University of Southern Queensland

Faculty of Engineering and Surveying

Shear Strengthening and Model Testing of Concrete Bridge Headstocks

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, 2004

Abstract

It is inevitable that as concrete bridges begin to age deterioration will become a problem. This may be caused by chemical attack, low quality construction materials, increased traffic flows or overloading. Regardless of the cause of the deterioration, the structures will require restoration. A variety of concrete rehabilitation methods can be used, however further research into the area could provide more suitable and cost efficient techniques. This research project is aimed at developing an effective means of rehabilitating concrete headstocks and relating it to the specific case of the Tenthill Creek Bridge. The research also aims to accurately model the future behaviour of these headstocks in order to gain a better understanding of the deteriorated member.

To achieve the aims of the research, three model specimens of the Tenthill Creek Bridge headstocks were developed. These specimens were then preloaded to simulate the bridge condition and then rehabilitated using epoxy crack injection and external post-tensioning. Once rehabilitated, the specimens were again tested. The results obtained from these tests could then be compared against the results obtained from a control specimen. This comparison provided an understanding of the effectiveness of the restoration techniques.

From the analysis undertaken on the data that was obtained from the experimental research, it has been discovered the by combining epoxy crack injection and external post-tensioning as a form of rehabilitation, a substantial increase in shear capacity can be expected. By utilising external post-tensioning alone, a small increase in the member’s capacity would be expected, however existing shear cracks have major influence on the effectiveness of the system. Hence by initially repairing these cracks by injecting them with epoxy resins, they will no longer have an effect on the post-tensioning system. It has been discovered by this research, the combination of epoxy crack injection and external post-tension, form a cost effective form of shear restoration for concrete members. This rehabilitation system is expected to become more prevalent in the future.

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Date

Acknowledgements

This research involved an extensive amount of experimental work, hence a considerable amount of assistance was required during the process. The author wishes to express his deepest gratitude to his supervisor Dr. Thiru Aravinthan, for his continued support and guidance throughout the course of the research.

Many thanks are also directed to the laboratory staff from Z1.101 at the University of Southern Queensland. This includes Mr Glen Bartkowski and Mr Bernard Black for their assistance during the construction and testing stages of the research. Special thanks is also expressed to Mr Mohan Trada, for his valuable advise and assistance during testing.

The author also wishes to acknowledge his fellow peers, Mr Lindsay Jobling and Mr Murshedul Alam, for their valuable assistance through the course of the research.

Finally many thanks is expressed to the author's family for all of the support that they have provided throughout the year.

The author apologises if anybody has been overlooked, however deepest appreciation is expressed.

Mr Evan Woods
October, 2004.

Contents

Chapter 3 - *Design Methodology*

Chapter 4 - *Construction & Testing*

List of Figures

List of Tables

Nomenclature

The following list of notation has been utilised throughout the text. Where possible, notation for the Concrete Structures Standard, AS3600, has been used.

A = *area of section in second moment of area calculations*

A_g = *gross area of a concrete cross section*

A_{sc} = *cross-sectional area of compression reinforcement*

A_{st} = *cross-sectional area of tension reinforcement*

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

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

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

a_v = *distance from load point to the face of the nearest support*

b = *width of a rectangular cross-section*

b_v = *effective width of web for shear (equal to b for rect. cross- section)*

C_c = *compressive force in the concrete of a cross-section*

C_s = *compressive force in the compressive reinforcement*

D = *overall depth of section*

d = *depth to resultant force in tensile steel at* M_u

d_c = *distance from extreme fibre to force* C_c

d_n = *depth to the 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_{st} = *depth to centre of tensile reinforcement*

e = *eccentricity of the prestressing force from the neutral axis*

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

f_{sy} = *yield strength of reinforcing steel*

I_g = *second moment of area of the gross cross-section about centroidal axis*

I_{xx} = *second moment of area of a cross section about the principal X-axis*

L = *centre-to-centre distance between supports*

L_{eff} = *effective span of a beam or slab*

L_n = *length of the clear span, measured face-to-face of supports*

M_{dec} = *decompression moment*

M_u = *ultimate strength in bending*

P = *prestressing force*

P_u = *ultimate theoretical load capacity of a member*

P_{ue} = *ultimate experimental load capacity of a member*

P_v = *vertical component of prestressing force*

s = *centre-to-centre spacing of shear reinforcement*

T_p = *tensile force in prestressing steel*

T_s = *tensile force in tension reinforcement*

V_{dec} = *shear force at the decompression moment*

V_u = *ultimate shear strength*

V_{uc} = *ultimate shear strength of the concrete alone*

V_{us} = *contribution by shear reinforcement to the ultimate shear strength*

y_b = *distance from centroidal axis to bottom fibre*

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

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

ε = *strain*

ε_{cp} = *concrete elastic strain at tendon level due to prestress*

ε_{sc} = *compressive strain in the compression reinforcement*

ε_{st} = *tensile strain in the tension 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 member*

σ = *stress*

σ_{pu} = *stress in the prestressing steel at* M_u

σ_{sc} = *compressive stress in the compression reinforcement*

σ_{st} = *tensile stress in the tension reinforcement*

CHAPTER 1

INTRODUCTION

1.1. Background

Over the past two decades the rapid deterioration of concrete bridge structures has become a serious problem in many countries. Approximately half of the modern concrete bridges in the world are approximately over 45 years old, therefore there is a growing concern regarding the structural quality of these bridges. There are a number of reasons for the deterioration of these structures including increased traffic flows and weight of vehicles, influence of environmental pollution, low quality structural materials and limited maintenance programs (Radomski, 2002). The most common cause of deterioration in bridges today is overloading. In engineering terms, *overloading* refers to applying loads to a member that are greater than that which they are designed for. In many cases the overloading is caused by the increased weight of transportation vehicles, such as trucks. This problem illustrates a clear need to develop a cost effective rehabilitation technique to restore the structures to their original working status.

This research is focused on developing a rehabilitation technique to repair damaged bridge headstocks. It was mentioned earlier that a substantial amount of concrete bridge deterioration is due to overloading, as is the case with the Tenthill Creek Bridge. The headstocks within the bridge are severely cracked, which dramatically decreases the load carrying capacity of the bridge. In order to restore the structure to its original working condition, rehabilitation will be required. The objective of this research is to develop and analyse a method of repairing headstocks that could be utilised in the rehabilitation of concrete bridges in the future.

1.2. Tenthill Creek Bridge

The Tenthill Creek Bridge is located approximately three kilometres west of Gatton in south-east Queensland. The bridge is situated on the Gatton-Helidon Road which was previously known as the Warrego Highway. Appendix A – *Bridge Location Map* shows the locality of the bridge. The bridge spans approximately 28 metres and is supported in three spans by column bent piers. These piers consist of a cap beam, or headstock, supported by two rectangular shaped piers, as seen in Figure 1.1. The piers are constructed on small spread footings which are supported by octagonal prestressed piles. Column bent piers are commonly used in highway bridges and are generally utilised where moderate clearance is required.

Figure 1.1: *Tenthill Creek Bridge Headstock Configuration*

Transportation of the Millmerran Power Station across the Tenthill Creek Bridge has caused substantial shear cracking in the headstocks. The severity of this cracking can be seen in Figure 1.2. These cracks extend vertically from the edge of the pier through the headstock and are approximately 1600mm long and 0.5mm wide. Since the headstocks are only 1700mm high, the cracks have almost extended the full height of the member. Consequently, almost the entire shear load is being carried by the reinforcement. If this load is greater than the yield strength of the reinforcement, the headstocks will almost certainly fail causing the bridge to collapse. Flexural cracking, also caused by overloading, can also be seen in the headstocks, however the cracks are far less critical. The flexural cracks will also require attention to ensure that they do not propagate any further into the member. It is obvious, that prompt attention is required to ensure the safety of motorists using the bridge.

Figure 1.2: *Cracking in Headstocks*

The south-east Queensland division of Main Roads has recently analysed the condition of the headstocks within the Tenthill Creek Bridge. Earlier, Main Roads rehabilitated the concrete girders within the bridge that were also showing signs of deterioration. This was completed by applying external post-tensioning to the girders to increase their load carrying capacity. At the stage of this restoration, the condition of the headstocks was acceptable, therefore rehabilitation was not required. However, since then the headstocks have deteriorated to a state that now requires immediate attention. Main Rods have recently completed upgrading the headstocks. Therefore the designs for the rehabilitation techniques used for this research have been based on the work that was carried out on the bridge.

1.3. Aims and Objectives

It has been mentioned above that many concrete bridges are deteriorating and will soon require some form of rehabilitation. The headstocks are a major structural member of these bridges, therefore they must be well maintained to ensure the stability of the bridges. The first aim of this research was to investigate the effectiveness of epoxy injection and external post-tensioning as a form of shear strengthening concrete bridge headstocks. The rehabilitation technique that was used to undertake the experimental investigation for this research involved the use of both epoxy injection and external post-tensioning together.

The second aim of the research was accurately model the behaviour of the deteriorated Tenthill Creek Bridge headstocks. This requires scaling the original headstocks down to a model size that can be tested to predict the member's behaviour in the future.

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

1. Research background information on the use of epoxy injection and external post-tensioning and determine the extent of their use within concrete bridge rehabilitation.
2. Investigate the state of the Tenthill Creek Bridge.
3. Design a model of the bridge headstocks.

4. Investigate and obtain a suitable epoxy resin to use in the experimental tests.
5. Design and develop a suitable external post-tensioning system.
6. Construct and test three specimens, one of which is used as a control headstock.
7. Finally, critically analyse the shear capacity of the test specimens and advise on a suitable rehabilitation technique based on the findings.

1.4. Effects of Research

In attempting to rehabilitate concrete bridges, it will inevitably cause some consequential effects. Through examining all of these effects that can be caused directly and indirectly, the consequences can be minimised. When examining the consequential effects of research such as this, three major aspects need to be addressed. These include aspects of sustainability, ethical responsibility and safety aspects. All three of these have been examined below.

1.4.1. Sustainability

Sustainability is one of the most important aspects of any engineering research conducted in today's society. This includes environmental protection, protection of future generations and global protection. These all seem irrelevant when researching bridge rehabilitation, however with closer analysis many problems begin to emerge. Bridges are already set into the environment therefore rehabilitation should have a minimal impact on the surroundings however, actions such as flora clearing, to carry out the work will impact on the environment. Also materials used to complete the work are produced from a finite source, meaning that materials to undertake the work are non renewable. Bridge deterioration is a global problem, therefore outcomes from any research will produce effects on a worldwide scale.

Conversely, not all of the sustainability issues will have a negative impact. For example the rehabilitation of these structures will preserve them for the use of future generations. Additionally, without some form for rehabilitation the bridges become disposable and therefore the impact on sustainability is much greater. The rehabilitation of concrete bridges will ultimately result in the utilisation of less finite resources and therefore improving sustainability aspects. Sustainability is a very controversial issue and many strong arguments can be made for and against the rehabilitation of concrete bridges. Transportation however is one of the most important aspects of the human race and without bridges, transportation would be severely confined.

1.4.2. Ethical Responsibility

It is important that all engineers abide by the appropriate code of ethics. Engineers are responsible for any actions that they undertake, even if they are directed to do so. Therefore all work needs to be closely analysed to ensure that it is not in breach of the code of ethics. The research surrounding the rehabilitation of bridge headstocks does not appear to breach any tenets of the code. With the implementation of the techniques into the field, the code may need to be more closely examined. It is important that this is done, as breaching any tenets of the code can generate serious consequences.

1.4.3. Safety

Finally, every engineering activity involves some form of physical risk to those who are directly and indirectly involved. When considering concrete bridge rehabilitation, these people would include construction workers undertaking the rehabilitation and also the general public who use the bridge. Short term measures must be implemented to guarantee the safety of those undertaking the rehabilitation work. Some of these measures would include-

- Briefing all workers to ensure that they understand the safety issues relating to the project,
- Making certain that all workers are qualified to carry out specified tasks, this is vital when constructing prestressing systems, and
- Minimising all hazards within the workplace.

These are a few general measures that would need to be considered when undertaking a rehabilitation project. However, there are many more safety measures that would need to be considered surrounding a specific project.

When analysing the safety of those not directly involved with the project, such as the public, long term safety effects would need to be considered. Such effects would include-

- Any effects from pollution, for example long term chemical reaction of epoxy,
- Removal of any hazardous material, for example protruding steel, any chemicals used in construction,
- Ensuring the safety of the surrounding environment, this would include flora and fauna, etc.

Again, these are only a few general safety issues that would to be attended to, there would be many more site specific issues that would require analysis.

1.5. Rehabilitation Techniques

The main focus of the research was to test the effectiveness of external post-tensioning on the shear strength of concrete headstocks under varying crack conditions. As stated above, the two rehabilitation techniques were used in this research included external post-tensioning and epoxy crack injection. Both of these techniques are introduced below.

1.5.1. Post-tensioning

External post-tensioning is a method of reinforcing or strengthening concrete members with high strength steel strands. To counteract shear cracking, the post-tensioning would work most effectively if was positioned vertically on the headstock (Figure 1.3(a)). Due accessibility problems associated with bridge headstocks, the post-tensioning is placed horizontally (Figure 1.3(b)). By applying the compressive force to the headstock, the shear cracks will close and by doing so it is hoped that the aggregate interlock will be reformed. This process has been used for a number of years and the results have found to be effective. However how far can the extent of the shear cracking propagate before the aggregate interlock between the crack faces can not be reformed?

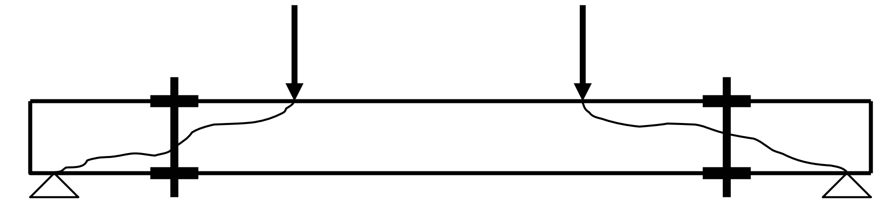


Figure 1.3(a): *Vertical Post-tensioning*

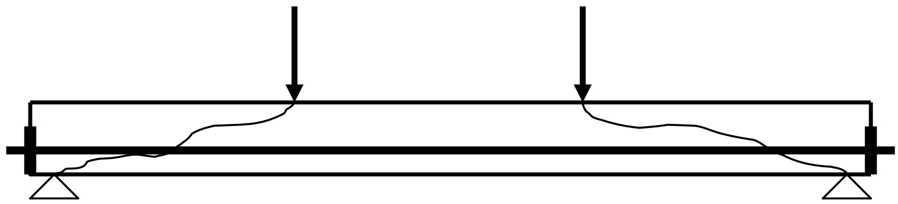


Figure 1.3(b): *Horizontal Post-tensioning*

1.5.2. Epoxy Injection

Obviously once a shear crack becomes large enough, simply applying post-tensioning will not reform the aggregate bond between the crack faces as some movement will occur. Therefore another form of rehabilitation must be utilised to regenerate the bond. This can be done with the use of epoxy injection. The epoxy is a resin that is pumped into the crack that once cured will form a bond between the crack faces. The epoxy will restore the structural quality of the member and in most conditions it will make the structure 'as good as new'. Another major advantage of the epoxy injection, is that by filling the cracks, penetration of moisture salts and other chemicals is prevented. Hence this will protect against the premature deterioration of the reinforcing and in some cases this is a crucial part of the rehabilitation process. This maybe a quality attribute of the epoxy, but the resins bonding properties is what was tested during this research. Below in Figure 1.4, an example of epoxy injection can be seen.

Figure 1.4: *Epoxy Crack Injection*

1.6. Summary

The deterioration of concrete bridges is a problem that cannot be ignored, therefore a suitable method to counteract this problem is required. This research aimed to develop a suitable technique for the restoration of concrete bridge headstocks that can be utilised in the future. The project is also related to the case of the Tenthill Creek Bridge that is currently being restored. Many different forms of member rehabilitation techniques have been developed, however not many have utilised both epoxy and external post-tensioning together.

CHAPTER 2

LITERATURE REVIEW

2.1. Introduction

Through researching this project it has been discovered that large amounts of work has been conducted in the area of bridge rehabilitation. This chapter will commence by presenting a brief overview of this work. The following section will identify work that has been undertaken in relation to external post-tensioning and epoxy crack injection.

2.2. Concrete Headstock Rehabilitation

There is over 33,000 bridges in Australia alone, 50% of which are constructed of reinforced or prestressed concrete (Austroads 2002). It is estimated that in Australia alone, approximately $85 million is spent annually repairing or replacing aging concrete bridges (Austroads 2002). For this reason an extensive amount of research has been devoted to developing a viable and cost effective method of upgrading these aging structures.

It has been noted, by a number of publications, that the deterioration of concrete bridges is caused by a number of different reasons. Akasha and Farkas (1998) report that a majority of the concrete bridge deterioration is the result of poor maintenance, increased legal load limits, insufficient reinforcement, excessive deflections, structural damage, corrosive attack or any combination of these. A majority of these structures are classified as deficient and in need of strengthening or replacement. The design life of the structure is an important factor in the decision of whether to restore or replace a structure. Such a decision is generally influenced by site conditions, operational requirements, possible technical solutions and the cost-benefit relationship.

The strengthening of structural members can be achieved through a number of different processes. Some of these include replacing defective or poor material, attaching additional load bearing material (for example: reinforcement, high strength concrete, thin bonded straps, post-tensioning or a combination of these) or redistributing imposed loads (Dywidag-Systems International n.d.). All of these methods can be utilised as an effective form of rehabilitation, however the correct technique needs to be selected to suit the member's requirements. It is noted by Pisani (1999), that external post-tensioning appears to be the most viable and cost effective form of bridge rehabilitation. In most cases a concrete bridge headstock is a relatively simple member to strengthen. The member is generally exposed on all sides apart from the top. This allows ample access to be able to apply many different rehabilitation techniques. Due to this exposure external post-tensioning can be easily fitted to most concrete bridge headstocks.

External post-tensioning is a rehabilitation technique that has been utilised for approximately sixty years (Stresscrete n.d.). The systems structural quality has been proven throughout this time. Conversely, epoxy injection is a relatively new system that has been introduced to the market in the past decade. In the past decade the system has proven to be a quality form of restoration. Research however into the use of epoxy injection and external post-tensioning together is limited. Consequently, the available literature can be broken down into two categories, rehabilitation with external post-tensioning and that with epoxy crack injection.

2.3. External Post-tensioning

2.3.1. What is external post-tensioning?

The Post-Tensioning Institute (2000) reports that external post-tensioning is an effective method of strengthening concrete. It uses high strength steel strands or bars referred to as tendons (Figure 2.1). This form of reinforcement is known as active reinforcement as it is prestressed, therefore it is effective before and after the member is cracked. Conventional reinforcement is known as passive reinforcement and only carries load after the member has cracked. Pisani (1999) identifies that external post-tensioning appears to be the most promising form of rehabilitation or strengthening of statically determinant structures, particularly bridges. Hwee (1997) believes that post-tension is an attractive method of strengthening concrete structures as it adds little weight to the original structure, its application poses little disturbance to the members surroundings and it allows the monitoring, re-stressing and replacement of tendons. Hwee (1997) has recently completed a three year study into the efficiency of the application of external prestressing in relation to beam strengthening. Through the course of his experimental work he concluded that by applying a prestress force to a beam with a small span-to-effective depth ratio, the likelihood of shear failure is dramatically decreased. Therefore it can be concluded that by applying post-tensioning to a bridge headstock, the members shear capacity will be increase.

Figure 2.1: *External Post-tensioning Used to Strengthen Bridge Headstock*

Pisani (1999) discovered that a member subjected to a single overloading case will not require rehabilitation as long as the cyclic loading produced by transport does not generate stress greater than the yield stress of the reinforcement. This may appear logical, however cyclic loading can cause strange outcomes within reinforcement and Pisani has only tested his theory to 10 000 cycles. Pisani (1999) also reports that if the above case is fulfilled, the member can be strengthened by means of prestressing and the load carrying capacity will be dramatically increased.

Dywidag-Systems International (n.d.) has reported that bridges of any material can be strengthened by adding external post-tensioning tendons. The influence of this rehabilitation on serviceability and ultimate limit states can be varied within wide limits by selecting different methods of introducing prestressing forces and using various tendon profiles. The correct tendon profile however needs to be selected to suit the rehabilitation required. This presents the effect of eccentricity. If the prestress force is applied above or below the centroid of the section by a certain eccentricity, a moment will be induced in the member. This moment is generally used to counteract moments induced by loading. When a member with external tendons deflects however, the tendon remains straight between the deviators or anchorages, hence altering the eccentricity (Figure 2.2). The change in eccentricity at a section becomes significant when the deflection of the section is relatively large compared to the members span (Ariyawardena & Ghali 2002). The effects of eccentricity variation should be accounted for when large deflections are expected. This is accomplished by considering geometric nonlinearity in the analysis.

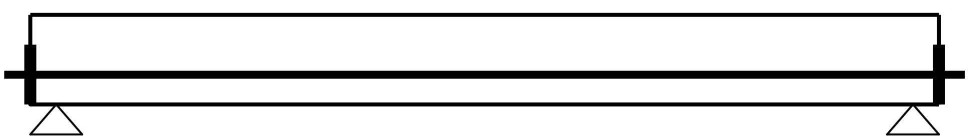


Figure 2.2(a): *Post-tensioned Member Before Loading*

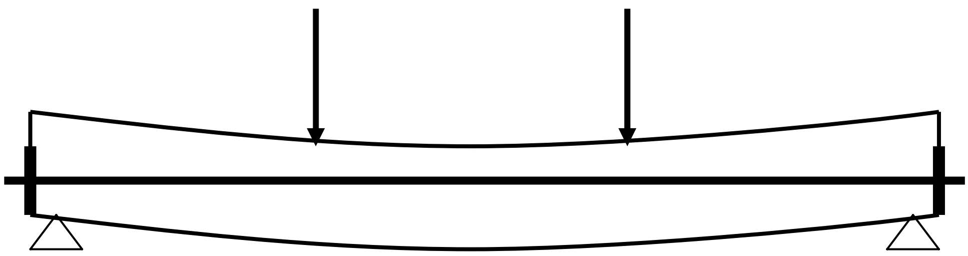


Figure 2.2(b): *Deflected Member*

(Note the change in eccentricity at the centre section)

2.3.2. Problems Associated With External Post-tensioning

One of the major problems associated with prestress in general is its susceptibility to creep. The force in a prestressing tendon begins to diminish from the instance when the steel is first tensioned and continues throughout the life of the prestressed member (Warner et al 1998). Several factors contribute to the problem of creep. Some of these factors include:

1. Short term losses:
   - elastic deformation of the concrete,
   - elastic stretching of the tendon,
   - friction between the post-tension tendon and the ducting, and
   - slip of the anchorages.
2. Long term losses:
   - stress relaxation of the tendons, and
   - shrinkage of the concrete due to the prolonged applied load.

During the design of the prestressing system, allowances should be made for these losses. This done by initially over-stressing the tendons to allow for the amount of creep that is expected (Warner et al 1998).

Another major problem connected to external post-tensioning is the effects of corrosion. Corrosion can be highly detrimental to prestressing as the tendons are under significant mechanical stress (Whiting, Corley & Tabatabai n.d.). Whiting, Corley and Tabatabai (n.d.) undertook various field investigations and experimental testing into the corrosive problems associated with prestressing.

From their analysis they have recommended that periodic repair and reapplication of protective systems may be necessary to guarantee the structural quality of the prestressing systems. Where such periodic repairs are difficult to implement, they suggest that complete replacement of distressed member may be a long-term cost effective alternative. Proper repair and rehabilitation of deteriorating prestress systems is a primary concern of owners, therefore it is crucial that the effects of corrosion are not ignored.

In past decade, the quantity of post-tensioning systems sold has almost doubled and the industry is continuing to grow rapidly (Post-Tensioning Institute 2000). As the industry continues to grow, the cost efficiency will rapidly increase therefore making the system more viable for restoration applications. Many cases have been investigated after the application of the post-tensioning and it has been reported that the rehabilitation technique has been effective (Dywidag-Systems International n.d.). It is obvious therefore that this form of rehabilitation should be utilised in the future.

2.4. Epoxy Crack Injection

The use of epoxy resins as a form of crack rehabilitation is relatively new to the construction world, however their use is becoming more and more popular (Figure 2.3). Epoxy injection is one of the most versatile problem solving products on today’s market. Epoxy Systems (2004) states that the structural restoration of concrete by epoxy injection is very often the only alternative to complete replacement. This technique can therefore lead to dramatic economic savings.

Figure 2.3: *Epoxy Crack Injection*

(Epoxy Systems 2004)

Epoxy Systems (2004) notes that epoxy injection serves two purposes:

1. It effectively seals the cracks to prevent damaging moisture entering, and
2. It monolithically welds the structure together.

The sealing properties of the injection, prevents premature failure of the reinforcing. This can be of equal or sometimes greater importance than the epoxy's structural welding properties. Cracks left unprepared allow moisture, road salts and other contaminants to attack the reinforcing. Once the crack is penetrated, the rebar will deteriorate and hence loose its structural value. In extreme cases losing the entire structure can be the result. Therefore sealing these cracks is generally high on the list of priorities.

The other major purpose for using epoxy crack injection is the resins exceptional bonding properties. Epoxy Systems (2004) report that the epoxy's bonding properties are good enough to restore a structure to it original condition. Minoru et al (2001) have undertaken laboratory tests on the bonding qualities of epoxy injection. From their analysis they have concluded that the resins will bond exceptional well to crack faces, however for precast segments the smooth face should be removed before the resins are applied. Furthermore as a result of their analysis it could be concluded that depending on the quality of the resins and the application, a superior bond can be expected when crack injection is undertaken.

One of the most important considerations that should be noted when utilising epoxy crack injection, is that the resins will repair the cracks, however they will not fix the cause of the cracking. This information is noted in almost all of the literature relating to this restoration technique. Demetrios and Tonias (1995) have discovered that a member subjected to an overloading condition which induces cracking can be repaired using the injection of epoxy resins. The technique will only be effective however, if the overloading condition was a one off occurrence. If the crack is constantly reopened from the effect of general live loading, then crack injection will be ineffective as the crack will reform. Demetrios and Tonias (1995) recommend that preloading the member until the injection material has reached a sufficient strength may help to overcome this problem. Generally it is essential however that the cause of the cracking be rectified during the rehabilitation process. This will eliminate the threat of the crack being reformed.

