.21 mm deflection, as the beam had obviously failed.

5.4.3 Specimen B3

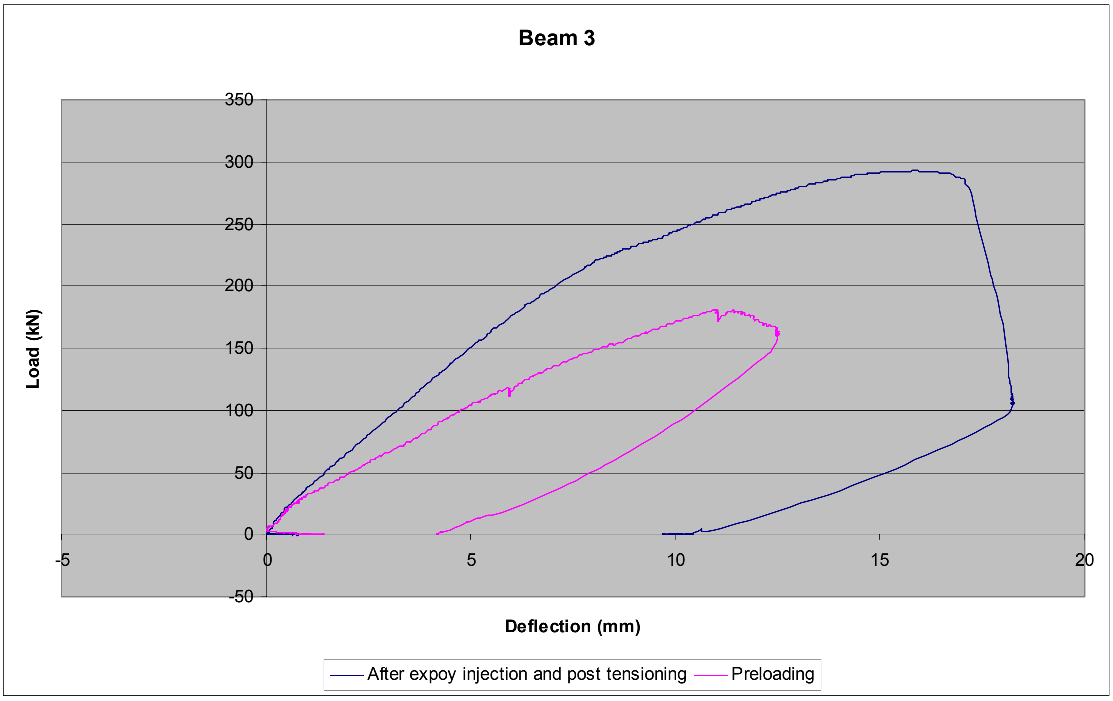

Figure 5.10 Load-deflection plot of Specimen B3

Figure 5.10 shows the load versus deflection graph for the beam that had its shear cracks repaired with epoxy injection and was then post-tensioned. The graph shows a linear shape up until 115 kN load, when the shear cracks began to form. The preloading was taken to 181 kN, with 11.53 mm deflection.

The beam had its shear cracks repaired with epoxy injection, and was then post tensioned. The first shear cracks formed in the repaired beam at 230 kN. This is evident on the graph by the end of the linear section of load versus deflection. The slope of the graph after this point is much flatter, as the load is predominantly being taken by the ligatures. The maximum load taken by the beam was 310 kN at 13.84 mm deflection. This is a 58% increase in strength from the reinforced control beam. The sharp drop in load after the maximum is due to a sudden shear-compression failure occurring. After this occurred, almost the entire load of 162 kN was being taken by the ligatures.

