BEHAVIOUR OF COMPOSITE CROSS-SECTIONS
WITH PARTIAL SHEAR CONNECTION

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BEHAVIOUR OF COMPOSITE CROSS-SECTIONS WITH PARTIAL SHEAR CONNECTION

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ABSTRACT

This report considers the ductility and behaviour of composite cross-sections with partial shear connection in sagging bending. A numerical method of analysis has been developed to study the full range of the cross-section response through its moment versus curvature relationship with partial shear connection. Parametric studies are made of the influence of material strength, various cross-section geometries, reinforcement in the slab and slip strain between the slab and joist. The ramifications of these on the strength and ductility of composite beams are discussed.

KEY WORDS

Composite beams, cross-section behaviour, ductility, numerical model, partial shear connection, rigid plastic analysis, strength.
Contents

1. INTRODUCTION 3
2. RIGID PLASTIC CROSS-SECTION ANALYSIS UNDER PSC 5
3. CROSS-SECTION ANALYSIS 6
   3.1 General 6
   3.2 Analysis Procedure 7
   3.3 Validation of Numerical Procedure 9
4. PARAMETRIC STUDY 10
   4.1 General 10
   4.2 Effect of Degree of Shear Connection 11
   4.3 Effect of Concrete Strength 12
   4.4 Effect of Slab Dimensions 13
   4.5 Effect of Steel Joist Size 13
   4.6 Effect of Slab Reinforcement 13
   4.7 Slip Strain Between Joist and Slab 14
5. CONCLUSIONS 14
6. ACKNOWLEDGEMENTS 15
7. REFERENCES 16
1. INTRODUCTION

This report addresses the issue of simplified rigid-plastic assumptions in the ultimate strength analysis of composite steel-concrete T-beams. The advantages that accrue to plastic analysis of statically indeterminate structures result from the ductility of plastic hinges, and their ability to redistribute bending moment while rotating without softening behaviour until a global plastic collapse mechanism has formed. For steel structures, the major requirements for hinge ductility are for the cross-sections to be compact with sufficient lateral bracing, and for reinforced concrete structures ductility is enforced by limiting the depth of the neutral axis at attainment of the ultimate moment. However, at ultimate conditions composite T-beams require a transfer of force between the steel and concrete at the interface to be effected by the shear connection. If the strength of the shear connection limits this force transfer, a condition of partial shear connection (PSC) is developed. The fully plastic moment under a condition of PSC is calculated easily by making rigid plastic constitutive assumptions for the steel, concrete and shear connection (Oehlers and Bradford 1995, 1999). However, the ductility of cross-sections with PSC whose strength has been determined using idealised simple rigid plastic analysis does not appear to have been studied in detail, and this issue is the subject of the present work.

Owing to the monosymmetry of the cross-sections of composite T-beams in both their geometric and constitutive properties, the fully plastic moments in negative (hogging) and positive (sagging) bending are different, as are the moment-curvature responses. Provided that instability effects are eliminated (Bradford and Kemp 2000), hogging plastic hinges are often thought of as ‘hardening’, in that the properties of the strain-hardening steel joist and reinforcement govern the moment - rotation characteristics of the hogging hinge and thereby usually permit the necessary rotation in the formation of a plastic mechanism without unloading. On the other hand, sagging plastic hinges are often thought of as ‘softening’, since the concrete component of the composite cross-section does not often display ductile behaviour. When using plastic analysis at ultimate load conditions, the formation of a plastic mechanism is reliant upon the fully plastic moment in sagging bending to be attained and to possess a quantifiable and sufficient rotation capacity, that is, to have sufficient ductility.
The ductility of composite cross-sections in sagging bending was studied by Rotter and Ansourian (1979), and based on work by Ansourian (1982a,b) the limitation of a contrived ductility parameter was proposed to allow the necessary rotation capacity at a sagging hinge to develop so that a plastic collapse mechanism is formed. The ductility of composite cross-sections with prestressing in the tensile region under sagging was also considered by Uy and Bradford (1993), but these studies did not consider the effect of PSC in the generation of the moment-curvature curves that were used to assess ductility. In a composite beam, the degree of shear connection varies along the beam from a (theoretical) value of zero at a simple support where the strength of the beam is that of the steel joist alone, to a position of full shear connection (FSC) where a plastic hinge may form without being limited by the strength of the shear connection, or to a position of PSC where a plastic hinge may form nevertheless but whose strength is governed necessarily by the strength of the shear connection. Economic constraints or physical constraints in the detailing of the shear connection topology often limit a composite beam to a condition of PSC along its entire length, and for such beams it is necessary to demonstrate the ductility in sagging bending for plastic analysis to be applicable.