Epoxy resins are new in the field of concrete bridge rehabilitation therefore further research into this area may uncover more effective uses for the materials. This research is aimed at furthering knowledge of epoxy resins by using them in conjunction with external post-tensioning.

2.5. Summary

External post-tensioning is a proven form of concrete rehabilitation. Under extreme crack conditions however, external post-tension becomes less effective due to the loss of aggregate bond between the crack faces. Epoxy injection is a relative modern technique for repairing cracks in concrete members. This form of rehabilitation is capable of reforming the aggregate bond that is lost between the crack faces. Consequently if the to techniques are combined, the load carrying capacity of the restored concrete member should be dramatically increased. This theory was tested through the course of this research.

CHAPTER 3

DESIGN OF THE SPECIMENS

3.1. Introduction

This chapter presents an overview of the design work that has been undertaken during the course of the research. The section below presents the procedure used to develop a model the Tenthill Creek Bridge headstocks.

3.2. Bridge Headstocks

It was noted earlier that the bridge is supported by column bent piers. The headstocks are a rectangular shape and have a trapezoidal cross section. The headstocks measure 1678mm high, 1068mm wide at the top and 686mm wide at the bottom. The overall length of the headstock is 9m. The types of reinforcement used in the headstocks are equivalent to Y32 and Y16 rebar. Appendix C contains a copy of the bridge plans from when it was constructed. From these plans note the trapezoidal shape of the headstock. It is also shown that drainage provisions have been made within the headstock and bearing pads have been used to support the girders. Figure 3.1 below shows the constructed configuration of the headstocks.

Figure 3.1: *Tenthill Creek Bridge Headstocks*

3.3. The Model

The design of the test specimens was produced by scaling the bridge headstocks down to a size that could be tested. Two different scales were initially chosen, ⅓ and ¼. The capacities of these two designs were then calculated and the ¼ size design was adopted. The capacity calculations are included in following sections. The adopted specimen size and section details are shown below in Figure 3.2.

Figure 3.2(a): *Plan of Model Specimen*

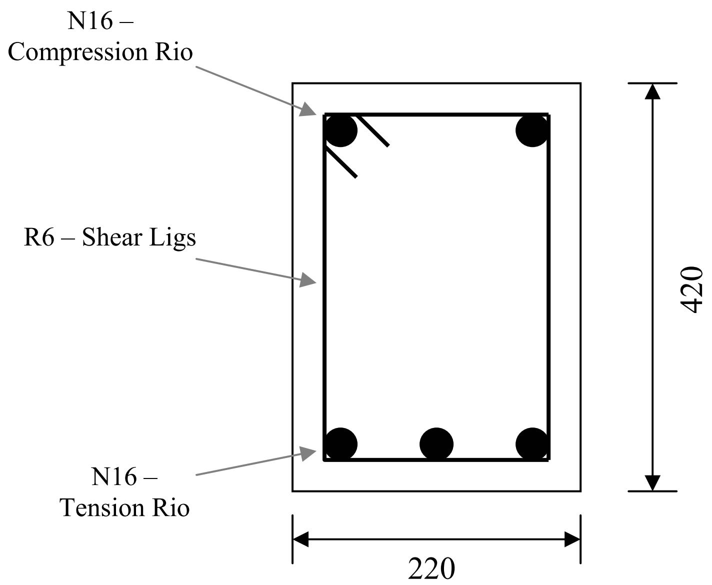


Figure 3.2(b): *Section Layout for Model Specimen*

It can be seen in Figure 3.2(b), the section has been simplified from a trapezoidal shape to a rectangular shape for the model. This was completed to simplify the required design calculations and also the construction work. An average of the top and bottom distances of the trapezoid was taken to produce the rectangular section. Additionally the dimensions were then scaled by ¼ to develop the model. The reinforcement contained in the bridge was then scaled by an area ratio of 1:16 to determine the area of steel required in the specimen.

3.3.1. 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 (M_u), the resultant compressive force C, consists of two component forces, C_c in the concrete and C_s in the compressive steel (Figure 3.3). Therefore when determining M_u the two forces are treated separately. After determining all of the forces and their point of action, M_u can be determined by assessing moments about the top most fibre.

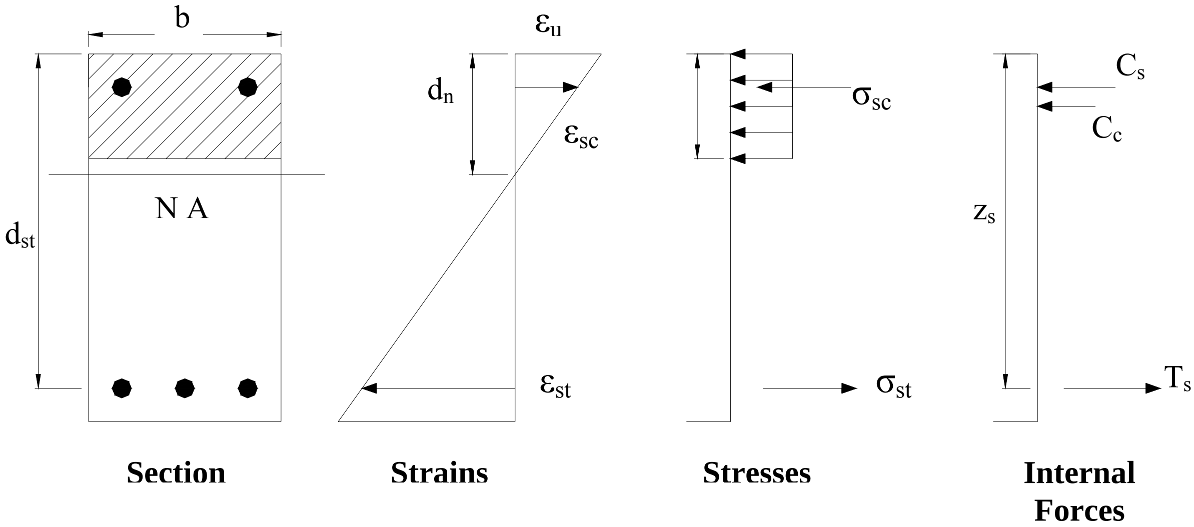


Figure 3.3: *Doubly Reinforced Section at Ultimate Moment*

In an accurate proportioned section, the tensile steel will yield when M_u is reached. Whether or not the compressive reinforcement will yield however will depend on the depth d_{sc} relative to d_n. Therefore when determining M_u for a section, it is reasonable to assume that both the compression and tension steel yield to determine d_n. Once d_n is determined, the section can be reassessed to determine if the assumption was correct. If the compression steel has yielded d_n will be correct, however if the yield point has not been reached the steel will still be in its elastic region and d_n can be determined by using trial strain distributions. This was required to determine M_u for the model specimen and the calculations can be seen below.

- ***Mu for the Model Section***

Section Properties

$$f_c' = 32MPa$$

$$\gamma = 0.85 - 0.007(f_c' - 28)$$

$$= 0.85 - 0.007(32 - 28) = 0.822$$

Reinforcing Properties

$$A_{st} = 600mm^2$$

$$A_{sc} = 400mm^2$$

$$f_{sy} = 500MPa$$

Initially assume that all reinforcement yields before M_u is reached.

Therefore Forces Equal:

Tensile Steel Force:

$$T = f_{sy} A_{st} = 500 \times 600 = 300 \times 10^3 N$$

Compressive Steel Force:

$$C_s = f_{sy} A_{sc} = 500 \times 400 = 200 \times 10^3 N$$

Concrete Compressive Force:

$$C_c = 0.85 f_c' \gamma b d_n = 0.85 \times 32 \times 0.822 \times 220 d_n$$
$$= 4918.85 d_n N$$

$\sum H = 0 \rightarrow$ $$4918.85 d_n + 200 \times 10^3 = 300 \times 10^3$$

$$d_n = 20.33mm$$

Therefore the tensile steel clearly yields, however the compressive steel:

$$\varepsilon_{sc} = \varepsilon_u \frac{d_n - d_{sc}}{d_n}$$

$$= 0.003\left(\frac{20.33 - 44}{20.33}\right) = -0.0035 < 0.0025(\varepsilon_{sy})$$

The assumption that the compressive steel had yielded is invalid, hence d_n is solved by a trial and error approach, iterating on assumed values of d_n until the equilibrium condition of $C_c + C_s - T = 0$.

Note an example of the working for the trial and error process can be seen in Appendix D.

Try $d_n = 52.833mm$

Compressive Steel Strain:

$$\varepsilon_{sc} = 0.003\left(\frac{52.833 - 44}{52.833}\right) = 0.0005$$

Compressive Steel Force:

$$C_s = 0.0005 \times 200000 \times 400 = 40124.92N$$

Concrete Compressive Force:

$$C_c = 4918.85 d_n = 4918.85 \times 52.833 = 259877.5N$$

Tensile Steel Force (As Before):

$$T = 300 \times 10^3 N$$

Check Equilibrium:

$$\sum H = 259877.5 + 40124.92 - 300 \times 10^3 = 2.42 \approx 0$$

From these values (C_c, C_s and T), M_u can be calculated by assessing the moments about the top fibres.

Leaver Arms:

$$d_c = \frac{0.822 \times 52.833}{2} = 21.71mm$$

d_{sc} *= 44mm* *(ie. Depth to compressive steel)*

d_{st} *= 420 – 30 – 6 – 8 = 376mm* *(ie. Depth to the tensile steel)*

Calculate M_u:

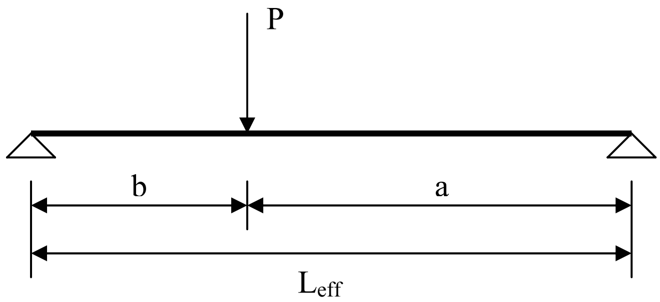
$$M_u = Td_{st} - C_c d_c - C_s d_{sc}$$

$$M_u = \left(300 \times 10^3 \times 376\right) - \left(259877.5 \times 21.7\right) - \left(40124.92 \times 44\right)$$

$$M_u = 105.40kNm$$

From M_u the ultimate force required to produce the moment can be determined from:

$$P_u = \frac{M_u L_{eff}}{ab}$$



Where:

$$L_{eff} = Min(L; L_n + D)$$

Therefore:

$$L_{eff} = L = 2300 - (190 \times 2) = 1920mm$$

The specimens were loaded at ⅔ of the top distance (ie. 775mm) along the specimen. Therefore:

$$a = 1920 - 585 = 1335mm$$

$$b = 775 - 190 = 585mm$$

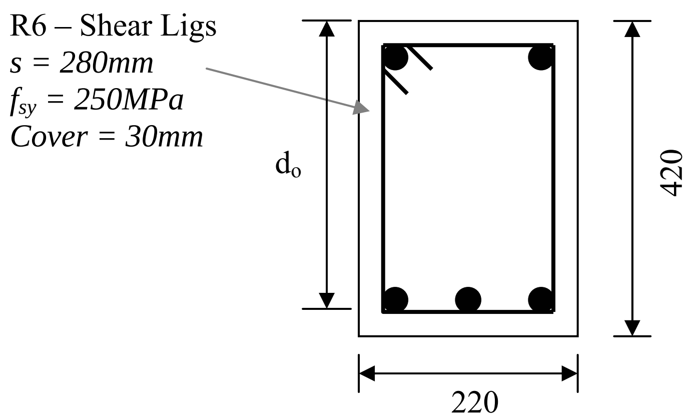
Calculate P_u:

$$P_u = \frac{105.40 \times 10^6 \times 1920}{1335 \times 585} = 259kN$$

This force can now be assessed against the ultimate shear capacity of the member to determine the mode of failure during loading. The research is based on shear analysis, therefore the specimens should be designed to ensure that shear failure does occur. This force also needs to be checked against the ultimate capacity of the loading equipment to determine if the specimens can be tested.

3.3.2. Shear Capacity

Determine the ultimate shear strength (V_u) of the section.



- ***Ultimate Shear Strength (V_u)***

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

Ultimate Shear Strength of the Concrete:

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o \left[\frac{A_{st} f'_c}{b_v d_o} \right]^{\frac{1}{3}}$$

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

$$\beta_1 = 1.1\left(1.6 - \frac{376}{1000}\right) = 1.346$$

- The factor β_2 is used to account for the effect of an axial load such as prestressing, however in this case no axial load is present, therefore β_2 = 1.0.
- The factor β_3 is used to account for large concentrated loads near the support of the member. Hence due the configuration of the specimen, $\beta_3 \neq$ 1.0. β_3 is calculated by the formula:

  $$\beta_3 = \left(\frac{2d_o}{a_v}\right) \leq 2.0$$

  Where a_v is the distance from the inside of the support to the load point. Therefore:

  $$a_v = 395mm$$

  $$\beta_3 = \left(\frac{2 \times 376}{395}\right) = 1.904$$

Therefore:

$$V_{uc} = 1.346 \times 1.0 \times 1.904 \times 220 \times 376 \times \left(\frac{600 \times 32}{220 \times 376}\right)^{\frac{1}{3}}$$

$$V_{uc} = 130.29kN$$

Ultimate Shear Strength of Reinforcement:

$$V_{us} = \frac{A_{sv}}{s} f_{sy} d_o \cot \theta_v$$

Determine the angle between the concrete compression strut and the longitudinal axis of the member (θ_v). The A_{sv}, the area of the stirrup, is known, θ_v can be calculated using:

$$\theta_v = 30^o + 15^o \left[\frac{A_{sv} - A_{sv-\min}}{A_{sv-\max} - A_{sv-\min}} \right]$$

Calculated the maximum and minium stirrup areas:

$$A_{sv-\min} = 0.35 b_v \frac{s}{f_{sy}} = 0.35 \times 220 \times \frac{280}{250} = 86mm^2$$

$A_{sv\text{-}min}$ is large due to the conservative yield stress of the R6, hence the quantity of steel used in the beam is less than $A_{sv\text{-}min}$. As a result, θ_v can be calculated within a range of values from 30^o to 45^o. This is not conventional, however as A_{sv} is small, when using the equation above, θ_v is approximated at less than 30^o. Therefore by calculating V_u over a range of values an approximate idea of the ultimate shear strength will be given.

For $\theta_v = 30^o$:

$$V_{us} = \frac{62}{280} \times 250 \times 376 \times \cot(30) = 36.05kN$$

For $\theta_v = 45^o$:

$$V_{us} = \frac{62}{280} \times 250 \times 376 \times \cot(45) = 20.81kN$$

Calculate V_u:

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

For $\theta_v = 30^o$:

$$V_u = 130.29 + 36.05 = 166.34kN$$

For $\theta_v = 45^o$:

$$V_u = 130.29 + 20.81 = 151.10kN$$

Note that the ultimate shear strength (V_u) of the specimen is dramatically smaller than the load required to induce the ultimate moment (M_u). The design has been developed in this way to ensure that when the testing is undertaken, the specimen will fail in shear. Additionally, the area of the shear reinforcement has been directly scaled from the bridges reinforcement. Therefore the area of the shear reinforcement in the bridge is also low, resulting in the development of the shear cracks.

3.4. Prestress Design

The load capacity of the specimens is expected to increase after the application of the prestressing. Consequently, the ultimate capacities after the application of the prestress needs to be determined to ensure that the available testing equipment is capable of failing the specimens. The sections below present the analysis undertaken to determine these capacities.

The force induced in the external post-tensioning, for this research was developed by scaling down the force that was applied to the rehabilitation of the Tenthill Creek Bridge. A total of four tendons were attached to the bridge and each was stressed to 1000kN, inducing a total force of 4000kN. This force was scale by the ratio of 1:16 to produce a force of 250kN be applied to the test specimens.

3.4.1. Flexural Capacity

Again the flexural capacity of the specimens after prestressing needs to be assessed to ensure that shear failure will occur.

When analysing the effects of prestressing on a beam, it generally assumed that any change in concrete strain will also occur in the steel. In the case of external unbonded tendons, the steel stress remains constant over the full length of the beam. Consequently, various conservative approximations have been developed to allow the stress in the unbonded tendons at ultimate moment to be estimated. In AS3600 Clause 8.1.6, two expressions are specified for approximation this tendon stress. For the purpose of this research, it has been assumed in the analysis that the stress in the external tendons does not increase as the member deflects. This will provide conservative results that are accurate enough for the purpose of this analysis.

The analysis used to determine M_u for the prestressed section is similar to that used above in section 3.3.1. The analysis can be seen below.

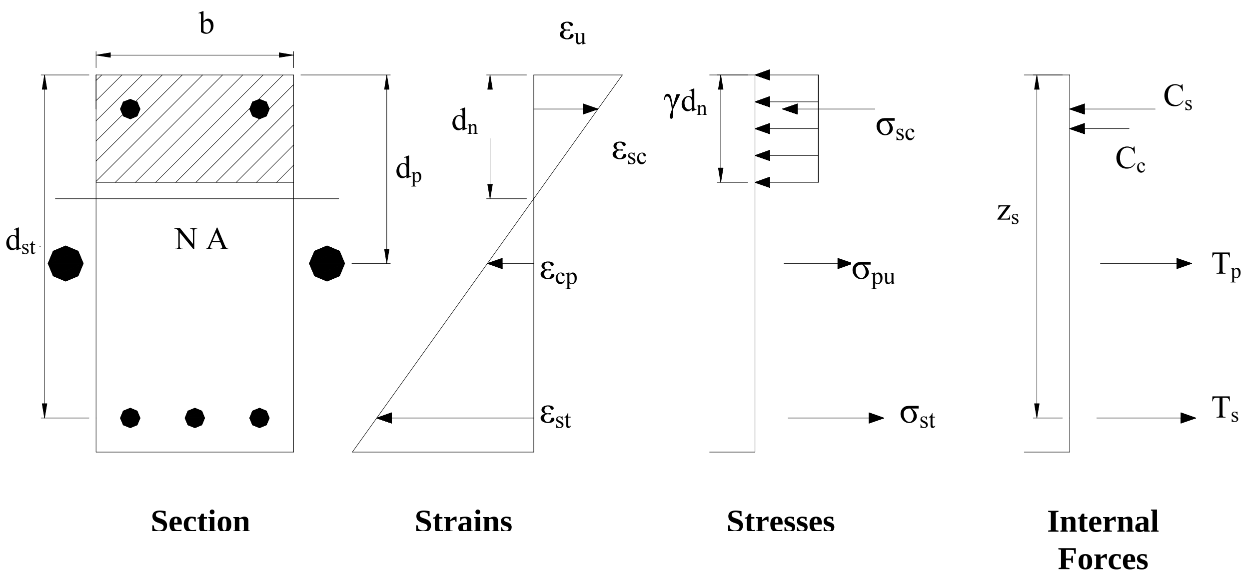


Figure 3.4: *Doubly Reinforced Section with External PT at M_u*

The section properties and reinforcing properties are the same as that in section 3.3.1 above.

Initially assume that all reinforcement yields before M_u is reached.

Determine the forces:

$$T_s = f_{sy} A_{st} = 500 \times 600 = 300 \times 10^3\, N$$

$$C_s = f_{sy} A_{sc} = 500 \times 400 = 200 \times 10^3\, N$$

$$C_c = 0.85\, f_c' \gamma b d_n = 0.85 \times 32 \times 0.822 \times 220 d_n$$

$$= 4918.85 d_n N$$

$$T_p = 250 \times 10^3\, N$$

$\sum H = 0 \rightarrow$ $$4918.85 d_n + 200 \times 10^3 = 300 \times 10^3 + 250 \times 10^3$$

$$d_n = 71.15mm$$

Therefore the tensile steel clearly yields, however the compressive steel:

$$\varepsilon_{sc} = 0.003\left(\frac{71.15 - 44}{71.15}\right) = 0.0011 < 0.0025(\varepsilon_{sy})$$

The assumption that the compressive steel had yielded is invalid, hence d_n is solved by a trial and error approach, iterating on assumed values of d_n until the equilibrium condition of $C_c + C_s - T = 0$.

Note an example of the working for the trial and error process can be seen in Appendix D.

From trial and error, $d_n = 87.545mm$

The forces equal:

$C_s = 119376.3N$

$C_c = 430620.5N$

$T_s = 300 \times 10^3 N$

$T_p = 250 \times 10^3 N$

Calculate M_u using these forces, by taking moments about the top most fibres.

Leaver Arms:

$$d_c = \frac{0.822 \times 87.545}{2} = 35.98mm$$

$d_{sc} = 44mm$

$d_{st} = 376mm$

$d_p = 220mm$ *(ie. depth to the prestress force)*

Calculate M_u

$$M_u = T_p d_p + T_s d_{st} - C_c d_c - C_s d_{sc}$$

$$M_u = \left(250 \times 10^3 \times 220\right) + \left(300 \times 10^3 \times 376\right) - \left(430620.5 \times 35.98\right) - \left(119376.3 \times 44\right)$$

$$M_u = 147.05kNm$$

Calculate the ultimate load to induce M_u from the equation set out above in section 3.3.1.

$$P_u = \frac{147.05 \times 10^6 \times 1920}{1335 \times 585} = 361.52kN$$

Note that this load is substantially larger than the load that was calculated above in section 3.3.1. This increase in strength is due to the effects of the external post-tensioning. Theoretically, by applying the post-tensioning system, in increase in strength of around 35% can be expected. Additionally, note that P_u is less than the capacity of the testing equipment that is to be utilised.

3.4.2. Shear Capacity

Once external post-tensioning is applied to a member, flexure-shear cracking becomes evident. The crack is initiated by flexure, however as the load is increased, the crack begins to turn towards the load point and shear cracking becomes evident. An estimate of the load required to produce the shear crack can be obtained by determining the load at which the flexural crack occurs and adding to it an estimate of the load required to turn the crack. There is no reliable way of determining the load that is required to turn the crack, however a simplified approach has been adopted in AS3600. The equation for V_{uc} given in the Standards is:

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o \left[\frac{\left(A_{st} + A_{pt}\right) f_c'}{b_v d_o} \right]^{\frac{1}{3}} + V_{dec} + P_v$$

V_{dec} is the shear force at which a state of decompression occurs and can be calculated by:

$$V_{dec} = \frac{M_{dec}}{\left(\frac{M*}{V*}\right)}$$

Where the decompression moment M_{dec} is calculated by:

$$M_{dec} = \left[\frac{P}{A_g} + \frac{Pe}{I_g} y_b \right] \frac{I_g}{y_b}$$

In the case off the research specimens, the eccentricity of the prestressing force is equal to zero, therefore the equation can be simplified to:

$$M_{dec} = \frac{P}{A_g} . \frac{I_g}{y_b}$$

Where:

$$I_g = \frac{bd^3}{12} = \frac{220 \times 420^3}{12} = 1.358 \times 10^9 mm^4$$

Therefore:

$$M_{dec} = \frac{250 \times 10^3}{220 \times 420} \times \frac{1.358 \times 10^9}{210} = 17.50 kNm$$

Calculate V_{dec} using the moment and shear capacities determined in sections 3.3.1 and 3.3.2.

$$V_{dec} = \frac{17.50 \times 10^6}{\left(\dfrac{105.40 \times 10^6}{104.48 \times 10^3}\right)} = 17.35kN$$

P_v is used to account for the vertical component of the prestressing, however in this case $P_v = 0$ as the profile of the tendons is horizontal.
Calculate the shear strength contributed by the concrete:

$$V_{uc} = 1.346 \times 1.0 \times 1.904 \times 220 \times 376 \times \left(\frac{(600 + 531) \times 32}{220 \times 376}\right)^{\frac{1}{3}} + 17.35 \times 10^3$$

$$V_{uc} = 178.29kN$$

From above, the shear strength contributed by the reinforcement (V_{us}) is:

For $\theta_v = 30^o$:

$$V_{us} = 36.05kN$$

For $\theta_v = 45^o$:

$$V_{us} = 20.81kN$$

Calculate V_u:

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

For $\theta_v = 30^o$:

$$V_u = 178.29 + 36.05 = 214.34kN$$

For $\theta_v = 45^o$:

$$V_u = 178.29 + 20.81 = 199.10kN$$

Again observe that the ultimate shear capacity of the prestress section is less than the load to induce the ultimate moment. This will ensure that even after prestressing, the specimen should still fail in shear. Based on the theoretical analysis, an increase in the order of 35% can be expected in the shear capacity of the section after prestressing is applied.

3.5. Second Moment of Area

The second moment of area (I) of a section represents the sections stiffness. Therefore it is essential when modelling a section, the I value of both the actual and model sections are similar. The calculations below show the analysis undertaken to determine the (I) values for the Tenthill Creek Bridge headstocks and also the model headstocks.

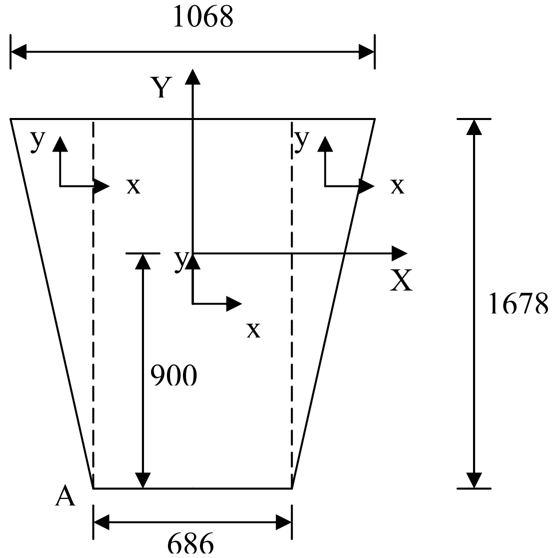


Figure 3.5: *Second Moment of Area Analysis*

Figure 3.5 above portrays the section of the Tenthill Creek Bridge Headstocks. The centroid of the section is marked by the principal axes X-Y in Figure 3.5. The centroid was found by dividing the section into simple shapes and deriving the point from these shapes using the equation:

$$\bar{y} = \frac{\sum (A_i x_i)}{\left(\sum A_i\right)}$$

Note the section is symmetrical about the Y-axis therefore calculations were not required to find the location of the centroid in the X direction.

The working for this process can be seen in Appendix D.

The second moment of area (I) can now be determined about the principal axes using the equation:

$$I_{xx} = \sum \left(I_{xi} + A_i y_i^{\ 2}\right)$$

Note the I-value about the X-axis is only required as this will provide enough information to compare the actual and model sections.

The working for determining I_{xx} can be seen in Appendix D.