5.4.4 Specimen B4

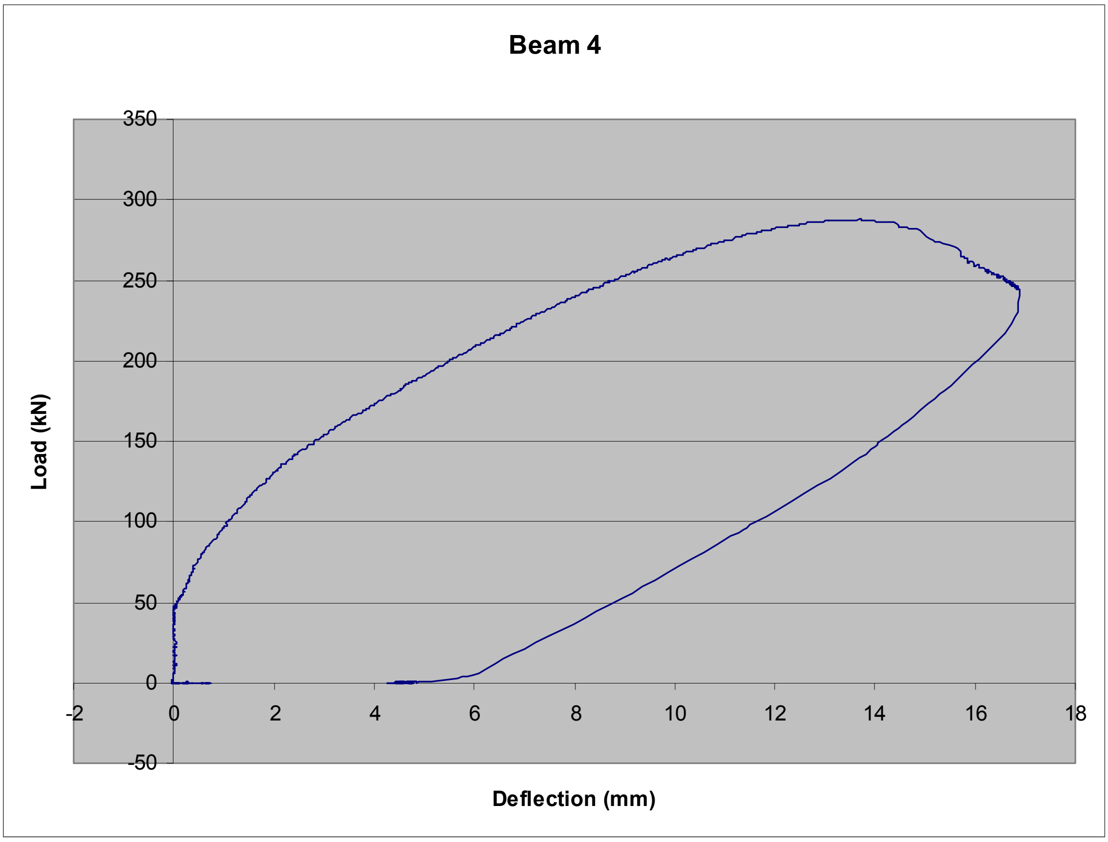

Figure 5.11 Load-deflection plot of Specimen B4

The load versus deflection graph for the post-tensioned control beam, Specimen B4, is shown in Figure 5.12. The beam was post-tensioned before loading, and was then loaded until failure. The post-tensioning caused the beam to deflect upwards by 0.38 mm. The graph shows a linear shape up until 180 kN load, when the shear cracks began to form. The slope of the graph is then flatter up until the maximum load of 354 kN. This load is 81% higher than for the reinforced control beam. The sharp drop in load after the maximum is again due to a sudden shear-compression failure occurring. This behaviour is very similar to that exhibited by Specimen B3.

5.5 Comparison of Load – Deflection Characteristics

The beam that was repaired only with post-tensioning did not gain any strength compared to the reinforced control beam. This is in contrast to the beam that was repaired with epoxy injection and then post-tensioned, which had a 58% increase in strength. This compared to the post-tensioned control beam which had an 81% increase in strength from the reinforced control beam. The shape of the load versus deflection graph for the post-tensioned control beam, Specimen B4, is very similar to that of the epoxy repaired beam, Specimen B3. The only significant difference is that Specimen B4 continued to be loaded to 354 kN, where Specimen B3 failed at 310 kN. The reason the repaired beam did not reach as high a failure load is that it probably received minor damages in preloading, that have caused it to fail earlier than the post-tensioned control beam. This is due to the small cracks and damages acting as initiators for the shear cracks. This shows that the beam that was epoxy injected and post-tensioned behaved very similarly to the post-tensioned control beam, except it did not gain the entire strength of the new member. The testing has also shown that epoxy injection of shear cracks combined with external post-tensioning substantially increases a beam’s shear capacity.