Rigid plastic analysis is an idealisation that the load versus deformation characteristics of a material are such that it is not stressed at all, or if it deforms that it is stressed at its ultimate strength with infinite deformation without softening. While this idealisation may have some justification for mild steel sections, it is questionable when applied to concrete, and to stud shear connectors which have limited slip capacities (Johnson and Molenstra 1991; Oehler and Sved 1995; Oehler and Bradford 1995, 1999; Uy et al. 1998; Diedricks et al. 1999). The rational nonlinear analysis of a composite beam that incorporates the material nonlinearities associated with the steel joist, reinforced concrete slab and stud shear connectors is complicated, as geometric nonlinearities arise owing to interface slip which is dependent on the load-slip characteristics of the connectors, and the rate of change of slip with respect to the length of the beam (slip strains) associated with the flexural deformations of the beam. Because of this, it has been proposed (Uy et al. 2000) that rigid plastic analysis of a beam incorporating PSC is admissible, provided that the moment to cause fracture of the stud shear connectors caused by excessive slip can be determined. In this way, the interaction of structural behaviour in the plane of the cross-section (plastic moments of resistance under PSC), and longitudinally out of the plane of the cross-section (slip deformations along the length of the beam), are eliminated.
Although the attainment of the collapse load of a composite beam is dependent on the interaction of the behaviour at the cross-section level and at the beam level, this report addresses the ductility of a specific cross-section with PSC. The behaviour of the shear connection is assumed to be rigid plastic herein, however realistic constitutive properties for the steel joist and concrete are used in the development of the model for the moment-curvature response of a cross-section, and from this conclusions are drawn regarding the attainment of the rigid plastic moment and deformation characteristics of the plastic hinge. This philosophy of analysis is called a mixed-analysis approach (Oehlers and Sved 1995). Parametric studies are made of the influence of material strength, various cross-section geometries and reinforcement in the slab. The ramifications of these on the strength and ductility of composite T-beams are discussed.

2. RIGID PLASTIC CROSS-SECTION ANALYSIS UNDER PSC

At this point, it is appropriate to define the flexural strength \((M_p)_{\text{PSC}}\) of a composite cross-section in sagging bending with PSC based on simplified rigid plastic assumptions. Figure 1 shows schematically the rigid plastic load versus deformation response of the constituent materials of the cross-section. For the steel joist, the strength is assumed to be the yield stress \(f_y\) in compression and \(-f_y\) in tension, for the concrete slab the strength is assumed to be the concrete strength \(0.85f_c\) (where \(f_c\) is the short-term cylinder strength) in compression and zero in tension (Warner et al. 1998), and at the cross-section under consideration, the strength of the shear connection is defined as

\[
(P_{sh})_{\text{PSC}} = \eta (P_{sh})_{\text{FSC}}
\]

(1)

where \((P_{sh})_{\text{FSC}}\) is the strength of the cross-section with FSC, and \(0 \leq \eta \leq 1\) is defined as the degree of shear connection. The strength \((P_{sh})_{\text{FSC}}\) is taken to be the minimum strength of the shear connection for which failure of the steel joist or concrete slab precedes failure of the shear connection in a rigid plastic analysis.

Figure 2 shows a cross-section in flexure with PSC at ultimate, assuming rigid plastic behaviour. Since the strength of the shear connection \((P_{sh})_{\text{FSC}}\) governs, and from equilibrium
shear connection for which failure of the steel joist or concrete slab precedes failure of the shear connection in a rigid plastic analysis.

