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STRUCTURAL BEHAVIOUR OF COMPOSITE CONCRETE-STEEL SLABS

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SYNOPSIS

Reinforced concrete structural elements can be strengthened by bonding external steel plates onto the concrete surface by means of an epoxy adhesive and are referred to as composite structural members.

The objectives of this Master’s research project on the *Structural Behaviour of Composite Concrete-Steel Slabs* were as follows:

- To determine whether steel plates bonded externally to existing reinforced concrete structural elements can control the deflection
- To determine whether steel plates bonded externally to existing reinforced concrete structural elements can increase the flexural resistance
- To determine whether the various debonding mechanisms of the steel plate from the concrete surface of existing reinforced concrete structural elements can be theoretically predicted by existing theories.

Twenty-four reinforced concrete specimens were constructed using 25 MPa concrete and 5 Y12 internal bars in the longitudinal direction. The concrete and steel plate surfaces were prepared by scabbling and sandblasting respectively. The steel plates, which varied in thickness (from 6 mm to 8 mm), width (from 110 mm to 150 mm) and number (1 or 2), were bonded externally to the soffit of the concrete structural elements by means of epoxy glue. These composite concrete-steel specimens were simply supported and loaded until destruction by applying either a third-span line load (TSLL), mimicking a uniformly distributed load, or a mid-span line load (MSLL). The applied load, vertical deflection and strain on the bonded steel plates at mid-span were electronically logged.

The vertical deflection within the elastic range of the unplated and plated reinforced concrete structural elements was obtained using the double integration method. The cracked second moment of area ($I_{cr}$) was calculated by transforming the cross-sectional area of the steel reinforcement to an equivalent area of concrete. It was found that the deflection of composite concrete structural elements decreased as the cross-sectional area of the bonded steel plates increased. A comparison between the theoretically analysed and experimentally measured deflections was done as part of this research study.

The findings of this study indicate that steel plates externally bonded to reinforced concrete structural elements increase the flexural and shear resistance of the members. The externally bonded plates are not, however, enclosed by the concrete as in the case of internal reinforcement and are therefore not as well anchored. This results in premature debonding of the steel plate from the concrete surface. Two design philosophies for the theoretical prediction of the flexural resistance were considered. The first was the anchorage design philosophy developed in Europe (fib-14, 2001) where the tension face plates are terminated in the uncracked region, which is beyond or at least to the point of contraflexure. The second is the hinge design philosophy developed in Hong Kong and Australia (Handbook 305-2008, Standards Australia) whereby the tension face plates are terminated short of the points of contraflexure; this latter philosophy was adopted as the bonded steel plates stopped 250 mm short of the support, which is also the point of contraflexure for simply supported structural elements. The following debonding mechanisms due to flexure and shear were investigated:
- Intermediate crack debonding (IC), which is associated with the flexural behaviour of the structural element
- Critical diagonal crack debonding (CDC), which is associated with the shear capacity of the structural element
- Plate end debonding (PE), which is associated with the curvature of the structural element
- Interface shear stress \((V_{Ay}/Ib)\) debonding, which is associated with the tensile strength of the concrete.

Theoretical calculations indicate that a third-span line load will cause plate end debonding and that intermediate crack debonding will occur when composite members are subjected to a mid-span line load.
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NOMENCLATURE AND ABBREVIATIONS

\( A_p \) Cross-sectional area of the plate
\( A_r \) Cross-sectional area of reinforcement bars
\( A_{r(min)} \) Minimum cross-sectional area of longitudinal reinforcement
\( A_{r(prov)} \) Cross-sectional area of longitudinal reinforcement provided
\( b_c \) Width of the structural element
\( b_{c(w_{eff})} \) Effective width due to shear lag
\( b_f \) Width of the failure plane
\( b_p \) Width of the bonded plate
\( d_f \) Depth of the failure plane
\( E_c \) Elasticity modulus of concrete
\( E_{I_{cr.pl}} \) Flexural rigidity of the cracked plated cross-section
\( E_p \) Elasticity modulus of the plate
\( E_r \) Elasticity modulus of reinforcement bars
\( f'_c \) Cylinder compressive strength of concrete
\( f_{ct} \) Concrete tensile splitting strength
\( f_{cu} \) Concrete compressive strength
\( f_r \) Force in reinforcement bars
\( f_{y} \) Stress in reinforcement bars
\( f_{yc} \) Yield stress of steel at compression
\( I_{cr} \) Cracked transformed second moment of area for a reinforced concrete beam
\( L \) Span length of beam
\( L_b \) Minimum bond length from the “crack face”
\( L_{per} \) Perimeter length of the cross-section of the failure plane
\( [(M_{PE})_{fp}]_{ch} \) Moment due to plate end debonding of the tension face plate
\( M_{plate} \) Moment resistance of a plated structural element
\( N_{plate} \) Normal interface tensile and compression forces in bonded plates
\( P \) Applied point load
\( P_{anal} \) Theoretically analysed point load
\( P_{appl} \) Experimentally tested point load
\( P_{IC} \) Axial force causing debonding of the plate
\( P_{ICbeam} \) Axial debonding force of a plate externally bonded to a beam with a single crack
\( P_{ICpp} \) Axial debonding force of the plate in a push-pull test
\( [(P_{IC})_{hp}]_{EB} \) Axial debonding force of an externally bonded plate in a push-pull test
\( P_{plate} \) Axial force in the plate
\( T_{adj} \) Distance between plates
\( T_{edge} \) Distance of bonded plate from edge of structural element
\( t_p \) Thickness of the bonded plate
\( t_{fp} \) Thickness of the tension face plate
\( V_{plate} \) Shear force transferred by the plate
\( V_{uc} \) Shear force capacity of reinforcement exceeded
\( W \) Uniformly distributed load
\( Y \) Vertical deflection
Greek letters

$\alpha_{\text{EB}}$  Constant (mean is 0.427) (95 percentile value is 0.315)
$\gamma_m$  Material safety factor of steel
$\delta$  Interface slip between concrete and plate
$\delta_f$  Interface slip between concrete and plate when debonding occurs
$\Delta V_{uc}$  Increase in the shear capacity of the reinforced concrete structural element
$\varepsilon_c$  Crushing strain of concrete (0.0035)
$\varepsilon_{c0}$  Plastic strain of concrete
$\varepsilon_{cu}$  Beam strain at failure
$\varepsilon_{db}$  Plate debonding strain
$\varepsilon_{\text{frac}}$  Plate fracturing at a strain
$\varepsilon_{IC}$  Intermediate crack strain
$(\varepsilon_{\text{pivot}})_{\text{res}}$  Strain difference between the plate and the adjacent concrete surface
$\varepsilon_r$  Reinforcement strain
$\varepsilon_{\text{rebar}}$  Reinforcement bars fracturing strain
$\tau_f$  Maximum interface slip
Chapter 1  Background and Introduction

1.1  Background

Reinforced concrete structural elements sometimes need to be strengthened due to (1) construction errors whereby too few reinforcement bars were placed in the concrete, or (2) when the maintenance of a reinforced concrete structure has been neglected, which can result in the erosion of the reinforcing bars, or (3) when there has been a change in the function of the structure whereby the applied load has been increased.

The choice between replacing or strengthening a reinforced concrete structure depends on factors such as the cost of construction materials, the cost of labour, the duration of the construction work, disruption of other facilities and environmental aspects.

A method for strengthening an existing reinforced concrete structural member is to bond steel plates to the tension face of the flexural member by means of adhesive materials. With the development of strong structural adhesive materials (e.g. epoxy) in recent years, the composite steel plate-bonding technique is now recognised as an effective, convenient and economical method for increasing the load-bearing capacity of structural elements. The advantages of this bonding technique are the relative simplicity of the application, the speed of construction and the small resulting changes in structural weights and sizes.

1.2  Purpose of the study

This method of bonding steel plates to the tension face of existing reinforced concrete flexural members by means of epoxy glue in order to increase the load-bearing capacity is not described in either of the South African National Standards:

- The Structural use of Steel, SANS 10162-1:2005
- The Structural use of Concrete, SANS 10100-1:2000.

The purpose of this study is therefore to document the structural behaviour of composite concrete-steel structural elements whereby composite action in the reinforced concrete slabs is achieved by bonding steel plates to the tension face of the concrete slab using epoxy glue.

This technique requires trustworthy information on the adhesion of steel to concrete and a sound understanding of the

- bonding process of the steel plate to the existing reinforced concrete structural member
- stresses and strains that will develop within the composite structural element
- limits within which this strengthening method can be used.

The civil engineering industry will benefit from this research project as an economical design method will be described on how to increase the load-bearing capacity of existing reinforced concrete beams and slabs by bonding steel plates to the concrete surface.
1.3 Research objectives

The objectives of this research project are:

- To determine whether steel plates bonded externally to existing reinforced concrete structural elements can control the deflection
- To determine whether steel plates bonded externally to existing reinforced concrete structural elements can increase the load-bearing resistance
- To determine whether the various debonding mechanisms of the steel plate from the concrete surface of existing reinforced structural elements can be theoretically predicted by existing theories.

The above research objectives were tested by comparing the experimentally applied loads and measured deflections with the theoretically predicted loads and deflections.

1.4 Brief overview of chapters

The research study is divided into seven chapters and an annexure (Chapter 8).

Chapter 1: Background and Introduction

This chapter describes the reason for creating composite concrete-steel structural elements, the purpose of this research study and the research objectives.

Chapter 2: Literature Review

Chapter 2 deals with research previously undertaken on the strengthening technique of bonding steel plates externally to existing reinforced concrete structural members to create composite concrete-steel structural members.

Chapter 3: Methodology

This chapter describes how the reinforced concrete slabs used for this research study were constructed, the process followed to glue the steel plates to the surface of the concrete and the equipment used for testing the composite concrete-steel structural elements.

Chapter 4: Analysis

This chapter lists the material properties of the concrete, steel reinforcement and steel plates used to construct the composite concrete-steel structural elements. The various debonding mechanisms of the steel plate from the concrete surface are described. The analysis methods for calculating the deflection and the flexural capacity of both the unplated reinforced concrete structural elements and the plated reinforced concrete structural elements are also presented in this chapter.

Chapter 5: Findings and Comparisons

This chapter presents the findings of this research study and compares the experimentally measured deflections and applied loads with the theoretically predicted deflections and loads.
Chapter 6: Conclusions

Chapter 6 gives the conclusions reached and proposes various options for further research in this field.

Chapter 7: References

Chapter 7 lists all the references used in undertaking this research study.

Annexure (Chapter 8)

The annexure gives the experimental results (in graphic form) of both the unplated and plated reinforced concrete structural elements.
Chapter 2  Literature Review

2.1  Introduction

This chapter deals with research previously undertaken on the strengthening technique of bonding steel plates externally to reinforced concrete flexural members to create composite concrete-steel structural members.

The earliest publications found on this plate-bonding technique were by the following three researchers:

- Flemming and King (1967) described this method of strengthening beams when reinforced bars had inadvertently been omitted due to faulty construction, resulting in excessive cracking upon loading, and its use for strengthening reinforced concrete beams when necessary due to increased loads which had not been foreseen in the design stages.
- Lerchenthal (1967) described the bonding of external steel reinforcement to existing reinforced concrete structural elements.
- Kajfasz (1986) presented a paper at the RILEM International Symposium on the adhesion between polymers and concrete.

This literature review is divided into topics, all related to the structural behaviour of composite concrete structural members.

2.2  Bonding techniques

The surface preparation of both the concrete and the steel plates is vital in order to transfer the forces between the two materials (steel and concrete), thus enabling composite action.

2.2.1  Concrete surface preparation

Flemming and King (1967) considered the soundness of the concrete surface. Experimental work was undertaken with regard to the cleaning and preparation of the concrete surface onto which the steel plates would be bonded. It was found that shutter oils penetrate more than 1 mm into the concrete and would present inadequate bonding. Mechanical cleaning of the contaminated surface proved to be a better solution than chemical cleaning.

Four methods of mechanical cleaning were investigated. Of these four, chipping the concrete surface with a blunt tool proved to be the most effective. Similar cleaning undertaken with sharp-pointed tools gave inferior results.

The results produced by sandblasting tended to be variable, particularly when metamorphosed aggregates were used in the concrete. Wire brushing of green concrete was only satisfactory when undertaken by skilled personnel who took particular care to avoid loosening the large aggregate. Wire brushing of a cured concrete surface only served to remove loose surface contaminants and did not yield good results.

Jones et al. (1980) prepared the concrete surface by abrading the face with a disk grinder to remove all the laitance and expose the aggregates. It was then mechanically wire brushed to
remove all loose particles. Finally, the beam was sanded with 100 grid emery cloth. All remaining dust and debris were removed by blowing with an inert gas.

Macdonald (1982) prepared the concrete surface ready to receive the steel plate by sandblasting using chilled cast iron grit. The surface of the concrete was removed until the large aggregate was revealed and a coarse surface was provided without undercutting the large aggregate.

### 2.2.2 Steel plate surface preparation

Flemming and King (1967) stated that the steel plates used were sandblasted in all tests performed in order to remove the mill scale.

Swamy et al. (1987) prepared the steel plates by first degreasing them, then grid blasting the surface with 34 µm mean size steel grid. The surfaces were protected from corrosion and contamination until bonding had taken place.

Jones et al. (1980) described the surface preparation of the steel plates used for their research as grid blasting to a uniform grade. The glue was applied to both the steel plate and the prepared surface of the concrete beam.

As the literature did not provide evidence of other surface preparation techniques (apart from sandblasting) having been used in previous research, sandblasting was used in this study to provide a rough surface capable of providing mechanical grip to the epoxy.

### 2.3 Epoxy thickness to be used for steel-to-concrete bonding

In order to ensure a proper bond between the steel plate and the concrete surface, the thickness of the adhesive (epoxy) was considered. Swamy et al. (1987) indicated that the first visual crack load increases with as thin a glue thickness as possible for a 6.0 mm plate thickness as indicated in Figure 2.1. As this research study was done with 6 mm and 8 mm steel plates, the glue thickness was kept as thin as possible in order to obtain a good bond between the steel plate and the concrete surface.
Jones et al. (1980) described a method for determining the shear stress with different glue thicknesses for the lapping of steel plates (see Figure 2.2). Although no lapping of steel plates was performed during this research, it was interesting to note that the results obtained confirm those of Swamy et al. (1987), namely that the best shear stress transfer was obtained from as thin a glue layer as possible.
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Figure 2.2: Double lap shear test specimens and test results (Jones et al., 1980)

The uneven scabbed surface of the concrete provided good mechanical grip for the epoxy. For the purpose of this research study, the epoxy thickness was kept to a minimum in order to obtain the best bond as found by other researchers.

2.4 Debonding design philosophies

Two distinct design philosophies have been developed, namely the anchorage design and the hinge design. These are discussed below.

2.4.1 Anchorage design philosophy

The anchorage design philosophy was developed in Europe (fib-14, 2001), whereby the tension face plates are terminated in the uncracked region. This implies that the bonded plates should extend at least to the point of contraflexure, as shown in Figure 2.3.
Due to practical considerations, the bonded steel plates terminate short of the supports for this research study. As the concrete slabs were tested as simply supported, the points of contraflexure coincide with the supports and therefore the anchorage design philosophy was not applicable to this research study.

### 2.4.2 Hinge design philosophy

The hinge design philosophy was developed by researchers from Hong Kong and Australia (HB 305-2008, Oehlers et al., 2008), whereby the tension face plates are terminated short of the points of contraflexure as indicated in Figure 2.4. This design philosophy was used for this research study as the steel plates stopped 200 mm short of the supports, i.e. the points of contraflexure, of the simply supported slabs.

![Hinge design philosophy](image)

**Figure 2.4:** Hinge design philosophy (Oehlers et al., 2008)
2.5 Debonding mechanisms in adhesively bonded plates

The steel plate bonded to the concrete surface of the structural element is not fully enclosed by the concrete as in the case of reinforcement bars and is therefore not as well anchored. The result is that premature separation of the steel plate from the concrete surface can occur before the composite concrete structural element reaches its full flexural strength. Oehlers et al. (2004) recommend that the tensile strength of the epoxy adhesive used to bond a plate to the concrete structural element should be much stronger than that of the concrete so that debonding or peeling occurs within the concrete element.