5.6 Increases in External Post-tension Force

The applied prestressing force increased as beams were loaded. This increase can be explained by when the beams were loaded and began to deflect, the tension side of the beam increased in length. This caused the post-tensioning rods to also increase in length, which therefore increased the force in the rods. Figure 5.32 shows the increase in post-tension force as the beams have been loaded. Note that the deflection shown is the deflection from when the post-tensioned beam has begun loading.

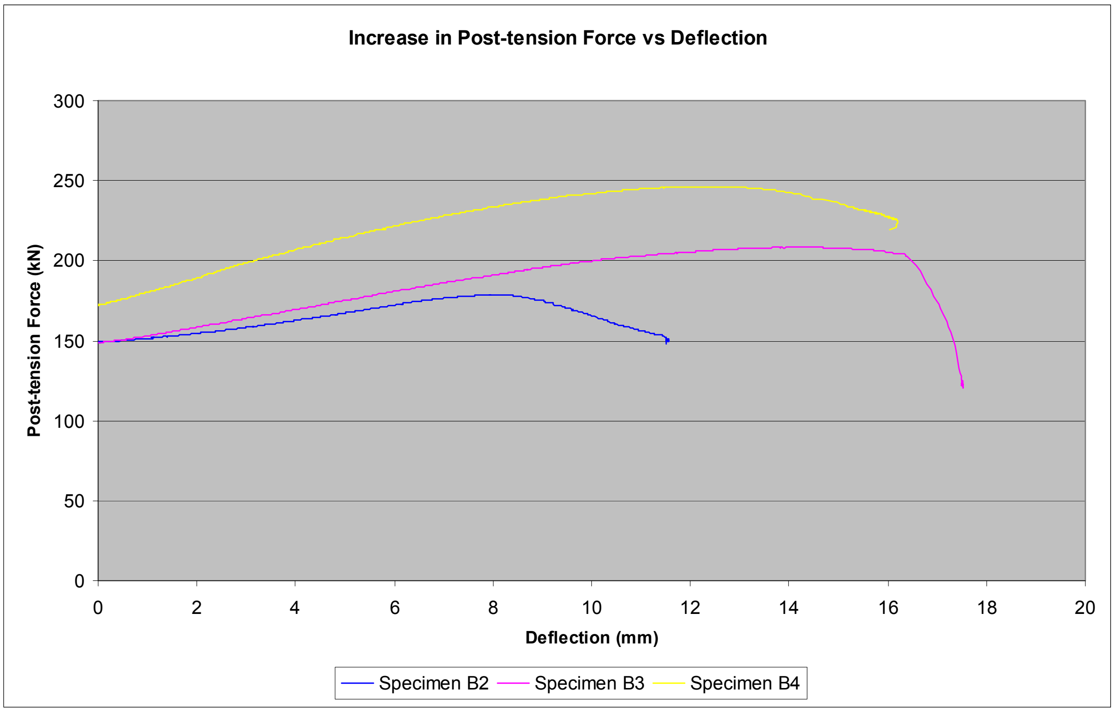

Figure 5.12: Increase in Post-tension Force as the Beams are Loaded

The graph shows that the post-tensioned force in Specimens B3 and B4 has increased more than Specimen B2. Specimen B2 had no crack repair which allowed for a sliding action between the two unboned surfaces. This action meant that the tension side of the beam has not increased in length as much as for B3 and B4, as the deflection has occurred due to the crack width increasing, not the beam bending as a whole. The post-tension force has increased in Specimens B3 and B4 up until failure, as these two beams were bending as a whole, and therefore increasing the rods' length.

Specimen B4 had a larger increase than B3, as it took a higher load and sustained a larger deflection.. The percentage increase in post-tensioning force for each of the beams is shown in Table 5.3. The percentage increase of post-tensioning force, combined with the shape of the graph, indicates that the epoxy repaired beam, Specimen B3, is behaving almost as a new condition member. it proved that the behaviour of the beam can be improved by a proper repair of existing cracks prior to strengthening.

Specimen	Initial Force (kN)	Maximum Force (kN)	Percentage Increase (%)
B2	148.7	178.7	20.2
B3	149.2	208.2	39.5
B4	172.21	246.3	43.0

Table 5.4: Percentage Increase in Post-tensioning Force

5.7 Section Capacities Based on Actual Material Properties

The section capacities of the test beams were calculated in Chapter 3 using theoretical material properties. Material tests conducted have shown material strengths different to the expected. To accurately compare the practical test results with AS3600 prediction equations, the section capacities need to be recalculated using the observed material properties.