Figure 2 shows a cross-section in flexure with PSC at ultimate, assuming rigid plastic behaviour. Since the strength of the shear connection \((P_{sh})_{PSC}\) governs, and from equilibrium at the steel/concrete interface, the force in the concrete \(F_c = (P_{sh})_{PSC}\) and the force in the steel is \(-F_s = -(P_{sh})_{PSC}\). The rigid plastic assumption for the concrete implies that it has no flexural strength, and when coupled with the PSC condition the implication is that not all of the slab is stressed at 0.85\(f_c\) as shown in Fig. 2. Further, the rigid plastic assumption and the condition of PSC imply that the steel is not fully stressed (in tension) at \(-f_r\) as shown in Fig. 2, and so some of the steel is stressed at \(f_s\) and some at \(-f_s\) producing an axial stress resultant of \(-(P_{sh})_{PSC}\) and a bending stress resultant of \(M_s\). If \(h\) is the distance between the axial stress resultants of \((P_{sh})_{PSC}\) in the concrete and \(-(P_{sh})_{PSC}\) in the steel, then the flexural strength of the cross-section with PSC \((M_p)_{PSC}\) is

\[
(M_p)_{PSC} = M_s + \eta(P_{sh})_{PSC} h
\]  

(2)

With a knowledge of \(\eta\) and of the geometry of the concrete slab and steel joist, the position of the neutral axis in the slab may be determined readily, as may be that in the steel joist. The lever arm \(h\) is then defined, and hence so is \((M_p)_{PSC}\) from eqn. (2). Note that the rigid plastic assumptions with \(0 < \eta < 1\) necessarily lead to the existence of two neutral axes.

3. CROSS-SECTION ANALYSIS

3.1 General

A cross-sectional analysis method was developed to model the full moment-curvature behaviour of the beam. The curvature \(\kappa\) is increased monotonically and the moment is computed in a similar fashion to that described by Rotter and Anisourian (1979) and Uy and Bradford (1993), but the effects of PSC with these procedures are augmented herein. The analysis makes the following assumptions:
hardening moduli respectively, \( \varepsilon_y = f_y/E \) is the yield strain and \( \gamma \) defines the strain at the onset of strain hardening.

- The rigid plastic assumption in Fig. 4 is made for the shear connection as the analysis is for a cross-section at which both the slip and slip strain are unknown (since these depend on a lengthwise member analysis as well). In reality, the slip deformation of stud shear connectors is limited by fracture within the slab (Oehlers and Bradford 1995, Oehlers and Sved 1995), but is taken herein to be infinite.

- The concrete is modelled using the CEB-FIP constitutive representation (CEB-FIP 1970), shown in Fig. 4, where \( \sigma_Y = 0.85f_c \) represents the peak stress on the stress-strain, and the short-term cylinder strength \( f_c \) is expressed in units of MPa (N/mm²).

- Secondary failures of the shear connection are prevented and the web of the beam is adequate to resist the transverse shear.

The degree of shear connection \( \eta \) is assumed to be known for the cross-section \textit{a priori}, and so the value of \( (P_{sl})_{PSC} \) is known from eqn. (1).

### 3.2 Analysis Procedure

The strain diagram for a composite cross-section with PSC subjected to a curvature \( \kappa \) is illustrated in Fig. 6, where the positions of the two neutral axes (as argued in the construction of Fig. 2) are defined by the depths \( d_{nc} \) and \( d_{ns} \). At any value of \( \kappa \), the strain distribution is defined by the position of the neutral axes as