I_{xx} for the bridge section:

$$I_{xx} = 3.393 \times 10^{11} mm^4$$

Calculate I_{xx} for the model section:

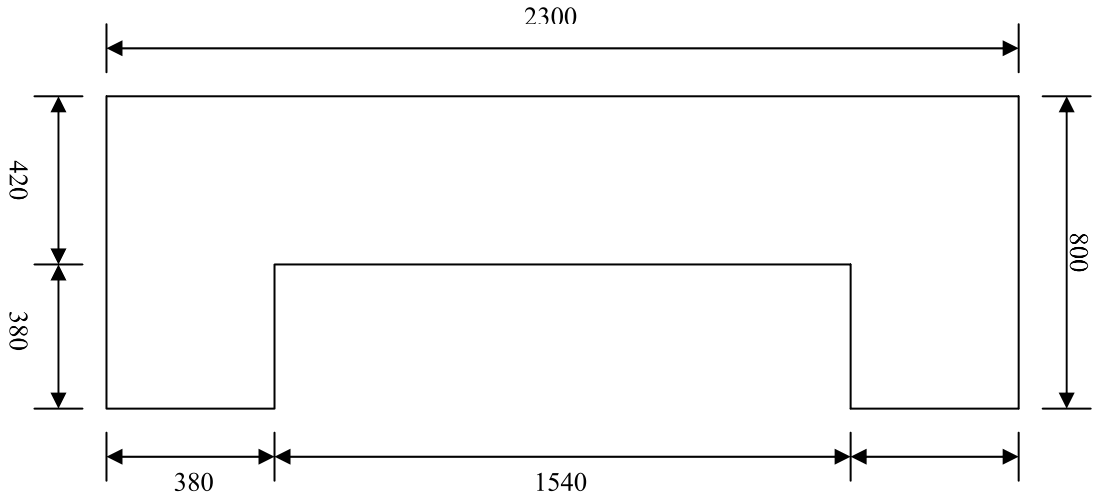
$$I_{xx} = \frac{bd^3}{12} = \frac{220 \times 420^3}{12} = 1.358 \times 10^9 mm^4$$

A scale factor of 1:250 needs to be applied to the model I-value to compare it with the actual I-value.

$$I_{xx-adj} = \left(1.358 \times 10^9\right) \times 250 = 3.395 \times 10^{11} mm^4$$

From this adjusted I-value, it is evident that by modelling the member as a rectangular section, the stiffness will only alter by a minor amount. Therefore the results obtain from the model testing are comparable to the actual bridge headstocks.

3.6. Design Summary



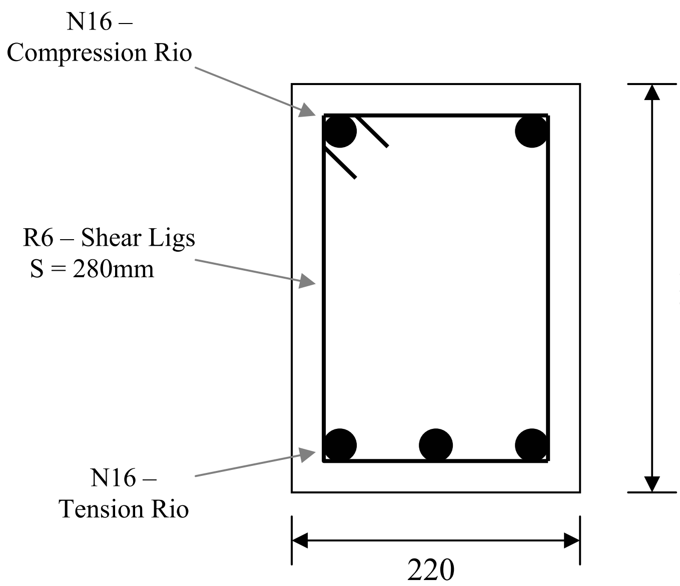


Figure 3.6: *Specimen Design*

Figure 3.6 above shows the design that was used to construct the three specimen to carry out the research. The theoretical strengths of the specimens before and after prestressing according to AS3600 are shown in Table 3.1 below.

Table 3.1: *Summary of Design Capacities*

Strength Property	*Strength Before Prestressing (kN)*	*Strength After Prestressing (kN)*	*Theoretical Strength Gain (%)*
Flexural	259	362	40
Shear	151 - 166	199 - 214	30.5

Note from these values that before and after prestressing, the shear strength is much lower than the flexural strength. The section has been designed in this manner as the objective of the research is top examine shear strengths. Additionally, the maximum strengths are all below the theoretical capacity of the testing equipment. In this chapter the stiffness of the bridge headstock has been examined against that of the model. The results are shown below. It can be seen that the stiffness of the model is almost equal to that of the bridge.

$I_{xx\text{-}bridge} = 3.393 \times 10^{11} mm^4$

$I_{xx\text{-}model} = 1.358 \times 10^{9} mm^4$

$I_{xx\text{-}adj} = 3.395 \times 10^{11} mm^4$

Notice that $I_{xx\text{-}bridge}$ and $I_{xx\text{-}adj}$ are similar, this shows that by modelling the headstocks with a rectangular section have little effects on the overall section properties.

From this design, the specimens can be constructed and tested. The procedure undertaken to carry out this process will discussed in the next chapter.

CHAPTER 4

CONSTRUCTION & TESTING

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.

4.2. Construction Methodology

4.2.1. Introduction

It was decided that three test specimens would be used to undertake the research. These specimens were constructed using the design discussed in the previous chapter. The specimens were then left to cure for at least 28 days to reach an optimum strength. The following sections outline the processes involved in constructing the model headstocks.

4.2.2. Formwork

The formwork was constructed by the university's lab staff from 12mm form-ply, obtained from local suppliers. It had to be designed to ensure that it was capable of construction a minimum of five specimens. Consideration was also required to ensure that the specimens could easily be removed without totally stripping the mould. It was decided that the specimens would be poured on their side and they would be removed from the top. This meant that the piers extensions incorporated in the specimen would need to be removed without stripping the formwork around them. For this reason, the sides of the formwork were slightly tapered towards the base of the pier to ensure that the entire specimen could be dragged out. The tapper was kept to a minimum to ensure that it did not interfere with the post-tensioning anchorages that were used during testing. The formwork was also greased before each specimen was poured to ensure that the concrete did not bond to the mould. Figure 4.1 below shows the formwork.

Figure 4.1: *Formwork Used to Construct Specimens*

4.2.3. Reinforcing

The reinforcing used in the specimens was obtained from a local supplier. It was ordered from the design shown in Appendix E – *Reinforcing Layout.* All of the bars were provided with at least 20mm of cover to ensure that the reinforcing did not break out of the specimen during loading. The reinforcing was delivered bent to shape and cut to length. This included the two U-shape bars positioned at the top of the specimen and the fifteen shear ligatures that were used in each of the specimens. This was done to decrease the time required for construction. The reinforcing cages were then tied using reinforcing ties. It was necessary to slightly modify the reinforcing layout when it was discovered when the three tensile bars were to be position in the cage. The two outside bars butted into the U-shape bar, which meant they could not be tied together. The problem was overcome by setting the cage in place and welding the two together. In the future however, the tensile bars could be bent with a small leg at either end so that they can be tied in. Figure 4.2 below shows a tied cage ready to be positioned in the formwork. The cage was supported with 20mm bar stools or chairs in the formwork. This provided the required cover on both sides of the cage. Once the cages were positioned the concrete could be poured.

Figure 4.2: *Reinforcing Cage Used in Specimens*

4.2.4. Strain Gauges

4.2.4.1. Steel Gauges

The steel strain gauges were used to obtain strain data from the reinforcing, hence they had to be fixed to the cages before the concrete was poured. The gauges used were obtained from TML, a Japanese company developing strain measurement products. The gauges used on the reinforcing were FLA-2-11, which are specially designed for use on metal, glass and ceramic. The gauges are 2mm long and 1.5mm wide and have a resistance of 120Ω. They are capable of operating between -20 and 80ºC. The gauges are constructed with an epoxy backing which is approximately 0.005mm thick. An adhesive known as CN Adhesive, also developed by TML, was used to bond the gauges to the reinforcing. CN Adhesive is similar to super glue, in that it cures in around one minute under finger pressure. Once the gauge is bonded to the reinforcing, wax was melted over it and then it was covered with a waterproof tape, known as VM Tape. This is carried out to ensure that the gauges are not affected during the pouring of the concrete. Once this is finished the gauges were tested to determine if they were functioning properly using a multimeter. If the gauge reads approximately 120Ω of resistance, then it is operation correctly. Data sheets attaining to these products can be seen in Appendix F. Below in Figure 4.3, an example of the steel strain gauges can be seen.

Figure 4.3: *Steel Strain Gauge (FLA-2-11)*

4.2.4.2. Concrete Gauges

The strains on the face of the specimen during testing were recorded using PFL-30-11 strain gauges. These gauges were fitted after the concrete had cured, just prior to testing. The gauges are also produced by TML and are mainly used on mortar or concrete. The gauges are 30mm long and 2.3mm wide and have a resistance of 120Ω. PFL-30-11's are capable of operating at temperatures similar to that of the steel gauges above. These gauges were bonded using the same adhesive as the steel strain gauges, however the concrete surface had to be smoothed off by sanding it before the gauges could be applied. The concrete gauges were position in a rosette formation, as can be seen in Figure 5.15 in the next chapter. This was done to try and determine the strains in each direction of the concrete. Another concrete gauge was positioned below the load point and was used to analyse the maximum stress in the specimen. Data sheets attaining to these strain gauges can be seen in Appendix F.

4.2.4.3. Obtaining Data

The data measured by the strain gauges was recorded using a data logger. To connect the gauges to the logger, a connection plug had to be soldered to the strain gauge wires. This involved stripping back the two wires that attached to each strain gauge and soldering them to the plugs. Other pins on the plug also had to be bridged so the data logger could function correctly. The soldering was a time consuming process as approximately ten gauges had to be fitted with plugs before each test.

4.2.5. Pouring the Concrete

The concrete used to construct the test specimens was obtained from a local supplier. The properties of the ordered concrete were: 80mm slump, 20mm aggregate and 32MPa strength. The concrete that was delivered however had extremely different properties and these differences can be seen in the results section. Due to the time constraints, the delivered concrete was utilised and the specimens were poured. A concrete vibrator was used to ensure that the concrete was correctly compacted. Care had to be taken to ensure that the wet concrete did not laminate (ie. the aggregate did not sink to the bottom of the specimen) during vibrating. After the concrete was placed, it was screeded and trowelled off to achieve a smooth finish. The concrete was then covered and allowed to cure. During this time, the surface of the specimen was continually moistened, for example every 4 hours, to make sure the specimens did not dry out to quickly and crack during curing. Below in Figure 4.4 is an example of a freshly poured specimen.

Figure 4.4: *Freshly Cast Specimen*

Before the concrete was poured, a slump test was completed on a sample from the batch to examine its quality. The results from these tests can be seen in Chapter 5. From these results, it is easy to see that the concrete that was obtained was of very pour quality. On the day of pouring each specimen, a number of test cylinders were poured; these were later used to determine the strength of the concrete on the day of testing. A set of these cylinders can be seen in Figure 4.5.

Figure 4.5: *Test Cylinders*

4.2.6. Stripping & Moving

The formwork was stripped by removing the front section to expose the very top of the specimen, see Figure 4.6. Once this was done, the specimen could be dragged out. In contrary to theory, when removing the first specimen, it was noticed that a bond had been formed between the concrete and the formwork down the sides. Therefore these sections of the formwork had to be loosened and then the specimens could be removed. Once they were removed, the formwork had to be prepared for the next specimen to be poured. This involved, cleaning off all of the concrete that split over the edges, refitting the removed formwork and greasing the mould. After all of this was completed, the next reinforcing cage could be positioned in the formwork, ready for the concrete to be poured.

The specimens were moved using a forklift obtained from the university. Threaded dowels were set into the concrete in the top of the headstocks. These were used to screw 16mm lifting eyes into, through which a D-shackle could be placed, see Figure 4.6. A chain was then threaded through the shackles at either end of the headstock and around the forks on the forklift. Once this was completed, the specimen could be dragged back out of the formwork and then stood up. Once standing, the forklift was used to manoeuvrer the specimens into an area for storage. The forklift was then later required to transport the specimens to the testing area.

Figure 4.6: *Removing Specimen from Formwork*

At the time that the specimens were stripped, the test cylinders also had to be removed. The two processes had to be undertaken at the same time to ensure that they both cured in the same conditions. This task involved removing the cylinders from their moulds and storing them with the headstocks. After stripping the moulds had to be brushed clean and greased so they could be refilled when the next headstock was poured.

4.2.7. Post-tensioning System

This section will give an overview of the post-tensioning system used to undertake the research. It will incorporate the type of system used and the way in which it was tensioned.

4.2.7.1. Tendons

The tendons used the post-tensioning system consisted of two 26mm threaded bar type tendons. The tensile force was applied to the tendons using a 60 tonne hollow core hydraulic jack. As stated in Chapter 3, the tendons were tensioned to apply a 250kN compressive force to the specimen. To do this, each tendon was stressed to around 125kN. Both tendons had to be stressed at the same time to ensure that the anchorages were not pulled off, however only one jack was available to undertake the stressing. This was overcome by swapping the jack from tendon-to-tendon and applying a small amount of load to each one until the desired force was reached. The load applied to the tendons was measured using load cells that were placed in front of one of the anchorages. These sent data to the data logger, were the force could be read off as it was applied. A spacing plate was constructed for the back of the jack to allow a nut to be positioned between the jack and the anchorage, see Figure 4.7. A number of flat plates were also used at either end of the tendons to support the load. Once the stressing was complete, the specimens were loaded. The safety issues relating to the post-tensioning system will be examined at the conclusion of this chapter.

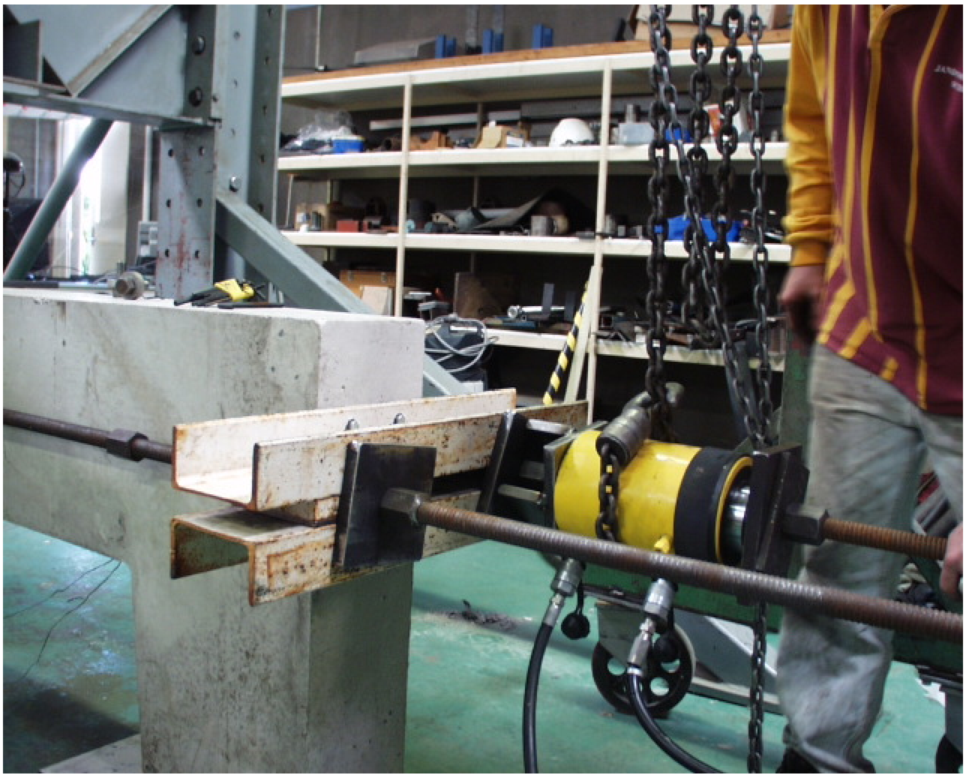


Figure 4.7: *Stressing of Post-tensioning Tendons*

4.2.7.2. Anchorages

End anchorages were used to transfer the tensile forces from the tendons to the specimen. They consisted of two 8x150mm C-sections, approximately 450mm long, that were bolted together back-to-back with a 38mm steel plate in between them, as can be seen in Figure 4.7. High strength nuts were used to transfer the tensile force from the bars to the anchorages. The anchorages were supported during stressing by 10mm threaded dowels that were cast into the ends of the specimen (Figure 4.8). The support was provided by placing a 10mm bolt through predrilled holes in the anchorages, then screwing them into these dowels. These anchorages had been used for a number of research applications in the past at the university.

Figure 4.8: *Dowels Used to Support Anchorages*

4.2.8. Epoxy Injection

To carry out the epoxy repair of the cracked specimens, a two part epoxy system developed by Parchem was used. This system involved a two step process, first a two part epoxy crack sealant, Lokfix E, was applied and then the cracks were injected with a second two part epoxy, Nitofill LV, see Appendix G for fact sheets relating to the resins. Both of these processes are discussed below.

All of the Parchem products required to undertake the epoxy injection process are included in Table 4.1.

Table 4.1: *Products Required for Epoxy Injection.*

Parchem Product	*Use For Product*	*Ordering Notes*
Nitofill LV Resin	Epoxy injected to repair crack	450mL Double Resin Cartridge
Lokfix E	Seals crack before injecting Nitofill LV	450mL Double Resin Cartridge
Gun	Double barrel corking gun	Single Item
Static Mixer/Hose	Used to mix two part epoxies	10 per bag
Injection Flanges	Used to inject Nitofill LV	50 per bag
Adaptors	Used to connect mixer to injection flange	10 per bag
Solvent 10	Used to clean equipment	4L tin

4.2.8.1. Lokfix E

Lokfix E is a two part epoxy specially developed for fast and permanent patching repairs to concrete structures. Its main application is for the repair of concrete substances where strength, impermeability and resistance to chemical attack are required. In this case, only the epoxy's impermeability characteristic was required to prevent the Nitofill LV from escaping from the cracks.

The Lokfix E was obtained in a 450mL double cartridge, which was inserted into a double barrel corking gun specially design for Parchem products. A static mixing hose was then attached to the front of the cartridge. This tube mixes the two resins to form a grey epoxy. The epoxy was squirted over the external crack line and was spread using a putty knife. Small 10mm holes were drilled into the crack line on the face of the concrete over which the injection flanges were glued using the Lokfix E. These small holes behind the flanges makes it easier to inject the Nitofill LV. Care had to be taken to ensure that the Lokfix E did not run in behind the flanges and block them off. A short length of tape was used to prevent the flanges from sliding down the face of the concrete. Figure 4.9 below, shows the applied Lokfix E and the injection flanges being held by the tape. After the flanges were fitted, the epoxy was allowed to cure for 3-5 days before the Nitofill LV was injected.

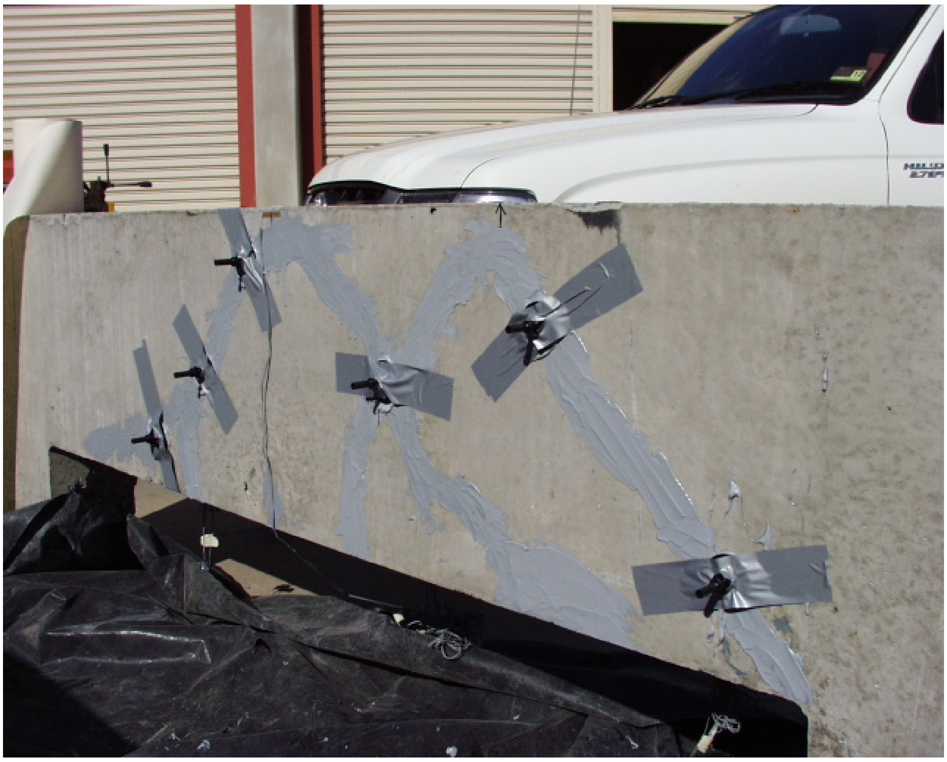


Figure 4.9: *Application of Lokfix E*

4.2.8.2. Nitofill LV

Nitofill LV is an epoxy resin specially designed for injecting cracks in concrete and masonry. The resin has an extremely low viscosity and is capable of filling cracks as small as 0.01mm. Nitofill LV is ideal for small scale, on-site repairs and is also suitable for precast elements. This epoxy resin is capable of restoring a cracked member to its original condition.

The Nitofill LV was obtained in a 450mL double cartridge, similar to the Lokfix E. The resin was injected by attaching a static mixing hose to the front of the cartridge, which was in turn connected to the injection flanges on the specimen using an adaptor. Initially the system was connected to one of the lower flanges on the specimen. Once connected, the gun was slowly pumped until the resin began to leak from another flange on the crack (Figure 4.10). The leaking flange could then be closed and the injection of the resin was continued. This process was repeated until epoxy had leaked from all of the flanges on the crack. A number of problems were encountered during the injection process. Some of these included flanges that were glued open or closed, small cracks that could not be injected and under high pressure some holes appeared in the Lokfix E. These problems however were easily overcome and they did not cause any major difficulties.

Figure 4.10: *Injection of Nitofill LV*

To test the quality of the epoxy resin, specimen S2 was loaded until it had totally failed, similar to the specimen in Figure 4.11. It was failed to the extent that the two crack faces had slid past each other. After the failure, the beam was repaired using Nitofill LV and then tested again. The epoxy formed an exceptional bond between the crack faces, sufficient enough to produce a new failure crack. This proves that Nitofill LV is capable of rejuvenating members to their original condition.

Figure 4.11: *Specimen After Failure*

4.2.9. Summary

The above information provides a brief overview of the construction methodology that was undertaken to complete this research. The next section will outline the testing procedure that was carried out.

4.3. Testing Methodology

4.3.1. Introduction

It was noted before that three test specimens were used to carry out the research. The specimens were tested as shown in Table 4.2. The following section provides an overview of the work that was undertaken to experimentally test these specimens.

Table 4.2: *Tests carried out on the specimens.*

Specimen	*Preloaded*	*Post-tensioned*	*Epoxy Injected*
S1	Yes	No	No
S2(a)	Yes	Yes	No
S2(b)	Yes	Yes	Yes
S3	Yes	Yes	Yes

4.3.2. Tests Undertaken

The shear rehabilitation techniques tested in this research involved the use of epoxy injection and external post-tensioning. The main focus of the experimental investigation however was to test the effectiveness of post-tensioning under varying shear crack conditions. As stated above, specimen S1 was used as a control specimen to determine the ultimate strength. Specimen S2 was firstly preloaded to simulate the actual bridge condition. The magnitude of the preload was determined by inspection of the forming cracks as the load increased. Once the desired crack condition had developed, post-tensioning was applied to the specimen and it was again loaded. This time the load was increased until the specimen failed. Next epoxy was injected into the failed member and after curing the specimen was post-tension and loaded until failure occurred. Specimen S3 was tested slightly differently, in that it was preload to form the initial cracks. After preloading, the cracks were repaired using epoxy injection and then the specimen was post-tensioned and loaded until it failed. The significance of this test was that it represented a model of the rehabilitation technique that had been applied to the Tenthill Creek Bridge. The results from all these tests can be seen in Chapter 5.

4.3.3. Test Configuration

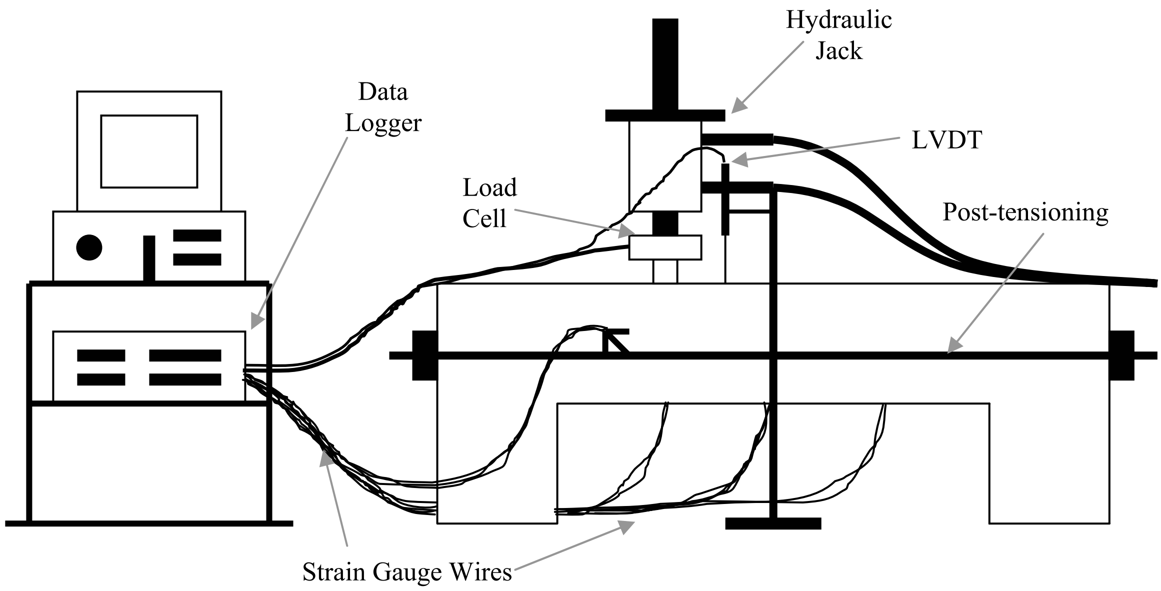


Figure 4.12: *Test Configuration Used to Load the Specimens*

The loading was conducted in a three point load configuration at ⅓ of the distance along the specimen. This distance was chosen as it is the location of one of the girders on the bridge, see Appendix C - *Tenthill Creek Bridge Plans*. Since the testing equipment did not have the capacity to undertake four point loading, a three point configuration positioned off centre was used to simulate the conditions, see Figure 4.12.