The following section will show the calculations of the shear capacities of the beams, using actual material properties. The flexural capacity of the beams will not be recalculated, as each the test beams failed in shear.

5.7.1 Control Beam

Section Properties

$$f'c = 29 \text{ MPa}$$

Shear strength of the concrete:

$$V_{uc} = 1.48 \times 1 \times 1 \times 150 \times 257 \times \left(\frac{900 \times 29}{150 \times 257} \right)^{\frac{1}{3}}$$

= 50.01 kN

Calculating the reinforced concrete beam’s ultimate shear capacity:

$$V_u = V_{uc} + V_{us}$$

=50.01 + 34.87

= 84.88 kN

As four point loading is used, the ultimate shear capacity load, Pu.s, is calculated as:

$$P_{u.s} = 2 \times 84.88$$

= 169.76 kN

5.7.2 Initial Cracking

Section Properties

$f'c$ = 30 MPa

Shear strength of the concrete:

$$V_{uc} = 1.48 \times 1 \times 1 \times 150 \times 257 \times \left(\frac{900 \times 30}{150 \times 257} \right)^{\frac{1}{3}}$$

= 50.67 kN

Calculating the reinforced concrete beam’s ultimate shear capacity:

$$V_u = V_{uc} + V_{us}$$

=50.67 + 34.87

= 85.54 kN

As four point loading is used, the ultimate shear capacity load, Pu.s, is calculated as:

$$P_{u.s} = 2 \times 85.54$$

= 171.08 kN

5.7.3 After Repair

Section Properties

$f'c$ = 34.5 MPa

Shear strength of the concrete:

$$= 1.477 \times 1 \times 1 \times 150 \times 257 \times \left(\frac{(900 + 1061.9) \times 34.5}{150 \times 257} \right)^{\frac{1}{3}} + 40 \times 10^3 + 0$$

= 108.69 kN

Calculating the reinforced concrete beam's ultimate shear capacity:

$$V_u = V_{uc} + V_{us}$$

= 108.69 + 34.87

= 143.56 kN

As four point loading is used, the ultimate shear capacity load, Pu.s, is calculated as:

$$P_{u.s} = 2 \times 143.56$$

= 287.12 kN

5.8 Comparison of Practical Results with AS3600 Predictions

This section compares the capacities of the beams found from the experimental testing with the recalculated theoretical section capacities found in section 5.8. This will indicate the accuracy of the prediction equations for this testing. The comparison for the four specimens is shown in Table 5.5.

Specimen No.	Recalculated Theoretical Capacity	Experimental Capacity (kN)	Percentage Difference %
B1	169.78	197.8	17
B2	287.12	173.13	-39.6
B3	287.12	292.69	1.9
B4	287.12	287.58	0.01

Table 5.5: Comparison of Theoretical and Experimental Failure Loads

It is apparent that the experimental capacities were approximately close to the theoretical capacities from AS3600. For Specimen B1, the reinforced control beam, AS3600 slightly under predicted the failure load. The AS3600 predications should show a conservative capacity and this is shown by the 17% under predication.

Specimen B2 had an actual capacity 39.6% less than the theoretical prediction for the post-tensioned beam. This was expected, as the AS3600 predictions are based on a new condition post-tensioned member. V_{uc} has been taken at 100% where in fact due to shear cracks V_{uc} would be much lower than this.

The beam that had its shear cracks epoxy repaired and strengthened by post-tension ding, Specimen B3, showed an actual capacity very close than the predicted. This shows evidence that a beam repaired by this method can increase the behaviour of the beam similar to that of a new beam.

The post-tensioned control beam, Specimen B4, also displayed experimental capacities similar to that of the AS36000 predication capacities.

AS3600 proved to have good predications for the ultimate capacity of post-tensioned beams.

5.9 Summary of Practical Results

Table 5.7 shows a summary of the experimental shear capacities of the four beams. Specimen B2 showed a significant decrease in capacity. It can be seen that Specimen B3 had a 62% increase in capacity

Specimen No.	Shear Capacity	Percentage Increase
B1	197.8	N/A
B2	173.1	-17.3
B3	292.7	62
B4	287.6	N/A

Table 5.6: Strength Increase of Post-tensioned Beams

5.10 Comparison of Results with Previous Research

This chapter will attempt to make a comparison to the work done by Luther (2005). The research investigated by Luther (2005) was with minium shear reinforcement. Minium shear reinforcement has ligatures spacing of 250mm. This allowed 1 ligature in the shear zone of the experiment with 2 ligatures under the loading points.