\[
\varepsilon = \varepsilon(\kappa, d_{nc}, d_{ns}) \tag{3}
\]
and from the constitutive relationships for the steel and concrete, the stresses throughout the cross-section may be determined and hence the axial force \( N(\kappa, d_{nc}, d_{nt}) \) by numerical integration of the stresses over the cross-section. This numerical process is performed by subdividing the cross-section into a large number of small slices of depth \( \Delta y \) whose stress is assumed constant throughout the depth at the value of the mid-height of the slice (shown in Fig. 6(a)). Starting with a condition of FSC at zero applied curvature, an initial assumption of FSC that \( d_{as} = d_{nt} \) is made, and the curvature \( \kappa \) is incremented in small steps from zero. For each value of curvature \( \kappa \), a trial position of the coincident neutral axes is selected and this position is iterated by using the bisections algorithm until the equilibrium condition that \( N = 0 \) defines the position of the coincident neutral axes \( d_{nc} = d_{as} \), shown in Fig. 6(b). Integration of the first moment of the stresses about the top of the cross-section produces the bending moment \( M \).

At each computed value of \( \kappa \) incremented from zero, the forces in the concrete slab \( F_c \) of area \( A_c \) and in the steel joist \( F_s \) of area \( A_s \) given by

\[
F_c = \int \sigma_c(\varepsilon) \, dA
\]

and

\[
F_s = \int \sigma_s(\varepsilon) \, dA
\]

are computed, and the condition that

\[
F_c \leq (P_{cr})_{PSC} \quad \text{and} \quad F_s \leq (P_{cr})_{PSC}
\]

is then tested. When eqn. (6) is violated, the force in the concrete or steel exceeds that which can be transmitted through the shear connection, and an inadmissible strength condition is established. However, when the curvature reaches a critical value \( \kappa_{PSC} \) such that the inequality in eqn. (6) is just violated, a condition of PSC is reached. This critical curvature
can again be determined numerically by a simple bisections algorithm. For incrementing curvature beyond $\kappa_{PSC}$, the two equilibrium conditions that

$$ N = 0 \quad \text{and} \quad F_r = (P_m)_{PSC} \quad (7) $$

must be enforced and for this to occur the positions of the neutral axes $d_{nc}$ and $d_{ns}$ must be different, as shown in Fig. 6(c). The above solution procedure therefore involves a two-step iterative process to determine the location of the two neutral axes $d_{nc}$ and $d_{ns}$ once $\kappa > \kappa_{PSC}$ and the cross-section has entered a condition of PSC. This can be done simply by use of a bisections algorithm in two dimensions, so that eqns. (7) are satisfied to an acceptable tolerance.

### 3.3 Validation of Numerical Procedure

In order to demonstrate the accuracy of the computer model, the results from the method herein have been compared with independent numerical results presented by Rotter and Ansourian (1979), and experimental results reported by Uy and Sloane (1998).

In the work of Rotter and Ansourian (1979), all of the composite cross-sections were assumed to have full shear connection, and in one set of graphs, partial interaction was considered by expressing the slip strain $\varepsilon_s$ as a linear function of the curvature $\kappa$ by

$$ \varepsilon_s = \delta \kappa \left( h_c + \frac{h_s}{2} \right) \quad (8) $$

where $h_c$ and $h_s$ are depths of slab and steel joist respectively, as shown in Fig. 2, $\delta$ is used to define loosely the degree of interaction (in deference to the degree of shear connection), and varies from 0 to 0.5 in the curves in Rotter and Ansourian. For comparison, the cross-section geometry and material strengths used in this illustration are identical to those used by Rotter and Ansourian (1979), using a 200UB29.8 joist with yield stress $f_y = 350$ MPa, and two slab sizes ($915\times102$ mm$^2$ and $1830\times102$ mm$^2$) with a concrete strength $f_c = 32$ MPa. The moment-curvature curves with slip strain defined by eqn. (8) are shown in Figs. 7 and 8, which are the
same those obtained by Rotter and Ansourian (1979). With an increase of the slip parameter from no slip ($\delta = 0$) to partial slip ($\delta = 0.5$), the strength decreases, but the ductility increases.