The various debonding mechanisms in adhesively bonded plates are described by Oehlers et al. (2004) and are discussed below (refer to Figure 2.5).

An intermediate crack (IC) is any crack that intercepts the longitudinally bonded plates. When such an IC is widened due to flexural deformations, it can cause the plate to debond from the concrete surface. This is referred to as “IC debonding” and is discussed in Section 2.5.1.

The critical diagonal crack (CDC) is commonly associated with shear deformation. When the CDC is widened due to shear deformation, it can cause the plate to debond from the concrete surface. This is referred to as “CDC debonding” and is discussed in Section 2.5.2.

As the curvature of a composite concrete structural element increases, the bonded plate tries to remain straight, which causes debonding cracks to start at the plate end (PE) and propagate inwards. This is referred to as “PE debonding” and is discussed in Section 2.5.3.

If the interface shear stresses between the plate and the concrete are exceeded, this can cause tensile failure of the concrete. This is referred to as “interface shear” or $\frac{V_{Ay}}{I_b}$ debonding. This debonding mechanism is discussed in Section 2.5.4.

The above-mentioned cracks are illustrated in Figure 2.5. They apply to all forms of plating and to all material types of the bonded plate.
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2.5.1 Intermediate crack (IC) debonding

Oehlers & Seracino (2004) report that the IC, shown in Figure 2.6, can be any type of crack that intercepts the longitudinally bonded plate, for example flexural cracks, flexural-shear cracks and critical diagonal cracks. The widening of these ICs is due to flexural or shear deformations.

![Diagram of Intermediate Crack Debonding](image)

**Figure 2.5:** Cracking of longitudinal tension face plates (Oehlers et al., 2004)

When any IC, shown vertically in Figure 2.6, intercepts a bonded plate, localised debonding accompanied by interface slip (δ) occurs at the concrete/plate interface. If the IC interface cracks, shown horizontally, are small, they have little effect on the structural behaviour of the plated concrete. If these IC interface cracks are allowed to elongate, they can join together and cause the plate to debond at an axial force $P_{IC}$ or an axial strain $\varepsilon_{IC}$, which is referred to as the IC debonding resistance.

The IC debonding resistance ($P_{IC}$) therefore depends on the interface bond/slip characteristics ($\tau/\delta$) between the concrete and the bonded plate, which are shown in Figure 2.7. The peak
interface shear stress is indicated as $\tau_f$ and the maximum interface slip that resists shear beyond which debonding occurs is indicated as $\delta_f$.

**Figure 2.7:** Fundamental interface bond/slip characteristics (Oehlers et al., 2004)

These material interface characteristics $\tau_f$ and $\delta_f$ are derived directly from the restrained push-pull tests as shown in Figure 2.8, where:

- $t_p$ is the thickness of the bonded plate
- $A_p$ is the cross-sectional area of the bonded plate
- $P$ is the applied force
- $L$ is the bonded length of the plate from the crack face to the plate end
- $\delta$ is the increased elongation

**Figure 2.8:** IC Interface crack single-shear push-pull test (Oehlers & Seracino, 2004)

The maximum IC debonding resistance in a push-pull test $P_{IC,pp}$ represents the debonding resistance in a beam between two cracks and is described by the following two researchers:

- Seracino et al. (2007)
- Teng et al. (2002)

A cross-sectional view of the push-pull test is shown in Figure 2.9, where:

- $b_p$ is the plate width
- $t_p$ is the plate thickness
- $d_f$ is the depth of failure plane
- $L_{per}$ is the perimeter length of the cross-section of the failure plane

$$L_{per} = b_p + 1\ mm + 1\ mm + d_f + d_f.$$
To achieve the maximum debonding resistance \( (P_{IC})_{pp} \) requires a minimum bond length from the crack face as shown in Figure 2.10. Seracino et al. (2007) assumed that the variation of the IC debonding resistance is linear, reaching a maximum of \( (P_{IC})_{pp} \) at a length of \( L_b \) and remaining constant at \( (P_{IC})_{pp} \) if the length exceeds \( L_b \), as indicated in Figure 2.10.

\[
(P_{IC})_{pp} = \sqrt{\tau_f \delta_f} \sqrt{L_{per} E_p A_p} \text{ (N)}
\]  
(2.5-1)

where:
- \( L_{per} \) is the perimeter length of the cross-section of the failure plane (mm) as shown in Figure 2.9
- \( E_p \) is the elasticity modulus of the plate (MPa)
- \( A_p \) is the cross-sectional area of the plate (mm\(^2\))
- \( \tau_f \delta_f \) is as per the formula below

\[
\tau_f \delta_f = 0.73 \left( \frac{d_f}{b_f} \right)^{0.5} \left( f'_c \right)^{0.67}
\]  
(2.5-2)

where:
- \( d_f \) is the depth of the failure plane (mm) as per Figure 2.9
- \( b_f \) is the width of the failure plane (mm) as per Figure 2.9
- \( f'_c \) is the cylinder compressive strength of concrete (MPa)

The mean value is 0.853 and the 95 percentile value is 0.73.
To achieve the maximum basic debonding resistance \((P_{ic})_{pp}\), Seracino et al. (2007) proposed a minimum bond length from the “crack face” as:

\[
L_b = \frac{\pi}{2} \left( \frac{\tau_f \delta_f}{\sqrt{\tau_f \delta_f \varepsilon_p}} \right) \text{ (mm)}
\]  
\[\text{(2.5-3)}\]

where:
\(\tau_f\) is as per formula (2.5-4)
\(\delta_f\) is as per formula (2.5-5)

\[
\tau_f = \left( 0.8 + 0.078 \frac{d_f}{b_f} \right) (f'_c)^{0.6}
\]  
\[\text{(2.5-4)}\]

\[
\delta_f = \frac{0.73 \left( \frac{d_f}{b_f} \right)^{0.5} (f'_c)^{0.67}}{\tau_f}
\]  
\[\text{(2.5-5)}\]

The stress in the bonded plate at debonding is:

\[
(\sigma_{IC})_{pp} = \frac{(P_{IC})_{pp}}{A_p}
\]  
\[\text{(2.5-6)}\]

where:
\(A_p\) is the cross-sectional area of the plate

The strain in the bonded plate at debonding is:

\[
\varepsilon_{db} = \frac{(\sigma_{IC})_{pp}}{E_p}
\]  
\[\text{(2.5-7)}\]

where:
\(E_p\) is the elasticity modulus of the plate

Teng et al. (2002) proposed the following IC debonding resistance for push-pull tests:

\[
(P_{IC})_{pp} = \alpha_{EB} \beta_p b_p \sqrt{E_p t_p \sqrt{f'_c}}
\]  
\[\text{(2.5-8)}\]

where:
\(\beta_p\) is as per formula (2.5-7)
\(b_p\) is the width of the bonded plate (mm)
\(t_p\) is the thickness of the bonded plate (mm)

The mean value is of \(\alpha_{EB}\) is 0.427 and the 95 percentile value is 0.315.

\[
\beta_p = \frac{\sqrt{2-b_p/b_c}}{\sqrt{1+b_p/b_c}}
\]  
\[\text{(2.5-9)}\]

where:
\(b_c\) is width of the concrete structural element
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For the purpose of this research study, equation 2.5-10 is used which is a revision of equation 2.5-8.

\[
(\sigma_{IC})_{\text{beam}} = \alpha_{EB} \beta_p \beta_L \sqrt{\frac{E_P f_{IC}}{t_p}} \quad \text{(N and mm)} 
\]

(2.5-10)

where:

- \( \beta_b \) is as per equation 2.5-9
- \( \beta_L \) is as per equation 2.5-11

\[
\beta_L = \begin{cases} 
1, & \text{if } L_b \geq L_e, \text{ see Figure 2.10 where } l_b \text{ and } L_e \text{ are described} \\
\sin \left( \frac{\pi L_e}{2L_b} \right), & \text{if } L_b < L_e 
\end{cases} 
\]

(2.5-11)

It is suggested that all plates should be designed as fully anchored \( (l_b > L_e) \), as is the case for this research study. Therefore \( \beta_L \) is taken as 1.

The strain in the bonded plate at debonding is:

\[
\varepsilon_{db} = \frac{(\sigma_{IC})_{\text{beam}}}{E_P} 
\]

where:

- \( E_P \) is the elasticity modulus of the plate

If the intermediate crack, as described above, widens due to flexural deformation and causes debonding, this is referred to as “flexural intermediate crack (FIC) debonding”. Figure 2.11, from HB 305-2008 (Oehlers et al., 2008), indicates where these cracks would appear on a continuous beam.

![Figure 2.11: FIC debonding in a continuous beam (HB 305-2008, Oehlers et al., 2008)](image-url)
The plate length ($L$) taken from the position of maximum moment, as shown in Figure 2.11, is usually larger than the minimum bond length ($L_b$) as proposed by Seracino et al. (2007). The FIC debonding resistance of a composite concrete-steel beam ($P_{IC}$) is therefore greater than or equal to the basic IC debonding resistance ($P_{ICpp}$) proposed by Seracino et al. (2007) and Teng et al. (2002). At the support of a continuous beam, where a steep moment gradient occurs, only one flexural crack will appear; the debonding resistance ($P_{ICbeam}$) is therefore equal to the basic debonding resistance ($P_{ICpp}$). In a continuous beam, the moment between the maximum negative, at the support, and the maximum positive, at mid-span, the gradient reduces, allowing more than one flexural or flexural-shear cracks. The FIC debonding resistance then generally increases and ($P_{ICbeam} > (P_{ICpp}$). In the constant moment regions of the continuous beam at a mid-span, where the maximum positive bending moment occurs, the FIC debonding resistance reduces back to that of a beam with a single crack and ($P_{ICbeam} = (P_{ICpp}$).

For epoxy-bonded (EB) plates, Teng et al. (2002) propose the following formula for FIC debonding resistance:

$$
(P_{IC})_{beam} = \alpha_{EB} \beta_p b_p \sqrt{E_p t_p \sqrt{f'_c}}
$$

(2.5-8)

The mean value is 0.887 and the 95 percentile value is 0.379.

$$
\beta_p = \sqrt{\frac{2-\frac{b_p}{b_c}}{1+\frac{b_p}{b_c}}}
$$

(2.5-9)

### 2.5.2 Critical diagonal crack (CDC) debonding (Oehlers & et al., 2004)

If the critical diagonal crack (CDC), caused by shear deformation, is widened due to aggregate interlock and causes the plate to debond, it is referred to as “CDC debonding”, as illustrated in Figure 2.12 (Oehlers & Seracino, 2004). The CDC causes the plate to debond from the root of the crack at B towards the plate end at C. As with all shear failures, this failure mode is brittle without prior warning. According to Oehlers & Seracino (2004), tests on plated beams have shown that the presence of stirrups does not affect the shear load that causes debonding. This is because the stirrups that cross the diagonal crack have to be stretched before they can contribute to the shear strength. Steel plates, however, are fairly rigid and debond as soon as the sliding action occurs.
The direct approach of Oehlers & Seracino (2004) determines the increase in the concrete structural element’s shear capacity $\Delta V_{uc}$ according to equation 2.5-10. This approach can only be used in the plated region of the concrete structural element (negative or positive moment) where the shear forces are capable of forming a CDC, i.e. where $V_{uc}$ is exceeded. For example, it can be used if the longitudinal plates are extended up to the points of contraflexure or to a region where the transverse shear force is low and beyond an anchorage length.

The increase in the shear capacity of the concrete structural element is:

$$\Delta V_{uc} = 0.15 \sum_{i=1}^{n} P_{plate}$$  \hspace{1cm} (2.5-10)

For each individual plate, $P_{plate}$ is the lesser of the yield strength, fracture strength or basic IC debonding resistance.

### 2.5.3 Plate end (PE) debonding

According to Oehler & Seracino (2004), the ability to transfer forces ($V_{plate}$) in the longitudinal tension face plate is governed by the PE debonding resistance. PE debonding is often associated with the interface normal stresses that are induced by the curvature of the plate.

The PE debonding mechanism is illustrated for a tension face plate in Figure 2.13. As the curvature on the composite structural element increases, as in Figure 2.13, a moment ($M_{plate}$) and an axial force ($P_{plate}$) are induced. The plate tries to stay straight, in contact and at its original length, which induces normal interface tensile and compressive forces ($N_{plate}$). This, in turn, may result in interface cracks spreading inwards, causing the plate to debond.
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Figure 2.13: Plate end (PE) debonding mechanism (Oehlers & Seracino, 2004)

The PE debonding starts at the stress concentration at the plate end, due to the plate discontinuity, as shown in Figure 2.13. The plate strain at the plate end is zero and the debonding interface crack then propagates inwards. PE debonding can be prevented by terminating the plate at the point of contraflexure where the curvature is low, or on the compression face in a continuous concrete structural element. For externally bonded tension face plates, the moment at the plate end that will cause PE debonding is given by the following capacity:

\[
(M_{PE})_{tfp}^{ch} = \frac{0.53(EI)_{cr.pl}f_{cb}}{0.474E_p tfp} \tag{2.5-11}
\]

where:
- \((EI)_{cr.pl}\) is the flexural rigidity of the cracked plated cross-section adjacent to the plate end but on the plated side. EI should be calculated for the short-term loads and is based on a cracked section.
- \(f_{cb}\) is the split tensile strength of the concrete
- \(E_p\) is the elasticity modulus of the plate (MPa)
- \(tfp\) is the thickness of the tension face plate

2.5.4 Interface shear stress \((VAy/Ib)\) debonding

The interface shear stress between the plate and the concrete structural element can be derived from structural mechanics using the well-known equation \(VAy/Ib\), which depends on the vertical shear force \((V)\) at a section of a beam. Oehlers & Seracino (2004) tested many plated beams and none failed by this mechanism. There is, however, a possibility that the interface shear stress \(VAy/Ib\) may cause debonding when thick plates are used, for instance if plating is used at serviceability levels to reduce deflections. The \(VAy/Ib\) shear stress may cause debonding in plated prestressed concrete structural elements as the prestress force will prevent the formation of intermediate and critical diagonal cracks and delay both IC and CDC debonding. Oehlers & Seracino (2004) suggest that it is good practice to check the \(VAy/Ib\) shear stress and to restrict it to less than the tensile strength of the concrete.
2.6 Flexural capacity of adhesively bonded plates

Oehlers & Seracino (2004) describe the flexural analysis of unplated and plated reinforced concrete structural elements: the plated elements could be propped or unpropped. In a propped structure the stresses are removed prior to plating, whereas an unpropped structure is plated while carrying its self-weight and some applied loads – hence the stresses prior to bonding must be taken into account. These three methods are described in detail in Chapter 4. The failure mechanism of the structural element is one of the following, whichever occurs first:

- Crushing of concrete at a strain ($\varepsilon_c$) of 0.0035
- Plate debonding at a strain ($\varepsilon_{db}$)
- Plate fracturing at a strain ($\varepsilon_{frac}$)
- Reinforcement bars fracturing at a strain ($\varepsilon_{rebar}$)

The plate debonding strain ($\varepsilon_{db}$) is calculated using the following three theories:

- Hinge approach, Seracino et al.’s (2007) push-pull theory
- Hinge approach, Teng et al.’s (2002) flexural theory
- Hinge approach, EB steel plates

2.7 Limiting plate thickness and width

According to Swamy et al. (1987), there is a limiting overall plate thickness that would prevent premature debonding of the steel plate from the concrete surface prior to yielding. In order to ensure ductile behaviour and flexural failure, the following must be adhered to:

- The bond stresses at the plate/glue/concrete interfaces should be significantly small so that neither peeling nor tearing of the plates occurs under high loads. These bond stresses are influenced by both the width ($b$) and the thickness of the plate ($t$). Beams with plates (125 mm x 1.5 mm) $\frac{b}{t} = 83.3$ all failed in flexure without any bond distress. Beams with plates (125 mm x 3 mm) $\frac{b}{t} = 41.7$ all showed shear/bond failure, although they achieved nearly all of the theoretical flexural strength. It therefore appears that a plate width/thickness ratio of not less than 50 should be used.
- Even if the plates satisfy the above requirement, the plate thickness limitation must also be applied as it is the increase in the cross-sectional area of the steel plate that transforms a conventionally designed under-reinforced beam into an over-reinforced member.