The member in this testing repaired only with post-tensioning actually had a 1% decrease in ultimate capacity. In this research there was a significant decrease in shear capacity when no crack repair was undertaken. This research has indicated that combining epoxy injection of cracks with external post-tensioning will increase the shear capacity of a concrete girder.

Previous research has been done" by Luther (2005) and there was found to be a 58% increase in shear capacity when the post tensioning technique was used in conjunction with the epoxy resin, while without the epoxy there was no effect of shear strengthening to be seen. This can be compared to this research with a more shear reinforcement found a to be a 62 % increase in shear capacity.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

This research project has investigated the effect of shear reinforcement ratio in the behaviour of reinforced concrete beams strengthened with external post-tensioning.
. The research was based on the experimental testing of four model beams. This section will outline the achievement of specified objectives, conclusions reached from the investigation, and possible areas for further research.

6.2 Achievement of Objectives

The following section provides an overview of the achieved objectives that were established during the initial stages of the research.

1. *Research background information on the use of epoxy injection and external post-tensioning*

At the commencement, of this research, a literature review was undertaken to gain an understanding of the extent of the work that had previously been conducted in the area of bridge rehabilitation (Chapter 2). It was discovered that external post-tensioning is a proven form of concrete restoration. Under increasing crack conditions however, the system becomes less effective due to the loss of aggregate interlock. Epoxy crack injection is capable of reforming this bond to between the crack faces. Consequently, by combining the two techniques, a vastly more efficient method of rehabilitation could be produced.

2. *Design model test beams for experimental investigations, taking into account previous test results.*

The model test beams were designed to fail in shear over flexure, and this was the case in testing. The process used in the design of the test beams was shown in Chapter 3.

3. *Prepare model beams, and arrange testing devices.*

The model beams were successfully constructed and set up for testing. The steps involved in the construction of the specimens, and the test set up used were discussed in Chapter 4.

4. *Conduct tests on the model beams, and record observed results.*

The four model beams were successfully tested, with observations and test data recorded. Three of the beams had post-tensioning applied, and one of the beams had its shear cracks repaired with epoxy injection. The testing of the model beams was discussed in Chapter 4, and the observed results were discussed in Chapter 5.

5. *Evaluate and analyse the test results of the different model beams.*

The results from the testing of the four model beams have been discussed in Chapter 5. These results have been analysed to see the effect of the epoxy injection, and the external post-tensioning on the shear strengthening of the model test beams.

6. *Arrive at a conclusion for the project, which will better explain the shear behaviour of rehabilitated girders using epoxy injection and external post tensioning.*

Comparisons have been reached on the shear strengthening of concrete girders with epoxy injection of cracks and external post-tensioning with the work completed by Luther (2005). These have been discussed in Chapter 5.

6.3 Conclusions

The results of the experimental investigation have shown that by repairing existing shear cracks with epoxy injection, concrete girders can then be shear strengthened by external post-tensioning. The amount of shear reinforcement can affect the amount of strength regained in this rehabilitation technique.

Previous research has been done Luther (2005) and there was found to be a 58% increase in shear capacity when the post tensioning was used in conjunction with the epoxy resin, while without the epoxy there was no effect of shear strengthening to be seen. The research investigated by Luther (2005) was with. This can be compared to this research with a more shear reinforcement found to be a 62 % increase in shear capacity.

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APPENDIX A

Project Specification

University of Southern Queensland

Faculty of Engineering and Surveying

ENG 4111/4122 Research Project
PROJECT SPECIFICATION

FOR: **Todd Whittaker**

TOPIC: Effect of shear reinforcement ratio on crack widths and epoxy injection in structural rehabilitation with external post-tensioning.

SUPERVISOR: Dr Thiru Aravinthan

SPONSERSHIP: Faculty of Engineering and Surveying USQ

1. This project is a continuation of projects done my Steven Luther and Paul Bolger. The background information on shear strengthening of concrete girders using epoxy injection and external post-tensioning will largely based on their research.