The test results reported by Uy and Sloane (1998) were selected here because they were of full-scale beams and the material properties were well-documented. In their experiments, the beam B1 had a degree of PSC of $\eta = 0.5$. The computer method developed herein is compared with the test results for this beam in Fig. 9 for its moment-curvature response and in Fig. 10 for the development of slip strain. It can be seen that the model is very accurate for stiffness, however the comparisons for the ultimate strength and ductility of the beam B1 is not relevant for larger curvatures as it was reported that the test was stopped prematurely due to physical constraints. Before the premature halting of the test, there is good agreement between the two curves in Figs. 9 and 10; the experimental curve is slightly more rounded than the computer prediction because the numerical model does not account for residual stresses in the steel beam. It can also be seen from Fig. 9 that the shear connection starts to fail at a curvature of around $15 \times 10^{-6}$ mm$^{-1}$. This is evident in Fig. 10, where the theory predicts well the curvature at which the slip strain increases suddenly as the PSC response at ultimate is reached.

4. PARAMETRIC STUDY

4.1 General

The parameters investigated in the study here include: the degree of shear connection, the concrete strength, the yield stress of steel, the geometric proportions of the concrete slab, the geometric proportions of the steel I-section joist, the reinforcement in the slab and the slip strain between the joist and slab. The effects of variations in these parameters on a composite cross-section are illustrated in the following.

The steel joist chosen is an Australian 200UB29.8 and a concrete slab is 915 mm wide by 102 mm thick for most studies. For transparency in the comparison, the moment $M$ is normalised with respect to $M_{\text{PSC}}$ which is the PSC moment capacity obtained from eqn. (2), and the curvature $\kappa$ is normalised with respect to $\kappa_{\text{PSC}}$, where $\kappa_{\text{PSC}} = M_{\text{PSC}}/(E_{s}I_{s})$ in which $E_{s}I_{s}$ is the
flexural rigidity of the composite section computed using transformed area principles (Oehlers and Bradford 1995, 1999).

4.2 Effect of Degree of Shear Connection

Figures 11, 12 and 13 show the effect of the degree of shear connection $\eta$ on the moment-curvature responses with different concrete and steel material properties. In this study, the value of $\eta$ was varied from 0.2 to 1, and the normalised PSC curvature $\kappa_{PSC}$ has been determined for FSC throughout, using $\eta = 1.0$. In Fig. 11, $f_c = 32$ MPa, $f_y = 250$ MPa, in Fig. 12 a higher steel yield strength $f_y = 350$ MPa is used, and in Fig. 13 a higher concrete strength $f_c = 50$ MPa is used. It can be seen from these figures that:

- The flexural capacity $M_{PSC}$ calculated from simple rigid plastic analysis is mostly attained for all degrees of shear connection, except for the higher strength steel with a high degree of shear connection, and indeed in Fig. 12 it is clear that the simple rigid plastic strength is not reached for values of $\eta$ greater than 0.6.

- The beams with lower degrees of shear connection display a substantial strain-hardening response, which may be described as very ductile; while the beams with higher degrees of shear connection exhibit a shorter strain-hardening region or no strain-hardening range (again as shown in Fig. 12, when $\eta > 0.6$), which may be described as much less ductile. It can therefore be deduced that the beams with lower degrees of shear connection tend to provide greater ductility which is required for plastic design.

- The moment capacity of beams with lower degrees of shear connection can be significantly higher than that calculated using rigid plastic analysis due to strain-hardening behaviour. However, the use of a higher moment capacity above $M_{PSC}$ in these cases is undesirable because the attainment of this maximum moment is characterised by abrupt crushing of the concrete with an accompanying brittle response.
The effects of the steel yield stress on the moment-curvature responses of the beam with $\eta = 0.4$, $0.6$ and $0.8$ are as shown in Figs. 14, 15 and 16 respectively, in which $f_s = 32$ MPa. The value of $\kappa_{pse}$ has been determined in the figures for $f_s = 250$ MPa. These figures show that:

- The steel yield strength has a strong influence on the beam behaviour. The beam with lower $f_s$ possesses a greater strain-hardening region and higher moment capacity because the concrete in the slab can still balance the force in the steel. Increasing $f_s$ results in decreases in both the ductility and the normalised moment capacity $M/M_{pse}$ owing to the brittle nature of failure by concrete crushing.