The results of Swamy et al. (1987) show that beams with 125 mm x 6 mm plates failed prematurely, achieving only about 70% of the theoretical flexural strength as a result of debonding, and exhibited sudden, brittle failure with little ductility. Such a failure mode is structurally undesirable.

Macdonald (1982) selected $\frac{b}{t}$ values to achieve a reasonable range and a cross-sectional area of the plate similar to the cross-sectional area of the internal reinforcement. The resulting plate sizes and other sectional properties are given in Table 2.1.
Table 2.1: Variations in plate geometry (Macdonald, 1982)

<table>
<thead>
<tr>
<th>Test number</th>
<th>Plate size (mm)</th>
<th>b/t</th>
<th>Area plated (mm²)</th>
<th>Area reinforcement (mm²)</th>
<th>Area plated/area reinforced</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>57 x 4.75</td>
<td>12</td>
<td>270.75</td>
<td>226.20</td>
<td>1.20</td>
</tr>
<tr>
<td>6</td>
<td>85 x 3.60</td>
<td>24</td>
<td>306.00</td>
<td>226.20</td>
<td>1.35</td>
</tr>
<tr>
<td>11</td>
<td>123 x 2.14</td>
<td>57</td>
<td>263.22</td>
<td>226.20</td>
<td>1.16</td>
</tr>
<tr>
<td>10</td>
<td>150 x 1.06</td>
<td>142</td>
<td>159.20</td>
<td>226.20</td>
<td>0.70</td>
</tr>
</tbody>
</table>

The deflection results for plated beams with different b/t plate dimensions are shown in Figure 2.14.

![Load-deflection curves for 3.5 m plated beams, one adhesive, variations in plate geometry (Macdonald, 1982)](image)

**Figure 2.14:** Load-deflection curves for 3.5 m plated beams, one adhesive, variations in plate geometry (Macdonald, 1982)

The proportions of the plates, although of similar cross-sectional area and providing similar increases in maximum loads, also had a marked effect. Narrow plates permitted the development of sufficient horizontal shear stress to cause a concrete failure, whereas wide plates remained bonded to the beam at the maximum sustained load. Wide plates not only reduced the horizontal shear stress in the concrete, but also reduced peel stresses due to their lower inherent stiffness. The optimum width/thickness ratio \( \frac{b}{t} \) was about 60.

Huovinen (1997) found that reinforced concrete beams with bonded steel plates constitute an effective and economical method of strengthening concrete beams and slabs. The method can
also be used for steel beams. The test results showed that the ratio between the width \( b \) and thickness \( t \) of the bonded plate should be greater than \( \frac{b}{t} = 20 \).

### 2.8 Full-scale tests on composite reinforced concrete structures

The increase in the flexural capacity of full-scale structural elements strengthened with externally bonded steel plates was demonstrated by Taljsten (1994) who reported that an existing portal frame bridge with a free span of 21 m, located in Stora Hoga 40 km south of Gothenburg, Sweden, had to be demolished due to the building of a new highway. The bridge was tested before demolition in order to check how the ultimate shear capacity compared with theoretical calculations and code provisions. Due to the design of the bridge, bending failure would occur before a shear failure and thus the bending capacity had to be increased if a shear failure was to be obtained. For this reason the bridge was strengthened for bending with externally bonded steel plates. Theoretical calculations showed that the bending strength of the bridge had to be increased from 3.1 MNm to 11.7 MNm. Twelve steel plates with a nominal tensile strength of 600 MPa and a cross-section of 6 x 260 mm were used for strengthening.

During testing the bridge deck showed extensive cracks on the tensile side (underside). The initial cracks due to bending did not propagate because the steel plates kept the concrete together. The interface between the steel plates and the concrete showed no signs of debonding before large combined bending and shear cracks appeared and the bridge deck collapsed.

### 2.9 Concluding summary

The anchorage design philosophy was developed in Europe (fib-14, 2001), whereby the tension face plates are terminated in the uncracked region beyond the points of contraflexure. The hinge design philosophy was developed in Hong Kong and Australia (HB 305-2008), whereby the tension face plates are terminated short of the points of contraflexure.

The four debonding mechanisms in adhesively bonded plates described in this chapter are:
- Intermediate crack debonding (IC)
- Critical diagonal crack debonding (CDC)
- Plate end debonding (PE)
- Interface shear stress debonding.
Chapter 3  Methodology

3.1  Introduction

The research specimens constructed were reinforced concrete slabs 4 800 mm long, 1 000 mm wide and 125 mm thick. Some specimens were kept as is to act as control slabs, and steel plates were bonded to the other specimens to create composite concrete-steel slabs. These specimens were constructed to large sizes in order to represent actual slab elements.

The construction procedure for the composite concrete-steel slab specimens prepared in the laboratory was kept the same as on a building site and was as follows:

- Setting up the formwork
- Fixing the reinforcement bars and placing the mat into the formwork
- Casting the ready-mixed concrete into the prepared formwork
- Sandblasting the surfaces of the steel plates ready for bonding
- After 28 days, when the concrete had reached adequate strength, removing the slabs from the formwork and crushing the test cubes to determine the concrete strength
- Preparing the concrete slab surface by scabbling, ready for bonding with primer
- Bonding the steel plates to the concrete slab surface using epoxy glue
- Testing of the composite concrete-steel slabs.

3.2  Construction of the research specimens

3.2.1  Formwork used to construct research specimens

The formwork used to construct the research specimens was reusable steel shutters which allowed multiple slabs to be cast with the same dimensions. The dimensions of the formwork were 4 800 mm long, 1 000 mm wide and 125 mm deep. Before new reinforced concrete specimens were cast, the shutters were oiled for easy releasing of the formwork. This applied shutter oil has no effect on the bonding between the steel plate and the concrete as the cement laitance was removed by scabbling, as described in Section 3.2.5, to a sufficient depth to expose the well-bonded large aggregate.

3.2.2  Steel reinforcement placed in research specimens

The steel reinforcement used to construct the research specimens was high-tensile bars with a minimum yield stress of 450 MPa. The minimum allowable cross-sectional area of longitudinal reinforcement \( A_{s(min)} \) used in reinforced concrete slabs, as per SANS 10100–1, is 0.13% of the slab’s cross-section.

\[
A_{s(min)} = \frac{0.13}{100} \times 1\,000 \times 125 = 162.5\,\text{mm}^2
\]

The cross-sectional area of the longitudinal reinforcement provided \( A_{s(prov)} \) was five Y12 bars spaced at 200 mm intervals \((565\,\text{mm}^2)\), which is more than the minimum allowable cross-sectional area \( A_{s(min)} \) of 162.5 mm². The transverse reinforcement used was Y12 bars spaced at 100 mm intervals. The cover to the longitudinal reinforcement bars was 25 mm. Figure 3.1 shows the formwork with the reinforcement mat in place before the concrete was poured.
Figure 3.1: Steel shutters with reinforcement ready for concrete to be poured

3.2.3 Concrete used to construct research specimens

The research specimens were constructed using ready-mixed concrete to ensure consistency. The concrete specifications for the research specimens were kept the same as for most commercial structures. The specified concrete strength ($f_{cu}$) for all research specimens was 25 MPa with a 75 mm slump. The concrete strength ($f_{cu}$) was determined by crushing 100 mm test cubes according to SANS 5861-3: 2006. Figure 3.2 shows the truck delivering the ready-mixed concrete, with the completed formwork ready to receive the concrete in the foreground.
Figure 3.2: Ready-mix truck with 4 m³ of concrete

Figure 3.3 shows the concrete being poured into the formwork and how it was vibrated.
3.2.4 Steel plates used to construct research specimens

Grade 350W steel plates were bonded to the soffit of the research specimens. The cross-sectional dimensions of the steel plates were either 6 mm thick and 110 mm wide, or 8 mm thick and 150 mm wide. The steel plates were 4 000 mm long. As the research specimens were 4 800 mm long, the steel plates stopped 400 mm short from the end of the specimen, allowing space for supports.

The epoxy manufacturer, StonCor Africa, specifies that the surface of the steel plates must be prepared as described below when using their Pro-Struct 617NS non-sag epoxy. The mild steel plates must be dry grit blasted (sandblasted) to a white metal finish according to ISO 8501 SA 21/2 to obtain a 100–140 µm blast profile. The plates must be bonded on the same day that the sandblasting has been done in order to prevent any rust from forming on the steel surface which could weaken the bond between the epoxy and the steel plate.

3.2.5 Concrete surface preparation of the research specimens

According to StonCor Africa’s specifications, the concrete surface must be prepared as described below before applying their Pro-Struct 618LV primer and Pro-Struct 617NS non-sag epoxy. The concrete must be at least 28 days old and have a minimum compressive strength of 25 MPa. The cement laitance must be removed by scabbling or sandblasting, as shown in Figure 3.4, to a sufficient depth in order to expose the well-bonded large aggregate. After removal of the cement laitance, the bonding surface must be dusted and dried.
3.2.6 Primer adhesive used to construct research specimens

The primer used, Pro-Struct 618LV, meets the ASTM C881 Types I and IV, Grade I, Class B & C specifications and was supplied by StonCor. It is applied to the scabbled concrete surface and penetrates into the hairline cracks of the concrete to provide an adhesive surface for the epoxy. The primer consists of two components. These were mixed in a ratio of 47 g to 100 g ratio and were spread at 3–4 m²/litre. Figure 3.5 shows the application of the primer.
3.2.7 Epoxy adhesive used to construct research specimens

The epoxy adhesive used to bond the steel plates to the concrete surface was Pro-Struct 617NS, which meets the ASTM C881 Types I, II, IV and V, Grade 3, Class B and C specifications. See Figure 3.6 where the epoxy is being applied to a steel plate.

![Figure 3.6: Application of Pro-Struct non-sag epoxy adhesive 617NS to a steel plate](image)

The two components of the Pro-Struct epoxy 617NS were thoroughly mixed on a clean, oil-free, flat board for 4 to 5 minutes. The epoxy adhesive was then applied to the steel plate in a triangular wedge, with the apex of the adhesive at the centre of the plate. The steel plates were propped to the soffit of the concrete slab by means of 2 ton hydraulic car jacks as indicated in Figure 3.7.
Figure 3.7: Plates propped to concrete surface by means of hydraulic jacks

3.3 Number of research specimens constructed

The research specimens were subjected to either a third-span line load (TSLL), which mimics a uniformly distributed load, or to a mid-span line load (MSLL) acting as a point load. Table 3.1 lists the number of research specimens cast, the number and sizes of the bonded plates and the applied load.
### Table 3.1: Research specimens, applied load and number of bonded plate/s

<table>
<thead>
<tr>
<th>Number and size of bonded plate</th>
<th>Slab name</th>
<th>Applied load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control slabs</td>
<td>Contr 1-T</td>
<td>TSLL</td>
</tr>
<tr>
<td></td>
<td>Contr 2-T</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Contr 3-T</td>
<td></td>
</tr>
<tr>
<td>Composite slabs</td>
<td>Comp 1-T</td>
<td></td>
</tr>
<tr>
<td>1 x 110 x 6 mm plate</td>
<td>Comp 2-T</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Comp 3-T</td>
<td></td>
</tr>
<tr>
<td>Composite slabs</td>
<td>Comp 4-T</td>
<td></td>
</tr>
<tr>
<td>2 x 110 x 6 mm plates</td>
<td>Comp 5-T</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Comp 6-T</td>
<td></td>
</tr>
<tr>
<td>Composite slabs</td>
<td>Comp 7-T</td>
<td></td>
</tr>
<tr>
<td>1 x 150 x 8 mm plate</td>
<td>Comp 8-T</td>
<td></td>
</tr>
<tr>
<td>Composite slabs</td>
<td>Comp 9-T</td>
<td></td>
</tr>
<tr>
<td>2 x 150 x 8 mm plates</td>
<td>Comp 10-T</td>
<td></td>
</tr>
<tr>
<td>Control slabs</td>
<td>Contr 1-M</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Contr 2-M</td>
<td></td>
</tr>
<tr>
<td>Composite slabs</td>
<td>Comp 1-M</td>
<td></td>
</tr>
<tr>
<td>1 x 110 x 6 mm plate</td>
<td>Comp 2-M</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Comp 3-M</td>
<td></td>
</tr>
<tr>
<td>Composite slabs</td>
<td>Comp 4-M</td>
<td></td>
</tr>
<tr>
<td>2 x 110 x 6 mm plates</td>
<td>Comp 5-M</td>
<td></td>
</tr>
<tr>
<td>Composite slabs</td>
<td>Comp 6-M</td>
<td></td>
</tr>
<tr>
<td>1 x 150 x 8 mm plate</td>
<td>Comp 7-M</td>
<td></td>
</tr>
<tr>
<td>Composite slabs</td>
<td>Comp 8-M</td>
<td></td>
</tr>
<tr>
<td>2 x 150 x 8 mm plates</td>
<td>Comp 9-M</td>
<td></td>
</tr>
</tbody>
</table>

### 3.4 Load-applying research apparatus

The load-applying apparatus can be seen in Figure 3.8 (applying a TSLL) and Figure 3.9 (applying an MSLL). The apparatus is hydraulically operated and electronically controlled in order to apply the displacement at a constant rate of 2 mm/minute.

The supports for the 4 800 mm long research specimens were spaced at 4 500 mm. The spacing of the TSLLs was 1 500 mm apart, which meant that the line load loads were applied at a third of the span. The MSLLs were applied at mid-span.
Chapter 3: Methodology

Figure 3.8: Load-applying apparatus – TSLL

Figure 3.9: Load-applying apparatus – MSLL
3.5 Electronic data measuring equipment

The loads applied to the research specimens were measured by means of a 50 ton (500 kN) Load Tech load cell. The vertical displacement was measured at mid-span by means of an Opkon linear resistive positive transducer (LRPT) with a 400 mm range and an accuracy of 0.5%. The LRPT, which measured the vertical displacement of a test specimen with a TSLL, was placed as indicated in Figure 3.10. When specimens were being tested with an MSLL, the LRPT was placed next to the load spreader; therefore the vertical deflection recorded was 75 mm off-centre from the mid-span. The vertical deformation of the supports was not considered as it would not meaningfully influence the results of this research study.

![Figure 3.10: Position of LRPT measuring vertical deflection](image)

The strain of the externally bonded steel plates was measured at mid-span by means of strain gauges. All these electronic data were logged by a National Instruments data logger and were captured on a computer by means of Signal Express software, as indicated in Figure 3.11.
3.6 Determination of material properties

The material properties for both the concrete and steel were determined as described below:

3.6.1 Concrete compressive strength

The concrete compressive strength was determined as per SABS Standard Method 863 (1994).

3.6.2 Steel tensile strength

The tensile strength of both the steel plates and the reinforcement bars was determined as per SANS 920: 2011

Figure 3.12 shows some of the steel coupon samples prepared in order to determine the tensile stress and strain properties of the reinforcement bars, the 6 mm steel plates and the 8 mm steel plates.
Figure 3.12: Steel samples
Chapter 4  Analysis

4.1 Material properties of concrete

4.1.1 Compressive strength of concrete

The compressive strength of concrete is defined as the maximum strength reached in a compression test on a standard specimen. The standard specimens used for this research study were 100 mm cubes and they were tested in accordance with SABS Standard Method 863 (1994).