Note from the diagram, the position of the Load Variable Displacement Transducer (LVDT) has been moved to the right to simplify the picture. Under actual test conditions, the LVDT was positioned at the load point to ensure the maximum deflection was measured.

4.3.4. Testing Equipment

Figure 4.12 shows a majority of the equipment that was required to carry out the testing. The major equipment used included, a loading frame, a hydraulic jack and a data logger. The loading frame was required to apply a load to the specimens. An overhead frame located in the laboratory at the university was used to carry out the testing. Figure 4.12 illustrates the hydraulic jack attached to the loading frame. The jack used to undertake the test was a 60 tonne hollow core hydraulic jack. This jack was chosen as it provided a sufficient capacity to load the specimens and as it has a hollow core, it could also be used to apply the post-tensioning. Two brackets were constructed to attach to the jack. The first was used to bolt the jack to the load frame. This ensured that the jack was protected during loading. The other bracket was used to undertake the prestressing, as was mentioned in Section 4.7.1 and can be seen in Figure 4.7. The use of the data logger will be discussed in the next section. Along with this equipment, other specialised tools were required and a list of these can be seen in Appendix I – *Resource Analysis.*

4.3.5. Data Logging

The data from the tests was collected using a System 5000 Data Logger. The system was capable of supporting fifteen inputs, which consisted of the LVDT, load cells and strain gauges. The LVDT was positioned at the load point to determine the maximum deflection of the specimen during testing. A maximum of three load cells were used during test, one was positioned at the load point to register the applied load. The other two were used on the post-tensioning bars to measure the tensile forces applied to them. These were continually monitored during the loading of the specimen to determine the increase in post-tension force due to deflection. It was noted above that strain gauges were placed on the reinforcement and on the concrete. The dagger logger was used to record the strains measured by these gauges. The System 5000 limited the amount of data that could be recorded during testing, as only 15 ports could be utilised. As such, care had to be taken to use these ports effectively to maximise to data collection.

4.3.6. Setting Up

The setting up of the test specimen was undertaken prior to loading. To begin setting up the specimen, initially the plugs to connect the strain gauges to the data logger had to be soldered on. After this was complete, the specimen was moved into place. The location of the specimen under the loading frame was determined by lowering a plumb-bob to find the exact centre of the jack on the floor. Once this point was located, the position of the specimen could be marked by measuring in either direction from the point. The specimen was then moved in and located over the marks. After positioning the specimen, the strain gauges were connected to the data logger, the jack was bolted to the loading frame and the LVDT and the load cells were positioned ready for loading to begin.

More work was involved in setting up specimens S2 and S3, as they had to be rehabilitated before they were loaded. For the tests that involved epoxy injection, once all of the resins had cured the specimens had to be prepared for testing. This preparation involved removing the injection flanges and also removing the Lokfix E so that new cracks could be located during testing. The injection flanges were simply knocked off using a hammer and the Lokfix E was removed using an angle grinder (Figure 4.13). When the specimens required post-tensioning, anchorages had to be bolted to the ends of the beams, the bars were positioned through the anchorages and the load cells were positioned on the bars. Once the load cells were calibrated, the anchorage nuts were screwed on and the system was stressed. As noted above, the tendons were stressed equally until the desired load was reached. Once all of the rehabilitation was complete, the specimen could be loaded.

Figure 4.13: *Removal of Lokfix E Before Testing*

4.3.7. Loading

Loading the specimen at a controlled rate proved to be challenging, as the oil was supplied to the jack via a hand pump (Figure 4.14). Therefore the person pumping the jack had difficulties in providing a constant supply of oil. This did not however cause any major problems, as each specimen was loaded at a steady pace to ensure that no sudden increase in load was applied.

Figure 4.14: *Hydraulic Jack Used During Loading*

4.3.8. Material Testing

4.3.8.1. Concrete Tests

It was mentioned before that during casting a number of concrete test cylinders were poured. Approximately seven 100 x 200mm cylinders were filled. These cylinders were allowed to cure in exactly the same conditions as the specimens. On the day of testing several small cylinders were tested to determine the compressive strength of the concrete (Figure 4.15). The results from these tests can be seen in Chapter 5.

Figure 4.15: *Concrete Compressive Test*

4.3.8.2. Reinforcing Tests

During the testing, a sample of the N16 rebar and the R6 bar was subjected to a tensile test. This was undertaken to determine the yield strength of the reinforcing used in the model specimens. Both bar types were tested until they failed (note the R6 was tested a number of times and the results were averaged). The results from these tests can be seen in Chapter 5.

4.4. Safety

This research was conducted with the aid of experimental work, therefore a number of safety issues needed to be addressed. The issues can be divided into three sections safety relating to construction, external post-tensioning and epoxy injection. Each have been examined in the sections below.

4.4.1. Construction

Wet concrete is a hazardous material, it can burn the skin and cause respiratory problems. For these reasons, before construction commenced, workers were briefed on the safety issues relating to the concrete. Hence during pouring care was taken to ensure that only people briefed in concrete safety were in the vicinity of the construction. Personal protective equipment had to be worn to ensure that the concrete did not burn skin. If concrete happened to splash onto exposed skin, it had to be removed immediately. During the movement of the specimens care was taken to ensure that everyone remained well clear whilst the specimen were raised and moved. Personal protective equipment was also worn during the process (for example safety boots). Once these simple steps were followed, no safety issues were uncovered. Other safety issues relating to the construction of the specimens can be seen in Appendix H.

4.4.2. External Post-tensioning

The tendons used in the external post-tensioning system were loaded with high forces, therefore a number of safety issues surrounding the system had to be addressed. Before stressing nuts were placed inside the anchorages to ensure that if a tendon failed it could not 'shoot out'. Once the tendons were stressed, care was taken to ensure that nobody moved in front of a tendon in case failure occurred. Everybody within the vicinity of the experimental work was briefed before testing commenced on the dangers of prestressing. Due that the high stresses induced in the system, a tendon is capable of killing a person if failure was to occur. Therefore once the system was stressed it was treated with extreme care.

4.4.3. Epoxy Injection

Epoxy resins are toxic products, therefore care must be taken when utilising them. Care needs to be taken to ensure that the vapours are not inhaled, the resins are not swallowed and they do not come in contact with the skin or eyes. Therefore before the injection process commenced, all workers were briefed on the safety issues surrounding the products and everyone was supplied with the MSDS – Material Safety Data Sheets. The MSDS supplied can be seen at the Parchem website. Extreme care was taken during the injection process and no problems were encountered.

All of these issues were examined and management procedures were implemented. A table clearly presenting all of the safety information can be seen in Appendix H – *Risk Assessment and Safety Issues.*

CHAPTER 5

RESULTS & DISCUSSION

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

Prior to each specimens being poured, a slump test was conducted on the mix to gain some indication of the wet concrete's properties. The results from each batch of concrete are provided in Table 5.1.

Table 5.1: *Concrete Slumps*

Concrete Batch No.	*Slump (mm)*	*Average Slump (mm)*
1	145	130
	125	
2	125	122
	118	
3	173	169
	165	

Within the construction section, it has been noted that the concrete had been ordered with a slump of 80mm and a strength of 32MPa. From slump results (Table 3.1) 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 significantly decreased due to the high moisture content.

5.2.2. Concrete Compressive Strengths

It was mentioned earlier that on the day of testing 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 5.2). Through examining the f'_c from each test in the table, it becomes obvious that only very marginal increase in concrete strength, throughout testing, has occurred. This can be explained, as all of the testing was conducted after 28 days of curing, therefore the specimens had almost reached their optimum strength. The gain in concrete strength after 28 days is small, hence the marginal increase in strength during testing.

Table 5.2: *Concrete Compressive Strengths*

Date of Testing	*Concrete Batch No.*	*Cylinder Sample No.*	*Average Dia (mm)*	*Compressive Strength (kN)*	f'_c *(MPa)*	*Average* f'_c *(MPa)*
10.8.04	1	1	101.25	185	23.0	22.2
		2	101.1	175	21.8	
		3	101.5	175	21.8	
12.8.04	2	1	100.1	145	18.4	18.8
		2	99.8	150	19.0	
		3	100.5	143	18.1	
10.9.04	2	4	100.2	152	19.3	
		5	99.7	148	18.8	
		6	101.4	155	19.4	
19.8.04	3	1	100.0	180	22.9	27.0
		2	100.6	230	28.9	
		3	100.3	200	25.3	
6.9.04	3	4	101.5	243	30.0	
		5	101.3	225	28.0	

The concrete was ordered with a strength of 32MPa, however due to the high slumps, the actual strength of the concrete was dramatically reduced (Table 5.2). Normally it's expected that as the moisture content increases, the strength decreases. In this case however, by examining Tables 5.1 and 5.2, it is evident that as the moisture content is increased (slump is increased), so too is the compressive strength. This may be caused by slow curing conditions or superior compaction of the concrete. These are board assumptions, as only three batches of concrete have been used. Consequently, the extent of the results is limited and difficult to draw any conclusions from.

For the theoretical analysis in Chapter 3, a concrete strength of 32MPa was used. This is above all of the experimental concrete strengths, therefore the strength of the specimens may slightly decrease, due to these differences. This will be examined towards the end of the chapter.

5.2.3. Reinforcing Strengths

Tensile tests were undertaken on samples of the two types of reinforcing (N16 and R6) used in the specimens. As a result of these tests the yield stress for both samples could be determined and compared against those used in the analytical analysis. The graphs and working relating to this analysis can be seen in Appendix F, however Table 5.3 below summarises the results.

Table 5.3: *Steel Tensile Tests*

Test Specimen	***Actual Diameter (mm)***	***Yield Load (kN)***	***Yield Stress (MPa)***	***Theoretical Yield Stress (MPa)***
R6	6.15	13.8	440	250
	6.56	14.1		
N16	16.0	119.0	592	500

These results show that the yield stresses of 250MPa and 500MPa used in the analytical analysis in Chapter 3 are somewhat conservative. Due to these results, the actual strength of the specimens should be increased, when compared to the theoretical analysis

From this testing it is also interesting to note the failure strain of the reinforcing. This can then be compared against the strains measured in the specimen during testing. The failure strains have been summarised below in Table 5.4.

Table 5.4: *Strain Capacity of Reinforcing*

Test Specimen	Maximum Load (kN)	Failure Strain (Micro Strain)
R6	17.1	2670
	16.8	
N16	134.0	3335

Subsequently it can be assumed that if the stains produced in the reinforcing steels become larger than those in Table 5.4, failure will occur.

5.3. Crack Analysis

Whilst loading each of the specimens, the crack pattern was observed to analyse the failure mode that was occurring. The following sections present these observations for each of the specimens.

5.3.1. Cracking in S1

The crack pattern observed in the control headstock was surprising. Initially it was thought that a shear crack would form on the shorter shear span of the specimen and as the load increased, the member would fail along this crack (Figure 5.1). As the load was applied, the crack did appear in the short shear span, however as the load was increased, a new crack developed in the long shear span and the specimen and then continued on to fail. This failure mode could be explained by what is known as a compression strut forming in the short shear span. The span-to-depth ratio in this span is small, therefore once the initial crack forms, this span goes into compression. The crushing stress of the concrete is larger than the stress required to shear the member over the long shear span. This is because a tensile force is generated in the longer shear span of the member and as the tensile strength of concrete is small it is inevitable that the section will fail. Hence the weaker failure mode becomes evident and the member shears. From these findings, it could be inferred that if the load was increased on the Tenthill Creek Bridge, the headstocks would fail in a similar manner.

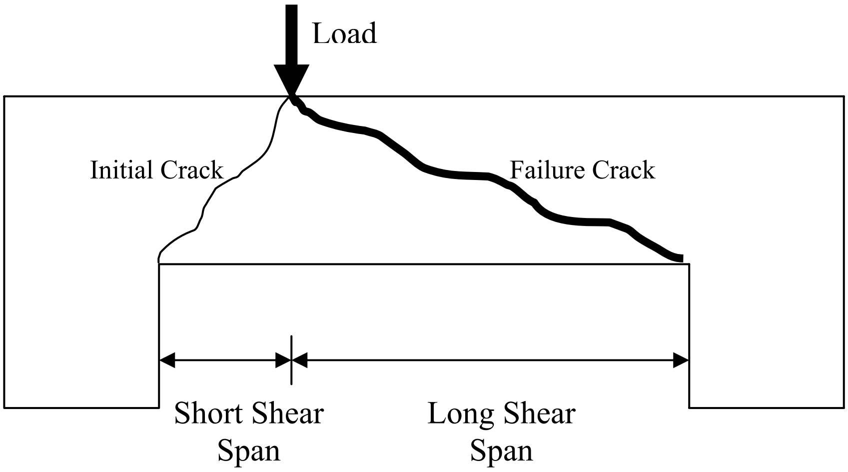


Figure 5.1: *Observed Crack Patterns*

Figures 5.2(a) and 5.2(b) show the cracks that formed on the control specimen as it was loaded. Figure 5.2(a) shows the initial crack that formed in the short shear span. It began to propagate from the inside of the support when the load reached around 125kN. The crack continued to form until it reached the load point. For this to occur, the load had to be increased to approximately 215kN. At this point the crack was approximately 0.5mm wide (Figure 5.2(a)). As the load was increased, this crack became 'stagnant' and no longer opened, this explains the development of the compressive strut. Notice, this crack is similar to that observed in the bridge.

Figure 5.2(a): *Initial Crack in S1*

When the load was increased to approximately 230kN, the crack in the longer shear span appeared (Figure 5.2(b)). This crack rapidly opened as the load was increased. When the load reached 366kN, the member failed along this crack plane. The remaining shear crack was approximately 4mm wide at failure.

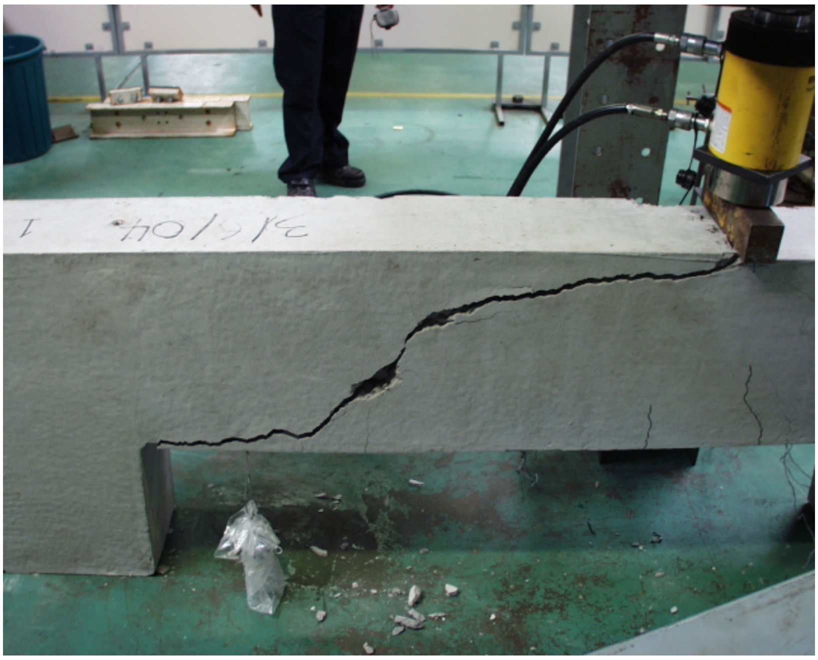


Figure 5.2(b): *Failure Shear Crack in S1*

Small flexural cracks can also be seen marked in Figure5.2(b). These cracks appeared at roughly 80kN of applied load, when the specimen began to deflect. These cracks did not propagate after the initial shear crack appeared.

5.3.2. Cracking in S2

Specimen S2 was loaded on three occasions, it was initially preloaded, then prestressed and loaded and finally epoxy injected, prestressed and loaded again. The cracking that occurred during preloading was very similar to that seen in specimen S1. Since the concrete strength of specimen S2 was less than that of specimen S1, the initial shear crack became evident when the load reached approximately 110kN. Once the preloading was complete, this crack had opened to approximately 0.4mm. Flexural cracking was noticed in the specimen at around 95kN. When the load was increased to about 175kN, the shear crack in the longer span began to rapidly propagate. This load is much less than the one observed in specimen S1 and can again be explained by the difference in concrete strengths (Table 5.2). The load was then increased to 250kN, where the crack opened to approximately 2mm wide. Proceeding this, the specimen was unloaded and the external post-tensioning was applied.

After the specimen was post-tensioned with a 250kN compressive force, the major shear crack in the long span closed to around 0.2mm (Figure 5.3(a)).

Figure 5.3(a): *Shear Crack in S2 after Post-tensioning*

Once post-tensioned, the specimen was again loaded and almost immediately the cracks began to reopen. The major shear crack continually opened during loading until the failure load of 333kN was reached. At this point the major shear crack had opened to around 7mm. This failure crack appeared to be very similar to the one generated in specimen S1 (Figure 5.2(b)). It was also interesting to note that whilst loading, the crack in the short shear span also reopened. Once loading had ceased, the width of this crack was in the vicinity of 0.9mm wide. Most of this deformation occurred shortly after loading commenced, then the compression strut became evident and the change in the crack after this point was minor.

After the specimen had failed, the crack was injected with epoxy resin, post-tensioned and tested again. During this test an entirely new failure mode was discovered. Once loading had commenced, it was found that the specimen was now much stronger. Initial flexural cracks became evident at around 200kN, however they did not propagate far. When the load reached 260kN, the initial crack in the short shear span appeared. In this test however, as the load was increased, the crack continued to gradually open. The shear in the longer span did not develop in this test. As the load was increased to 400kN, the concrete in the short shear span began to crush (Figure 5.3(b)). From this figure, it is evident that the epoxy injected crack in long shear span did not reopen. It can be concluded from this that the injection process reformed the bond between the crack faces.

Figure 5.3(b): *Failure of S2 after Epoxy Injection and Post-tensioning*

5.3.3. Cracking in S3

Specimen S3 was loaded twice, it was initially preloaded to produce the shear cracks which were then epoxy injected. Once the resins were cured, the specimen was post-tensioned and loaded again. The crack pattern that was produced during preloading, was similar to that seen in specimens S1 and S2. Initial flexural cracks became evident during preloading at about 100kN. The initial shear crack then appeared at around 174kN. These loads are larger than that experienced in the previous specimens as the strength of the concrete was greater in specimen S3 (Table 5.2). To produce a major shear crack that was similar to the previous specimens, the load had to be increased to approximately 400kN. The loading was ceased at 423kN and the cracks were injected with epoxy resins.

After the resins had cured, the specimen was post-tensioned and tested again. Similar to specimen S2, the failure mode that occurred was in the short shear span (Figure 5.4). Hence, if a post-tension force was applied to an uncracked specimen, the failure mode would be similar to what has been experienced in the epoxy injected specimens. This can be assumed because by injecting the cracks with epoxy resins, the specimen is effectively restored to a new member. The effect of the post-tensioning force therefore prevents the large tensile forces being formed in the long shear span, and the weak point is then shifted to the failure of the concrete in the shorter shear span.

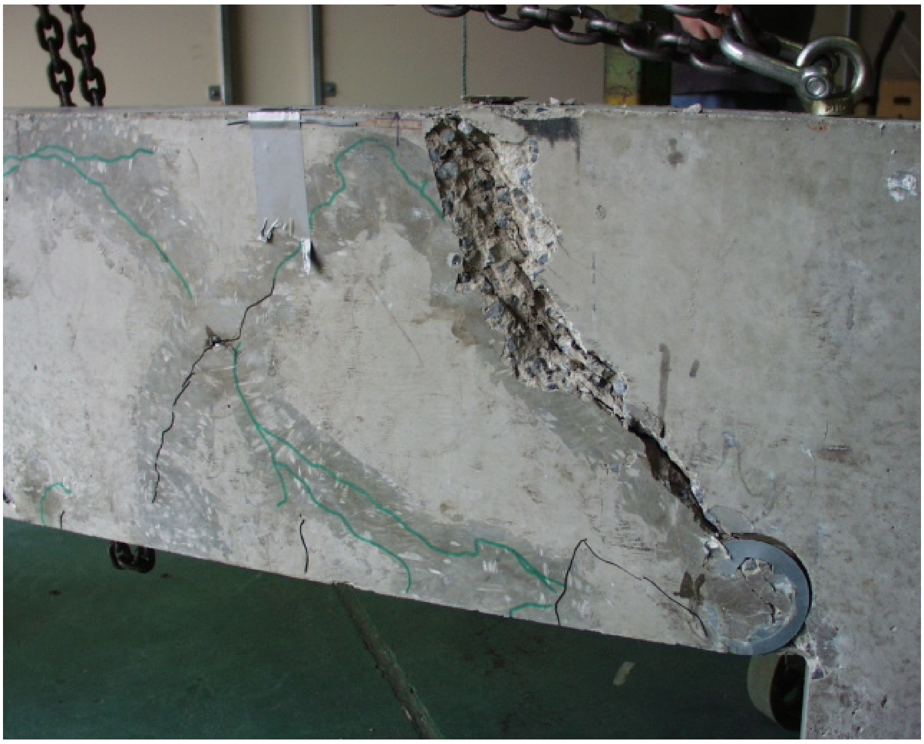


Figure 5.4: *Failure of S3 after Epoxy Injection and Post-tensioning*

5.3.4. Cracking in Supports

During loading, a difficulty was encountered, in that as the load increased, a section of the pier at the end of the long shear span began to fail (Figure 5.5). It is believed that the problem was caused by rotation of the member. As the specimen was loaded and it began to deflect and a small amount of rotation is caused in the pier. This rotation was sufficient enough to cause a stress concentration on the inside of the pier. It is also thought that a small imperfection on the loading floor may have contributed to this failure. Consequently, the support failed. The failure did not interfere with the testing, as each time the section cracked away, the remainder of the support carried the load. This problem is believed to have a minimal effect on the results obtained, however a slight increase in the deflection will have been recorded when the section broke away.

Figure 5.5: *Failure of the Support*

5.4. The Effects of Epoxy Injection

Specimens S2 and S3 were utilised to test the effects of epoxy crack injection. From the experiments conducted, it was discovered that the resins were capable to restoring the specimens to their original condition. In both tests, the cracks that were injected were not able to reform. Figure 5.6 below show specimen S3 after it had been rehabilitated with epoxy resins and external post-tension and tested. In the picture, the green lines represent cracks that have been injected with epoxy resins and the black lines indicate the new cracks that formed during loading. From this it can be seen that none of the injected cracks reformed. It is difficult to see, however on a number of occasions, the shear failure crack actually crossed the initial crack formed during preloading. It can then be inferred that the initial crack had been repaired to its original condition, as a new crack cannot propagate through an existing crack. From all of the experimental analysis conducted throughout this research, it has been found that epoxy crack injection is a quality method of repairing concrete structures.

Figure 5.6: *Effectiveness of Epoxy Injection*

5.5. Load – Deflection Characteristics

From the data that was obtained during testing, a simple load-deflection relationship can be produced for each specimen. Figure 5.7 below shows a plot of the expected load-deflection relationship for the specimens. 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 and is represented by the red circle on the plot below. 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.

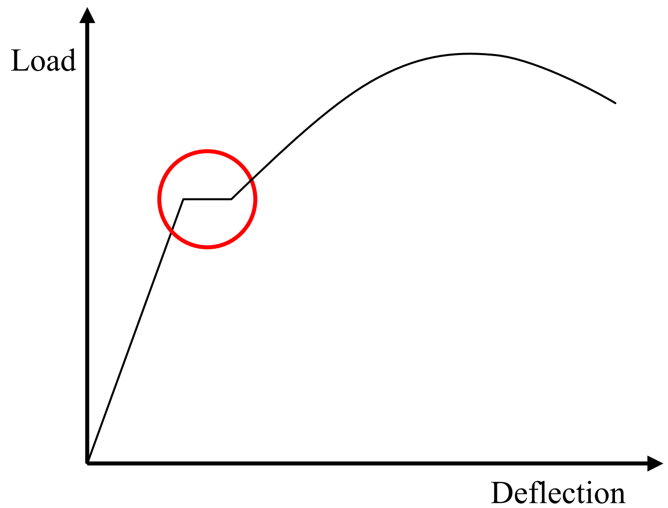


Figure 5.7: *Expected Load vs Deflection Plot for Specimens*

The following sections show the load versus deflection relationships observed during each test.

5.5.1. Specimen S1

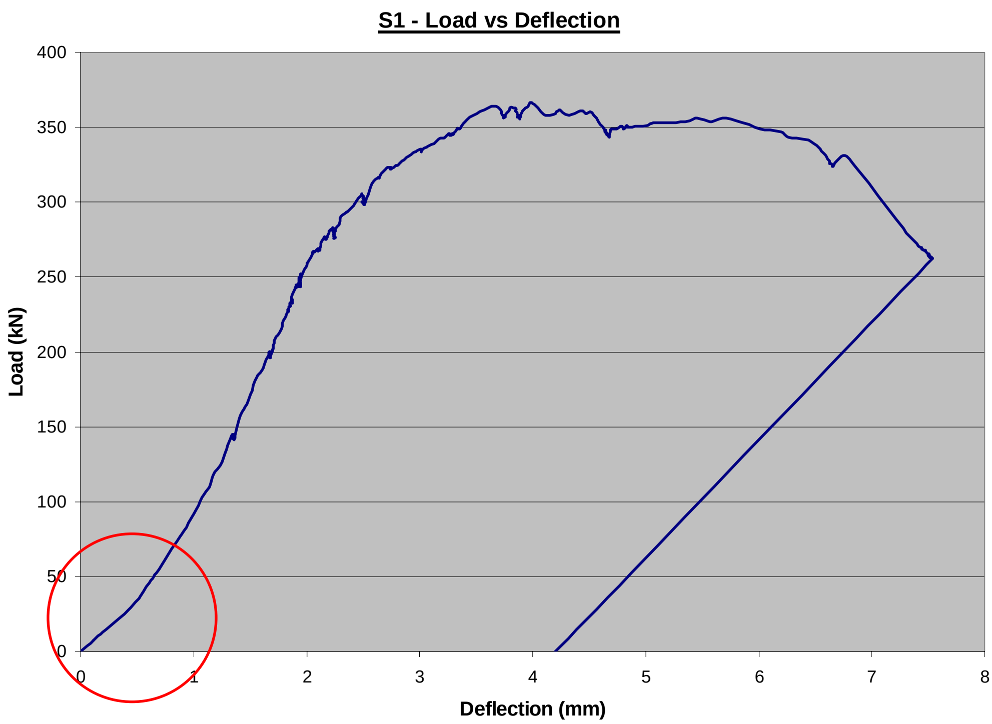


Figure 5.8: *Load –deflection Relationship for S1 (Control Test)*

The plot in Figure 5.8 above represents the load-deflection relationship for specimen S1 (the control headstock). From the plot a linear section can be observed as the loading commences. Small irregularities or dips can be seen in the linear section from around 150kN onwards. These dips represent the opening of cracks as the load is increased. Between 250kN and 300kN, a number of these dips are evident and the slope of the plot begins to decrease. 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 350kN, the specimen continues deform with no increase in load. This implies that the concrete is beginning to fail and as the load is applied the cracks continue, thus no increase in load is recorded. 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 350kN and 5.5mm of deflection. From here the load rapidly drops off as the member deforms.