2. The project will investigate the effect of increasing the shear reinforcement in the experimental procedures and the benefits in rehabilitation.

3. Design model beams with an increased shear reinforcement ratio for experimental investigations, taking into account previous test results.

4. Prepare model beams, and arrange testing devices.

5. Conduct tests on the model beams, and record observed results.

6. Evaluate and analyse the test results of the different model beams.

7. Arrive at a conclusion for the project, which will better explain the shear behaviour of rehabilitated girders using epoxy injection and external post tensioning.

AGREED: ______________________ (Student) ______________________ (Supervisor)

__ / __ / __ __ / __ / __

APPENDIX B

Epoxy Resin Data

1. Lokfix E Fact Sheet

2. Nitofill LV Fact Sheet

FOSROC

Lokset E

1/1999

Lokset E

Structural epoxy adhesive paste and filler

Uses

For speedy and permanent patching repairs to concrete structures; bonding of precast concrete components and all repair work to concrete cementitious substrates where strength, impermeability to water, and resistance to aggressive chemicals is essential; emergency repairs to concrete structures, sea walls, and industrial floors in chemical handling and process areas.

The thixotropic nature of Lokset E makes the product ideal for setting starter bars, dowels, holding down bolts and anchoring in general.

Advantages

- Early development of initial hardness, minimises maintenance disruption
- Pre-weighed quality controlled materials ensure consistency and reduce risk of site errors
- Two pack colour coding gives visual check on correct mixing
- Unaffected by a wide range of acids, alkalis and industrial chemicals
- 3 to 4 times stronger than typical concrete. Excellent resistance to abrasion and impact
- Natural grey colour sympathetic to aesthetic requirements

Description

Lokset E is a two-component, epoxy paste consistency, structural adhesive/filler. It cures, with minimal shrinkage, at temperatures above 5°C to a very strong, dense solid.

The mixed material is applied to a suitably prepared surface and quickly cures to form a complete impermeable repair unaffected by many forms of chemical attack.

It is supplied as a two pack colour coded material in pre-weighed quantities ready for on-site mixing and use.

Technical support

Parbury Technologies offers a comprehensive range of high quality, high performance construction products. In addition, Parbury Technologies offers technical support and on-site service to specifiers, end-users and contractors.

Properties

Data quoted is typical for this product but does not constitute a specification.

Pot life:	1.5 litre mix at 25°C – 25-35 minutes. Note: To obtain maximum pot life, spread Lokset E into a thin (less than 10 mm) layer immediately after mixing.
Initial hardness:	24 hours.
Full cure:	7 days. Below 20°C the curing time will be increased.
Minimum application temperature:	5°C.
Maximum service temperature:	50°C.
Specific gravity (mixed):	1.7 (approx.)
Chemical resistance:	
Citric Acid 10%	Excellent
Tartaric Acid 10%	Excellent
Sodium Hydroxide 50%	Excellent
Diesel Fuel/Petrol	Excellent
Sugar Solutions	Very Good
Lactic Acid 10%	Very Good
Hydrocarbons	Very Good
Phosphoric Acid 50%	Very Good
Colour:	Grey, when mixed (may yellow and/or darken when exposed to sunlight or certain chemicals.

Instructions for use

Preparation

All grease, oil, chemical contamination, dust, laitance and loose concrete must be removed by scabbling or light bush hammering to provide a sound substrate.

All concrete must be at least 14 days old prior to treatment.

Steel surfaces should be grit blasted to white metal. Surfaces which show any traces of oil must be degreased with a chemical degreaser prior to grit blasting.

FOSROC

Mixing

Thoroughly mix resin (white) and curing agent (black) until an even grey colour is obtained. Mix for minimum 3 - 5 minutes.

Application

Apply the mixed Lokset E with a notched trowel, putty knife, caulking gun, twin cartridge gun etc., depending upon the application. Bonded surfaces should be held rigidly together until the Lokset E has set.

Cleaning

All tools and equipment should be cleaned immediately after use with Fosroc Solvent 20. Hardened material can only be removed mechanically.

Estimating

Supply

Lokset E is supplied in 1.5 litre and 6 litre two component packs and a convenient 450 ml twin cartridge pack.

Quantity estimating guide

Table indicates volume of Lokset E in ml/100 mm bond.