4.1.2 Tensile strength of concrete

The split cylinder test was used to determine the tensile strength of the concrete. This test is executed by loading a concrete cylinder 150 mm in diameter x 300 mm along its length, as shown in Figure 4.1. Assuming that the stresses are elastic just before failure, the tensile splitting strength $f_{ct}$ is determined from the stresses over the central portion of the cylinder.

$$f_{ct} = \frac{2P}{\pi LD} \quad (4.1-1)$$

![Figure 4.1: Split cylinder (indirect tension) test](image)

According to Illston (1994), the tensile strength of concrete ($f_{ct}$) is related to its compressive strength, as indicated in Figure 4.2.
According to Eurocode 4 (1994), the direct tensile split strength of concrete \( f_{ct} \) is a function of the compression strength of concrete, as follows:

\[
f_{ct} = 0.3f'_c \frac{2}{3}
\]  

(4.1-2)

4.1.3 Stress-strain relationship for concrete

A concrete cube or cylinder subjected to uniaxial compression has an almost uniform compressive strain, while the strain in the beam in flexure is distributed triangularly. Hognestad et al. (1955) carried out a study to compare the stress-strain relationship and concluded that the shape of the stress-strain relationship in the compression zone of a beam is almost identical to that of a uniaxially loaded cylinder, except that the maximum stress in the beam is less than the cylinder strength \( f'_c \) (see Figure 4.3) and that the maximum strain in the beam at failure \( \varepsilon_{cu} \) can be significantly greater than the maximum strain measured in an axially loaded cylinder.
The ultimate design strength in flexure, expressed in terms of the cube stress, is shown in Figure 4.3.

\[ f_c' \text{ (cylinder stress)} = 0.8 f_{cu} \text{ (cube stress)} \] (Robberts & Marshall, 2008)

\[ f \text{ (flexure stress)} = 0.85 f_c' \text{ (cylinder stress)} \] (Robberts & Marshall, 2008)

Therefore the compressive stress \((f)\) in flexure = \(0.85 \times 0.8 f_{cu} = 0.68 f_{cu}\).

The short term stress-strain relationship for concrete in flexure is per Eurocode 2 (1992). The specific stress-strain relationship used was chosen for ease of programming. The curve is linear elastic up to a strain of \(\varepsilon_{c0}\); for strains larger than \(\varepsilon_{c0}\) but smaller than \(\varepsilon_{cu}\), a constant strain is assumed, as shown in Figure 4.4.

![Figure 4.4: Triangular-rectangular stress-strain relationship for concrete in flexure](image)

Note that the material safety factor for concrete \((\gamma_m)\) is 1.5, but for the purpose of this research study it is taken as 1.0.

### 4.1.4 Modulus of elasticity for concrete

The stress-strain curve for concrete as per Figure 4.3 is not linear, and therefore the modulus of elasticity must be defined at different stresses.

The modulus of elasticity is dependent on many factors, the most important being the type of aggregate and the concrete cube strength, as per Figure 4.5.

The age of the concrete influences the modulus of elasticity of the concrete. According to Robberts and Marshall (2008), an approximation of the modulus of elasticity of concrete at 28 days is as follows:

\[ E_{c,28} = K_o + 0.2 f_{cu,28} \text{ (GPa)} \] \(\text{(4.1-3)}\)

where:

- \(K_o\) is a constant closely related to the modulus of elasticity of the aggregate. If the properties of the aggregate are unknown, a value of 20 GPa can be assumed.
\( f_{cu,28} \) is the concrete cube strength at 28 days

According to Robberts and Marshall (2008), an approximation of the modulus of elasticity of concrete older than 28 days is as follows:

\[
E_{c,t} = E_{c,28} \left( 0.4 + 0.6 \frac{f_{cu,t}}{f_{cu,28}} \right)
\]

where:

\( f_{cu,t} \) is the concrete cube strength at time \( t \)

\[\text{Figure 4.5: Short-term (static) modulus of elasticity for concrete made from different aggregates and cube strengths (Alexander & Davis, 1992)}\]

4.2 Material properties of steel reinforcement

4.2.1 Stress-strain relationship for steel reinforcement

The stress-strain relationship for steel reinforcement is given in SANS 10100 (2000) and is shown in Figure 4.6. Note that the stress-strain behaviour differs between compression and tension. The behaviour is linear elastic for stresses less than \( f_y/\gamma_m \) and greater than \( f_{yc} \). The stress-strain relationship is expressed as:

\[
f_s = \begin{cases} 
  f_y/\gamma_m & \text{if } \varepsilon_s \geq \varepsilon_y \\
  \varepsilon_sE_s & \text{if } \varepsilon_{yc} < \varepsilon_s < \varepsilon_y \\
  f_{yc} & \text{if } \varepsilon_s \leq \varepsilon_{yc}
\end{cases}
\]

\[\text{Figure 4.6: Stress-strain relationship for steel reinforcement (SANS 10100, 2000)}\]
Chapter 4: Analysis

4.2.2 Modulus of elasticity for steel reinforcement

According to SANS 10100 (2000), the modulus of elasticity of steel reinforcement bars is 200 GPa.

4.3 Deflection control of concrete structural elements with adhesively bonded plates

The formulas for determining the deflection of both the TSLL and MSLL are derived using the double integration method. Figure 4.7 shows the layout of a typical structural element with a TSLL.

![Diagram of a typical structural element with a TSLL](image)

Figure 4.7: Loading configuration of a TSLL structural element

The vertical deflection ($y$) at mid-span ($L/2$) due to the applied load is:

$$y = -\frac{1}{E_{\text{eff}}} \left(\frac{23P L^3}{1296}\right)$$  \hspace{1cm} (4.3-1)
Similarly to the above, Figure 4.8 shows the layout of a typical structural element with an MSLL.

\[ y = -\frac{1}{E_{\text{eff}}} \left( \frac{P L^3}{48} \right) \]  

(4.3-2)

The effective secant stiffness \( I_{\text{eff}} \) due to tension stiffening in the deflection equations for TSLL (4.3-1) and MSLL (4.3-2) is as given by Branson (1963, 1977) and is indicated in equation 4.3-3.

\[ I_{\text{eff}} = \left( \frac{M_{\text{cr}}}{M} \right)^3 I_{\text{co}} + \left( 1 - \frac{M_{\text{cr}}}{M} \right)^3 I_{\text{cr}} \leq I_{\text{co}} \]  

(4.3-3)

where:
- \( M \) is the applied moment
- \( M_{\text{cr}} \) is the moment at first cracking
- \( I_{\text{co}} \) is the second moment of area of the uncracked transformed section (equation 4.3-4)
- \( I_{\text{cr}} \) is the second moment of area of the cracked transformed section (equation 4.3-2)

\[ I_{\text{co}} = \frac{b x_{co}^3}{3} + \frac{b (h - x_{co})^3}{3} + (n - 1) A_r (d_r - x_{co})^2 + n A_p (d_p - x_{co})^2 \]  

(4.3-4)

\[ I_{\text{cr}} = \frac{b x_{cr}^3}{3} + n A_r (d_r - x_{cr})^2 + n A_p (d_p - x_{cr})^2 \]  

(4.3-5)

where:
- \( b \) is the width of the section
- \( x_{co} \) is the depth to the neutral axis of the uncracked transformed section
- \( x_{cr} \) is the depth to the neutral axis of the cracked transformed section
- \( h \) is the height of the section
- \( n \) is the modular ratio \( = \frac{E_s}{E_c} \)
- \( A_r \) is the total cross-sectional area of tension reinforcement
- \( A_p \) is the total cross-sectional area of the bonded plates
- \( d_r \) is the depth of the tension reinforcement bar/s
- \( d_p \) is the depth of the bonded plate/s
Figure 4.9: Strains, stresses and forces on a cracked transformed rectangular section

The strain in the steel reinforcement is assumed to be the same as that in the surrounding concrete, as indicated in Figure 4.9 (c).

\[ \varepsilon_r = \varepsilon_{rec} \]  

(4.3-6)

The force in the equivalent concrete must be the same as that in the reinforcement bars in order to maintain an equilibrium of forces, as indicated in Figure 4.9 (d).

\[ F_r = F_{rec} \]  

(4.3-7)

\[ A_r f_r = A_{rec} f_{rec} \]  

(4.3-8)

Assuming all materials are within their elastic ranges, then

\[ A_r (\varepsilon_r E_s) = A_{rec} (\varepsilon_{rec} E_c) \]  

(4.3-9)

\[ A_{rec} = \frac{A_r E_s}{E_c} = nA_r \]  

(4.3-10)

The above equation implies that the cross-sectional area of the concrete is equivalent to the cross-sectional area of the reinforcement bars multiplied by a modular ratio \((n)\).

The position of the neutral axis for a reinforced concrete structural element with a steel plate bonded to the soffit can be determined by conventional elastic analysis by setting the first moment of area about the neutral axis to zero. \(x\) can be determined by solving the quadratic equation:

\[ (bx) x/2 = nA_r (d_r - x) + nA_p (d_p - x) \]  

(4.3-11)

The cracked transformed second moment of area for a reinforced concrete structural element with a steel plate bonded to the soffit is given by:

\[ I_{cr} = \frac{bx^3}{3} + nA_r (d_r - x)^2 + nA_p (d_p - x)^2 \]  

(4.3.12)
4.4 Flexural capacity of concrete structural elements strengthened with adhesively bonded plates

The anchorage design philosophy is based on the principle that the bonded plates must terminate in an uncracked region, meaning that the plates should be extended beyond the point of contraflexure. The test specimens for this research study are simply supported and so the points of contraflexure are at the supports. The anchorage design philosophy is therefore not applicable to simply supported structural elements.

The hinge design philosophy, on the other hand, is based on the principle that the bonded plates must terminate short of the point of contraflexure, which means the supports in the case of simply supported structural elements. This philosophy was therefore used in this research study.

The attached plate is not enclosed by the concrete as in the case of the steel reinforcement bars and therefore not as well “gripped”. This may result in premature separation of the steel plate from the concrete surface of the reinforced concrete structural element.

Oehlers & Seracino (2004) propose the following debonding mechanisms of plated concrete structural elements:

- Intermediate crack (IC) debonding is usually associated with the flexural behaviour.
- Critical diagonal crack (CDC) debonding is usually associated with the shear capacity.
- Plate end (PE) debonding is usually associated with the curvature.
- \( VAy/Ib \) (interface shear stress) debonding is usually associated with the shear capacity.

4.4.1 Moment resistance of a structural element due to intermediate cracking

The following analysis methods are described in the sections that follow:

- Unplated reinforced concrete structural elements
- Propped plated reinforced concrete structural elements
- Unpropped plated reinforced concrete structural elements

4.4.1.1 Flexural analysis of unplated reinforced concrete structural elements

The flexural analysis of an unplated reinforced concrete structural element (control slab) is first revisited. This standard approach is then adapted to analyse plates adhesively bonded to reinforced concrete structural elements.

Figure 4.10 illustrates the flexural analysis of an unplated reinforced concrete structural element:

- The cross-section of a reinforced concrete structural element is shown in (a).
- The longitudinal strain distribution is shown in (b).
- The profile of the longitudinal stress is shown in (c).
- The longitudinal forces are shown in (d).
The strain capacity of the longitudinal reinforcement bars is large compared with that of concrete. This implies that whether or not the longitudinal bars have yielded, eventual failure of the beam is caused by concrete crushing at strain $\varepsilon_c$ (0.0035). Therefore one point on the strain profile is known and is referred to as the “pivotal point” in Figure 4.10 (b). The correct strain profile can be any strain profile that pivots like a pendulum about $\varepsilon_c$ (0.0035). The strain in the longitudinal reinforcement bars ($\varepsilon_b$) can be guessed and the neutral axis $k_{ad}$ calculated.

From the strain profile in Figure 4.10 (b) and by knowing the stress/strain relationship of the materials, the stress profile in Figure 4.10 (c) can be derived. The concrete stress profile is considered as elastic (triangular shape) if $\varepsilon_c$ does not exceed $\varepsilon_{c0}$ (0.00175), and plastic if $\varepsilon_c$ does exceed $\varepsilon_{c0}$ (0.00175) (rectangular shape), as indicated in Figure 4.10 (c). The tensile strength of the concrete below the neutral axis is ignored due to the large strains associated with failure, resulting in cracking.

Integrating the stresses over the areas in which they act gives the resultant forces ($F$) and their positions in the force diagram indicated in Figure 4.10 (d). The sum of the longitudinal forces must be zero in order to establish longitudinal force equilibrium. This can be achieved by altering the guessed strain in the longitudinal reinforcement bars ($\varepsilon_b$).

The moment capacity can be derived from the force distribution in Figure 4.10 (d) by taking moments about any convenient point, and the external theoretical load ($P_{anal}$) can be derived from the layout of the structural element. The theoretical load ($P_{anal}$) is compared with the load determined by experimental tests ($P_{appl}$).
4.4.1.2 Flexural analysis of a propped plated reinforced concrete structural element

The standard approach of an unplated reinforced concrete structural element (control slab) described in Section 4.4.1.1 was adapted to analyse composite reinforced concrete structural elements with adhesively bonded plates that are propped. A plated propped reinforced concrete structural element will be without any residual stresses in the tension face of the concrete where plating is done. Figure 4.11 illustrates the flexural analysis of such an element:

- The cross-section of a plated reinforced concrete structural element is shown in (a).
- Possible pivotal points of the strain profile are shown in (b).
- The longitudinal strain distribution is shown in (c).
- The profile of the longitudinal stress is shown in (d).
- The longitudinal forces are shown in (e).

![Diagram of flexural analysis](image)

Debonding of the plate usually occurs before the concrete reaches its crushing strain ($\varepsilon_c = 0.0035$) and therefore a good initial assumption for the pivotal point is that the debonding strain $\varepsilon_{db}$ will be equal to the intermediate crack strain $\varepsilon_{IC}$. If the pivotal point is incorrectly chosen as $\varepsilon_{db}$ on the strain profile and the guessed strain in the concrete ($\varepsilon_c$) exceeds 0.0035 in order to establish longitudinal force equilibrium, the pivotal point must be altered to the concrete crushing strain ($\varepsilon_c = 0.0035$) as no debonding of the plate will occur.

Once the debonding strain ($\varepsilon_{db}$) has been calculated and set as the pivotal point, the strain profile pivots like a pendulum about $\varepsilon_{db}$. The strain of the concrete ($\varepsilon_c$) can then be guessed. From the strain profile in Figure 4.11 (c) and knowing the material’s stress/strain relationships, the stress profile in Figure 4.11 (d) can be derived. The concrete stress profile is considered as elastic (triangular shape) if $\varepsilon_c$ does not exceed $\varepsilon_{c0}$ (0.00175), and plastic (rectangular shape) if $\varepsilon_c$ exceeds $\varepsilon_{c0}$ (0.00175), as indicated in Figure 4.11 (d). The tensile strength of the concrete below...
the neutral axis is ignored due to the large strains associated with failure, resulting in cracking of the concrete.

Integrating the stresses over the areas in which they act gives the resultant forces \( F \) and their positions in the force diagram shown in Figure 4.11 (e). The resultant of the longitudinal forces must sum to zero in order to establish longitudinal force equilibrium. This can be achieved by altering the strain in the concrete \( \varepsilon_c \).

The moment capacity can be derived from the force distribution in Figure 4.11 (e) by taking moments about any convenient point, and the theoretical applied load \( P_{anul} \) can be derived from the layout of the structural element. The theoretical load \( P_{anul} \) is compared with the load determined by experimental tests \( P_{appl} \).