A seating error is evident in Figure 5.8 when the loading commences, it has been highlighted by the red circle. This error has been caused by the ‘bedding in’ of a number of spacing plates used between the jack and the specimen. The seating of the piers on the loading floor may have also contributed to this error. In subsequent test, a square of fibro sheeting was placed under the piers in an attempt to lessen the initial errors. The linear section in Figure 5.8 should coincide with the zero point as can be seen in Figure 5.7.

5.5.2. Specimen S2

Figure 5.9 displays the load versus deflection plot for specimen S2. From the plot, the three separate tests are visually evident. A considerable increase in strength can be seen as each restoration technique has been applied to the specimen. By comparing the preloading plot from Figures 5.9, with the plot in Figure 5.8, it can be seen that specimen S2 is much weaker than specimen S1. The maximum load recorded during preloading of S2 was 250kN, whereas S1 recorded a maximum load of 366kN. After post-tensioning however, the strength of S2 was recorded at 333kN. Hence a strength increase of 33% was gained by applying the post-tensioning. Once all of the cracks were epoxy injected and the specimen was post-tensioned, a strength increase of 68% to 420kN was achieved over the preloading force. Therefore, it is obvious that after properly repairing the cracks in a member, external post-tensioning will substantially increase the members load carrying capacity.

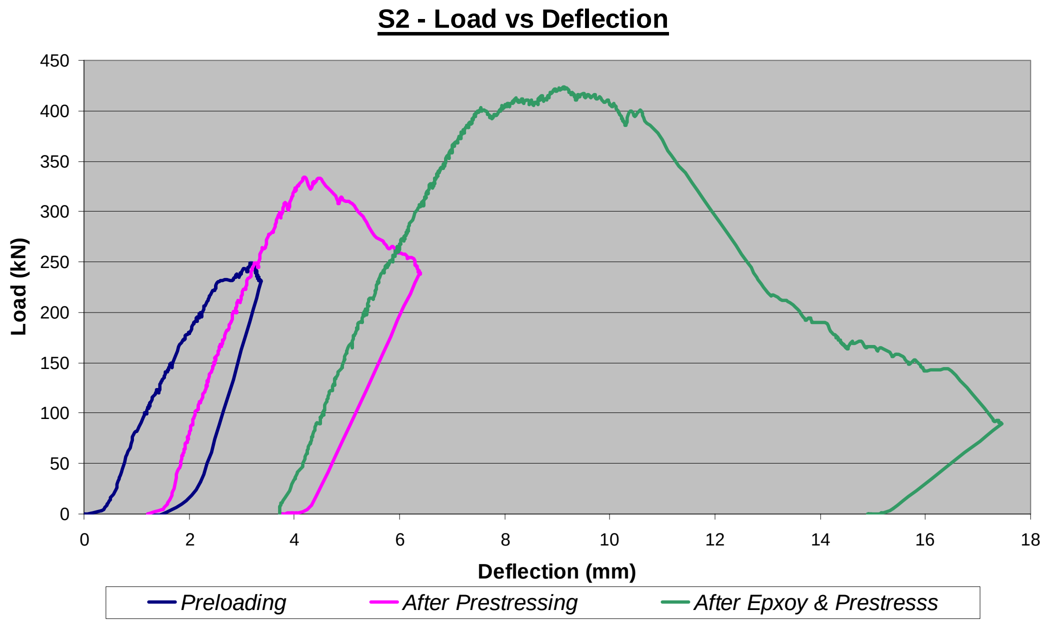


Figure 5.9: *Load- Deflection Relationship for S2*

From the plots of the testing undertaken on S2, it can be seen that after testing is complete a residual deflection is sustained within the specimen. This is caused by cracks that are not able to close properly and also some reinforcing steel may have undergone a small amount of plastic deformation. After the restoration of the member from its previous loading, this residual deflection remains in the specimen. Hence the plots in Figure 5.9 carry on for the previous loadings residual deflection.

By comparing the three linear regions in the plots for specimen S2, slight changes in the stiffness of the member are evident. An increase in the members stiffness can be seen after the application of post-tensioning as would be expected. Conversely, after the member was epoxied and post-tensioned, the stiffness is slightly decreased. This is difficult to explain however, the change is very slight and is almost comparable to the stiffness of the member during preload. Seating errors can again be seen as loading is commenced during preloading and after the application of the post-tensioning. These would again be caused by the 'bedding in' of the loading packers and the seating of the piers.

5.5.3. Specimen S3

The load versus deflection plots for the testing conducted on specimen S3 can be seen below in Figure 5.10. From the plot it can be seen that the concrete used to construct this specimen was stronger than any of the other batches of concrete. For this specimen, the preload had to be increased to 420kN before any major cracks appeared. Once the specimen was injected with epoxies and post-tensioned, a strength increase of approximately 33% was gained to 546kN. This is much less than the increase that was achieved in specimen S2. This could be due to the increased concrete strength of S3 and the decreased stiffness of S2 may have also contributed to this difference. A substantial decrease in load is evident subsequent to the rehabilitated member reaching its peak, this is explained by the failure of one of the shear ligatures passing through the shear crack. The flat section in the plot at 450kN represents the point at which the shear ligature began to yield.

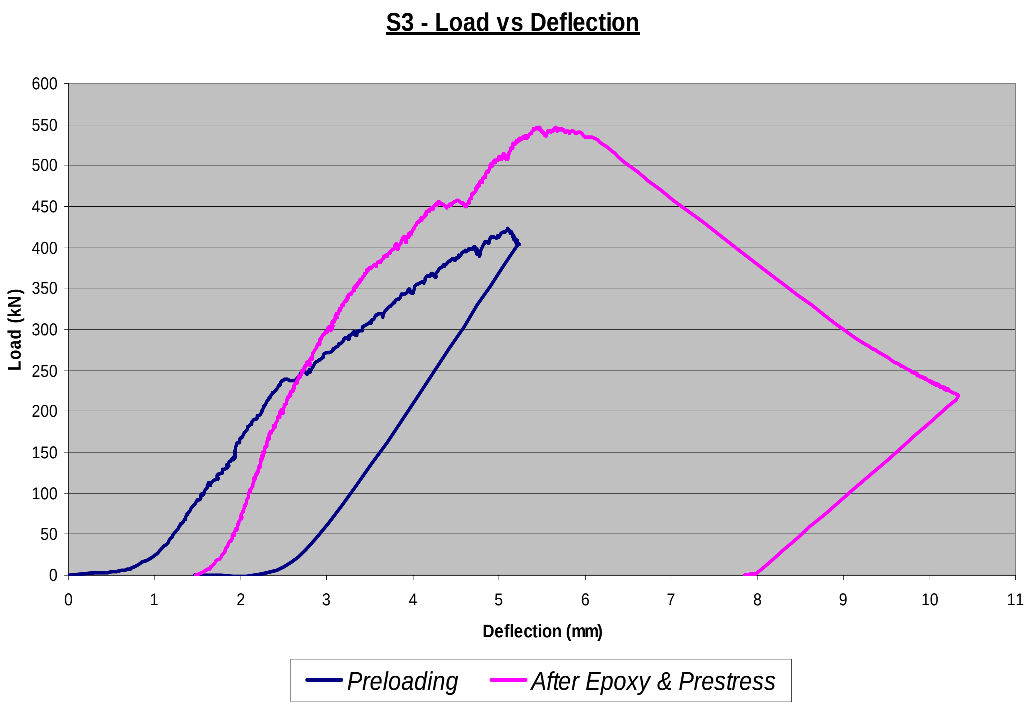


Figure 5.10: *Load- Deflection Relationship for S3*

By comparing the two plots in Figure 5.10, a large increase in stiffness can be seen after the specimen is rehabilitated. This is similar to the difference that would be expected after the application of post-tensioning to a member. It was thought that specimen S2 would exhibit a similar difference in stiffness to that seen in specimen S3. Large errors can again be seen in the preloading of this specimen. These are once more thought to be caused by seating problems in the testing arrangement.

5.5.4. Summary

Through comparing the load-deflection relationship for specimens S2 and S3, an accurate indication of the capable load increase that can be achieved by epoxy injection and external post-tension, becomes evident. From this experimental data it has been discovered that by applying these rehabilitation techniques, a substantial increase in capacity can be achieved.

5.6. Shear Ligatures

5.6.1. Specimen S1

It was noted in Section 5.3 that the major crack plane that specimen S1 failed along passed through the long shear span. When the specimen was setup, the strain gauges on the shear ligatures were positioned in the short shear span, were the specimen was expected to fail. Consequentially, the strains measured by the gauges were small, with a maximum of 805 micro strain. The yield strength of the shear ligatures was approximately 2200 micro strain, therefore the load in the ligatures was dramatically below there capacity. As a result, the ligatures remained in an elastic state and the plot of load vs micro strain exhibits a linear relationship.

5.6.2. Specimen S2

Figure 5.11 displays the plots of the strain in the shear ligatures during the three loading cases of specimen S2. Again, as the shear crack propagated over the long shear span during the preloading and after prestressing, the strains measured in the shear ligatures were small. During these tests, the ligatures remained in an elastic state. Conversely, after the specimen was epoxy injected and post-tensioned, the failure crack passed through the short shear span and much larger strains were recorded in the ligatures. During this test, the strain in the shear ligatures passes the yield strain of 2200 micro strain and continued on to fail as the load is increased (Figure 5.12). According to the material tests undertaken on the reinforcement, the R6 steel should fail at approximately 2670 micro strain. From Figure 5.11, it can be seen that the strain increased beyond this point to 3820 micro strain. This may have been caused by a difference in steel strengths.

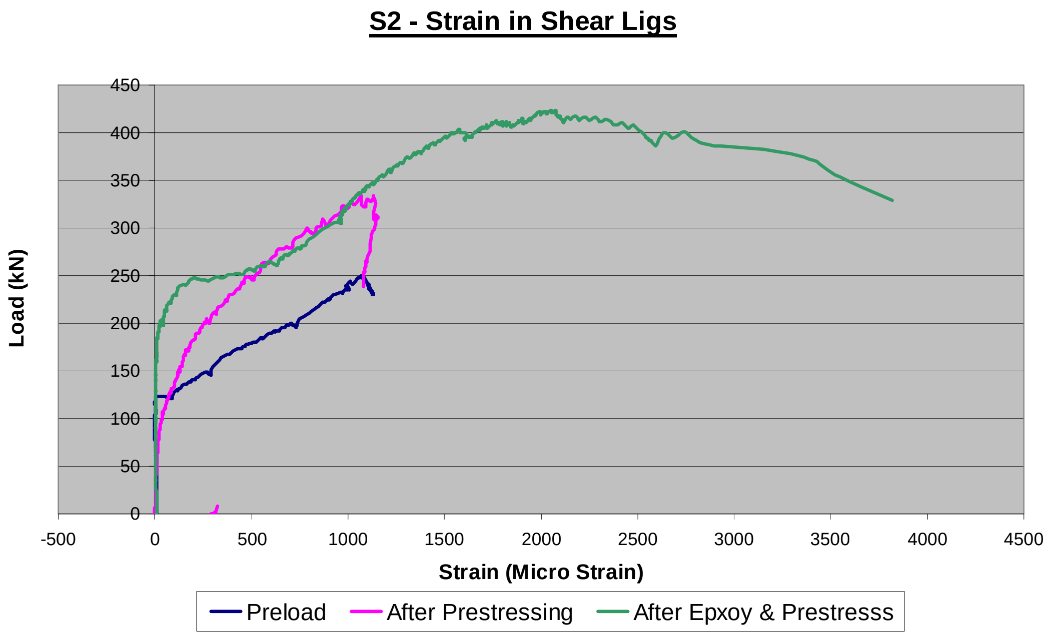


Figure 5.11: *Strain Measured in Shear Ligatures in S2*

Figure 5.12: *Failure of Shear Ligature after Epoxy Injection & Prestressing*

In Figure 5.11, a horizontal section can be seen in the strain plot for the epoxied and post-tensioned test. This section represents the formation of a major crack in the specimen. The crack opens up and transfers the strain to the shear ligatures whilst under a constant load. The formation of the cracks is also represented by the point at which the ligatures begin to load (when the plot leaves the y-axis). It can be seen that during preloading, the initial cracking occurred abruptly, as the plot initially extends from the load axis at 90º. It should be noted, these initial cracks are not the same as the initial cracks mentioned in Section 5.3. These strains only represent the cracks that pass through the shear ligatures.

5.6.3. Specimen S3

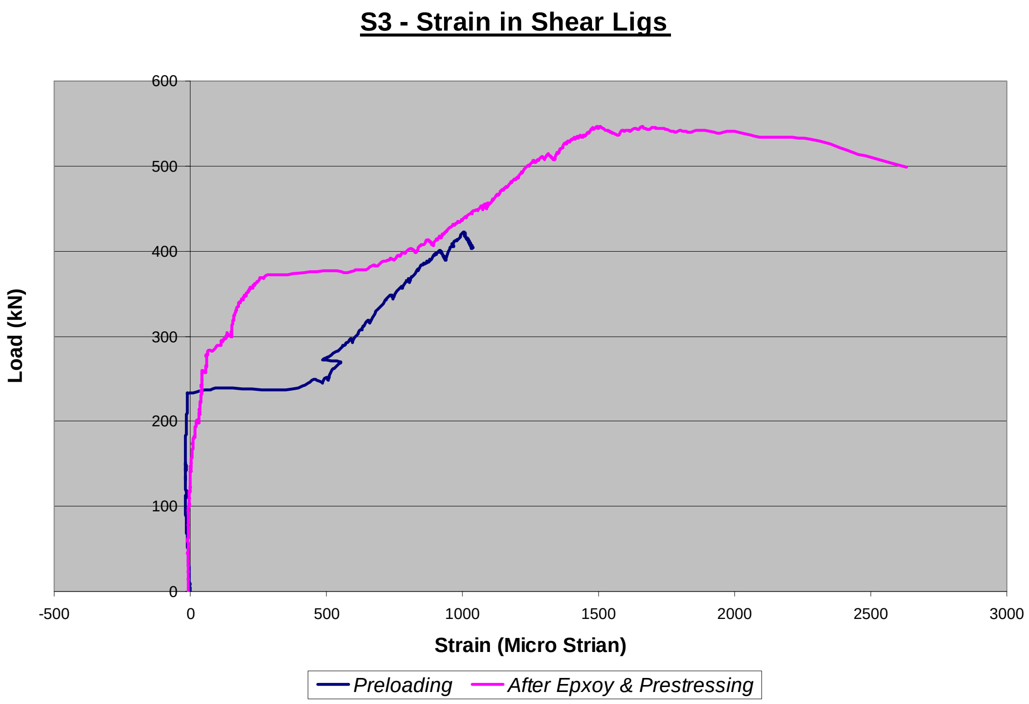


Figure 5.13: *Strain Measured in Shear Ligatures in S3*

Specimen S3 exhibited a similar strain pattern in the shear ligatures as seen in specimen S2. A plot of the stains can be seen in Figure 5.13. As with the previous two specimens, during preloading the strains that were read were small as the failure crack did not pass through the ligatures. When the load reached 250kN during preloading, a crack passing through the ligatures formed abruptly. This is characterised in Figure 5.13 by the large increase in strain in the shear ligatures. Once this crack opened, the compression strut formed and the increase in strain in the ligatures as the loading continued was minimal. The maximum strain in the shear ligatures during preloading was approximately 1032 micro strain, which is far below the yield strength of the steel.

Once the member was rehabilitated, the failure crack passed through the short shear span, similar to specimen S2. From the plot it can be seen that the crack passing through the ligatures formed at a smaller load than that during preloading. Conversely, after rehabilitating, the crack opened much more steadily. This may have been caused by the effects of the prestressing force on the member, hence preventing the crack from opening. The strains produced in the shear ligatures after the specimen was rehabilitated, increased to 2630 micro strain. Consequently, the steel passed its yield point, however it did not fail, as the failure strain is around 2670 micro strain.

5.7. Tensile Reinforcement

The tests specimens were designed to ensure that they failed in shear when loaded. For this reason the flexural reinforcement used in the specimens was over-designed. Due to this over-design, the strains measured in the tensile steel were small. In specimen S1, the maximum strain measured during loading was 1866 micro strain. Consequently, the steel was far below its yield strain of 2960 micro strain. The plot of load vs strain is linear elastic, hence the residual strain the in the steel after loading was almost zero. The following two sections explain the strain that was recorded in the tensile steel during the testing of specimens S2 and S3.

5.7.1. Specimen S2

The strain in the tensile steel was recorded during the three tests undertaken on specimen S2 and can be seen plotted against the applied load in Figure 5.13. From the plot it is evident that by post-tensioning the specimen, a compressive force is induced in the tensile reinforcement. Clearly the steel remained linear elastic during all of the tests. A small horizontal section can be seen in plot of the stains after rehabilitation has occurred, this is marked by the red circle in Figure 5.13. It is assumed that this represents the stage at which the concrete crushed in the short shear span (Figure 5.3(b)), as the sections either side of horizontal part are almost parallel. From the figure, note that that the prestress is applied alone, the plot has a different slope to the other two plots. Obviously there has been some change in the stiffness of the section after applying the prestress. This contradicts theory, in that it would be assumed that after the member is cracked, more strain would be carried by the reinforcement. Hence after prestressing it would be predicted that the plot would have a decreased slope when compared to the preload plot.

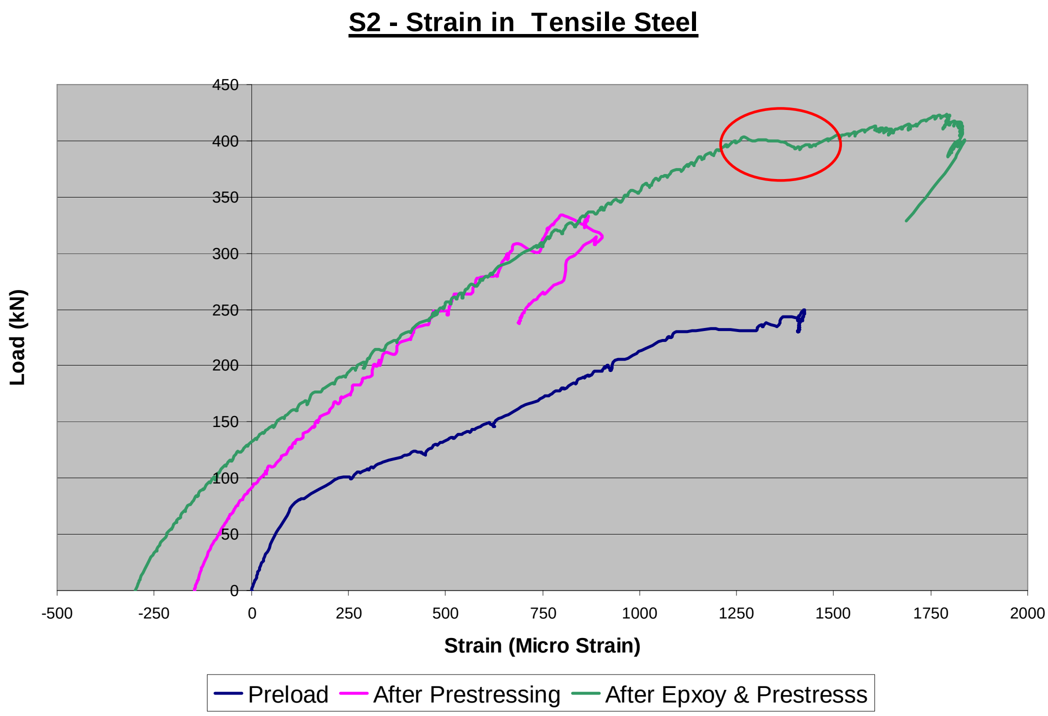


Figure 5.13: *Strain Measured in Tensile Steel of S2*

5.7.2. Specimen S3

Figure 5.14 shows the plots of the measured stain in the tensile steel during the testing of specimen S3. These plots are very similar to what was predicted to occur in the tensile steel. In both loading cases, the steel remained in its linear elastic region. After post-tensioning, some of the tensile strain in the member has been supported of the compressive force. This can be seen in the plot by comparing the strains generated in the preloading with the strain generated in the rehabilitated specimen. Also notice that after post-tensioning, a compressive strain is developed in the steel, similar to specimen S2. Additionally, a substantial change in slope is evident at the commencement of loading in the plot from the rehabilitated member, marked by the red circle in Figure 5.14. This distinguishes the point at which the load was transferred from the concrete to the steel.

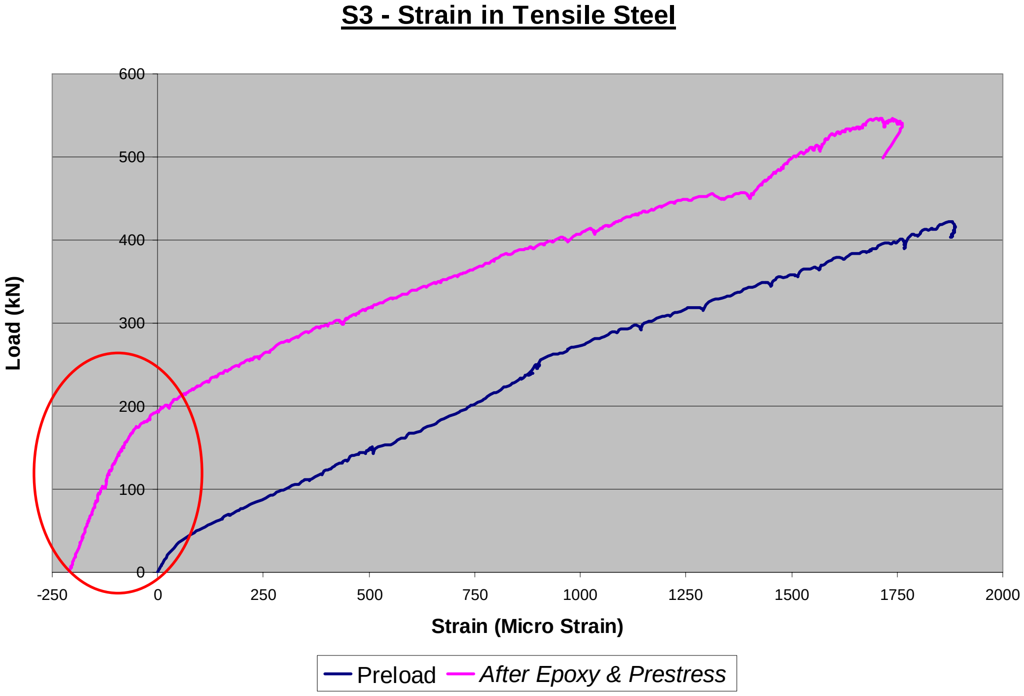


Figure 5.14: *Strain Measured in Tensile Steel of S3*

5.8. Compressive Reinforcement

The highest strain recorded in the compressive steel during all of the tests was 1865 micro strain. As the yield strain of the steel is approximately 2960 micro strain, the reinforcement always remained in its linear elastic region, similar to the tensile reinforcement. Again this was due to the over-design that was evident in the flexural reinforcement. All of the plots were examined from the compressive steel strains and they all exhibited an almost perfect linear plot. After unloading the specimen after each test, the strain in the compressive reinforcement returned to zero. This shows that no plastic deformation occurred in the steel.

5.9. Strain Gauge Rosette

A rosette of strain gauges was placed on the face of the concrete at the centre of the short shear span (Figure 5.15). The gauges were positioned in an attempt get the shear crack to pass through them, so as to determine the principle strains in the crack plain. Due to the altered crack pattern observed, the rosette was found to be ineffective as it only recorded small concrete strains. No analysis could be undertaken on these small strains, therefore due to the limitations of the data logger, the rosette gauges were not used in the final three tests.

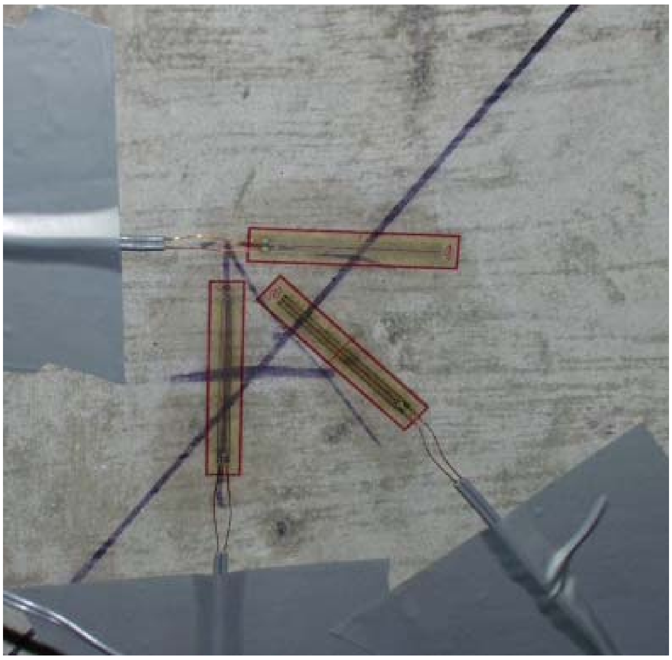


Figure 5.15: *Strain Gauge Rosette*

5.10. Strain in Concrete

A strain gauge was positioned on the concrete directly below the load point to determine the strain in the outer fibres of the specimen. When this strain was plotted against the load, an almost linear plot was gained for each specimen. It was initially thought that by assessing this data it would be able to be determined if the concrete had crushed at the load point. When the data was examined however, it was found that the gauge could not actually distinguish this point, as when the concrete did happen to crush the gauge remained on one side of the failure zone. Hence the data collected produced a linear relationship between the strain and the load (Figure 5.16).