- For a given concrete strength and a given degree of shear connection, every cross-section has a limiting value of yield stress above which the behaviour is strain-softening, which is a similar observation to that noted by Rotter and Ansourian (1979) for FSC. With the increase of value of $\eta$, the limiting value of $f_s$ inducing strain-softening behaviour decreases.

It can be seen that due to poor ductility and an inability to attain the plastic moment using rigid plastic assumptions, a beam with high steel strength is unfavourable for plastic design.

4.3 Effect of Concrete Strength

The moment-curvature curves for different concrete strengths with degree of shear connection $\eta = 0.4$, $0.6$ and $0.8$ are shown in Figs. 17, 18 and 19 respectively. In this investigation, the value of $\kappa_{pse}$ has been determined with $f_s = 17.5$ MPa. The concrete cylinder strength $f_c$ has only a slight effect on the ultimate moment of composite beams if the concrete is above $f_c = 32$ MPa. However, the higher strength concrete tends to produce a longer strain-hardening range and therefore an increase in ductility and the moment at crushing, which can be attributed to higher strength concrete allowing large curvatures to develop in the beam because the compressive force can be carried by a narrow slice of concrete at the top of the
slab (Rotter and Ansourian 1979). It can be seen that the effect of the concrete strength on the response of the beam is not as great as that of the steel yield strength.

4.4 Effect of Slab Dimensions

Figures 20 and 21 show the effect of the slab dimensions (width and depth) on the moment-curvature responses for \( \eta = 0.4 \) and 0.6 respectively. For this investigation, the steel beam was a 200UB29.8 with \( f_y = 350 \text{ MPa} \), and \( f_c = 32 \text{ MPa} \) in the slab, and the PSC curvature \( \kappa_{\text{PSC}} \) was determined for a 915×102 slab. It can be seen that the slab width affects substantially the beam behaviour both in terms of its ultimate strength and its ductility. An increase in slab width (from 915 mm to 1830 mm in this example) produces a larger strain-hardening region with improved ductility and a higher ultimate moment. However, the change of slab depth (from 102 mm to 203 mm) has little effect on the ductility and normalised moment \( M/M_{\text{PSC}} \) of composite cross-sections.

4.5 Effect of Steel Joist Size

The behaviour of the two joists 310UB46.2 and 200UB29.8 is shown in Figs. 22 and 23 with \( \eta = 0.4 \) and 0.6 respectively, in which \( f_y = 350 \text{ MPa} \) with a slab 915 mm wide by 102 mm thick with \( f_c = 32 \text{ MPa} \). The results show that the deeper steel joist size produces an earlier onset of strain-hardening and shorter yield plateau and strain-hardening regions, which represents a decrease in ductility.

4.6 Effect of Slab Reinforcement

Figures 24 and 25 demonstrate the influence of longitudinal reinforcement in the slab with the degrees of shear connection \( \eta = 0.6 \) and 0.8 respectively, where the area of reinforcement \( A_r \) is expressed as a proportion of the steel joist area \( A_s \). The reinforcement is placed in a single layer 25 mm below the top of the slab. Unlike the results of the beam with FSC (studied by Rotter and Ansourian 1979), the full plastic moment is unaffected by the addition of slab reinforcement from 0.1\( A_s \) to 0.5\( A_s \) for the beam with PSC. However, increasing the area of reinforcement \( A_r \) can change the cross-section behaviour from strain-softening to strain-
hardening (shown in Fig. 25) or can produce a longer strain-hardening range (in Fig 24). This results in an increase in ductility and an increase in the maximum moment before crushing occurs.