The debonding strain \( \varepsilon_{db} \) of plates externally bonded to concrete structural elements is calculated using the following three theories:

- Seracino et al.’s (2007) push-pull theory
- Teng et al.’s (2002) flexural theory
- Push-pull tests on EB steel plates

In sizing the composite structural element for effective width, shear lag was considered and the simplified Eurocode 4 model was used for this research study. The Eurocode 4 recommendation is that \( b_c(w_{eff}) \) should be calculated as follows:

\[
b_c(w_{eff}) = 0.25L_c \tag{4.4-1}\]

where:

\( L_c \) is the maximum distance between points of contraflexure

The following geometrical restraints apply to \( T_{adj} \) and \( T_{edge} \) (see Figure 4.12 for explanations):

\[
b_c(w_{eff}) \leq \frac{(T_{adj})^1 + (T_{adj})^2}{2} \tag{4.4-2}\]

and

\[
b_c(w_{eff}) \leq 2T_{edge} \tag{4.4-3}\]

\[
b_c(w_{eff}) \leq \frac{(T_{adj})^1 + (T_{adj})^2}{2} \tag{4.4-4}\]

\[\text{Figure 4.12: Effective width}\]

The \( a_{EB} \) coefficient was calibrated by Teng et al. (2002) on the basis of a large number of tests. The results are summarised in Table 4.1.
Table 4.1: IC debonding coefficient $a_{EB}$ (Teng et al., 2002)

<table>
<thead>
<tr>
<th>$a_{EB}$</th>
<th>Pull tests</th>
<th>Slabs</th>
<th>Beams</th>
<th>Slabs and beams</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beams</td>
<td>Steel</td>
<td>Steel and FRP</td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>0.448</td>
<td>0.401</td>
<td>0.427</td>
<td>0.720</td>
</tr>
<tr>
<td>95%</td>
<td>0.322</td>
<td>0.324</td>
<td>0.315</td>
<td>0.478</td>
</tr>
</tbody>
</table>

Neubauer and Rostasy (2001) determined the debonding strains ($\varepsilon_{db}$) of steel plates as in Table 4.2. No IC debonding strain values were determined for steel plates thicker than 3 mm. The available IC debonding strains (1–3 mm) become smaller as the plate thickness increases. If these data were to be extrapolated to the 6 mm and 8 mm steel plates used for this research study, a negative value would be obtained. The push-pull theory of Neubauer and Rostasy (2001) therefore cannot be applied to this research study.

Table 4.2: Typical IC debonding strains

<table>
<thead>
<tr>
<th>Source</th>
<th>IC debonding strains</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adelaide beam tests: 3 mm steel plates</td>
<td>0.0044–0.0213</td>
</tr>
<tr>
<td>Adelaide beam tests: 2 mm steel plates</td>
<td>0.0059</td>
</tr>
<tr>
<td>Adelaide beam tests: 1 mm steel plates</td>
<td>0.0149</td>
</tr>
</tbody>
</table>

4.4.1.3 Flexural analysis of an unpropped plated reinforced concrete structural element

The approach of a plated reinforced concrete structural element that is propped, described in Section 4.4.1.2, is adapted to analyse plates adhesively bonded to reinforced concrete structural elements that are unpropped.

A plated unpropped reinforced concrete structural element will have residual stresses in the tension face of the concrete where plating occurs. Figure 4.13 illustrates the flexural analysis of such an element:

- The cross-section of a plated reinforced concrete structural element is shown in (a).
- The longitudinal strain distribution is shown in (b).
- The profile of the longitudinal stress is shown in (c).
- The longitudinal forces are shown in (d).
When an unpropped concrete structural element is plated, before any further increase in load the plate strain is zero and the concrete strain adjacent to it is \( \varepsilon_{\text{pivot}} \). There will always be a difference in strain between the plate and the adjacent concrete surface. When the plated concrete structural element is loaded, the plate will eventually debond at a plate strain of \( \varepsilon_{\text{db}} = \varepsilon_{\text{IC}} \). Therefore the strain in the adjacent concrete is \( \varepsilon_{\text{db}} + (\varepsilon_{\text{pivot}})_{\text{res}} \), as shown in Figure 4.13 (b). This will become the pivotal point for the strain profile. With the pivotal point having been fixed, the longitudinal forces can be determined, the moment capacity can be calculated and the external load can be determined to be compared with the experimentally determined load.

The bending moment resistance can be determined as per Section 4.4.1.2.

### 4.4.2 Moment resistance of a structural element due to critical diagonal cracking

HB 305-2008 (Oehlers et al., 2008) indicates that the shear capacity for reinforced concrete structural members with bonded longitudinal plates is as follows:

\[
V_{\text{pl}} = V_{\text{uc}} + \Delta V_{\text{uc}}
\]  

(4.4-5)

The shear stress capacity of a concrete structural element without any shear reinforcement as per SANS 10100.2000 is:

\[
v_c = \frac{0.75}{\gamma_{\text{m}}}(\frac{f_{\text{cu}}}{25})^{1/3}(\frac{100A_s}{b_vd})^{1/3}(\frac{400}{d})^{1/4}
\]

(4.4-6)

where:
- \( \gamma_{\text{m}} \) is material safety factor taken as 1.0
- \( f_{\text{cu}} \) is the concrete cube strength (\( \leq 4.0 \) MPa)
- \( 100A_s/b_vd \) is the reinforcement ratio (\( \leq 3.0 \))
- \( A_s \) is the area of effectively anchored tension reinforcement
The shear force capacity of a concrete structural element without any shear reinforcement is:

\[ V_{uc} = v_c b_v d \]  \hspace{1cm} (4.4-7)

Equation 2.5-10 indicates the increase in the shear force capacity of the concrete structural element due to the bonded plate. The longitudinal plates must, however, extend at least an anchorage length beyond the point of moment contraflexure, or to a region where the transverse shear force is low. Due to the simply supported layout of the slabs being tested, the plates do not extend an anchorage length beyond the point of contraflexure, nor do they end in an area with a small shear force. Therefore the increase in the shear force capacity \((\Delta V_{uc})\) does not apply to this research study.

### 4.4.3 PE debonding

For an externally bonded tension face plate, the moment at the plate end to cause PE debonding is given by Equation 2.5-11.

### 4.4.4 \(\frac{VA_y}{I_b}\) debonding

The \(\frac{VA_y}{I_b}\) shear stress must be restricted to less than the tensile strength of the concrete.
Chapter 5  Findings and Comparisons

5.1  Material properties of composite specimens

5.1.1  Properties of the concrete

All the concrete used was ready mixed, specified to a cube strength \( f_{cu} \) of 25 MPa. It was a mix using aggregate from the Zimbiwa quarry situated near Benoni, Gauteng province. The aggregate mixture consists of 80% dolomite and 20% dolerite. The static modulus of elasticity of all concrete used in this research study was taken as 35 GPa per Alexander and Davis (1992) as no equipment was available to test it.

5.1.2  Properties of the reinforcing bars

The average material properties of three coupon samples of the 12 mm longitudinal high-yield reinforcing bars are shown in Table 5.1 and Figure 5.1.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>RE bar for control slabs</th>
<th>RE bar for composite slabs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2% yield stress</td>
<td>543 MPa</td>
<td>534 MPa</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>694 MPa</td>
<td>686 MPa</td>
</tr>
<tr>
<td>Yield strain</td>
<td>0.0028</td>
<td>0.0027</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>204.6MPa</td>
<td>197.8 MPa</td>
</tr>
</tbody>
</table>

Figure 5.1:  Reinforcing bar material, stress-strain graph

5.1.3  Properties of the steel plates

The average material properties of three coupon samples of the 6 mm and 8 mm steel plates are shown in Table 5.2 and Figures 5.2 and 5.3. These tests were also done at the laboratory of the Department of Metallurgical Engineering.
Table 5.2: Properties of the steel plate material

<table>
<thead>
<tr>
<th>Parameter</th>
<th>6 mm plate</th>
<th>8 mm plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensions</td>
<td>3.88 mm</td>
<td>3.96 mm</td>
</tr>
<tr>
<td>Cross-sectional area</td>
<td>11.82 mm²</td>
<td>12.32 mm²</td>
</tr>
<tr>
<td>Gauge length</td>
<td>17.76 mm</td>
<td>17.58 mm</td>
</tr>
<tr>
<td>Extension</td>
<td>4.14 mm</td>
<td>4.22 mm</td>
</tr>
<tr>
<td>0.2% yield stress</td>
<td>461 MPa</td>
<td>446 MPa</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>610 MPa</td>
<td>609 MPa</td>
</tr>
<tr>
<td>Yield strain</td>
<td>0.0023</td>
<td>0.0022</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>200.4 MPa</td>
<td>202.7 MPa</td>
</tr>
</tbody>
</table>

Figure 5.2: 6 mm steel plate, stress-strain graph

Figure 5.3: 8 mm steel plate, stress-strain graph
5.1.4 Summary of material properties

Table 5.3 lists the properties of the concrete used to construct the composite concrete-steel specimens.

**Table 5.3: Material properties of concrete used to construct the composite specimens**

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Elastic modulus of steel plate ($E_s$) (GPa)</th>
<th>Concrete cube strength ($f_{cu}$) (MPa)</th>
<th>Concrete cylinder strength ($f'c$) (MPa)</th>
<th>Elastic modulus of concrete ($E_c$) (GPa)</th>
<th>Concrete tensile cylinder split strength ($f_{ct}$) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Contr 1-T</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Contr 2-T</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Contr 3-T</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Comp 1-T</td>
<td>1, 110 x 6</td>
<td>200.4</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 2-T</td>
<td>1, 110 x 6</td>
<td>200.4</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 3-T</td>
<td>1, 110 x 6</td>
<td>200.4</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 4-T</td>
<td>2, 110 x 6</td>
<td>200.4</td>
<td>23.10</td>
<td>18.48</td>
<td>35</td>
<td>2.10</td>
</tr>
<tr>
<td>Comp 5-T</td>
<td>2, 110 x 6</td>
<td>200.4</td>
<td>23.10</td>
<td>18.48</td>
<td>35</td>
<td>2.10</td>
</tr>
<tr>
<td>Comp 6-T</td>
<td>2, 110 x 6</td>
<td>200.4</td>
<td>23.10</td>
<td>18.48</td>
<td>35</td>
<td>2.10</td>
</tr>
<tr>
<td>Comp 7-T</td>
<td>1, 150 x 8</td>
<td>202.7</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 8-T</td>
<td>1, 150 x 8</td>
<td>202.7</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 9-T</td>
<td>2, 150 x 8</td>
<td>202.7</td>
<td>23.10</td>
<td>18.48</td>
<td>35</td>
<td>2.10</td>
</tr>
<tr>
<td>Comp 10-T</td>
<td>2, 150 x 8</td>
<td>202.7</td>
<td>23.10</td>
<td>18.48</td>
<td>35</td>
<td>2.10</td>
</tr>
<tr>
<td>Contr 1-M</td>
<td></td>
<td>31.74</td>
<td>25.39</td>
<td>37</td>
<td>2.68</td>
<td></td>
</tr>
<tr>
<td>Contr 2-M</td>
<td></td>
<td>31.74</td>
<td>25.39</td>
<td>37</td>
<td>2.68</td>
<td></td>
</tr>
<tr>
<td>Comp 1-M</td>
<td>1, 110 x 6</td>
<td>200.4</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 2-M</td>
<td>1, 110 x 6</td>
<td>200.4</td>
<td>23.10</td>
<td>18.48</td>
<td>35</td>
<td>2.10</td>
</tr>
<tr>
<td>Comp 3-M</td>
<td>1, 110 x 6</td>
<td>200.4</td>
<td>30.60</td>
<td>24.48</td>
<td>35</td>
<td>2.53</td>
</tr>
<tr>
<td>Comp 4-M</td>
<td>2, 110 x 6</td>
<td>200.4</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 5-M</td>
<td>2, 110 x 6</td>
<td>200.4</td>
<td>23.10</td>
<td>18.48</td>
<td>35</td>
<td>2.10</td>
</tr>
<tr>
<td>Comp 6-M</td>
<td>1, 150 x 8</td>
<td>202.7</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 7-M</td>
<td>1, 150 x 8</td>
<td>202.7</td>
<td>30.60</td>
<td>24.48</td>
<td>35</td>
<td>2.53</td>
</tr>
<tr>
<td>Comp 8-M</td>
<td>2, 150 x 8</td>
<td>202.7</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
<tr>
<td>Comp 9-M</td>
<td>2, 150 x 8</td>
<td>202.7</td>
<td>24.70</td>
<td>19.76</td>
<td>35</td>
<td>2.19</td>
</tr>
</tbody>
</table>

5.2 Increase in load-bearing resistance from the control to the composite slabs

Table 5.4 presents the maximum applied load ($P_{appl}$) determined by experimental testing for both the control and composite slabs. A ratio larger than 1.0 indicates an increase in the load-bearing capacity from an unplated to a plated structural member.
Table 5.4: Experimental test results indicating the increase in load-bearing resistance

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Type of line load</th>
<th>$P_{appl, unplated}$ slab (kN)</th>
<th>$P_{appl, plated}$ slab (kN)</th>
<th>$P_{appl, plated} / P_{appl, unplated}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contr 1-T</td>
<td>-</td>
<td>TSLL</td>
<td>24.32</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Contr 2-T</td>
<td>-</td>
<td>TSLL</td>
<td>25.54</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Contr 3-T</td>
<td>TSLL</td>
<td></td>
<td>25.51</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Comp 1-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>52.72</td>
<td>2.10</td>
<td></td>
</tr>
<tr>
<td>Comp 2-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>60.79</td>
<td>2.42</td>
<td></td>
</tr>
<tr>
<td>Comp 3-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>52.26</td>
<td>2.08</td>
<td></td>
</tr>
<tr>
<td>Comp 4-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>83.42</td>
<td>3.32</td>
<td></td>
</tr>
<tr>
<td>Comp 5-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>80.99</td>
<td>3.22</td>
<td></td>
</tr>
<tr>
<td>Comp 6-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>88.14</td>
<td>3.51</td>
<td></td>
</tr>
<tr>
<td>Comp 7-T</td>
<td>1, 150 x 8</td>
<td>TSLL</td>
<td>54.72</td>
<td>2.18</td>
<td></td>
</tr>
<tr>
<td>Comp 8-T</td>
<td>1, 150 x 8</td>
<td>TSLL</td>
<td>59.61</td>
<td>2.37</td>
<td></td>
</tr>
<tr>
<td>Comp 9-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>79.25</td>
<td>3.15</td>
<td></td>
</tr>
<tr>
<td>Comp 10-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>89.00</td>
<td>3.54</td>
<td></td>
</tr>
<tr>
<td>Contr 1-M</td>
<td>MSLL</td>
<td></td>
<td>21.39</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Contr 2-M</td>
<td>MSLL</td>
<td></td>
<td>20.05</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Comp 2-M</td>
<td>1, 110 x 6</td>
<td>MSLL</td>
<td>54.36</td>
<td>2.62</td>
<td></td>
</tr>
<tr>
<td>Comp 3-M</td>
<td>1, 110 x 6</td>
<td>MSLL</td>
<td>52.24</td>
<td>2.52</td>
<td></td>
</tr>
<tr>
<td>Comp 4-M</td>
<td>2, 110 x 6</td>
<td>MSLL</td>
<td>56.23</td>
<td>2.71</td>
<td></td>
</tr>
<tr>
<td>Comp 5-M</td>
<td>2, 110 x 6</td>
<td>MSLL</td>
<td>61.69</td>
<td>2.98</td>
<td></td>
</tr>
<tr>
<td>Comp 6-M</td>
<td>1, 150 x 8</td>
<td>MSLL</td>
<td>64.09</td>
<td>3.09</td>
<td></td>
</tr>
<tr>
<td>Comp 7-M</td>
<td>1, 150 x 8</td>
<td>MSLL</td>
<td>49.60</td>
<td>2.39</td>
<td></td>
</tr>
<tr>
<td>Comp 8-M</td>
<td>2, 150 x 8</td>
<td>MSLL</td>
<td>60.93</td>
<td>2.94</td>
<td></td>
</tr>
<tr>
<td>Comp 9-M</td>
<td>2, 150 x 8</td>
<td>MSLL</td>
<td>89.58</td>
<td>4.32</td>
<td></td>
</tr>
</tbody>
</table>

The increase in the applied point load from the unplated to the plated reinforced concrete structural element with 6 mm plate/s attached and loaded at TSLL is expressed as the ratio in Figure 5.4 and Table 5.4 as follows:

- One 110 x 6 mm plate (TSLL), average ratio = 2.20
- Two 110 x 6 mm plates (TSLL), average ratio = 3.35
The increase in the applied point load from the unplated to the plated reinforced concrete structural element with 8 mm plate/s attached and loaded at TSLL is expressed as the $\frac{P_{\text{applied, plated}}}{P_{\text{applied, unplated}}}$ ratio in Figure 5.5 and Table 5.4 as follows:

- One 150 x 8 mm plate (TSLL), average ratio = 2.28
- Two 150 x 8 mm plates (TSLL), average ratio = 3.35

Figure 5.4: Load-deflection graph comparing TSLL unplated with 6 mm plated elements
The increase in the applied point load from the unplated to the plated reinforced concrete structural element with 6 mm plate/s attached and loaded at MSLL is expressed as the \( \frac{P_{\text{applied, plated}}}{P_{\text{applied, unplated}}} \) ratio in Figure 5.6 and Table 5.4 as follows:

- One 110 x 6 mm plate (MSLL), average ratio = 2.62
- Two 110 x 6 mm plates (MSLL), average ratio = 3.04
Figure 5.6: Load-deflection graph comparing MSLL unplated with 6 mm plated elements

The increase in the applied point load from the unplated to the plated reinforced concrete structural element with 8 mm plate/s attached and loaded at MSLL is expressed as the $\frac{P_{\text{applied, plated}}}{P_{\text{applied, un-plated}}}$ ratio in Figure 5.7 and Table 5.4 as follows:

- One 150 x 8 mm plate (MSLL), average ratio = 2.67
- Two 150 x 8 mm plates (MSLL), average ratio = 4.33
Chapter 5: Findings and Comparisons

Figure 5.7: Load-deflection graph comparing MSLL unplated with 8 mm plated elements

As expected, the creation of composite concrete structural elements by bonding external steel plates to the tension zone of the structural reinforced concrete element has a definite advantage in increasing the load-bearing resistance. The plated structural elements were stiffer (less deflection for the same applied load) than the unplated structural elements.