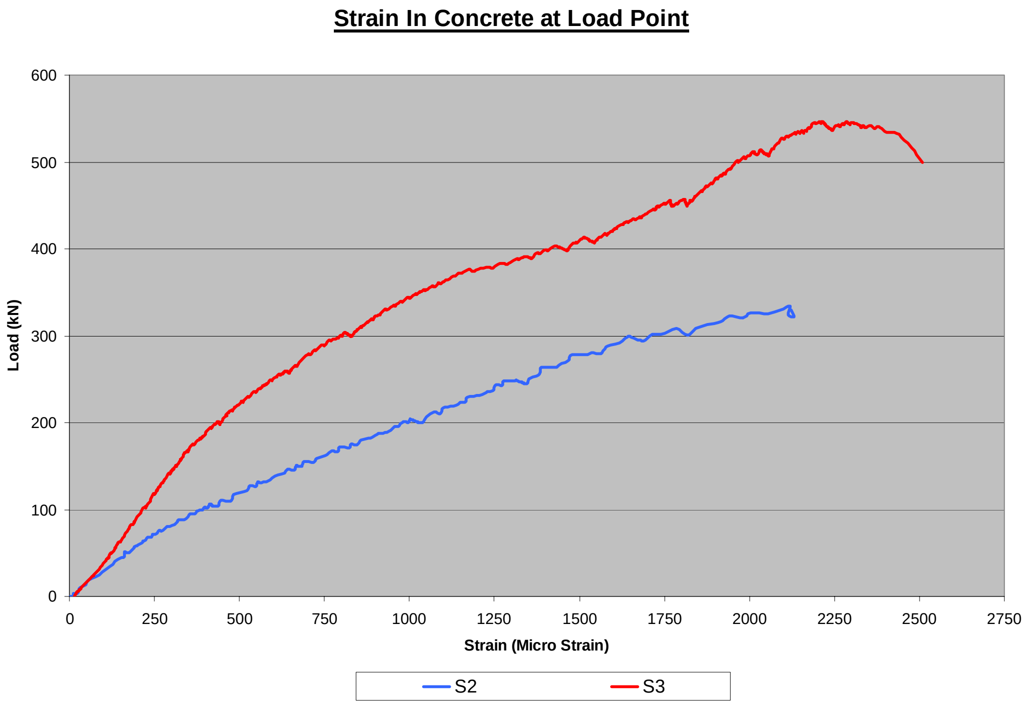


Figure 5.16: *Concrete Strain After the Rehabilitation of Specimens S2 and S3*

The plot in Figure 5.16, presents strain that was recorded in the concrete at the load point during the testing of specimens S2 and S3, after their rehabilitation with epoxy injection and external post-tensioning. It is evident that both specimens exhibit an almost linear plot of the load vs strain. The small dips in the line represent the loss of load through the opening of cracks in the specimen.

5.11. Increase in Prestress Force

As the specimen is loaded and begins to deflect, the overall length begins to slightly increase. Due to this extension, the force in the prestressing system is increased and can be seen in Figure 5.17. This increase in force will have an effect on the load capacity of the member. AS3600, supplies formulae for determining this force increase, however they have proven to be inaccurate. Hence for the purpose of the design calculations undertaken in this research, the increase prestressing force was assumed to be zero. Figure 5.17 shows that this assumption was incorrect.

From the figure it can be seen that each test has a different increase in prestress force. Specimen S3 after epoxy injection and prestressing, exhibits a smaller deflection than S2, however it generates a greater load in the prestressing. It is difficult to explain this irregularity, however it obvious that the material strengths of the specimens had and effect on the increase in prestressing force. Additionally, the stiffness of specimen S3 was greater than that of S2, therefore more of the load would be carried by the concrete during the testing of S3.

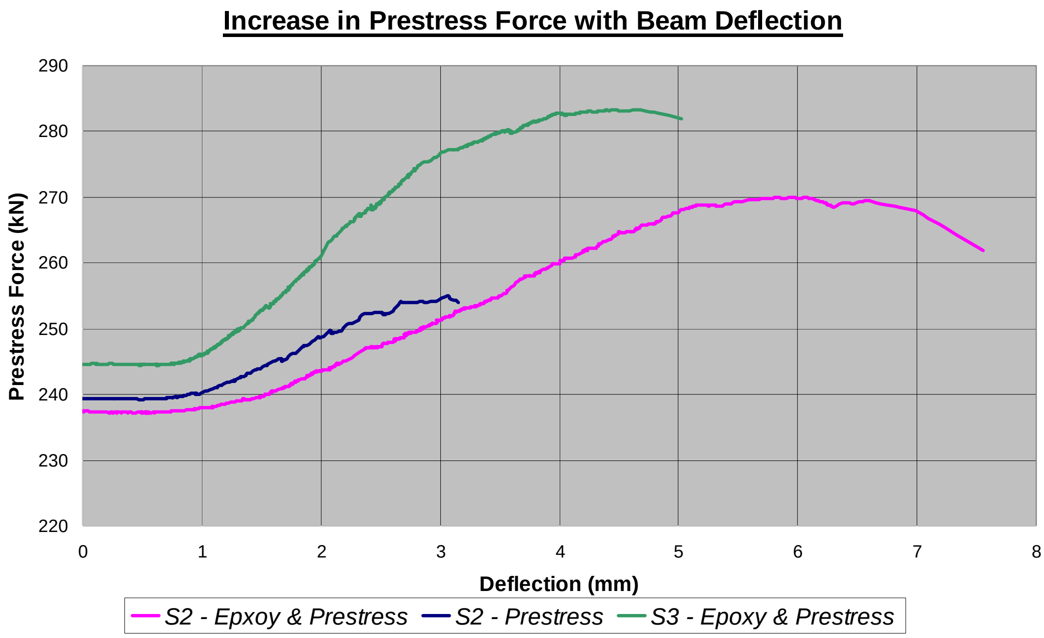


Figure 5.17: *Increase in Prestress Force as the Specimen is Loaded*

It is evident that the initial load applied to the prestressing system was not exactly 250kN. This was due to the way in which the load was applied, as it was difficult to precisely estimate the loss that would occur in the anchorage when to load was applied to it. Note from the plot that the scale on the load axis has been altered, therefore the plot have been exaggerated. Table 5.5 below shows the percentage gain in prestress force during loading.

Table 5.5: *Percentage Increase in Prestressing Force*

Specimen	***Initial Force (kN)***	***Maximum Force (kN)***	***Percentage Increase (%)***
S3	245	283	15.5
S2 (Prestress)	239	255	7
S2 (Epoxy & Prestress)	237	270	14

Note that when the theoretical calculations were undertaken in Chapter 3, it was assumed that the force in the prestress did not increase.

5.12. Altered Section Capacities

The theoretical section capacities of the model were determined in Chapter 3 based on theoretical material strengths. Through the course of the testing however, these material strengths have been found to be inaccurate. To directly compare the experimental capacities of the specimens with the capacities determined from AS3600, the specimen strengths need to be reanalysed with tested material properties. The following sections set out the process undertaken to determine the altered specimen capacities.

5.12.1. Before Prestressing

5.12.1.1. Specimen S1

- *Mu for the Model Section*

Section Properties

$$f_c' = 22.2MPa$$

Reinforcing Properties

$$A_{st} = 600mm^2$$

$$A_{sc} = 400mm^2$$

$$f_{sy} = 592MPa$$

Calculate M_u

$$M_u = T_s d_{st} - C_c d_c - C_s d_{sc}$$

$$M_u = (355.2\times10^3\times376)-(259041.1\times31.2)-(96150.4\times44)$$

$$M_u = 121.2kNm$$

Calculate the ultimate load to induce M_u from the equation set out in Section 3.3.1.

$$P_u = \frac{121.2 \times 10^6 \times 1920}{1335 \times 585} = 298kN$$

- *Ultimate Shear Strength:*

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

Ultimate Shear Strength of the Concrete:

Note: *As the shear crack propagated over the long shear span, the effects accounted for by β_3 are no longer evident, therefore this factor is equal to one for the calculation of V_{uc}.*

$$V_{uc} = 1.346 \times 1.0 \times 1.0 \times 220 \times 376 \times \left(\frac{600 \times 22.2}{220 \times 376} \right)^{\frac{1}{3}}$$

$$V_{uc} = 60.57kN$$

Ultimate Shear Strength of Reinforcement:

$$V_{us} = \frac{A_{sv}}{s} f_{sy} d_o \cot \theta_v$$

Determine the angle between the concrete compression strut and the longitudinal axis of the member (θ_v). As A_{sv}, the area of the stirrups, is known, θ_v can be calculated using:

$$\theta_v = 30^o + 15^o \left[\frac{A_{sv} - A_{sv-\min}}{A_{sv-\max} - A_{sv-\min}} \right]$$

Calculated the maximum and minium stirrup areas:

$$A_{sv-\min} = 0.35 b_v \frac{s}{f_{sy}} = 0.35 \times 220 \times \frac{280}{440} = 49mm^2$$

$$A_{sv-\max} = \frac{b_v s}{f_{sy}} \left[0.2 f_c' - \frac{V_{uc}}{b_v d_o} \right] = \frac{220 \times 280}{440} \left[0.2 \times 22.2 - \frac{115.33 \times 10^3}{220 \times 376} \right] = 426mm^2$$

$$\theta_v = 30^o + 15^o \left[\frac{62 - 49}{426 - 49} \right] = 30.52^o$$

Therefore:

$$V_{us} = \frac{62}{280} \times 250 \times 376 \times \cot(30.52) = 35.3kN$$

Calculate V_u:

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

$$V_u = 60.57 + 35.3 = 95.87kN$$

5.12.1.2. Specimen S2 & S3

The capacities for the unprestressed sections of specimens S2 and S3 are not required as each of these specimens were not failed during preloading. Their capacities after prestressing however, need to be determined to compare the experimental and theoretical results.

5.12.2. After Prestressing

5.12.2.1. Specimen S2

- *Calculate Mu*

$$M_u = T_p d_p + T_s d_{st} - C_c d_c - C_s d_{sc}$$

$$M_u = (270 \times 10^3 \times 220) + (300 \times 10^3 \times 376) - (454618.9 \times 64.7) - (170588 \times 44)$$

$$M_u = 151.65kNm$$

Calculate the ultimate load to induce M_u from the equation set out above in section 3.3.1.

$$P_u = \frac{151.65 \times 10^6 \times 1920}{1335 \times 585} = 373kN$$

- *Ultimate Shear Strength:*

$$M_{dec} = \frac{270 \times 10^3}{220 \times 420} \times \frac{1.358 \times 10^9}{210} = 18.9kNm$$

$$V_{dec} = \frac{18.9 \times 10^6}{\left(\frac{120.2 \times 10^6}{144.41 \times 10^3}\right)} = 22.7kN$$

Ultimate Shear Strength of the Concrete:

Note: *Specimen S2 was loaded on two separate occasions and each time a different failure mode was evident. It failed once over the long shear span and once over the short shear span. Hence to different values of V_{uc} will be required to analyse the specimen based on the value of β_3.*

Failure over the long shear span:

$\beta_3 = 1.0$

$$V_{uc} = 1.346 \times 1.0 \times 1.0 \times 220 \times 376 \times \left(\frac{(600 + 531) \times 18.8}{220 \times 376} \right)^{\frac{1}{3}} + 22.7 \times 10^3$$

$$V_{uc} = 93.49 kN$$

Failure over the short shear span:

$\beta_3 = 1.904$

$$V_{uc} = 1.346 \times 1.0 \times 1.904 \times 220 \times 376 \times \left(\frac{(600 + 531) \times 18.8}{220 \times 376} \right)^{\frac{1}{3}} + 22.7 \times 10^3$$

$$V_{uc} = 157.5 kN$$

Ultimate Shear Strength of Reinforcement:

$$V_{us} = 35.3 kN$$

Calculate V_u:

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

$$V_u = 93.49 + 35.3 = 128.8kN$$

$$V_u = 157.5 + 35.3 = 193kN$$

5.12.2.2. Specimen S3

- *Calculate Mu*

$$M_u = T_p d_p + T_s d_{st} - C_c d_c - C_s d_{sc}$$

$$M_u = \left(283 \times 10^3 \times 220\right) + \left(300 \times 10^3 \times 376\right) - \left(490578.5 \times 48.6\right) - \left(147619.6 \times 44\right)$$

$$M_u = 158.23 kNm$$

Calculate the ultimate load to induce M_u from the equation set out above in section 3.3.1.

$$P_u = \frac{158.23 \times 10^6 \times 1920}{1335 \times 585} = 389 kN$$

- *Ultimate Shear Strength:*

$$M_{dec} = \frac{283 \times 10^3}{220 \times 420} \times \frac{1.358 \times 10^9}{210} = 19.8 kNm$$

$$V_{dec} = \frac{19.8 \times 10^6}{\left(\dfrac{122.5 \times 10^6}{158.3 \times 10^3}\right)} = 25.6 kN$$

Ultimate Shear Strength of the Concrete:

> **Note:** *Specimen S3 only experience failure over the short shear span, therefore only one value of V_{uc} will be required that incorporates β_3.*

$$V_{uc} = 1.346 \times 1.0 \times 1.904 \times 220 \times 376 \times \left(\frac{(600 + 531) \times 27}{220 \times 376}\right)^{\frac{1}{3}} + 25.6 \times 10^3$$

$$V_{uc} = 177.7 kN$$

Ultimate Shear Strength of Reinforcement:

$$V_{us} = 35.3 kN$$

Calculate V_u:

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

$$V_u = 177.7 + 35.3 = 213 kN$$

5.12.3. Summary

Tables 5.6 and 5.7 below, summaries the corrected capacities of each specimen based on the material testing that was undertaken. Note that the only theoretical capacities that have been calculated are the ones that can be directly compared with the experimental data.

Table 5.6: *Corrected Flexural Capacities of Specimens*

Specimen No.	*Strength Before Prestressing (kN)*	*Strength After Prestressing (kN)*
S1	298	-
S2	-	373
S3	-	389

Table 5.7: *Corrected Shear Capacities of Specimens*

Specimen No.	*Failure Mode*	*Strength Before Prestressing (kN)*	*Strength After Prestressing (kN)*
S1	Long Shear Span	96	-
S2	Long Shear Span	-	129
S2	Short Shear Span	-	193
S3	Short Shear Span	-	213

From the shear capacities noted above in Table 5.7, the applied load (P_u) can be determined by:

$$P_u = \frac{V_u L_{eff}}{a} \qquad \text{or} \qquad P_u = \frac{V_u L_{eff}}{b}$$

Figure 5.18, portrays all of the variables for the equations. The equation is selected by the side of the load in which the failure cracks forms. Hence if the specimen failed over the long shear span, the first equation would be used (Figure 5.18(b)). These equations have been used to calculate the applied load (P_u) and the results can be seen in Table 5.8.

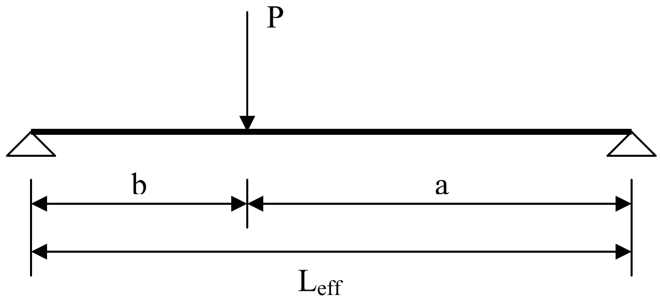


Figure 5.18(a): *Line Load Diagram*

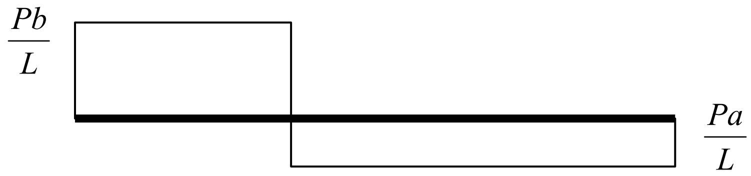


Figure 5.18(b): *Shear Force Diagram*

For the purpose of the shear calculations, L_{eff} is equal to the clear span between the supports.

Table 5.8: *Theoretical Ultimate Applied Load for Specimens*

Specimen No.	***Failure Mode***	***V_u (kN)***	***P_u (kN)***
S1	Long Shear Span	96	374
S2	Long Shear Span	129	503
S2	Short Shear Span	193	260
S3	Short Shear Span	213	286

The values of the ultimate load of the specimens can be directly compared to the experimental applied loads.

From Table 5.8, it can be seen that the ultimate loads to produce shear failure are greater than the loads to reach the ultimate moment (Table 5.6). Based on these values, it could be assumed that the specimens should fail in flexure. From the experimental analysis however, this has proven to be incorrect. It is assumed that is inaccuracy in the flexural strengths of the specimens has been caused by the assumption that the specimens act in a similar manner to a simply supported beam. In reality however, the specimens would react similar to an indeterminate beam with canter-leaver supports. This inaccuracy will have no effects on the overall results of the research, however in the future it should be addressed and additional calculations should be undertaken.

5.13. Comparison of Results

This section will compare the altered theoretical capacities from Section 5.12, with the capacities that were determined by the experimental analysis (Table 5.8). P_{ue} denotes the ultimate applied load during the testing of the specimens.

Table 5.9: *Experimental Specimen Capacities*

Specimen No.	*Failure Mode*	*P_u (kN)*	*P_{ue} (kN)*	*Percentage Difference*
S1	Long Shear Span	374	366	-2%
S2	Long Shear Span	503	334	-33%
S2	Short Shear Span	260	423	63%
S3	Short Shear Span	286	547	91%

Form Table 5.9, it can be seen that there are substantial differences between the theoretical ultimate loads, based on AS3600, and the ultimate loads that were applied during testing. For specimen S1, the control headstock, AS3600 has slightly over predicted the ultimate load. This over prediction is very minor and it can be assumed that AS3600's predicted value is accurate. When a prestress force is applied to specimen S2 without crack repair, AS3600 has over predicted the ultimate strength by a substantial amount. This would be expected however, because the equations provided by the code are not capable of coping with cracked members. The P_u equated from the code therefore, would represent a member without cracks that had a prestressing force applied to it and failed over the long shear span. Hence the initial cracking in specimen S2 reduced its ultimate load capacity.

The final two rows of Table 5.9 represent the two specimens that were rehabilitated with epoxy resins and external post-tensioning. From the 'percentage difference' column in the table, it is obvious that AS3600 supplied substantially conservative capacities for these tests. This conservativeness can be explained by the fact the equations in AS3600 for shear capacity assume a tensile shear failure within the member, similar to the preceding tests in this research. The epoxy injected beams however, experienced a shear compression failure (Figures 5.3(b) and 5.4). As concrete is significantly stronger in compression, the ultimate loads established during testing are considerably higher than the corresponding theoretical loads. AS3600 does provide a method for determining the capacity of a member that experiences web crushing, however intense analytical analysis is required to carryout the method. Consequently, future research may be devoted to simplifying this process.

By comparing both the rows from Table 5.9, that relate to specimen S2, it obvious why the specimens failed over the short shear span once their cracks were repaired by epoxy injection. Based on the theoretical analysis, if an uncracked specimen is prestressed, a load of 503kN would have to be applied for the specimen to fail over the long shear span. For the same specimen to fail over the short shear span, a load of 260kN would be required. Hence the reason for the different failure mode after the specimens were repaired with epoxy resins. When specimen S2 was preloaded and prestressed, it did not fail over the short shear span because the existing cracks simply continued to propagate as the load was applied. Hence the reason for the lower value of P_{ue}.

5.14. Conclusions

Firstly, from this experimental research, it can be seen that the design material strengths supplied by manufactures appear to be considerably conservative (Table 5.3). Due to this conservativeness, the capacities calculated using AS3600 are generally less the member’s ultimate capacity. By comparing the capacities determined in Chapter 3 for the unprestressed member and the relating capacities above, it appears that this is incorrect. The concrete strengths in this case however had considerable effects on the ultimate capacities of the specimens. Generally, therefore it can be assumed that by using the design material strengths in conjunction with formulae from AS3600, a conservative strength will be determined.

Secondly, by examining Table 5.9, it appears that AS3600 considerably under or over estimated, depending on the test, the capacity of the rehabilitated specimen used in this research. The equations defined in the code are designed to model an 'ideal' case. For this reason, these equations cannot accurately estimate the capacity of a rehabilitated member. Hence the significant differences in Table 5.9. AS3600 is not able to determine the capacity of a cracked section, as the extent of the aggregate loss between the crack faces is unknown. This is portrayed by the testing of specimen two with the application of prestress alone. Additionally the equations in the code have been developed for a tensile shear failure. Therefore, when the load is applied close to the support, the β_3 factor in the calculation of V_{uc}, is not sufficient to estimate the load required to cause a compressive shear failure. This explains the significant differences seen in the last two rows of Table 5.9. This form of failure is not common, however in many cases it is overlooked. This error could be overcome by undertaking the method set out in the code to determine the web crushing load, however this requires intense analytical analysis.

It can be concluded, based on the results from this research, that epoxy crack injection and external post-tensioning, together form a high quality form of concrete member shear rehabilitation. It has been found that by applying external post-tension to a concrete member alone, an increase in the members load carrying capacity can be expected. If however, the initial cracks are repaired before the application of the post-tension, a substantial increase in capacity will be achieved. This is reflected by the results from this experimental research.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1. 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 and determine the extent of their use within concrete bridge rehabilitation.

At the commencement, of this research, a detailed 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. *Investigate the state of the Tenthill Creek Bridge.*

Prior to commencing the design work for the research, an investigation was undertaken on the Tenthill Creek Bridge to determine the severity of the shear cracking within the headstocks. This investigation also proved useful throughout the course of the research, as a mental picture was gained of the actual structure. The bridge was revisited on a number of occasions to monitor the rehabilitation that was conducted by Main Roads.

3. *Design a model of the bridge headstocks.*

Subsequent to the initial investigation, a design for the model specimens was developed at a quarter of the size of the actual bridge headstocks. All dimensions were scaled down by the ratio 1:4, and the quantity of reinforcement was determined by an area ratio of 1:16 (Chapter 3). The theoretical capacity of the specimens was then calculated to gain an understanding of the model's capacity.

4. *Investigate and obtain a suitable epoxy resin to use in the experimental tests.*

The types of epoxy resins used during this research had been utilised in previous research at the university, therefore minimal investigation was required. The system included a two part epoxy resin that was injected into the cracks through injection flanges. These resins have been specially designed for the structural rehabilitation of concrete and masonry members.

5. *Design and develop a suitable external post-tensioning system.*

Similar to the epoxy resins, the external post-tensioning system that was utilised had previously been used in prior research. Minor changes however had to be made to the system to be able to apply it to the model. These changes were minor therefore with the assistance of the laboratory staff, the system was easily adapted to suit the specimens.

6. *Construct and test three specimens.*

Once the design work had been completed, the construction of the specimens was undertaken (Chapter 4). This involved building formwork, tying reinforcing cages and pouring concrete. Three specimens in total were constructed and then later tested. The testing was conducted over a number of weeks. The results obtained from these tests were to somewhat unexpected, which lead to interesting conclusions.

7. *Finally, critically analyse the shear capacity of the test specimens and advise on a suitable rehabilitation technique based on the findings.*

Subsequent to the data being collected and analysed, conclusions were drawn which are mentioned below in Section 6.2.

All of the objectives that were set during the initial stages of this research have been achieved. Thus the entire research has been successfully completed.

6.2. Conclusions

The results that were obtained throughout the course of this research were of a surprising nature. Initially, with the specimens failing over the long shear span and also the substantial differences that became evident between the theoretical and experimental capacities of the specimens. The following two sections summarise these findings and attempt to draw conclusions from them.

6.2.1. Rehabilitation Technique

Subsequent to conducting the research on the model specimens of the Tenthill Creek Bridge headstocks, it has been reiterated that external post-tensioning is an effective form of concrete rehabilitation. Conversely, it was discovered that the system has limits on the extent of the initial cracking that is present within the member. By preloading specimen S2 to simulate the bridge condition and then rehabilitating with external post-tensioning, it was established that once a substantial amount of the aggregate bond between the crack faces is lost, the compressive force on the member will only marginally increase its capacity. This can be overcome however by the used of epoxy crack injection. This system was found to be capable of restoring the member to its original structural condition. If the prestressing force is applied to a member that has been repaired by epoxy injection, a significant increase in capacity will be achieved. This increase in capacity is similar to what would be achieved if the prestressing force was applied to an uncracked member.

Throughout the course of this research, it was discovered that by using both epoxy crack injection and external post-tensioning together, an efficient increase in load capacity can be achieved. The combination of these systems is able of achieving sizeable increases in member capacity. Consequently, the used of these systems may become more prevalent in the future.

6.2.2. Tenthill Creek Bridge

The Tenthill Creek Bridge headstocks were in need of immediate rehabilitation. As noted in the introduction, the headstocks contained severe shear cracks that dramatically decreased the structures load capacity. Proceeding the testing of specimen S1, the control headstock, it was found that it experienced a tensile failure over the long shear span. This can be explained by the development of a compression strut in the short shear span. With the formation of this strut, the weaker point in the specimen became prominent over the long shear span. From these findings, it could be assumed that a similar crack pattern could also occur in the bridge headstocks if the applied load was further increased. This assumption is based on observations from the testing, therefore additional analysis would be required to validate the assumption.

Form the experimental results, it was discovered that by using both epoxy crack injection and external post-tensioning in combination, a sizable increase in load capacity could be achieved. Hence with application of these techniques to the Tenthill Creek Bridge headstocks (Figure 6.1), it can be assumed that the capacity of the members will satisfy the loads produced by the design traffic for the bridge.

Figure 6.1: *Application of Epoxy Crack Injection and External PT to Tenthill Creek Bridge Headstocks*

6.2.3. AS3600

From the results discussed in Chapter 5, it is portrayed that AS3600 was not able to accurately estimate the capacity of the test specimens. The main reason for these inaccuracies is due to the code being designed to model an 'ideal case', therefore it is not able to cope with the variances that have been discovered throughout this research. Initially, it is almost impossible to predict the shear capacity of a cracked section; therefore it is inevitable that differences between the theoretical and experimental capacities will be found under these conditions. Additionally, the method supplied by the code to evaluate shear compression failure, is of a complex nature, therefore it is difficult to evaluate the relating capacity without intense analytical analysis. AS3600's equations have been developed around a tensile shear failure, hence this explains the differences that were discovered between the theoretical and experimental capacities after they were rehabilitated. From this analysis, it is recommended that AS3600 be treated with caution when analysing problems similar to this.