Hole Diameter (mm)	Volume of grout for bolt diameter (mm) 12	16	20	25	32	40
20	25					
25	50	40	25			
32	80	70	60	40		
38		100	100	75	45	
45			150	130	100	45
50				180	150	90
62					280	225

These figures allow for a 25% wastage factor.

Storage

Lokset E has a shelf life of 12 months when stored in a dry place below 35°C in unopened containers.

Precautions

Health and safety

Prolonged and repeated skin contact with epoxy resins and curing agents may cause dermatitis in persons sensitive to these products. Gloves, barrier creams, protective clothing and eye protection should be worn when handling these products. If poisoning occurs, contact a doctor or Poisons Information Centre. If swallowed, do NOT induce vomiting - give a glass of water. If in eyes, hold eyes open, flood with water for at least 15 minutes. If skin contact occurs, remove contaminated clothing and wash skin thoroughly.

Material Safety Data Sheets (MSDS) are available to users of Parbury Technologies products on request to their nearest Parbury Technologies branch. Read MSDS, data sheet and label carefully before first use of any product.

Fire

Lokset E is non-flammable.

Additional information

Lokset E is only one product in the Parbury Technologies range of construction products. Ancillary products include concrete repair products, grouts, joint sealants, protective coatings, anchoring and flooring products, all ideally suited for industrial maintenance and construction requirements.

PARBURY TECHNOLOGIES

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Melbourne	(03) 9326 3100

7 days a week, 24 hour Technical Support Hotlines : 1800 817 779, 1800 812 864

Important note

Nitofill LV

6/1997

FOSROC

Nitofill LV

Pre-packaged low viscosity epoxy crack injection system

Uses

Nitofill LV is designed for injecting cracks in concrete and masonry wherever there is a need to consolidate a structure or exclude water and air from contact with the reinforcement.

Nitofill LV is a low viscosity system and is suitable for cracks down to 0.2 mm at the surface and cracks tapering internally down to 0.01 mm.

The Nitofill LV system is ideal for small scale repairs on site and is also suitable for insitu or precast concrete elements

Advantages

System includes everything necessary to complete the crack injection.

Convenient to use, disposable cartridge pack contains both base and hardener.

Safe and clean to use.

High strength, excellent bond to concrete and masonry.

Low viscosity allows cost effective and efficient repair.

Description

Nitofill LV crack injection system incorporates a two part epoxy base and hardener contained in a dual cartridge pack.

The Nitofill LV accessory items which are packed individually and complimentary to the cartridge pack are - a cartridge gun, injection flanges, static mixers and hoses, flange adaptors and a removing tool.

Technical support

Parbury Technologies offers a comprehensive range of high performance, high quality construction products. In addition, Parbury Technologies offers a technical support service to specifiers, end-users and and contractors, as well as on site technical support.

Design criteria

Nitofill LV is suitable for injecting cracks in concrete and masonry down to 0.2 mm at the surface and internal cracks tapering down to 0.01 mm.

The system should not be used for cracks where movement is expected to continue.

Properties

The following results are typical for the hardened Nitofill LV epoxy resin.

Usable life at	10°C:	40 minutes
	20°C:	20 minutes
	30°C:	10 minutes
Viscosity at	10°C:	250-450 cps
	20°C:	150-200 cps
	30°C:	50-100 cps
Compressive strength	1 day	57MPa
(BS 6319)	3 day	66MPa
	7 day	83MPa
Tensile strength (BS 6319):	>25 MPa	
Flexural strength (BS 6319):	>50 MPa	

Chemical resistance

The cured Nitofill LV epoxy is resistant to oil, grease, fats, most chemicals, mild acids and alkalis, fresh and sea water. Consult Parbury Technologies Technical Department when exposure to solvents or concentrated chemicals is anticipated.

Specification clauses

Low viscosity crack injection system

The crack injection system shall be Nitofill LV. It shall be applied strictly in accordance with the application instructions given in the product data sheet.

Instructions for use

Surface preparation

All contact surfaces must be free from oil, grease, free standing water or any loosely adherent material. All dust must be removed.

Mixing the surface sealant

Lokset E is used to bond the injection flanges to the substrate and to seal the face of the crack. Pour all the contents of the Lokset E hardener pack into the base container. Mix using a slow speed mixer until homogeneous.