5.3 Deflection control of structural elements strengthened with adhesively bonded plates

Table 5.5 shows the increase in the stiffness of the plated compared with the unplated structural elements in terms of the reduced deflection. In order to compare experimentally measured deflections an applied load of 15 kN is used, which is within the elastic range of both plated and unplated structural elements.
Table 5.5: Experimental test results showing the increase in stiffness

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Type of line load</th>
<th>Experimentally measured deflection (mm)</th>
<th>( y_{appl} ) (unplated)/ ( y_{appl} ) (plated)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contr 1-T</td>
<td>TSL</td>
<td>1 x 110 x 6</td>
<td>TSLL</td>
<td>11.10</td>
</tr>
<tr>
<td>Contr 2-T</td>
<td>TSL</td>
<td>1 x 110 x 6</td>
<td>TSLL</td>
<td>11.04</td>
</tr>
<tr>
<td>Contr 3-T</td>
<td>TSL</td>
<td>2 x 110 x 6</td>
<td>TSLL</td>
<td>8.92</td>
</tr>
<tr>
<td>Comp 1-T</td>
<td>TSL</td>
<td>2 x 110 x 6</td>
<td>TSLL</td>
<td>8.30</td>
</tr>
<tr>
<td>Comp 2-T</td>
<td>TSL</td>
<td>2 x 110 x 6</td>
<td>TSLL</td>
<td>8.40</td>
</tr>
<tr>
<td>Comp 3-T</td>
<td>TSL</td>
<td>1 x 150 x 8</td>
<td>TSLL</td>
<td>6.94</td>
</tr>
<tr>
<td>Comp 4-T</td>
<td>TSL</td>
<td>1 x 150 x 8</td>
<td>TSLL</td>
<td>6.74</td>
</tr>
<tr>
<td>Comp 5-T</td>
<td>TSL</td>
<td>2 x 150 x 8</td>
<td>TSLL</td>
<td>5.76</td>
</tr>
<tr>
<td>Comp 6-T</td>
<td>TSL</td>
<td>2 x 150 x 8</td>
<td>TSLL</td>
<td>7.46</td>
</tr>
<tr>
<td>Comp 7-M</td>
<td>MSL</td>
<td>1 x 110 x 6</td>
<td>MSL</td>
<td>13.29</td>
</tr>
<tr>
<td>Comp 8-M</td>
<td>MSL</td>
<td>1 x 110 x 6</td>
<td>MSL</td>
<td>16.97</td>
</tr>
<tr>
<td>Comp 9-M</td>
<td>MSL</td>
<td>1 x 110 x 6</td>
<td>MSL</td>
<td>18.37</td>
</tr>
<tr>
<td>Comp 10-M</td>
<td>MSL</td>
<td>2 x 110 x 6</td>
<td>MSL</td>
<td>12.70</td>
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<tr>
<td>Comp 11-M</td>
<td>MSL</td>
<td>2 x 110 x 6</td>
<td>MSL</td>
<td>14.30</td>
</tr>
<tr>
<td>Comp 12-M</td>
<td>MSL</td>
<td>1 x 150 x 8</td>
<td>MSL</td>
<td>12.80</td>
</tr>
<tr>
<td>Comp 13-M</td>
<td>MSL</td>
<td>1 x 150 x 8</td>
<td>MSL</td>
<td>12.95</td>
</tr>
<tr>
<td>Comp 14-M</td>
<td>MSL</td>
<td>2 x 150 x 8</td>
<td>MSL</td>
<td>10.94</td>
</tr>
<tr>
<td>Comp 15-M</td>
<td>MSL</td>
<td>2 x 150 x 8</td>
<td>MSL</td>
<td>9.69</td>
</tr>
</tbody>
</table>

It is evident from Table 5.5 that the stiffness of a structural member is increased, up to 453%, by bonding external reinforcement to an existing reinforced concrete member.

In order to compare the theoretically analysed deflections (\( y_{anal} \)), as per equations 4.3-1 and 4.3-2, with the experimentally measured deflections (\( y_{appl} \)) of plated composite concrete structural elements, an applied load of 15 kN, which is within the elastic range, was used for the calculations.

In Table 5.6 the theoretically analysed deflections (\( y_{anal} \)) and the experimentally measured deflections (\( y_{appl} \)) for unplated and plated reinforced concrete structural elements are compared.
Table 5.6: Comparison between the theoretically calculated and the experimentally measured deflections

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates</th>
<th>Type of line load</th>
<th>Moment of cracking moment (kNm)</th>
<th>$I_{cr}$ x10^6 (mm^4)</th>
<th>$I_{eff}$ x10^6 (mm^4)</th>
<th>$y_{anal}$ (mm)</th>
<th>$y_{appl}$ (mm)</th>
<th>$y_{anal}/y_{appl}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp 1-T</td>
<td>1x110x6 TSLL</td>
<td>9.34</td>
<td>308.30</td>
<td>89.36</td>
<td>176.86</td>
<td>7.02</td>
<td>11.10</td>
<td>0.63</td>
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<tr>
<td>Comp 2-T</td>
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<td>308.30</td>
<td>89.36</td>
<td>176.86</td>
<td>7.02</td>
<td>9.58</td>
<td>0.73</td>
</tr>
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<td>89.36</td>
<td>176.86</td>
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<td>0.64</td>
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<td>136.00</td>
<td>216.58</td>
<td>5.74</td>
<td>8.92</td>
<td>0.64</td>
</tr>
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<td>329.20</td>
<td>136.00</td>
<td>216.58</td>
<td>5.74</td>
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<td>236.40</td>
<td>5.25</td>
<td>6.94</td>
<td>0.76</td>
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<td>129.50</td>
<td>236.40</td>
<td>5.25</td>
<td>6.74</td>
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<td>203.10</td>
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<td>7.46</td>
<td>0.46</td>
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<td>89.36</td>
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<td>20.61</td>
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<td>308.30</td>
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<td>55.04</td>
<td>22.57</td>
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<td>16.73</td>
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<td>363.30</td>
<td>203.10</td>
<td>111.36</td>
<td>11.16</td>
<td>12.95</td>
<td>0.86</td>
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<td>363.30</td>
<td>203.10</td>
<td>128.71</td>
<td>9.65</td>
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<td>0.88</td>
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<td>203.10</td>
<td>128.71</td>
<td>9.65</td>
<td>9.69</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The moments due to the applied load (15 kN) are:
- TSLL = 11.25 kNm
- MSLL = 16.87 kNm

From a comparison of the theoretically analysed ($y_{anal}$) deflection with the experimentally measured deflection ($y_{appl}$) for reinforced concrete structural elements with externally bonded steel plates, the following is evident:

- For a TSLL, the theoretically analysed deflection ($y_{anal}$) is on average 34% lower than the measured deflection ($y_{appl}$).
- For an MSLL, the theoretically analysed deflection ($y_{anal}$) is on average 11% larger than the measured deflection ($y_{appl}$).

A possible cause of the difference between the experimentally measured deflection ($y_{appl}$) and the theoretically analysed deflection ($y_{anal}$) is that the elasticity modulus of concrete ($E$) was predicted and not measured as the necessary equipment was not available.

The elasticity modulus ($E$) of the control and composite slabs is calculated by measuring the vertical deflection ($y$) and applied load ($P$) at the yielding point of the slab and by substituting...
them into equations 4.3-1 for TSLL and equation 4.3-2 for MSLL. A comparison between the elasticity modulus of the control and composite slabs is drawn in Table 5.7.

### Table 5.7: Comparison of the elasticity modulus of control and composite slabs

<table>
<thead>
<tr>
<th>Slab Name</th>
<th>Number and Size of Steel Plates</th>
<th>Type of Line Load</th>
<th>P15 (kN)</th>
<th>I_eff x 10^6 (mm^4)</th>
<th>y_max (mm)</th>
<th>E (GPa)</th>
<th>E (plated) / E (unplated)</th>
<th>EI_eff (plated) / EI_eff (unplated)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contr 1-T</td>
<td>TSLL</td>
<td>15.00</td>
<td>200.52</td>
<td>44.25</td>
<td>4.90</td>
<td>-</td>
<td>982.59</td>
<td>-</td>
</tr>
<tr>
<td>Contr 2-T</td>
<td>TSLL</td>
<td>15.00</td>
<td>200.52</td>
<td>40.92</td>
<td>5.30</td>
<td>-</td>
<td>1062.55</td>
<td>-</td>
</tr>
<tr>
<td>Contr 3-T</td>
<td>TSLL</td>
<td>15.00</td>
<td>200.52</td>
<td>38.68</td>
<td>5.61</td>
<td>-</td>
<td>1124.08</td>
<td>-</td>
</tr>
<tr>
<td>Comp 1-T</td>
<td>1x110x6</td>
<td>TSLL</td>
<td>15.00</td>
<td>176.86</td>
<td>11.10</td>
<td>4.43</td>
<td>3917.07</td>
<td>3.91</td>
</tr>
<tr>
<td>Comp 2-T</td>
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<td>TSLL</td>
<td>15.00</td>
<td>176.86</td>
<td>9.58</td>
<td>4.47</td>
<td>5238.49</td>
<td>4.82</td>
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<td>176.86</td>
<td>11.04</td>
<td>5.11</td>
<td>3938.36</td>
<td>6.02</td>
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<td>TSLL</td>
<td>15.00</td>
<td>216.58</td>
<td>8.92</td>
<td>4.47</td>
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<td>4.82</td>
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<tr>
<td>Comp 5-T</td>
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<td>TSLL</td>
<td>15.00</td>
<td>216.58</td>
<td>8.30</td>
<td>4.47</td>
<td>5176.13</td>
<td>4.82</td>
</tr>
<tr>
<td>Comp 6-T</td>
<td>1x150x8</td>
<td>TSLL</td>
<td>15.00</td>
<td>236.40</td>
<td>6.94</td>
<td>5.11</td>
<td>6265.06</td>
<td>6.02</td>
</tr>
<tr>
<td>Comp 7-T</td>
<td>1x150x8</td>
<td>TSLL</td>
<td>15.00</td>
<td>236.40</td>
<td>6.74</td>
<td>5.11</td>
<td>5828.35</td>
<td>6.33</td>
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<tr>
<td>Comp 8-T</td>
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<td>TSLL</td>
<td>15.00</td>
<td>362.55</td>
<td>5.76</td>
<td>5.52</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Comp 9-T</td>
<td>2x150x8</td>
<td>TSLL</td>
<td>15.00</td>
<td>362.55</td>
<td>7.46</td>
<td>5.52</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Comp 10-T</td>
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<td>TSLL</td>
<td>15.00</td>
<td>362.55</td>
<td>16.08</td>
<td>5.52</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

From Table 5.7 it is evident that the elasticity modulus (E) and the flexural rigidity (EI) of the composite beams increases compared with the control beams.

### 5.4 Flexural capacity of unstrengthened structural elements

The analysed flexural resistance of an unplated reinforced concrete structural element \( M_{\text{anal}} \) and the point load \( P_{\text{anal}} \), as described in Section 4.4.1.1, are compared in Table 5.8 with that of the applied load \( P_{\text{appl}} \), which was experimentally determined (refer to Figures 5.4 and 5.6).
Table 5.8  Comparison between the analysed and applied loads due to the flexural resistance of the unplated element

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Type of line load</th>
<th>$M_{anal}$ (kNm)</th>
<th>$P_{anal}$ (kN)</th>
<th>$P_{appl}$ (kN)</th>
<th>$P_{appl}/P_{anal}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contr 1-T</td>
<td>TSLL</td>
<td>26.03</td>
<td>24.88</td>
<td>24.32</td>
<td>0.98</td>
</tr>
<tr>
<td>Contr 2-T</td>
<td>TSLL</td>
<td>26.03</td>
<td>24.88</td>
<td>25.54</td>
<td>1.03</td>
</tr>
<tr>
<td>Contr 3-T</td>
<td>TSLL</td>
<td>26.03</td>
<td>24.88</td>
<td>25.11</td>
<td>1.03</td>
</tr>
<tr>
<td>Contr 1-M</td>
<td>MSLL</td>
<td>26.03</td>
<td>17.14</td>
<td>21.39</td>
<td>1.25</td>
</tr>
<tr>
<td>Contr 2-M</td>
<td>MSLL</td>
<td>26.03</td>
<td>17.14</td>
<td>20.05</td>
<td>1.17</td>
</tr>
</tbody>
</table>

The analysed point load ($P_{anal}$) was calculated by considering the own weight (3.6 kN/m) of the slab and the weight of the load spreaders, which was 112.6 kg for the MSLL and 298.6 kg for the TSLL. From Table 5.8 it is evident that the theoretically analysed loads are accurate to within 25% of the loads experimentally applied to the control slabs.

5.5  Flexural capacity of structural elements strengthened with adhesively bonded plates

The analysed flexural resistance of the plated concrete structural element ($M_{anal}$) and the point load ($P_{anal}$), as described in Section 4.4, are compared with that of the applied load ($P_{appl}$), which was experimentally determined. The following debonding mechanisms of plates bonded to concrete structural elements were analysed and conclusions were drawn by comparing with the experimental results.

- Intermediate cracks (IC) are associated with crack forming due to flexural behaviour.
- Critical diagonal cracks (CDC) are associated with the shear capacity.
- Plate end debonding (PE) is associated with the curvature.
- $VAy/Ib$ debonding is associated with the tensile strength of the concrete.

5.5.1  Intermediate crack (IC) debonding

Table 5.9 compares the flexural analysed load ($P_{anal}$), calculated as per Section 4.4.1.2, with the applied load ($P_{appl}$), which was experimentally determined, of the plated reinforced concrete structural elements as per the hinge approach using Seracino et al.’s (2007) theory.
The analysed point load \( P_{\text{anal}} \) was calculated by considering the own weight (3.6 kN/m) of the slab and the weight of the load spreaders, which was 112.6 kg for the MSLL and 298.6 kg for the TSLL. From the comparison of the flexural analysed load \( P_{\text{anal}} \) with the applied load \( P_{\text{appl}} \) using Seracino et al.'s theory, it is evident that the analysed load \( P_{\text{anal}} \) is much smaller than the experimentally determined load \( P_{\text{appl}} \), which is an indication that Seracino et al.'s theory does not accurately predict the IC debonding resistance for 6 mm and 8 mm bonded steel plates.