4.7 Slip Strain Between Joist and Slab

The slip strain $\varepsilon_s$ is the step change in the strain profiles at the steel-concrete interface, as shown in Fig. 5(c). In the numerical procedure, once the iterative process for each curvature has been completed, the slip strain can be determined from

$$\varepsilon_s = (d_n - d_{nc}) \kappa$$  \hspace{1cm} (9)

The slip strain given by eqn. (9) is that caused by PSC at a cross-section only, and is not to be confused with that generated by lengthwise partial interaction of an entire beam. The relationships between the slip strain $\varepsilon_s$ and the curvature $\kappa$ are plotted in Fig. 26 with $f_s = 250$ MPa and Fig. 27 with $f_s = 350$ MPa, where the value of $\eta$ varies from 0.2 to 0.8. These curves were used previously in the validation of the numerical method for full shear connection. The unreinforced composite section used in this study is that adopted in the previous study. From the curves in Figs. 26 and 27, it can be seen that the composite section with lower degrees of shear connection undergoes larger slip strains before crushing, and which further illustrates the improved ductility of the section under lower degrees of shear connection. By comparing Figs. 26 and 27, increasing the steel yield strength $f_s$ results in a decrease in the slip strain of the cross-section.

5. CONCLUSIONS

In this report, the issue of the ductility and strength of composite cross-sections with PSC whose strength can be determined using simple rigid plastic analysis has been investigated. A numerical method has been developed for calculating the moment-curvature response of composite cross-sections with PSC and with realistic rather than idealised material behaviour. The numerical procedure has been validated against independent theoretical and experimental investigations. The effects of various parameters on the behaviour of a composite cross-
section have been studied. The general conclusions regarding the strength and ductility are: (1) the full plastic moment calculated from simple rigid plastic analysis is nearly reached for almost all degrees of shear connection except when higher strength steel with higher degrees of shear connection; and (2) beams with lower degrees of shear connection are very ductile while those with higher degrees of shear connection are far less ductile.

6. ACKNOWLEDGEMENTS

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7. REFERENCES


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Figure 1: Idealised rigid-plastic material properties

Figure 2: Rigid plastic behaviour of a cross-section with PSC
**Figure 3:** Stress-strain relationship for steel joist

**Figure 4:** Load-slip relationship for shear connection
Figure 5: CEB-FIP concrete stress-strain relationship

\[ \sigma = \sigma_0 \varepsilon \left( \frac{a - 206.200 \varepsilon}{1 + b \varepsilon} \right) \]

\[ a = 39,000 (\sigma_0 + 7.0)^{0.953} \]

\[ b = 65,000 (\sigma_0 = 10.0)^{-1.085} - 850.0 \]

\[ \sigma_0 = 0.85 f_c \]
Figure 6: Cross-section subdivision and strain diagrams
**Figure 7:** Effect of interface slip with 915×102 mm² slab

**Figure 8:** Effect of interface slip with 1830×102 mm² slab
Figure 9: Moment-curvature comparison of numerical model with tests of Uy and Sloane (1998)

Figure 10: Slip strain comparison of numerical model with tests of Uy and Sloane (1998)
Figure 11: Effect of degree of shear connection for $f_p = 250$ MPa

Figure 12: Effect of degree of shear connection for $f_p = 350$ MPa
Figure 13: Effect of degree of shear connection with $f_c = 50$ MPa

Figure 14: Effect of steel yield stress with $\eta = 0.4$
Figure 15: Effect of steel yield stress with $\eta = 0.6$

Figure 16: Effect of steel yield stress with $\eta = 0.8$

Figure 17: Effect of concrete strength with $\eta = 0.4$
Figure 18: Effect of concrete strength with $\eta = 0.6$

Figure 19: Effect of concrete strength with $\eta = 0.8$

Figure 20: Effect of slab size with $\eta = 0.4$
Figure 21: Effect of slab size with $\eta = 0.6$

Figure 22: Effect of joist size with $\eta = 0.4$

Figure 23: Effect of joist size with $\eta = 0.6$
Figure 24: Effect of slab reinforcement with $\eta = 0.6$

Figure 25: Effect of slab reinforcement with $\eta = 0.8$

Figure 26: Slip strain versus curvature for $f_y = 250$ MPa
Figure 27: Slip strain versus curvature for $f_y = 350$ MPa