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Application of the surface sealant

Immediately after mixing apply a small amount of product to the underside of each flange, making sure that the valve will not be blocked and place the flange centrally over the crack. Flanges should be placed between 200 mm and 500 mm apart dependent on crack size, along the length of the crack. Additional surface sealant should be applied around each flange edge and to the remainder of the crack between the flanges to ensure a resin tight seal to the substrate.

Where cracks can be sealed on one side only, flanges should be placed at centres which are 80% of the depth to which the resin is required to penetrate.

Application of the Nitofill LV injection resin can commence as soon as the Lokset E has fully hardened, (at least 12 hours at 20°C)

Injection of the Nitofill LV epoxy resin

The Nitofill LV static mixer/hose should be screwed onto the cartridge. The cartridge is then placed into the gun and the outlet end of the hose pushed onto the lowest flange using the adaptor.

The contents of the cartridge are then injected until the resin flows from an adjacent flange, or until firm and sustained hand pressure on the gun trigger signifies that no further resin will be accepted. Then pull the barb on the flange away from the base. Remove the liner strip out of the barb on the flange. Hold the base of the flange while removing the liner slip from the barb. This will ensure the flange is not accidentally removed from the substrate. The flange should be in the closed position when the liner slip is pulled totally away from the base. This will prevent material flowing out from the crack. The pressure should be released and the hose disconnected from the flange using the adaptor and tool.

The injection hose can then be refixed to an adjacent flange, and more Nitofill LV resin injected. Repeat the process until the entire length of crack has been injected.

In the case of cracks which go all the way through a wall or slab, the resin should be injected through alternate flanges on both sides where access is possible. In the case of slabs, injection from the underside takes precedence to top injection.

Making good

After the Nitofill LV injection resin has set, remove the flanges. These can be knocked off with a hammer. Make good any holes or voids with Lokset E.

The existing surface sealant can then be removed using a sharp broad-chisel or by grinding until the original substrate profile is restored.

Cleaning

All tools and equipment should be cleaned immediately after use with Fosroc Solvent 10.

Limitations

The Nitofill LV resin injection system should not be used for cracks where movement is expected to continue. Other measures should be taken to accommodate such movement, ie cutting and forming a movement joint.

Contact your local Parbury Technologies branch for further information.

Estimating

Nitofill LV resin:	450 ml pack (12 per carton)
Lokset E:	1.5 and 6 litre packs
Fosroc Solvent 10:	20 litre pails

Nitofill LV system accessory items

Nitofill LV Gun:	single item
Nitofill LV Flange:	50 per bag
Nitofill LV Adaptor:	10 per bag
Nitofill LV Static mixer/hose:	10 per bag
Nitofill LV Flange tool:	10 per bag

Storage

Nitofill LV resin, Lokset E and Fosroc Solvent 10 have a shelf life of 12 months if kept in dry conditions at 20°C.

Precautions

Fire

Fosroc Solvent 10 is flammable. In the event of fire extinguish with CO_2 or foam.

Health and safety

Nitofill LV resin and Lokset E contain resins which may cause sensitisation by skin contact. Avoid contact with skin and eyes and inhalation of vapour. Wear suitable protective clothing, gloves and eye/face protection. Barrier creams provide additional skin protection. Should accidental skin contact occur, remove immediately with a resin removing cream, followed by soap and water. Do not use solvent. In case of contact with eyes, rinse immediately with plenty of clean water and seek medical advice. If swallowed seek medical attention immediately - **do not** induce vomiting.

Nitofill LV

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Fosroc Solvent 10: Flammable liquid.
Flash point: 27°C.

Keep away from sources of ignition - no smoking. Wear suitable protective clothing, gloves and eye/face protection. Use only in well ventilated areas.

A product Material Safety Data Sheet is available to users on request to their nearest Parbury Technologies branch. Read MSDS and product data sheet carefully before first use. In emergency, contact any Poisons Information Centre.

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Important note

Parbury Technologies products are guaranteed against defective materials and manufacture and are sold subject to its standard terms and conditions of sale, copies of which may be obtained on request. Whilst the company endeavours to ensure that any advice, recommendation, specification or information it may give is accurate and correct, it cannot, because it has no direct or continuous control over where or how its products are applied, accept any liability either directly or indirectly arising from the use of its products, whether or not in accordance with any advice, specification, recommendation or information given by it.

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