In order to obtain more accurate results from Seracino et al.'s IC debonding theory, the mean constant of 0.853 in the IC debonding resistance equation \( ((P_{\text{IC}})_{pp})_{\text{charac}} \), as per equation 2.5-2, was recalculated. The new increased IC debonding resistance \( ((P_{\text{IC}})_{pp})_{\text{charac}} \) will result in a higher debonding strain \( \varepsilon_{\text{db}} \), which will increase the bending moment resistance and lead to a higher flexural analysed load \( P_{\text{anal}} \). Table 5.10 presents the new proposed mean constant calculated by setting the analysed load \( P_{\text{anal}} \) equal to the experimentally applied load \( P_{\text{appl}} \).
This new mean constant was calculated by setting the analysed point load ($P_{anal}$) equal to the applied point load ($P_{appl}$), which was experimentally determined. Taking the average from Table 5.10, a new mean constant of 3.48 for 6 mm plates and 2.97 for 8 mm plates is proposed for Seracino et al.’s theory. Table 5.15 indicates that composite sections, plated with 6 mm and 8 mm plates, loaded with an MSLL loaded failed due to IC debonding. This analysis is therefore valid.

Table 5.11 compares the flexural analysed load ($P_{anal}$), as per Section 4.4.1.2, with the applied load ($P_{appl}$) of the plated reinforced concrete structural elements as per the hinge approach using Teng et al.’s (2002) approach. The results of the various failure modes are compared in Table 5.15 (in Section 5.5.5) and the most likely failure mechanism predicted.

### Table 5.10: New proposed mean constant of plated elements due to intermediate crack (IC) debonding – Seracino et al.’s (2007) theory

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Type of line load</th>
<th>Concrete cube strength ($f_{cu}$)</th>
<th>$M_{anal}$ (kNm)</th>
<th>$P_{anal} = P_{appl}$ (kN)</th>
<th>Calculated constant</th>
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<tbody>
<tr>
<td>Comp 1-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>50.85</td>
<td>52.72</td>
<td>2.65</td>
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<tr>
<td>Comp 2-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>57.08</td>
<td>60.79</td>
<td>2.97</td>
</tr>
<tr>
<td>Comp 3-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
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<td>52.26</td>
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<td>2, 110 x 6</td>
<td>TSLL</td>
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<td>83.42</td>
<td>3.43</td>
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<td>2, 110 x 6</td>
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<td>2.24</td>
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<td>52.24</td>
<td>5.20</td>
</tr>
<tr>
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<td>78.87</td>
<td>60.93</td>
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<td>3.95</td>
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<td>24.70</td>
<td>111.23</td>
<td>89.70</td>
<td>3.95</td>
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</table>
Table 5.11: Comparison between the analysed and the applied load due to intermediate crack (IC) debonding—Teng et al.’s (2002) theory

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Type of line load</th>
<th>Concrete cube strength ( f_{cu} ) (kNm)</th>
<th>( M_{anal} ) (kN)</th>
<th>( P_{anal} ) (kN)</th>
<th>( P_{appl} ) (kN)</th>
<th>( P_{appl}/P_{anal} )</th>
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<td>79.95</td>
<td>88.14</td>
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<td>24.70</td>
<td>59.21</td>
<td>63.81</td>
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<td>59.21</td>
<td>63.81</td>
<td>59.61</td>
<td>0.93</td>
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<td>2, 150 x 8 TSLL</td>
<td>23.10</td>
<td>99.22</td>
<td>117.14</td>
<td>79.25</td>
<td>0.68</td>
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<td>99.22</td>
<td>117.14</td>
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<td>1, 110 x 6 MSSL</td>
<td>24.70</td>
<td>44.69</td>
<td>30.51</td>
<td>54.36</td>
<td>1.78</td>
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<td>52.24</td>
<td>1.77</td>
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<td>48.13</td>
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<tr>
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<td>1, 150 x 8 MSSL</td>
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<td>59.20</td>
<td>43.40</td>
<td>49.60</td>
<td>1.14</td>
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</tr>
<tr>
<td>Comp 7-M</td>
<td>1, 150 x 8 MSSL</td>
<td>30.60</td>
<td>63.72</td>
<td>47.43</td>
<td>60.93</td>
<td>1.28</td>
<td></td>
</tr>
<tr>
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<td>24.70</td>
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<td>1.10</td>
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</tr>
<tr>
<td>Comp 9-M</td>
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<td>24.70</td>
<td>101.88</td>
<td>81.34</td>
<td>89.70</td>
<td>1.10</td>
<td></td>
</tr>
</tbody>
</table>

From the \( P_{appl}/P_{anal} \) ratio in Table 5.11 it is evident that Teng et al.’s theory would yield more accurate results than Seracino et al.’s theory if the applied loads were to be compared with the analysed loads. The applied loads (\( P_{appl} \)) measured during the tests were larger than the theoretically analysed loads (\( P_{anal} \)), except for the 8 mm bonded plates at the TSLL. Further research is suggested to establish whether Teng et al.’s theory is applicable to all thicknesses of bonded plates, especially thicker plates. Table 5.15 (in Section 5.5.5) indicates that composite sections plated with 6 mm and 8 mm plates and loaded with an MSSL failed due to IC debonding.

Teng et al.’s theory is suitable for the design of composite structural elements with 6 mm bonded plates because the analysed load (\( P_{anal} \)) is larger than the applied load (\( P_{appl} \)) measured during experimental testing. The results of the various failure modes are compared in Table 5.15 (in Section 5.5.5) and the most likely failure mechanism predicted.

5.5.2 Critical diagonal cracks (CDC) due to shear crack deformation

Critical diagonal crack (CDC) debonding is induced when shear deformation widens a crack through aggregate interlock, as described in Section 4.4.2. In Table 5.12 a comparison is made between the analysed load (\( P_{anal} \)) causing CDC debonding and the experimentally applied load
The shear capacity of the concrete slab without any shear reinforcement ($v_{uc}$) and with five Y12 reinforcing bars for flexure is 0.7894 MPa. The shear force ($V_{uc}$) is therefore 93.94 kN.

Table 5.12: Comparison between the analysed and the applied load due to critical diagonal crack (CDC) debonding

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Type of line load</th>
<th>Concrete cube strength ($f_{cu}$)</th>
<th>$V_{uc}$ + $\Delta V_{uc}$ Oehler &amp; Seracino, 2004 (kN)</th>
<th>$P_{anal}$ Seracino, 2007 (kN)</th>
<th>$P_{appl}$ (kN)</th>
<th>$P_{appl}/P_{anal}$ Seracino et al., 2007</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp 1-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>107.05</td>
<td>194.91</td>
<td>52.72</td>
<td>0.27</td>
</tr>
<tr>
<td>Comp 2-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>107.05</td>
<td>194.91</td>
<td>60.79</td>
<td>0.31</td>
</tr>
<tr>
<td>Comp 3-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>107.05</td>
<td>194.91</td>
<td>52.26</td>
<td>0.27</td>
</tr>
<tr>
<td>Comp 4-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>23.10</td>
<td>109.99</td>
<td>200.80</td>
<td>83.42</td>
<td>0.42</td>
</tr>
<tr>
<td>Comp 5-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>23.10</td>
<td>109.99</td>
<td>200.80</td>
<td>80.99</td>
<td>0.40</td>
</tr>
<tr>
<td>Comp 6-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>23.10</td>
<td>109.99</td>
<td>200.80</td>
<td>88.14</td>
<td>0.44</td>
</tr>
<tr>
<td>Comp 7-T</td>
<td>1, 150 x 8</td>
<td>TSLL</td>
<td>24.70</td>
<td>112.97</td>
<td>206.76</td>
<td>54.72</td>
<td>0.26</td>
</tr>
<tr>
<td>Comp 8-T</td>
<td>1, 150 x 8</td>
<td>TSLL</td>
<td>24.70</td>
<td>112.97</td>
<td>206.76</td>
<td>59.61</td>
<td>0.29</td>
</tr>
<tr>
<td>Comp 9-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>23.10</td>
<td>118.18</td>
<td>217.18</td>
<td>79.25</td>
<td>0.36</td>
</tr>
<tr>
<td>Comp 10-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>23.10</td>
<td>118.18</td>
<td>217.18</td>
<td>89.00</td>
<td>0.41</td>
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<tr>
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<td>1, 110 x 6</td>
<td>MSLL</td>
<td>24.70</td>
<td>107.05</td>
<td>196.77</td>
<td>54.36</td>
<td>0.28</td>
</tr>
<tr>
<td>Comp 2-M</td>
<td>1, 110 x 6</td>
<td>MSLL</td>
<td>23.10</td>
<td>104.68</td>
<td>192.04</td>
<td>52.24</td>
<td>0.27</td>
</tr>
<tr>
<td>Comp 3-M</td>
<td>1, 110 x 6</td>
<td>MSLL</td>
<td>30.60</td>
<td>114.97</td>
<td>212.62</td>
<td>56.23</td>
<td>0.26</td>
</tr>
<tr>
<td>Comp 4-M</td>
<td>2, 110 x 6</td>
<td>MSLL</td>
<td>24.70</td>
<td>112.48</td>
<td>207.63</td>
<td>61.69</td>
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<tr>
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<td>MSLL</td>
<td>23.10</td>
<td>109.99</td>
<td>202.66</td>
<td>64.09</td>
<td>0.32</td>
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<tr>
<td>Comp 6-M</td>
<td>1, 150 x 8</td>
<td>MSLL</td>
<td>24.70</td>
<td>112.97</td>
<td>208.62</td>
<td>49.60</td>
<td>0.24</td>
</tr>
<tr>
<td>Comp 7-M</td>
<td>1, 150 x 8</td>
<td>MSLL</td>
<td>30.60</td>
<td>121.34</td>
<td>225.35</td>
<td>60.93</td>
<td>0.27</td>
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<tr>
<td>Comp 8-M</td>
<td>2, 150 x 8</td>
<td>MSLL</td>
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<td>89.58</td>
<td>0.40</td>
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<td>120.86</td>
<td>224.39</td>
<td>89.70</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Using Seracino et al.’s theory to compare the analysed load ($P_{anal}$) causing CDC debonding with the experimentally applied load ($P_{appl}$), as indicated in Table 5.12, it evident that the analysed load ($P_{anal}$) is larger than the experimentally applied load ($P_{appl}$). This leads to the assumption that failure of the composite structural elements is not likely to occur due to CDC debonding. The results of the various failure modes are compared in Table 5.15 (in Section 5.5.5) and the most likely failure mechanism predicted.

5.5.3 Plate end debonding (PE) due to curvature

Plate end debonding is induced by the interface normal stresses due to the discontinuity of the plate and is associated with the curvature of the bonded plate. In Table 5.13 a comparison is made between the analysed load ($P_{anal}$) causing the plate end to debond and the experimentally applied load ($P_{appl}$).
Table 5.13: Comparison between the analysed and the applied load due to plate end (PE) debonding

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Type of line load</th>
<th>Concrete cube strength (f_{cu})</th>
<th>M_{anal} at plate end (kNm)</th>
<th>P_{anal} (kN)</th>
<th>P_{appl} (kN)</th>
<th>P_{appl}/P_{anal}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp 1-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>5.98</td>
<td>41.32</td>
<td>52.72</td>
<td>1.28</td>
</tr>
<tr>
<td>Comp 2-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>5.98</td>
<td>41.32</td>
<td>60.79</td>
<td>1.47</td>
</tr>
<tr>
<td>Comp 3-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>5.98</td>
<td>41.32</td>
<td>52.26</td>
<td>1.26</td>
</tr>
<tr>
<td>Comp 4-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>23.10</td>
<td>8.59</td>
<td>67.43</td>
<td>83.42</td>
<td>1.24</td>
</tr>
<tr>
<td>Comp 5-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>23.10</td>
<td>8.59</td>
<td>67.43</td>
<td>80.99</td>
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</tr>
<tr>
<td>Comp 6-T</td>
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<td>TSLL</td>
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<td>8.59</td>
<td>67.43</td>
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<td>45.84</td>
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<td>54.36</td>
<td>1.26</td>
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<td>1.29</td>
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<td>73.21</td>
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<tr>
<td>Comp 5-M</td>
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<td>6.43</td>
<td>47.70</td>
<td>49.60</td>
<td>1.04</td>
</tr>
<tr>
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<td>1, 150 x 8</td>
<td>MSLL</td>
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<td>60.93</td>
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<td>MSLL</td>
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<td>9.89</td>
<td>82.31</td>
<td>89.70</td>
<td>1.09</td>
</tr>
</tbody>
</table>

From a comparison of the analysed load \( (P_{anal}) \) causing PE debonding and the experimentally applied load \( (P_{appl}) \) in Table 5.13 it can be seen that the variance between these values is small. This is an indication that PE debonding in composite structural elements is likely to occur. The results of the various failure modes are compared in Table 5.15 (in Section 5.5.5) and the most likely failure mechanism predicted.

5.5.4 \( VAy/Ib \) debonding due to shear capacity

The interface shear stress \( (\tau) \) induced by transverse or vertical shear forces must be limited to the tensile strength of the concrete as described in Section 4.4.4. In Table 5.14 a comparison is made between the analysed load \( (P_{anal}) \) causing the interface shear stress and the experimentally applied load \( (P_{appl}) \).
Table 5.14: Comparison between the analysed and the applied load due to $VA_y/I_b$ debonding

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Type of line load</th>
<th>Concrete cube strength ($f_{cu}$)</th>
<th>$I_e$ x $10^6$ (mm$^4$)</th>
<th>$V_{anal}$ (kNm)</th>
<th>$P_{anal}$ (kN)</th>
<th>$P_{appl}$ (kN)</th>
<th>$P_{appl}/P_{anal}$</th>
</tr>
</thead>
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<tr>
<td>Comp 1-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>24.70</td>
<td>114.80</td>
<td>48.46</td>
<td>77.74</td>
<td>52.72</td>
<td>0.68</td>
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<tr>
<td>Comp 2-T</td>
<td>1, 110 x 6</td>
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<td>24.70</td>
<td>114.80</td>
<td>48.46</td>
<td>77.74</td>
<td>60.79</td>
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</tr>
<tr>
<td>Comp 3-T</td>
<td>1, 110 x 6</td>
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<td>114.80</td>
<td>48.46</td>
<td>77.74</td>
<td>52.26</td>
<td>0.67</td>
</tr>
<tr>
<td>Comp 4-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>23.10</td>
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<td>75.96</td>
<td>132.73</td>
<td>83.42</td>
<td>0.63</td>
</tr>
<tr>
<td>Comp 5-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>23.10</td>
<td>172.40</td>
<td>75.96</td>
<td>132.73</td>
<td>80.99</td>
<td>0.61</td>
</tr>
<tr>
<td>Comp 6-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
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<td>75.96</td>
<td>132.73</td>
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<td>0.66</td>
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<tr>
<td>Comp 7-T</td>
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<td>91.99</td>
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<tr>
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<td>55.59</td>
<td>91.99</td>
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<tr>
<td>Comp 9-T</td>
<td>2, 150 x 8</td>
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<td>166.20</td>
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<td>MSLL</td>
<td>23.10</td>
<td>172.40</td>
<td>75.96</td>
<td>134.59</td>
<td>64.09</td>
<td>0.48</td>
</tr>
<tr>
<td>Comp 6-M</td>
<td>1, 150 x 8</td>
<td>MSLL</td>
<td>24.70</td>
<td>164.60</td>
<td>55.59</td>
<td>93.85</td>
<td>49.60</td>
<td>0.53</td>
</tr>
<tr>
<td>Comp 7-M</td>
<td>1, 150 x 8</td>
<td>MSLL</td>
<td>30.60</td>
<td>164.60</td>
<td>64.12</td>
<td>110.92</td>
<td>60.93</td>
<td>0.55</td>
</tr>
<tr>
<td>Comp 8-M</td>
<td>2, 150 x 8</td>
<td>MSLL</td>
<td>24.70</td>
<td>253.10</td>
<td>96.92</td>
<td>176.52</td>
<td>89.58</td>
<td>0.51</td>
</tr>
<tr>
<td>Comp 9-M</td>
<td>2, 150 x 8</td>
<td>MSLL</td>
<td>24.70</td>
<td>253.10</td>
<td>96.92</td>
<td>176.52</td>
<td>89.70</td>
<td>0.51</td>
</tr>
</tbody>
</table>

From this table it is evident that the analysed load ($P_{anal}$) is larger than the theoretically applied load ($P_{appl}$). This leads to the conclusion that failure of the composite structural elements is not likely to occur due to $VA_y/I_b$ debonding. The results of the various failure modes are compared in Table 5.15 (in Section 5.5.5) and the most likely failure mechanism predicted.