6.3. Recommendations

The deterioration of concrete bridges is a problem that cannot be ignored. Consequently, a cost effective rehabilitation technique for these structures is required. Many different forms of concrete member rehabilitation systems have been developed, however the used use of both epoxy crack injection and external post-tension together is a relatively new technique. From this research, it has been discovered that this form of restoration can be highly effective and may become more prominent in the future.

This combined technique is relatively new to the construction industry, therefore a considerable amount of research needs to be conducted to refine the system. Future research could be focused on predicting the shear capacity of a cracked section, as this is one of the major limitations related to concrete member rehabilitation. Additionally, the loading pattern used on the specimens is thought to have a major effect on the acquired results. Hence in the future, research could be focused on the effects of these loading patterns. Finally, an improved understanding of the systems could be gained through the use of numerical simulation of the system. Results from this analysis may uncover means by which the system can be refined. In conclusion, it is recommended that the use of epoxy crack injection and external post-tensioning, as a form of restoration is further investigated to ensure that it is utilised to its full potential.

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APPENDIX A

PROJECT SPECIFICATION

University Of Southern Queensland
FACULTY OF ENGINEERING AND SURVEYING

ENG 4111/4112 Research Project
PROJECT SPECIFICATION

FOR: ***Evan Woods***
Q1121090

TOPIC: Strengthening of headstocks using external prestressing

SUPERVISORS: Dr. Thiru Aravinthan
Dr. Tim Heldt

SPONSORSHIP: University of Southern Queensland

PROJECT AIM: This project aims to provide a cost effective method of upgrading and strengthening existing concrete bridges that are deemed to be unfit for service. The project will focus on the headstock of the bridge in an attempt to increase the life of the structure.

PROGRAMME: ***Issue A, 18 March 2004***

1. Research background information on external prestressing and its use to-date
within bridge rehabilitation. *(Finish early Semester 2, 2004)*
2. Investigate the state of the Tenthill Creek Bridge on the old Warrego Highway.
(Undertake ASAP)
3. Design a scale model of the headstocks within the Tenthill Creek Bridge. *(Undertake ASAP)*
4. Design suitable external prestressing for the headstocks. *(Finish early April)*
5. Investigate a suitable resin to impregnate shear and flexure cracks in the headstocks. *(Finish early April)*
6. Construct and test three concrete headstocks, each under different conditions, including – *(Finish early Semester 2, 2004)*
    a. A control headstock, later impregnated with resin and tested again.
    b. Two externally prestressed headstock.
    c. Headstocks from (b) will be impregnated if resin and tested again.
7. Critically analysis the shear and flexural strengths of the headstocks.
8. Analysis and discuss all findings from the tests and advise on suitable rehabilitation techniques. *(Finish for dissertation)*

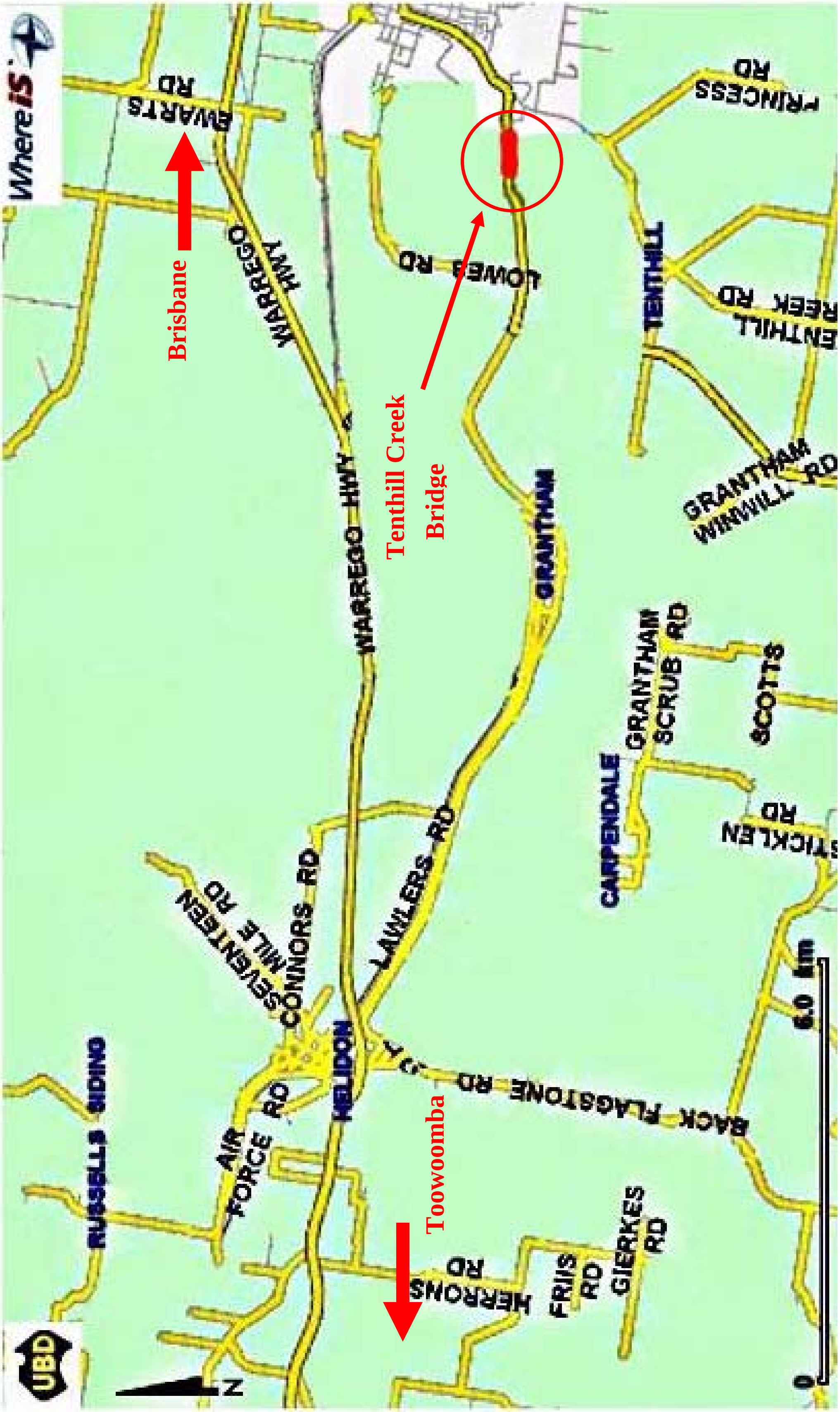
AGREED:___________________(student) __________________ (supervisor)

__ / __ / __ __ / __ / __

APPENDIX B

BRIDGE LOCATION MAP

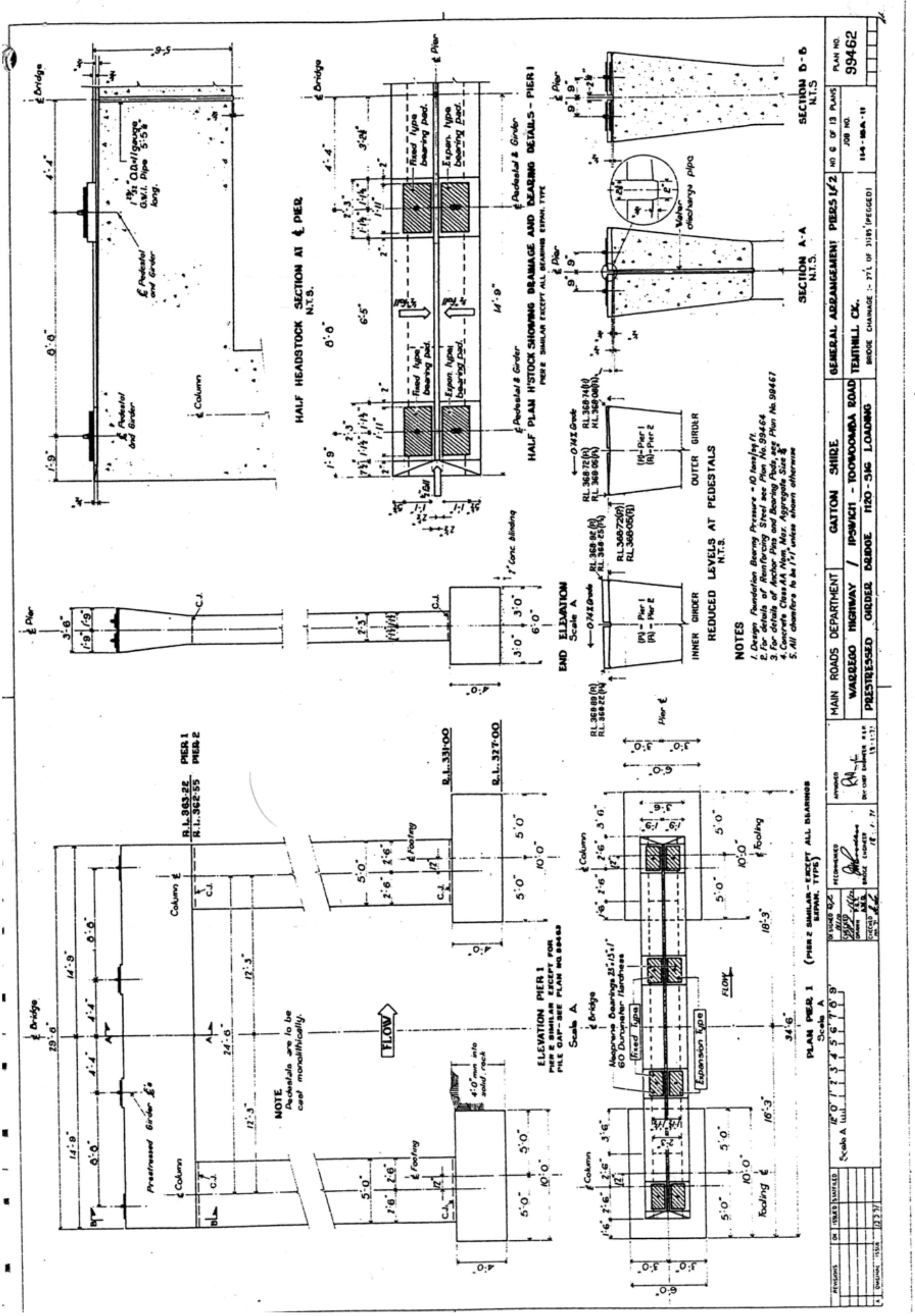
(UBD 2002)

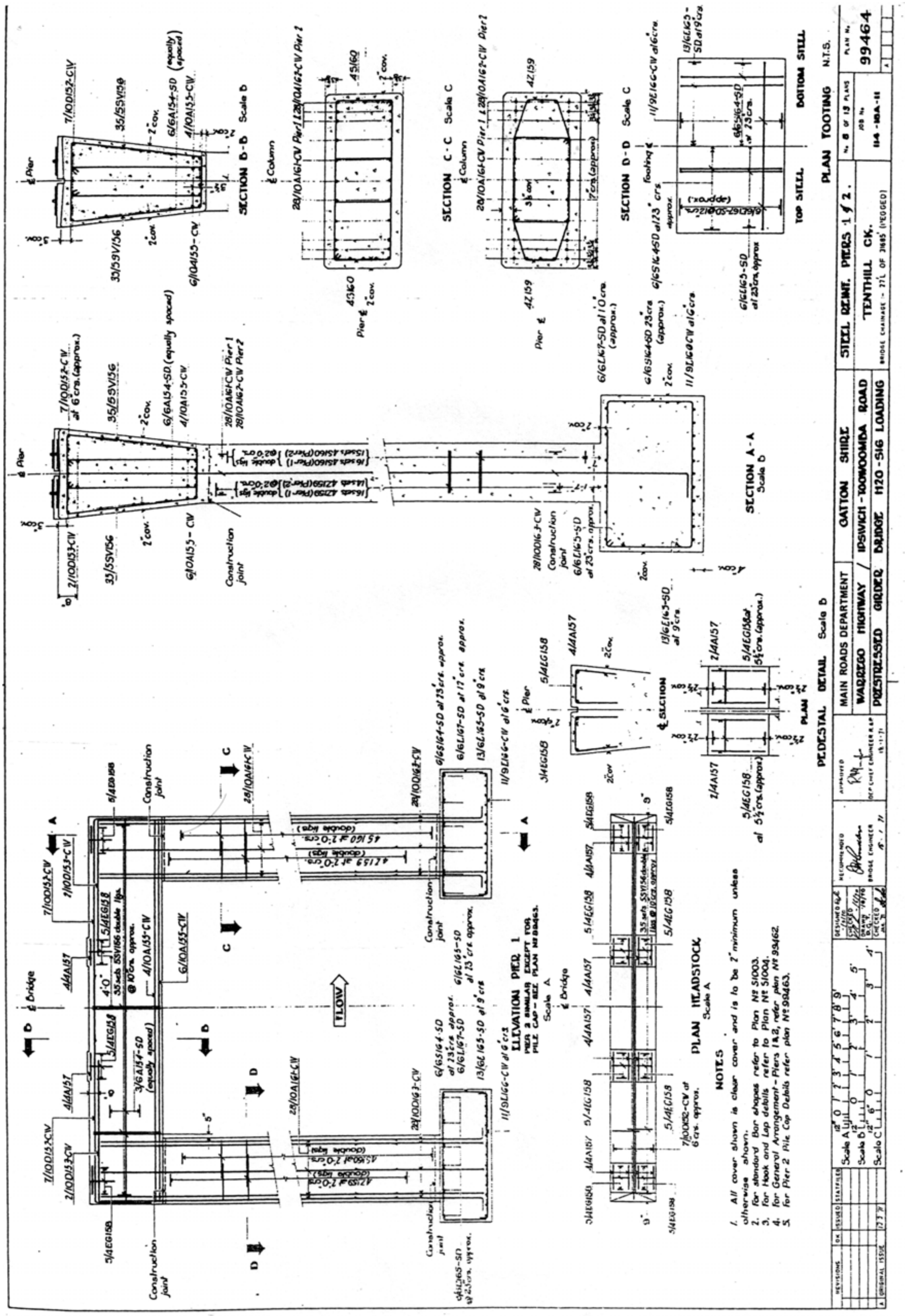

WhereIS
Brisbane
Tenthill Creek
Bridge
Toowoomba
EWARTS RD
WARREGO HWY
LOWES RD
PRINCESS RD
TENTHILL
TENTHILL CREEK RD
GRANTHAM WINWILL RD
GRANTHAM
GRANTHAM SCRUB RD
SCOTTS
CARPENDALE
STICKLEN RD
LAWLERS RD
SEVENTEEN MILE RD
CONNORS RD
HELIDON
BACK FLAGSTONE RD
AIR FORCE RD
RUSSELLS SIDING
HERRONS RD
FRIIS RD
GIERKES RD
UBD
N
0
6.0 km

APPENDIX C

TENTHILL CREEK BRIDGE HEADSTOCK & PIER PLANS

(Obtained from Main Roads Queensland)


HALF HEADSTOCK SECTION AT ℄ PIER
HALF PLAN H'STOCK SHOWING DRAINAGE AND BEARING DETAILS - PIER 1
SECTION A-A
SECTION B-B
END ELEVATION
REDUCED LEVELS AT PEDESTALS
ELEVATION PIER 1
PLAN PIER 1
FLOW
MAIN ROADS DEPARTMENT
GATTON SHIRE
GENERAL ARRANGEMENT PIERS 1 & 2
WARREGO HIGHWAY / IPSWICH - TOOWOOMBA ROAD
TENTHILL CK.
PRESTRESSED GIRDER BRIDGE
H20-S16 LOADING
99462


MAIN ROADS DEPARTMENT
WARREGO HIGHWAY / IPSWICH - TOOWOOMBA ROAD
GATTON SHIRE
PRESTRESSED GIRDER BRIDGE H20-S16 LOADING
STEEL REINF. PIERS 1 & 2.
TENTHILL CK.
99464
ELEVATION PIER 1
PLAN HEADSTOCK
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
PEDESTAL DETAIL
PLAN FOOTING
NOTES

APPENDIX D

DESIGN CALCULATIONS

1. **Trial & Error Calculations To Determine d_n**
2. **Calculations To Determine Centroid Of Bridge Headstock**
3. **Calculations To Determine I_{xx} For Bridge Headstocks**

1. Trial & Error Calculations To Determine d_n

The following calculations set out the trial and error procedure used to determine the depth of the neutral axis for the test specimens. This is required as the compressive reinforcement within the specimen does not yield. Two sets of calculations are presented:

- Before Post-tensioning, and
- After post-tensioning.

- ***Before Post-tensioning***

Determined in Chapter 3:

$T = 300 x10^3 N$

$C_s = 200 \times 10^3 N$

$C_c = 4918.85 d_n N$

$d_n = 20.33mm$

(Compressive steel does not yield, therefore $d_n \neq 20.33mm$)

Try d_n = 50mm

$$\varepsilon_{sc} = 0.003\left(\frac{50-44}{50}\right) = 0.0004 < 0.0025(\varepsilon_{sy})$$

$C_s = 0.0004 \times 200{,}000 \times 400 = 28.8 \times 10^3 N$

$C_c = 4918.85 \times 50 = 245942.4N$

$T = 300 \times 10^3 N$

Check Equilibrium:

$$\sum H = 245942.4 + 28.8\times10^3 - 300\times10^3 = -25257.6N$$

Therefore the estimated value of d_n is incorrect as $\sum H \neq 0$, hence a larger value of d_n is required.

Try d_n = 55mm

$\varepsilon_{sc} = 0.0006$

$C_s = 48 \times 10^3 N$

$C_c = 270536.6N$

$T = 300 \times 10^3 N$

$$\sum H = 18536.64N$$

Hence d_n is to large

Try d_n = 52.833mm

ε_{sc} = 0.0005

C_s = $40.1 \times 10^3 N$

C_c = 259877.5N

T = $300 \times 10^3 N$

$$\sum H = 2.418N \approx 0$$

Therefore d_n = 52.833mm should be used to continue the ultimate moment capacity calculations for the specimen. The remain calculations can be seen in Chapter 3.

Note: This exact value of d_n was determined using a spreadsheet.

- ***After Post-tensioning***

Determined in Chapter 3:

$T_p = 250 \times 10^3 N$

T = $300 \times 10^3 N$

C_s = $200 \times 10^3 N$

C_c = $4918.85 d_n N$

d_n = 71.15mm

$$\varepsilon_{sc} = 0.003\left(\frac{71.15-44}{71.15}\right) = 0.0011 < 0.0025(\varepsilon_{sy})$$

(Compressive steel does not yield, therefore $d_n \neq 71.15mm$)

Try d_n = 100mm

ε_{sc} = 0.0017

C_s = $13.4 \times 10^3 N$

C_c = 491884.8N

T = $300 \times 10^3 N$

$T_p = 250 \times 10^3 N$

$$\sum H = 76284.8N$$

Therefore the estimated value of d_n is incorrect as $\sum H \neq 0$, hence a smaller value of d_n is required.

Try d_n = 87.545mm

$\varepsilon_{sc} = 0.0015$

$C_s = 11.9 \times 10^3 N$

$C_c = 430620.5N$

$T = 300 \times 10^3 N$

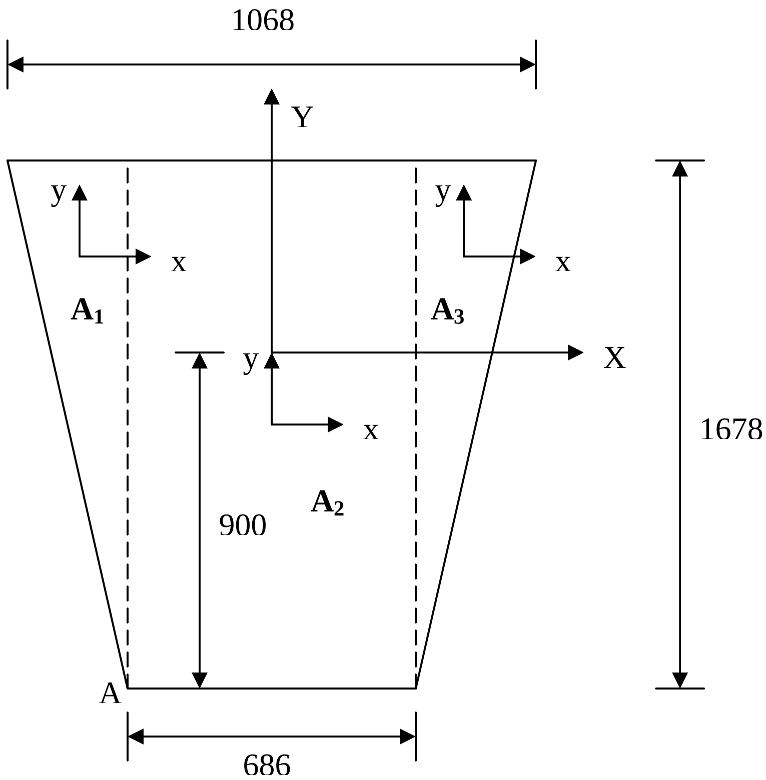
$T_p = 250 \times 10^3 N$

$$\sum H = -3.131N \approx 0$$

Therefore d_n = 87.545mm should be used to continue the ultimate moment capacity calculations for the post-tensioned specimen. The remain calculations can be seen in Chapter 3.

Note: This exact value of d_n was determined using a spreadsheet.

4. Calculations To Determine Centroid Of Bridge Headstock



The diagram shows the cross-section of the Tenthill Creek Bridge Headstock. The centroid of the section is required.

Note: *The section is symmetrical about the y-axis, therefore the centroid only needs to be calculated about the x-axis.*

- Initially arbitrary axes a chosen with the origin at point A.
- The section is subdivided into simple shapes shown by the dotted lines.
- The centroid of each shape is marked by the small axes marked on each one.
- The dimension of each shapes y position relative to point A is shown below.

A_1 = 1118.67mm

A_2 = 839mm

A_3 = 1118.67mm

- The area of each shape:

A_1 = 160 x 10^3mm^2

A_2 = 1.15 x 10^3mm^2

A_3 = 160 x 10^3mm^2

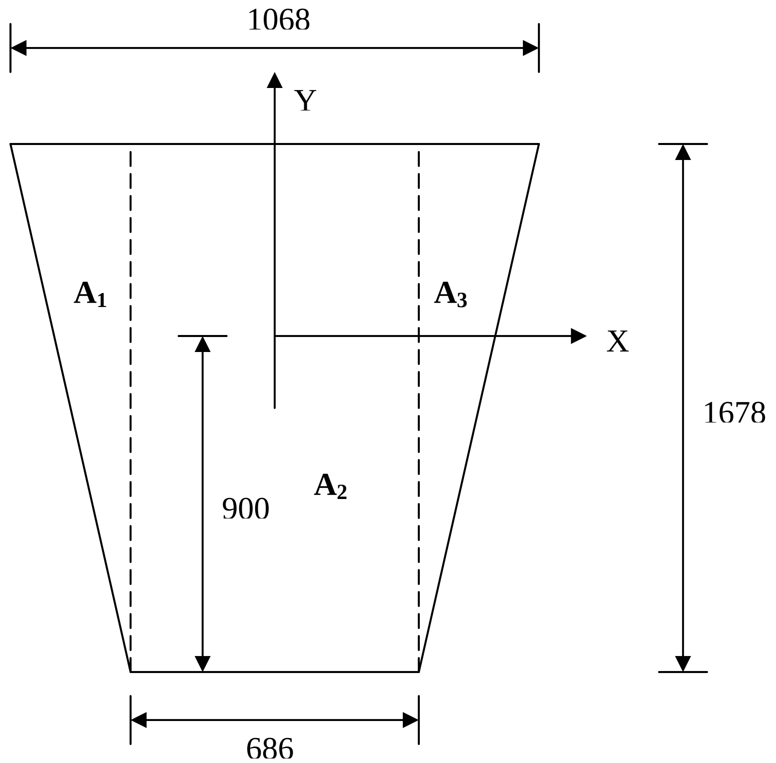
- Determine the y distance from point A o the centroid using:

$$\bar{y} = \frac{\sum (A_i x_i)}{(\sum A_i)}$$

$$\bar{y} = \frac{(1.15\times10^6 \times 839)+(160\times10^3 \times 1118.67\times 2)}{1.15\times10^6 + (160\times10^3 \times 2)} = 899.95mm$$

- Therefore the position of the centroid relative to point A is (343, 900) and can be seen on the diagram above.

5. Calculations To Determine I_{xx} For Bridge Headstocks



The second moment of area (I_{xx}) needs to be determined about the principal axes shown in the diagram above.