5.5.5 Determining the most likely debonding mechanism

A summary of the analysed loads ($P_{anal}$) for all the debonding mechanisms is given in Table 5.15. The smallest analysed load ($P_{anal}$) is the most likely debonding mechanism.
Table 5.15: Comparison of the debonding failure modes and the most likely debonding mechanisms

<table>
<thead>
<tr>
<th>Slab name</th>
<th>Number and size of steel plates (mm)</th>
<th>Type of line load</th>
<th>$P_{anal}$ IC debonding Seracino et al. (kN)</th>
<th>$P_{anal}$ IC debonding Teng et al. (kN)</th>
<th>$P_{anal}$ CDC debonding (kN)</th>
<th>$P_{anal}$ PE debonding (kN)</th>
<th>$P_{anal}$ VAy/Ib debonding (kN)</th>
<th>Most likely debonding mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>Comp 1-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>11.51</td>
<td>63.81</td>
<td>206.76</td>
<td>45.84</td>
<td>91.99</td>
<td>PE</td>
</tr>
<tr>
<td>Comp 2-T</td>
<td>1, 110 x 6</td>
<td>TSLL</td>
<td>4.87</td>
<td>33.56</td>
<td>212.62</td>
<td>52.36</td>
<td>94.47</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 3-T</td>
<td>2, 110 x 6</td>
<td>TSLL</td>
<td>8.39</td>
<td>47.43</td>
<td>224.39</td>
<td>82.31</td>
<td>176.52</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 4-T</td>
<td>1, 150 x 8</td>
<td>TSLL</td>
<td>10.71</td>
<td>81.34</td>
<td>224.39</td>
<td>82.31</td>
<td>176.52</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 5-T</td>
<td>1, 150 x 8</td>
<td>TSLL</td>
<td>10.71</td>
<td>81.34</td>
<td>224.39</td>
<td>82.31</td>
<td>176.52</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 6-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>10.71</td>
<td>81.34</td>
<td>224.39</td>
<td>82.31</td>
<td>176.52</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 7-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>10.71</td>
<td>81.34</td>
<td>224.39</td>
<td>82.31</td>
<td>176.52</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 8-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>10.71</td>
<td>81.34</td>
<td>224.39</td>
<td>82.31</td>
<td>176.52</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 9-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>10.71</td>
<td>81.34</td>
<td>224.39</td>
<td>82.31</td>
<td>176.52</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 10-T</td>
<td>2, 150 x 8</td>
<td>TSLL</td>
<td>10.71</td>
<td>81.34</td>
<td>224.39</td>
<td>82.31</td>
<td>176.52</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 1-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 2-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 3-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 4-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 5-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 6-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 7-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 8-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
<tr>
<td>Comp 9-M</td>
<td>1, 110 x 6</td>
<td>MSL</td>
<td>2.04</td>
<td>32.20</td>
<td>196.77</td>
<td>43.18</td>
<td>79.60</td>
<td>IC</td>
</tr>
</tbody>
</table>

In order to establish the most likely debonding mechanism, the smallest theoretically analysed point loads ($P_{anal}$) from the following debonding mechanisms were selected:
- IC debonding – Seracino et al.’s theory
- IC debonding – Teng et al.’s theory
- CDC debonding
- PE debonding
- VAy/Ib debonding

The results of Seracino et al.’s IC debonding theory were not considered as the analysed point load ($P_{anal}$) values were too small. In Table 5.15 the point loads due to all the debonding mechanisms and the most likely debonding mechanism are indicated.

It is evident that PE debonding is predicted when a TSLL is applied to the reinforced concrete structural element strengthened with externally bonded steel plate/s. This PE debonding failure mode is a result of a TSLL which presents a bending moment diagram similar to that of a
uniformly distributed load (UDL) load. As the curvature of a composite structural element increases and the plate tries to stay straight, normal interface tensile and compressive forces are formed which result in interface cracks and debonding of the steel plate end. This failure mode is described in Section 2.5.3.

IC debonding is predicted when an MSLL is applied to the reinforced concrete structural element. The mid-span moment due to an MSLL is larger than that due to a TSLL, and the point load in the form of an MSLL also occurs at mid-span. Flexural IC cracks form at mid-span, resulting in horizontal IC interface cracks when they intercept the bonded plates. When these cracks join together, debonding of the steel plate occurs. This failure mode is described in Section 2.5.1

Figure 5.8 illustrates the steel plate end debonding from the concrete surface and Figure 5.9 illustrates IC debonding. If debonding of the plate occurs due to both PE and IC, composite action is lost and the structural member behaves as an ordinary reinforced concrete structural element.
5.6 Failure stages of a composite slab

The different failure stages of a composite slab during loading are described using the graph of Comp 2-T in Figure 5.10. The composite slab under consideration was strengthened by one 110 x 6 mm plate and showed ductility due to the yielding of the steel plate and the reinforcement.
The typical failure stages of a composite concrete-steel structural element during loading are as follows:

1. At approximately 10 kN the concrete in the tension zone cracks.
2. At approximately 55 kN the steel reinforcement and/or steel plates start yielding.
3. At approximately 60 kN the steel plates debond and composite action is lost.
4. Once debonding has occurred, the load-bearing capacity of the slab decreases dramatically from approximately 60 kN as a composite member to approximately 10 kN as an reinforced concrete member.

Figure 5.10: Failure stages of a composite slab during loading
Chapter 6  Conclusions

6.1  Conclusions reached

The flexural strength of reinforced concrete structural elements increases significantly when steel plates are bonded to the concrete surface by means of epoxy as external reinforcement. The average increases are determined by comparing the experimentally measured point load acting on the unplated reinforced concrete slab with that acting on the plated slab and are as follows:

- TSLL with one 110 x 6 mm plate is 114%.
- TSLL with two 110 x 6 mm plates is 217%.
- TSLL with one 150 x 8 mm plate is 122%.
- TSLL with two 150 x 8 mm plates is 226%.
- MSLL with one 110 x 6 mm plate is 166%.
- MSLL with two 110 x 6 mm plates is 208%.
- MSLL with one 150 x 8 mm plate is 171%.
- MSLL with two 150 x 8 mm plates is 339%.

The deflection of composite structural elements is significantly stiffer than that of ordinary reinforced concrete structural elements. A comparison with a load of 15 kN applied to both types of element shows an average increase in stiffness as indicated below:

- TSLL with one 110 x 6 mm plate is 297%.
- TSLL with two 110 x 6 mm plates is 390%.
- TSLL with one 150 x 8 mm plate is 511%.
- TSLL with two 150 x 8 mm plates is 543%.
- MSLL with one 110 x 6 mm plate is 261%.
- MSLL with two 110 x 6 mm plates is 327%.
- MSLL with one 150 x 8 mm plate is 346%.
- MSLL with two 150 x 8 mm plates is 458%.

A comparison between the experimentally measured deflection ($y_{appl}$) and the theoretically analysed deflection ($y_{anal}$) reveals that when a TSLL is applied, the theoretically analysed deflection ($y_{anal}$) is on average 24% lower than the measured deflection ($y_{appl}$). With an MSLL applied, the theoretically analysed deflection ($y_{anal}$) is on average 11% higher than the measured deflection ($y_{appl}$).

The theoretically analysed values of the flexural, shear, curvature and concrete tensile strength resistance of plated slabs were compared with the experimentally measured values. This comparison was between the experimentally applied point loads ($P_{appl}$) and the analysed point loads ($P_{anal}$) associated with the theoretically calculated resistances. The following four plate debonding theories were considered:

- For flexure, the intermediate crack (IC) debonding theory of Seracino et al. (2007)
- For flexure, the intermediate crack (IC) debonding theory of Teng et al. (2002)
- For shear, the critical diagonal crack (CDC) debonding theory
- For curvature, the plate end (PE) debonding theory
For the tensile strength of concrete, the VAy/Ib debonding theory

For **IC debonding** the two different theories of Seracino et al. (2007) and Teng et al. (2002) were considered and it was found that Teng et al.’s theory yielded more accurate results (see Section 5.5.1):

- The theory of Seracino et al. did not yield accurate flexural results. The analysed point loads ($P_{anai}$) were on average 798% lower than the experimentally applied point loads ($P_{appl}$). A revised constant for Seracino et al.’s equation was calculated to suit the 6 mm and 8 mm bonded steel plates.
- The theoretically analysed point loads ($P_{anai}$), according to the IC debonding theory of Teng et al., yielded accurate results compared with the experimentally applied point loads ($P_{appl}$). For design purposes using steel plates 6 mm and 8 mm thick, this theory is recommended. IC debonding failure was evident in slabs with an MSLL due to large bending moments creating excessive cracking at mid-span.

**Critical diagonal crack (CDC) debonding** did not occur in this research study.

**Plate end (PE) debonding** was evident in this research study. PE debonding failure was evident in slabs with a TSLL.

**VAy/Ib debonding** did not occur in this research study. Previous research indicates that this failure mode is not critical.

The observed failure mechanisms correlate well with the analytically predicted mechanisms and the overall conclusions to be drawn from this research study are the following:

- **PE debonding** occurs when a TSLL is applied to a reinforced concrete structural element strengthened with externally bonded steel plate/s as per Table 5.15. This PE debonding failure mode is a result of a TSLL that presents a bending moment diagram similar to that of a UDL load. As the curvature of a composite structural element increases and the plate tries to stay straight, normal interface tensile and compressive forces are formed, resulting in interface cracks and debonding of the steel plate end. This failure mode is described in Section 2.5.3.
- **IC debonding** occurs when an MSLL is applied to a reinforced concrete structural element strengthened with externally bonded steel plate/s as per Table 5.15. This IC debonding failure mode is a result of flexural cracks forming in the vicinity of the point load in the form of an MSLL. When the vertical IC cracks widen due to increased loads and intercept with the bonded plates, horizontal IC interface cracks are formed; when these cracks join together, they cause debonding of the steel plate. This failure mode is described in Section 2.5.1.
6.2 Future research

From experience gained through this research study the necessity arises to undertake further research into the debonding mechanisms of steel plates thicker than 3 mm. Research has been conducted previously on structural elements with bonded steel plates having a thickness of up to 3 mm. It is proposed that structural elements with bonded steel plate/s thicker than 3 mm should be analysed for the following five debonding mechanisms:

- For flexure, the intermediate crack (IC) debonding theory of Seracino et al. (2007)
- For flexure, the intermediate crack (IC) debonding theory of Teng et al. (2002)
- For shear, the critical diagonal crack (CDC) debonding theory
- For curvature, the plate end (PE) debonding theory
- For the tensile strength of concrete, the \( \frac{V_{Ay}}{I_b} \) debonding theory.
Chapter 7 References


Chapter 8  Annexure

8.1  Control slab 1-T

Figure 8.1:  Control slab 1-T load-deflection graph
8.2 Control slab 2-T

![Control slab 2-T load-deflection graph](image)

Figure 8.2: Control slab 2-T load-deflection graph
8.3 Control slab 3-T

![Control slab 3-T load-deflection graph](image)

**Figure 8.3:** Control slab 3-T load-deflection graph
8.4 Composite slab 1-T

Figure 8.4: Composite slab 1-T load-deflection graph

Figure 8.5: Composite slab 1-T load-strain graph
8.5 Composite slab 2-T

Figure 8.6: Composite slab 2-T load-deflection graph

Figure 8.7: Composite slab 2-T load-strain graph
8.6 Composite slab 3-T

Figure 8.8: Composite slab 3-T load-deflection graph

Figure 8.9: Composite slab 3-T load-strain graph
8.7 Composite slab 4-T

Figure 8.10: Composite slab 4-T load-deflection graph

Figure 8.11: Composite slab 4-T load-strain graph
8.8 Composite slab 5-T

![Composite slab 5-T load-deflection graph](image1)

**Figure 8.12**: Composite slab 5-T load-deflection graph

![Composite slab 5-T load-strain graph 1](image2)

**Figure 8.13**: Composite slab 5-T load-strain graph 1
Figure 8.14: Composite slab 5-T load-strain graph 2
8.9 Composite slab 6-T

Figure 8.15: Composite slab 6-T load-deflection graph
Figure 8.16: Composite slab 6-T load-strain graph 1

Figure 8.17: Composite slab 6-T load-strain graph 2
8.10 Composite slab 7-T

Figure 8.18: Composite slab 7-T load-deflection graph

Figure 8.19: Composite slab 7-T load-strain graph
8.11 Composite slab 8-T

Figure 8.20: Composite slab 8-T load-deflection graph

Figure 8.21: Composite slab 8-T load-strain graph
8.12 Composite slab 9-T

Figure 8.22: Composite slab 9-T load-deflection graph

Figure 8.23: Composite slab 9-T load-strain graph 1
Figure 8.24: Composite slab 9-T load-strain graph 2

8.13 Composite slab 10-T

Figure 8.25: Composite slab 10-T load-deflection graph
Figure 8.26: Composite slab 10-T load-strain graph 1

Figure 8.27: Composite slab 10-T load-strain graph 2
8.14 Control slab 1-M

![Control slab 1-M load-deflection graph](image)

Figure 8.28: Control slab 1-M load-deflection graph
8.15 Control slab 2-M

Figure 8.29: Control slab 2-M load-deflection graph
8.16 Composite slab 1-M

Figure 8.30: Composite slab 1-M load-deflection graph

Figure 8.31: Composite slab 1-M load-strain graph
8.17 Composite slab 2-M

Figure 8.32: Composite slab 2-M load-deflection graph

Figure 8.33: Composite slab 2-M load-strain graph
8.18 Composite slab 3-M

Figure 8.34: Composite slab 3-M load-deflection graph

Figure 8.35: Composite slab 3-M load-strain graph
8.19 Composite slab 4-M

Figure 8.36: Composite slab 4-M load-deflection graph

Figure 8.37: Composite slab 4-M load-strain graph 1
Figure 8.38: Composite slab 4-M load-strain graph 2

8.20 Composite slab 5-M

Figure 8.39: Composite slab 5-M load-deflection graph
Figure 8.40: Composite slab 5-M load-strain graph 1

Figure 8.41: Composite slab 5-M load-strain graph 2
8.21 Composite slab 6-M

Figure 8.42: Composite slab 6-M load-deflection graph

Figure 8.43: Composite slab 6-M load-strain graph
8.22 Composite slab 7-M

Figure 8.44: Composite slab 7-M load-deflection graph

Figure 8.45: Composite slab 7-M load-strain graph
8.23 Composite slab 8-M

Figure 8.46: Composite slab 8-M load-deflection graph

Figure 8.47: Composite slab 8-M load-strain graph 1
Figure 8.48: Composite slab 8-M load-strain graph 2

8.24 Composite slab 9-M

Figure 8.49: Composite slab 9-M load-deflection graph
Figure 8.50: Composite slab 9-M load-strain graph 1

Figure 8.51: Composite slab 9-M load-strain graph 2