Note: *Only I_{xx} is required for comparison with the model.*

- The shape is again subdivided into three sections and the y position of each shape relative to the principal axes are :

  A_1 = 218.67mm

  A_2 = -61mm

  A_3 = 218.67mm

- The area of each shape:

  A_1 = $160 \times 10^3 mm^2$

  A_2 = $1.15 \times 10^3 mm^2$

  A_3 = $160 \times 10^3 mm^2$

- Calculate the second moment of area for each simple shape (I_x)

  Area 1 = Area 3:

$$I_x = \frac{bd^3}{36} = \frac{190 \times 1678^3}{36} = 25.0 \times 10^9 \, mm^4$$

  Area 2:

$$I_x = \frac{bd^3}{12} = \frac{686 \times 1678^3}{12} = 270 \times 10^9 \, mm^4$$

- Evaluate I_{xx} about the principal axes:

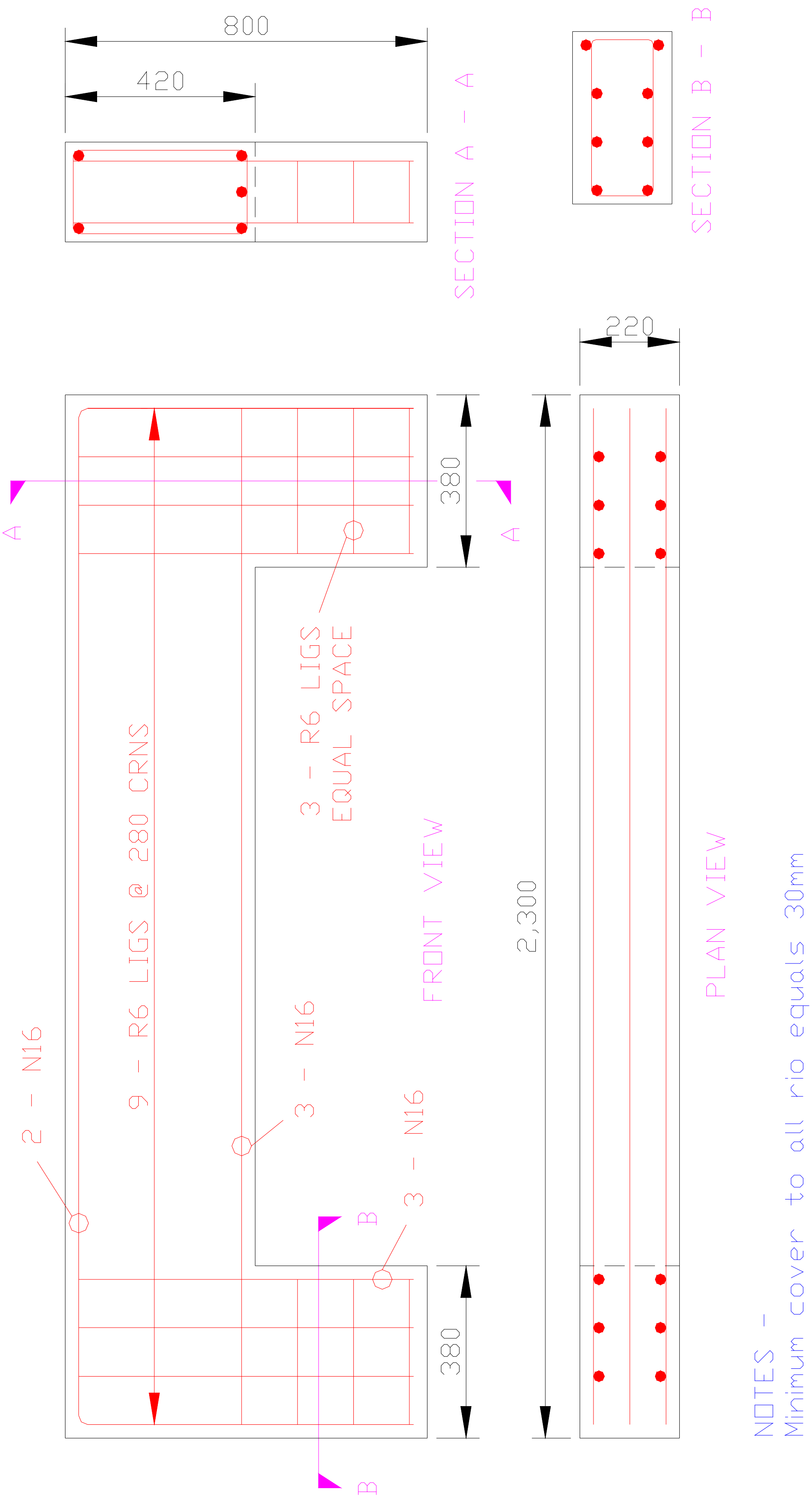
$$I_{xx} = \sum \left(I_{xi} + A_i y_i^2 \right)$$

$$I_{xx} = \left(270 \times 10^9 + 1.15 \times 10^6 \times -61^2\right) + \left(25 \times 10^9 + 160 \times 10^3 \times 218.67^2\right) \times 2$$

$$I_{xx} = 3.393 \times 10^{11} \, mm^4$$

APPENDIX E

SPECIMEN PLANS


800
420
SECTION A - A
SECTION B - B
220
380
380
2,300
A
A
B
B
9 - R6 LIGS @ 280 CRNS
3 - R6 LIGS
EQUAL SPACE
2 - N16
3 - N16
3 - N16
FRONT VIEW
PLAN VIEW
NOTES -
Minimum cover to all rio equals 30mm

APPENDIX F

STRAIN GAUGE DATA

1. **Steel Strain Gauge Data Sheet**
2. **Concrete Strain Gauge Data Sheet**
3. **Wax Coating Data Sheet**
4. **CN Adhesive Data Sheet**

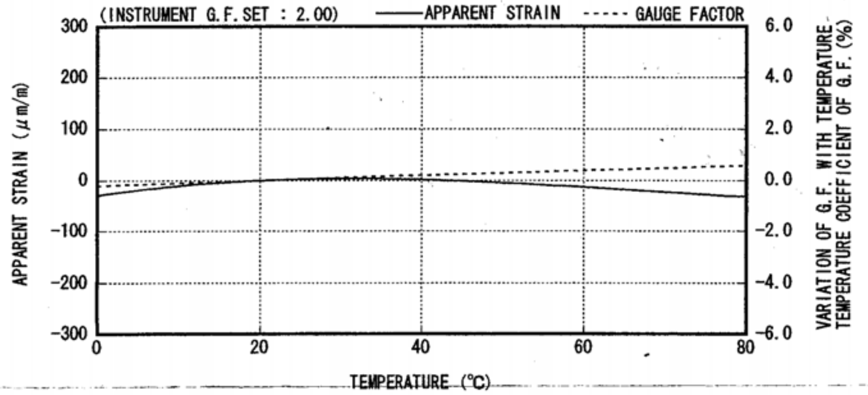
TML STRAIN GAUGE TEST DATA

GAUGE TYPE	: FLA-2-11	TESTED ON	: SS 400
LOT NO.	: A510411	COEFFICIENT OF THERMAL EXPANSION	: 11.8 $\times 10^{-6}$/℃
GAUGE FACTOR	: 2.13 ±1%	TEMPERATURE COEFFICIENT OF G.F.	: +0.1±0.05 %/10℃
ADHESIVE	: P-2	DATA NO.	: A0313

THERMAL OUTPUT (ε app : APPARENT STRAIN)

ε app $= -2.94\times10^{1}+2.32\times T^{1}-4.60\times10^{-2}\times T^{2}+1.67\times10^{-4}\times T^{3}+5.00\times10^{-7}\times T^{4}$ (μm/m)

TOLERANCE : ±0.85 [(μm/m)/℃] , T : TEMPERATURE



ひずみゲージ取扱いの注意事項

- 上記の特性データは、リード線の取付けによる影響を含んでおりません。裏面記載のリード線の測定値への影響に従って補正してください。
- ゲージの使用温度は、接着剤の耐熱温度などにより変わります。
- 絶縁抵抗などの点検は、印加電圧を５０Ｖ以下にしてください。
- ゲージリード線に無理な力を加えないでください。
- ゲージ裏面に接着剤を塗布して接着してください。
- ひずみゲージの裏面は脱脂洗浄してありますので、汚さないように取扱いしてください。
- ゲージの包装を開封後は、乾燥した場所で保管してください。
- ご使用に際してご不明な点などがございましたら、当社までお問い合わせください。

CAUTIONS ON HANDLING STRAIN GAUGES

- The above characteristic data do not include influence due to lead wires. Correct the data in accordance with the influence of lead wires on measured values described overleaf.
- The service temperature of strain gauge depends on the operating temperature of adhesive, etc.
- Check of insulation resistance, etc. should be made at a voltage of less than 50V.
- Do not apply an excessive force to the gauge leads.
- Apply an adhesive to the back of a strain gauge and stick the gauge to a specimen.
- As the back of strain gauge has been degreased and washed, do not contaminate it.
- After unpacking, store strain gauges in a dry place.
- If you have any questions on strain gauges or installation, contact TML or your local agent.

Made in Japan

TML 株式會社 東京測器研究所

〒 140-8560 東京都品川区南大井 6 - 8 - 2
TEL 03 - 3763 - 5611

Tokyo Sokki Kenkyujo Co., Ltd.

8 - 2, Minami - Ohi 6 - Chome
Shinagawa - ku, Tokyo 140-8560

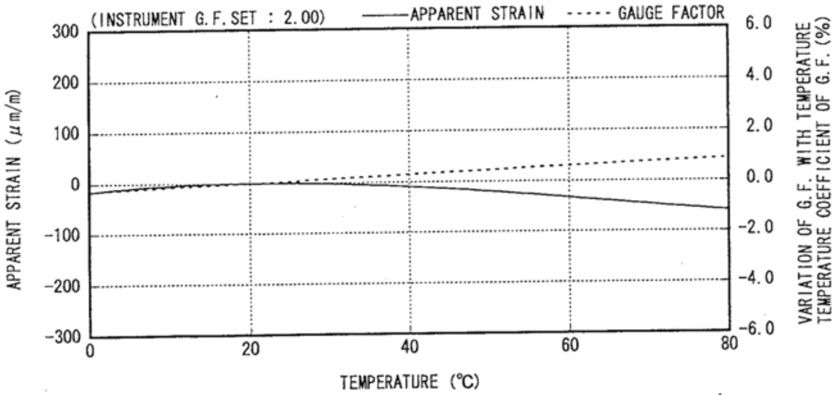
TML STRAIN GAUGE TEST DATA

GAUGE TYPE	: PFL-30-11	TESTED ON	: SS 400
LOT NO.	: A701611	COEFFICIENT OF THERMAL EXPANSION	: 11.8 $\times 10^{-6}$/℃
GAUGE FACTOR	: 2.13 ±1%	TEMPERATURE COEFFICIENT OF G.F.	: +0.15±0.05 %/10℃
ADHESIVE	: P-2	DATA NO.	: D0019

THERMAL OUTPUT (ε app : APPARENT STRAIN)

ε app $= -1.62\times10^{1}+1.63\times T^{1}-4.55\times10^{-2}\times T^{2}+2.27\times10^{-4}\times T^{3}+3.52\times10^{-8}\times T^{4}$ (μm/m)

TOLERANCE : ±0.85 [(μm/m)/℃] , T : TEMPERATURE



ひずみゲージ取扱いの注意事項

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- ゲージの使用温度は、接着剤の耐熱温度などにより変わります。
- 絶縁抵抗などの点検は、印加電圧を５０Ｖ以下にしてください。
- ゲージリード線に無理な力を加えないでください。
- ゲージ裏面に接着剤を塗布して接着してください。
- ひずみゲージの裏面は脱脂洗浄してありますので、汚さないように取扱いしてください。
- ゲージの包装を開封後は、乾燥した場所で保管してください。
- ご使用に際してご不明な点などがございましたら、当社までお問い合わせください。

CAUTIONS ON HANDLING STRAIN GAUGES

- The above characteristic data do not include influence due to lead wires. Correct the data in accordance with the influence of lead wires on measured values described overleaf.
- The service temperature of strain gauge depends on the operating temperature of adhesive, etc.
- Check of insulation resistance, etc. should be made at a voltage of less than 50V.
- Do not apply an excessive force to the gauge leads.
- Apply an adhesive to the back of a strain gauge and stick the gauge to a specimen.
- As the back of strain gauge has been degreased and washed, do not contaminate it.
- After unpacking, store strain gauges in a dry place.
- If you have any questions on strain gauges or installation, contact TML or your local agent.

Made in Japan

TML 株式會社 東京測器研究所

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OPERATION MANUAL OF TML COATING MATERIAL TYPE W-1

The W-1 is a coating material for TML strain gauges and used for moisture or water-proofing of strain gauges bonded on metal or nonmetal surface. The W-1 is light-yellow colored microcrystalline wax and can be used immediately after heat-melting.

1 SURFACE PREPARATION

The following work should be finished before gauge installation.

(1) Remove moisture, grease, scale, rust, paint, etc. from the bonding area to present the foundation of a test piece.

(2) Grind an area a bit larger than the bonding area uniformly with abrasive paper. Finish should be achieved with abrasive paper #80～120 for steel and #240～320 for aluminium. Consult supplier for other materials.

(3) Clean the abraded area with industrial tissue paper or cloth soaked in a small amount of chemical solvents such as acetone. Cleaning should be made until the tissue or cloth is kept contamination free.

2 GENERAL APPLICATION BONDING PROCEDURES

(1) Bond the strain gauges in the specific manner and connect lead wires.

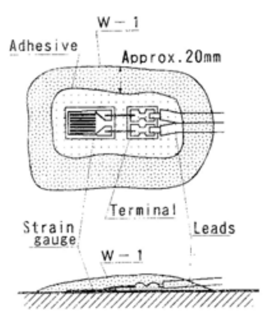
(2) Put the W-1 in an oil bath with temperature adjustment and the like and melt it perfectly at 100 ～ 120℃. The melted W-1 becomes light yellow transparent liquid. As a new W-1 may crackle in heating, leave it at 100 ～ 120℃ till the sound stops.

(3) Dip the tip of a brush in the melted W-1 for warming.

(4) Soak the brush tip in the melted W-1 and apply at once over the strain gauge and extruded adhesive. (Fig. 1) The W-1 hardens as soon as cooling and turns cloudy light yellow.

(5) As the once-used brush gets cool due to heat absorption of the test specimen, repeat the steps (3) and (4) till complete application.

Fig.1 Coating area of W-1



TML manu W1002E

3 STORAGE AND HANDLING

(1) After use, making sure that the melted W-1 is hardened at room temperature, store in a cool dry place without flame.

(2) After handling, wash hands well.

(3) If there is any question, consult TML or your nearest TML agent. A material safety data sheet is available on request.

(Table)

Contents	500g
Components	Microcrystalline wax (100%)
Operating temperature	0 ～ +50℃
Storing condition	A dark cool place (room temperature)

⚠ CAUTION

1. Avoid contact with skin, eyes and clothing.
2. Avoid direct heating for fear of ignition.
3. Melted W-1 may cause a burn. As occasion arises, wear a protect glove or goggle.
4. If melted W-1 adhered to skin, cool at once in cold water for more than 10 minutes and call for medical aid.
5. In case of getting in eyes, flush eyes well with water for more than 15 minutes and call for medical aid.
6. Use in a well-ventilated area.
7. Handle in a place without flame.
8. Store away from children.

Made in Japa

Tokyo Sokki Kenkyujo Co., Ltd.
8-2. Minami- Ohi 6- Chome

OPERATION MANUAL OF TML STRAIN GAUGE ADHESIVE TYPE CN

TML manu CN004E

The **CN** adhesive is a single component cement for strain gauges, and time required for bonding gauges is extremely short and handling is very easy. There are two types: **CN** (with green cap) for general purpose and **CN-E** (with white cap) featuring high viscosity.

1 UNPACKING

(1) Take off the adhesive cap and drill a minute hole on the top of nozzle with the supplied pin. (Fig. 1)
(2) Pull out the pin. Then, take care of the adhesive liquid which may jump out.
(3) Wipe off the adhesive attached to the nozzle with cloth, etc. and surely tighten the cap.
(4) If necessary, use the supplied upset-protect stand.

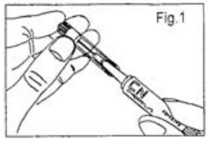


Fig.1

2 SURFACE PREPARATION

(1) Remove grease, scale, dust, paint, etc. from the bonding area to present a metal burnish (foundation).
(2) Grind an area somewhat larger than the bonding area uniformly and finely with abrasive paper.
Finish should be achieved with #120 ~ 180 abrasive paper for steel and #240 ~ 320 for aluminium. (Fig. 2)
(3) Clean the ground area with industrial tissue paper or cloth soaked in a small quantity of chemical solvents such as acetone. Cleaning should be made till the tissue or cloth is kept contamination free. (Fig. 3)
(4) After surface preparation, stick strain gauges before the prepared surface makes oxidizing membrane or is not contaminated.

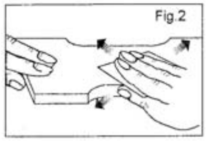


Fig.2

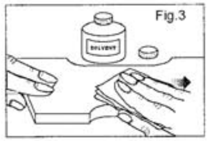


Fig.3

3 GENERAL BONDING PROCEDURES

(1) Position strain gauges on the bonding area and mark a guide line with a scriber or 4H pencil. (Fig. 4)
(2) Take out a strain gauge from the plastic binder. Then, you need not wipe the bonding surface with a solvent because the gauge is supplied in the thoroughly washed state.
(3) Drop a necessary amount of the adhesive on the back of the gauge base. The amount of adhesive is usually one drop but you may increase the number of drops according to the size of gauge. (Fig. 5)
(4) Spread the adhesive on the back of gauge thin and uniformly using the adhesive nozzle.

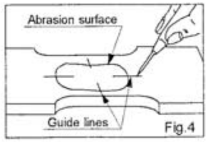


Fig.4

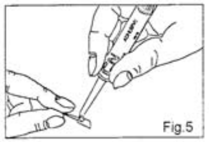


Fig.5

(5) Place the gauge to the guide mark, put a polyethylene sheet and press down the gauge constantly by thumb or a gauge pressure application device. This work should be done quickly as curing is completed very fast. (Fig. 6)
(6) The curing time is different depending on gauges, test pieces, temperature, humidity or pressing force. The curing time under normal conditions is 20 ~ 60 seconds for **CN** and 40 ~ 120 seconds for **CN-E**.
(7) Measurement gets ready about 15 minutes after hardening. For precise measurement or experiments involving temperature rise, however, leave for a few to 24 hours.

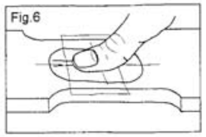


Fig.6

4 STORAGE AND HANDLING

(1) After use, wipe adhesive out of container and nozzle with cloth, etc. and replace the cap.
(2) After use, put back in the aluminum bag and store in a dry, cool and dark place.
(3) After handling, wash hands well.
(4) As curing speed gets slow in winter, it is recommended to warm the bonding surface or use a CN accelerator.
(5) For bonding on polyethylene (PE), polypropylene (PP), 4F ethylene (PTFE), etc., an exclusive surface treatment agent is required. If there is any question, consult TML or your nearest TML agent. A material safety data sheet is available on request.

Contents	2g × 5
Components	Synthetic resin (100 %) Cyanoacrylate
Operating temperature	− 30 ~ + 120 °C
Shelf life	6 months in a cool, dry and dark place such as refrigerator (5 ~ 10 °C, freezer prohibited)

⚠ CAUTION

1. Avoid contact with skin, eyes and clothing.
2. Should fingers be bonded, do not forcibly detach but slowly take off, rubbing in a bath.
3. Should adhesive enter eyes, flush eyes well with water and call for medical aid. Never forcibly detach nor rub eyes.
4. Do not use a textile glove in bonding work.
5. Use in a well ventilated room.
6. Handle in a place without flame.
7. Store away from children.

Made in Japan

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APPENDIX G

EPOXY RESIN FACT SHEET

(Parchem 2004)

Lokfix E

(formerly 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 Lokfix 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
- Excellent resistance to abrasion and impact
- Natural grey colour sympathetic to aesthetic requirements

Description

Lokfix 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

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

8/2003

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 Lokfix 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.

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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 Lokfix 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 Lokfix E has set.

Cleaning

All tools and equipment should be cleaned immediately after use with Solvent 10. Hardened material can only be removed mechanically.

Estimating

Supply

Lokfix 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 Lokfix 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

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

Precautions

Health and safety

Lokfix E & Solvent 10 are classed as hazardous under WorkSafe Australia guidelines. 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 - phone 13 11 26. 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 from your local Parchem sales office. Read MSDS, data sheet and label carefully before first use of any product.

Fire

Lokfix E is non-flammable.
Solvent 10 is flammable. Confined spaces must be well ventilated and no naked flame. Keep away from sources of ignition. No smoking. In the event of fire, extinguish with CO_2 or foam. Do not use a water jet.

Flash Point - Solvent 10: 27°C

Additional information

Lokfix E is only one product in the Parchem range of construction products.

parchem

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Email: technical@parchem.com.au
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7 days a week
Technical Support Hotline : 1800 812 864

Important note

AUS/0040/03A

FOSROC parchem

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

Parchem offers a comprehensive range of high performance, high quality construction products. In addition, Parchem 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 (BS 6319)	1 day	57MPa
	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 Parchem 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 acordance 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

Lokfix 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 Lokfix E hardener pack into the base container. Mix using a slow speed mixer until homogeneous.

Nitofill LV: 1/2003

FOSROC

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 Lokfix 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 Lokfix 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 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 Parchem sales office for further information.

Estimating

Nitofill LV resin:	450 ml pack (12 per carton)
Lokfix E:	1.5 and 6 litre packs
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, Lokfix E and Solvent 10 have a shelf life of 12 months if kept in dry conditions at 20°C.

Precautions

Fire

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

Health and safety

Nitofill LV resin and Lokfix 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.

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 from your local Parchem sales office. Read MSDS and product data sheet carefully before first use. In emergency, contact any Poisons Information Centre.

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Important note

Parchem Construction Products Pty Ltd 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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APPENDIX H

RISK ASSESSMENT & SAFETY ISSUES

This research project involves experimental research which is conducted within laboratories, therefore a considerable number of hazards exist. However any person undertaking work within the laboratories is required to have a sound knowledge of any perceivable hazards and should undertake their work in a manor that reduces any risks that may be involved. Below is a list of hazards and management principles that are specific to this experimental research.

DESCRIPTION OF HAZARD	PEOPLE AT RISK	INJURY TYPE OR CONSEQUENCE	RISK LEVEL	RISK MANAGEMENT
Handling concrete during construction	Up to 5 or more people in the vicinity during construction	Burns to skin and eyes	Slight – possible but unlikely	People in the vicinity should wear protective clothing.
Movement of headstocks	Anybody in the vicinity, generally 1 or 2 people	Crushed limbs due to slippage	Slight – possible but unlikely	Ensure all people are well clear of the headstocks whenever they are moved.
Use of power tools	Person using tool Surrounding people	Could injure any part of the body	Significant - possible	Most of the work conducted using power tools is done by trained staff. Any people in the vicinity must wear protective clothing.
Use of hand tools	Person using tool Surrounding people	Generally eyes, hands and limbs	Very slight – conceivable but unlikely	Person using tool must wear protective clothing.
Lifting heavy objects (eg. formwork, reinforcing)	Person doing the lifting	Generally back	Significant - possible	Person should be taught to lift with their knees.
Overhead weights (eg. Loading Frame)	Anybody near instrument	Crushing of body	Slight – possible but unlikely	Unauthorised people should stay well clear of instrument at all times. Authorised people should approach with caution when required and suitable. Nobody should approach instrument during testing.
Large forces in steel (eg. external post-tensioning)	Anybody in vicinity of materials	Server body damage	Significant - possible	Unauthorised people should stay well clear of instrument at all times. Authorised people should approach with caution when required.

DESCRIPTION OF HAZARD	PEOPLE AT RISK	INJURY TYPE OR CONSEQUENCE	RISK LEVEL	RISK MANAGEMENT
Use of adhesives (eg. epoxy, strain gauge adhesive)	Anybody in vicinity of materials	Respiratory problems Skin damage	Significant - possible	Suitable protect clothing must be worn. Trained personal must inform users of the dangers of any chemicals. MSDS must be addressed before material is used.
Use of solvent (eg. Cleaning equipment after epoxy)	Person using solvent People in vicinity	Skin damage Respiratory problems	Significant - possible	Suitable protect clothing must be worn. Trained personal must inform users of the dangers of any chemicals. MSDS must be addressed before material is used.
Removal of resins (eg removal of Lokfix E before testing)	Person undertaking the process	Respiratory problems	Significant - probable	Suitable PPE should be worn ie. Dust mask.
Loading of headstocks (eg. member may slip, member may fail abruptly)	Up to 8 to 10 people within the vicinity during testing	Server bodily injury	Significant - possible	All people should remain well clear during testing. Any people in the vicinity must wear suitable protective clothing.
Welding (eg. during reinforcing cage construction)	Generally 2 people in the vicinity of welder	Sever eye injury (eg. welding flash) Burns to body	Slight – possible but unlikely	Suitable protection clothing must be worn. Caution should be taken around hot materials.

PPE – Personal Protective Equipment.

Note – this is a comprehensive list of hazards that are involved with this experimental work, however some sources may have been overlooked.

APPENDIX I

RESOURCE ANALYSIS

This project involves experimental work, therefore a number of resources are required to undertake the necessary analysis. The table below identifies a majority to these resources and their current status.

Materials\Resources	***Notes***
Laboratories and Workshops	
Z1.101	*Laboratory attendant was approached and access has been granted for necessary work.* • Required for the construction of formwork • Required to tie steel reinforcing cages • Used as an area to pour the concrete
Z4	*Laboratory attendant was approached and access was granted for necessary work.* • Required for testing headstocks
Equipment and Machines	
60 Ton Hollow Core Hydraulic Jack	• Used to load the headstocks during testing. • Jack obtained from FCDD
Data Logger (System 5000)	• Used to log data transmitted by strain gauges • Availability needs to be determined
Concreting Equipment	• Minimal equipment is required • All equipment required was contained in Z1.101
Post-tensioning Jack	• Same jack used to load specimen also used for prestressing. • Jack obtained from FCDD
Forklift	• Required to move headstocks once cast • Glen Bartkowski organised the use of the forklift
Steel Tying Equipment	• Tools required such as nips and pliers • All required equipment contained in Z1.101
Soldering Iron	• Required to connect leads to strain gauges and to apply wax when fitting steel strain gauges • Available from Z1.101
Other Equipment to Fit Strain Gauges	• Requires equipment such as Stanley knife, angle grinder, etc. • All available from Z1.101
General Power Tools	• Such tools include angle grinders and drills • These were available from laboratory staff and would generally used by such people for tasks surrounding the project
General Hand Tools	• Such tools include hammers, sockets and spanners All were available from Z1.101
Gantry	• Small gantry was used to manoeuvre specimens into position before testing • Also used to relocate specimens before and after testing

MATERIALS\RESOURCES	*NOTES*
Materials and Other Items	
Formwork	• Was obtained and constructed by Glen Bartkowski (Z1.101 – Laboratory Attendant)
Reinforcing	• Quotations obtained from local suppliers • Reinforcing cages were constructed in Z1.101
Reinforcing Ties	• Ample stock in Z1.101
Concrete	• Quotations obtained by laboratory staff • Batches ordered as required
Strain Gauges and Adhesives	• Quotations have been obtained • Were fitted to reinforcing cage and face of concrete.
Post – tensioning System	*(Used system that was utilised in previous projects)* • Includes anchorages and post-tensioning bars • Examined and minimal modifications were required • Modifications undertaken by laboratory staff
Ferals	• Cast into concrete to support post-tensioning anchorage • Ample stock left from previous projects
Data Analysis Tools	• Microsoft Excel software package was used to undertake data analysis
Lifting Chains	• Required to move headstocks once cast • Organised by Glen Bartkowski
Lifting Eyes	• To be cast into headstock for lifting purposes • Ample stock left from previous projects
General Fasteners	• These include screws and nails that were required throughout the course of the project • All are available from Z1 Laboratory staff

Note – this is a comprehensive list of equipment and materials required to carry out the experimental work, however some items may have been overlooked.